Effect of Beach Slope and Grain-Stabilization on Coastal Sediment Transport: An Attempt to Overcome the Erosion Overestimation by XBeach.

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Abstract: The reproduction of seventeen laboratory tests for dune erosion by the surf-beat mode of the two-dimensional horizontal (2DH) model XBeach for storm wave conditions showed that the model overestimates the erosion and thus the deposition volumes. This overestimation becomes particularly significant for very large overtopping rates on coastal barriers, which are commonly induced by extreme storm surges. In this paper, the recent model improvements to overcome this problem are first summarised. Moreover, two physical reasons for these overestimations are identified: (i) The wave nonlinearity effect on sediment transport, which is described in XBeach by a calibration factor for the time-averaged flow depending on the wave skewness and asymmetry and (ii) The considerable excess of the actual shear stress required to initiate the sediment particle motion as compared to that predicted by the common Shields curve. To improve the prediction capability of XBeach in terms of erosion and overwash volumes, two new improvements, related to the two aforementioned reasons for the overestimation, are introduced and implemented in the model. The improved XBeach model is then tested for dune erosion, for barrier breaching as well as for a barrier island erosion and overwash under an extreme storm surge event. The results showed a very good prediction capability of the improved model. Particularly, the second model improvement opens the way toward further model improvements to account for spatially varying soil resistance, which is crucial for reliable prediction of a barrier breaching.

Keywords: XBeach; Wave nonlinearity; Grain-stabilization; Soil resistance; Dune erosion; Breaching; Numerical modelling

1. Introduction

Dune erosion, overwash and breaching are the major storm-induced threats to coastal safety and sustainable development in the areas where frontal dune systems or barrier islands are present [1]. Indeed, coastal dune systems provide a natural defence against erosion and coastal flooding. However, the dune crest height can be rapidly reduced, mainly in lower crest zones along the barrier during overwash and overflow regimes [2]. This is due to sediment being transported both landwards and seawards from the dune [3,4], which might lead to breaching of coastal barriers and catastrophic floods. Therefore, the need for predicting the response of coasts to storms is increasing steadily as the loads on the coastal systems increase due to rising sea levels and the possibility of more intense storms [5,6]. This need is also rising as the population in coastal areas continues to grow worldwide. 20 million people live under current high tide levels and 200 million under storm tide levels [7,8]. Moreover, approximately 10% of the world's coastline consists of barrier coasts [9], which are prone to beach/dune erosion and coastal flooding.

In the flooding prone European states, especially the Netherlands and England after the 1953 floods, holistic and empirical rules (e.g. [10]) and statistical approaches (e.g. [11]) were commonly used to assess coastal safety [12,13]. However, such approaches commonly use empirical models based on the assumption of alongshore uniformity that not necessarily holds along many coasts. Therefore, processes-based approaches and modelling are necessary to calculate erosion volumes considering the longshore variability of both hydraulic loading and resistance parameters of the coastal barriers [14]. Recently, considerable efforts have been made to better understand storm surge induced erosion, overwash and breaching of coastal dunes using physically-based numerical models. The 2DH model "XBeach" [15] represents one of the latest developments and an off-the-shelf model, which is able to

assess the vulnerability of coastal areas to coastal erosion induced by extreme storm surges and to evaluate the performance of existing and future coastal protection projects [16,17]. Moreover, it is an open-source and freely available code, which means that users can make changes in the code to suit the requirements of individual projects. Therefore, the model is being continuously improved through diverse applications in different coastal environments worldwide. Given the capabilities of the XBeach model (e.g. depth-averaged shallow water equations including subcritical and supercritical flow, time-varying wave action balance, depth-averaged advection-diffusion equation, morphological evolution and bed update), many studies worldwide (e.g. [18–23]) have been attempting and still attempt to further develop and extend the model in order to include further nearshore processes in addition to those initially considered in the model.

XBeach describes processes in the four flow regimes proposed by Sallenger [2], who defined an impact level to denote different regimes of impact on dune and barrier islands by storm surges: (i) swash regime, (ii) impact/collision regime, (iii) overwash regime and (iv) overflow/inundation regime. When a dune system is subject to any or to a combination of the latter three regimes, the likelihood of its failure increases. Thus, three failure mechanisms are possible for dune systems exposed to storm conditions [24]. The first mechanism is caused by the impact of breaking waves on the seaward slope during the collision regime when the water level is lower than the dune crest. This process may induce external erosion on the seaside in addition to dune crest lowering due to interaction between seaside avalanching and offshore sediment transport under return flow effect [22]. Consequently, breach development from the seaward side becomes possible. The second mechanism is caused by overtopping of waves during the overwash regime that may result in external erosion on the landward slope accompanied by lowering of the dune crest. As a third mechanism, overflow may occur during a storm when the mean water level exceeds the crest (inundation regime). In this case, the continuous erosion on the landward side is relatively intense and can lead to a breach. For both overwash and overflow, the breach initiation is on the landward slope of the dune while the eroded sediment deposits landward in the form of sediment fans [3,4,22].

The hydrodynamics and morphodynamics of XBeach were extensively calibrated and validated against (1D) flume experiments (e.g. [15,25,26]) and some (2DH) field cases (e.g. [15]): the model showed qualitatively a good agreement with measured cross-shore measurements of dune erosion and overwash. However, the overwash morphology is validated for the first time using the 2DH field case Santa Rosa Island under hurricane Ivan [27,28]. Such hurricane events often result in significant overtopping rates and large flow velocities, especially through breach induced inlets [29]. The results of the Santa Rosa simulations have shown that the XBeach substantially overestimates the erosion volumes and consequently the washover volumes [20,27,28]. As reported by McCall et al. [27] and De Vet [20], the overestimation of the erosion rates for the specific Santa Rosa case is certainly due to the limitations of the morphodynamic module of XBeach to reproduce sediment transport with sufficient accuracy rather than to the limitations of the hydrodynamic module. In fact, various studies (e.g. [15,16,20,30,31]) reported that XBeach is generally capable of predicting hydrodynamics properly. In some cases, however, the sediment transport rates are overestimated, especially during extreme storm surge conditions where significant overtopping rates and high flow velocities are expected. Therefore, McCall et al. [27] suggested that some artificial limiters for the sediment transport should be introduced in XBeach so that the transport rates induced by the high-velocity flow are properly simulated. As a result, they limited the equilibrium sediment concentration in XBeach under sheet flow conditions by reducing the stirring velocity in the sediment transport calculations. Although this suggestion is not physically-based, it allowed achieving an acceptable agreement between the observed and simulated post-storm profiles. On the other hand, the introduced limiter for the sediment transport showed unexpected side effects and have in some cases led to significantly worse predictions than without any limiter [20,32].

In another attempt, De Vet [20] suggested another artificial limiter by simply restricting the total sediment concentration, but this limiter also showed worse predictions. Therefore, De Vet attempted to introduce physically-based means to reduce the erosion rates. For instance, he implemented in

XBeach an improved formulation for the bed slope effect on incipient sediment motion as well as the concept of hindered erosion by soil dilatancy, which was formulated by [33,34]. The extended model of De Vet is tested using several laboratory and field studies (e.g. [35]), showing an improved prediction performance. However, in some cases, especially in the case of Santa Rosa Island, the prediction performance of XBeach with De Vet's extension was worse. Moreover, the results of the numerical reproduction of 17 available physical model experiments on dune erosion showed that the De Vet's extension does not solve fully and satisfactorily the overestimation problem.

Given that XBeach is recently involved in operational modelling applications systems for coastal flood forecasting and alerting in many European states [36–39] besides its recent use to estimate inland flow during events of storm surges [8,22,40,41], there is an urgent necessity to address the overestimation problem using new physically-based means so that reliable alerting and reasonable inland discharges can be achieved. Therefore, the purposes of this paper are (i) to review the recent state of the art attempts to overcome the erosion overestimation by XBeach (ii) to document the aforementioned numerical simulations, (iii) to propose two new improvements and to implement them in XBeach as a contribution to the solution of the overestimation problem and finally (iv) to validate the improved XBeach with the two implemented improvements. For this purpose, the theoretical background of the current XBeach code corresponding to the two new planned improvements are first briefly introduced in section 2, which is ended by review of recent attempts to solve the overestimation problem. The results of the numerical reproduction of the 17 physical tests are discussed in section 3. The two proposed improvements are then formulated in section 4. The validation of the improved model is performed in section 5. Finally, a discussion and conclusions of the results are provided in sections 6 and 7, respectively.

2. Theoretical Background

XBeach solves the nonlinear shallow water equations (NLSWEs), including the capabilities for time-varying wave action balance, roller energy balance and the advection-diffusion equation [15]. A full explanation of the hydrodynamic formulation can be found in [16,42]. However, in this paper, the focus will be on the sediment transport calculations, advection-diffusion equation and the effects of wave nonlinearity on coastal sediment transport.

2.1. Sediment Transport Formulation in the Current XBeach Model

Different sediment transport equations are implemented into XBeach (revision 4812) to calculate bed, suspended and total sediment loads that allow flexibility in modelling different types of environment. These are the Soulsby (1997 [43]) and the Van Rijn (2007 [44–47]) formulations, modified by Van Theil de Vries (2009 [26]). Model user selects one out of these formulations to determine the bed boundary condition (equilibrium sediment concentration C_{eq}) for the depth-averaged advection diffusion of Galappatti and Vreugdenhil (1985 [48]).

$$\frac{\partial d \cdot C}{\partial t} + \frac{\partial d \cdot C \cdot u^E}{\partial x} + \frac{\partial d \cdot C \cdot v^E}{\partial y} + \frac{\partial}{\partial x} \left(D_h d \frac{\partial C}{\partial x} \right) + \frac{\partial}{\partial y} \left(D_h d \frac{\partial C}{\partial y} \right) = \frac{d \cdot (C_{eq} - C)}{T_s} \quad (1)$$

Eq. (1) shows that the mass-balance of suspended sediment in flowing water can be expressed in the form of a partial differential equation that describes the processes of convection, turbulent diffusion and sediment mobilisation/deposition in terms of the local sediment concentration. In Eq. (1), C represents the depth-averaged sediment concentration which varies on the wave-group time scale, D_h is the sediment diffusion coefficient and d is the water depth while u^E and v^E are the Eulerian wave-induced flow velocities in cross-shore and longshore directions, respectively. u^E and v^E are defined as the short-wave-averaged velocity observed at a fixed point and are computed from the hydrodynamic module of XBeach, which solves a Generalized Lagrangian Mean (GLM) form of the NLSWEs [49,50]. The entrainment of sediment is represented by an adaptation time T_s (the elapsed time for C to reach C_{eq}), which is given by a simple approximation based on local water depth d and sediment fall velocity w_s as in Eq. (2). A small value of T_s corresponds to nearly instantaneous sediment response.

$$T_s = \max \left\{ T_s \cdot \frac{d}{w_s}, T_{s,min} \right\} \quad (2)$$

f_{T_s} is a correction and calibration factor to account for the fact that the mass-balance for a uniform suspended sediment of fall velocity w_s is determined based on depth-averaged flow conditions [20,48,51,52] (default value of f_{T_s} is 0.1) and the minimum adaptation time $T_{s,min}$ is 0.5 sec by default [51]. The entrainment or deposition of sediment is determined by the mismatch between the depth-averaged sediment concentration C and the equilibrium concentration C_{eq} that represents the source term in Eq. (1).

The differential equation for the advection-diffusion of sediment is solved to obtain C using a finite difference approach and a first order upwind scheme, with the water depths from previous time steps and the corresponding velocities at the updated time step [16,42,52]. As a result, the bed update is calculated based on continuity considerations (Exner Equation):

$$\frac{\partial z_b}{\partial t} = -\frac{f_{mor}}{1-n} \left(\frac{\partial q_{sx}}{\partial x} + \frac{\partial q_{sy}}{\partial y} \right) \quad (3)$$

z_b represents bed level, f_{mor} is a morphological factor added to the Exner equation to speed up the calculation, parameter n represents the porosity of the bed. q_{sx} and q_{sy} are the sediment transport rates in cross-shore and longshore directions, respectively, that are calculated based on the depth-averaged concentration C and the advection and dispersion fluxes as:

$$q_{sx} = d \cdot C \cdot u^E + D_h \cdot d \cdot \frac{\partial C}{\partial x} \quad \text{with} \quad q_{sy} = d \cdot C \cdot v^E + D_h \cdot d \cdot \frac{\partial C}{\partial y} \quad (4)$$

q_{sx} and q_{sy} are the volume flux of sediment, excluding pores, through a cross-section perpendicular to the axis indicated by the subscript while water depth d and flow velocities u^E and v^E are calculated from the NLSWEs.

Many studies (e.g. [20,53]) reported that the prediction of equilibrium sediment concentration C_{eq} is better using Van Rijn-Van Thiel formulation than using the Soulsby-Van Rijn formulation. However, C_{eq} is calculated from the outcomes of both formulations as follows

$$C_{eq} = \max \left\{ \min \left(C_{eq,b}, \frac{1}{2} C_{max} \right) + \min \left(C_{eq,s}, \frac{1}{2} C_{max} \right) \right. \\ \left. 0 \right\} \quad (5)$$

Where C_{max} is a user-defined parameter (By default $0.1 \text{ m}^3/\text{m}^3$) while the equilibrium sediment concentrations $C_{eq,b}$ and $C_{eq,s}$ represent, respectively, the bed and suspended loads contribution in the total equilibrium concentration C_{eq} and are calculated according to Soulsby-Van Rijn formulation as follows

$$C_{eq,b} = \frac{A_{sb}}{d} (U_{stirring} - U_{cr})^{2.4} \quad \text{with} \quad C_{eq,s} = \frac{A_{ss}}{d} (U_{stirring} - U_{cr})^{2.4} \quad (6)$$

Where $U_{stirring}$ is the depth-averaged sediment stirring velocity, U_{cr} is the critical stirring (threshold) flow velocity for sediment movement that is based on Shields curve [54] and A_{sb} and A_{ss} are the bed and suspended load coefficients, respectively (see e.g. [16] for details). If the Van Rijn-Van Thiel formulation is selected instead of Soulsby-Van Rijn formulation, then $C_{eq,b}$ and $C_{eq,s}$ take the following form

$$C_{eq,b} = \frac{A_{sb}}{d} (U_{stirring} - U_{cr})^{1.5} \quad \text{with} \quad C_{eq,s} = \frac{A_{ss}}{d} (U_{stirring} - U_{cr})^{2.4} \quad (7)$$

The critical velocity in Eq. (7) is computed as a weighted summation of the separate contributions of currents U_{cr_c} based on Shields curve [54] and waves U_{cr_w} based on Komar and Miller [55]

$$U_{cr} = \alpha \cdot U_{cr_c} + (1 - \alpha) \cdot U_{cr_w} \quad (8)$$

Where α is a weighting factor based on whether the long wave stirring is turned on or not? The stirring velocity $U_{stirring}$ is defined as in Eq. (9) for both Soulsby-Van Rijn and Van Thiel-Van Rijn formulations.

$$U_{stirring}^2 = \begin{cases} (u^E)^2 + (v^E)^2 + 0.018 \frac{u_{rms}^2}{c_d} & \text{(Soulsby-Van Rijn)} \\ (u^E)^2 + (v^E)^2 + 0.64 u_{rms}^2 & \text{(Van Thiel - Van Rijn)} \end{cases} \quad (9)$$

c_d is a drag coefficient and u_{rms} is the root-mean-squared orbital velocity or the at-bed short wave orbital velocity that is obtained from the wave group varying wave energy using the linear wave theory.

$U_{stirring}$, in addition, can be represented in terms of effective Shields parameter θ , median sediment size D_{50} , relative density of sediment s ($\rho s/\rho$), gravitational acceleration g and non-dimensional friction coefficient C_f as follows.

$$U_{stirring}^2 = \theta \cdot \frac{g D_{50} (s-1)}{C_f} \quad \text{with} \quad c_f = \sqrt{g/C_{ch}^2} \quad \text{or} \quad c_f = \sqrt{\frac{g n^2}{d^{12}}} \quad (10)$$

Where C_f can be calculated based on the roughness coefficient of Chezy c_{ch} or Manning n as in Eq. (10). The critical stirring velocity U_{cr} is mainly a function of the sediment properties (see e.g. [16] for the details). However, it can also be expressed as a function of the critical Shields parameter θ_{cr} as follows:

$$U_{cr}^2 = \theta_{cr} \cdot \frac{g D_{50} (s-1)}{C_f} \quad (11)$$

2.2. Effect of Wave Nonlinearity on Sediment Transport in the Current XBeach Model

In XBeach, one of the most important aspects of wave-induced sediment transport and cross-shore profile evolution is the mechanism through which asymmetric short waves transport sediment onshore to counteract the effects of gravity [18,56] (offshore sediment transport under the wave rundown effect) knowing that other return flow effects, e.g. undertow current, is directly included in XBeach through the depth-averaged flow equation [56,57]. Indeed, a key parameter for cross-shore sediment transport under breaking and near-breaking waves is the wave shape and the induced shape of the near-bed wave orbital velocity [58–61]. Generally, a harmonic velocity field will result in no cross-shore sediment transport because both the time history of flow velocity and acceleration are symmetric with respect to the positive and negative phase. However, a skewed velocity field can cause a net cross-shore transport of sediment (that moves onto the onshore direction).

XBeach considers the wave energy of short waves as averaged over their length, and hence does not directly simulate the wave shape [16,35]. By other words, XBeach does not explicitly compute short intra-wave processes since it only calculates the short-wave energy balance and does not contain phase information of the short waves [18,62]. However, wave shapes gradually change when they propagate to the nearshore due to the combined effect of wave shoaling, breaking and nonlinear interactions [59,61]. As waves propagate from deep water onto beaches, the water surface elevation becomes increasingly nonlinear due to the amplification of the higher harmonics. As a result, the time histories of both near-bed flow velocity and acceleration also become asymmetric; i.e. the skewness (velocity skewness) and asymmetry (acceleration skewness) also increase. Skewness is defined as the relative difference between the larger orbital velocities under the wave crest and the lower orbital velocities under the wave trough while asymmetry is defined as the relative measure of the larger accelerations between wave trough and crest as compared to the smaller accelerations between wave crest and trough [58]. The increase in skewness and asymmetry has important implications for the wave-induced net cross-shore sediment transport as it results in an increase of onshore sediment transport. In order to indirectly account for these changes of the wave shape within XBeach, Van Thiel de Vries [26] proposed a skewness and asymmetry discretization model that takes the following form

$$u_a = \gamma_{ua} u_{rms} (S_k - A_s) \quad (12)$$

Besides the sediment transport under the effect of depth-averaged flow, the model of Van Thiel de Vries in Eq. (12) adopts a net flow velocity u_a that stirs sediment to transport onshore under effect of nonlinear waves, where the values of wave skewness S_k and wave asymmetry A_s are calculated using either the formulation of Van Thiel de Vries [26] or the formulation of Ruessink et al. [63]. As a result, the flow velocity u_a can be calculated as a function of S_k , A_s , u_{rms} and a calibration factor γ_{ua} as in Eq. (12). In case of or on the locations where waves are linear (symmetric), u_a vanishes because both S_k and A_s equal zero. However, with highly nonlinear waves, higher values for u_a are expected since the difference between S_k and A_s is considerable. Consequently, higher values for u_a will result in higher onshore sediment transport [16,64]. The parameter γ_{ua} is defined in XBeach by the keyword *facua* (factor for u_a) and has a default value of 0.1. It represents in fact one of the most important parameters in XBeach as it is the only parameter mostly affecting the net cross-shore sediment transport [53,65]. Therefore, many studies (e.g. [35,53,66,67]) reported that *facua* might be one of the reasons behind the

overestimation problem by the current XBeach model that usually need to be increased (than the default value 0.1) in order to achieve better prediction of the erosion rates. Higher values for $facua$ would increase u_a . As a result, sediment transport due to the wave nonlinearity would accordingly increase in the onshore direction, thus resulting in a reduction the over-predicted erosion rates. The latter is possible because u_a is used in both the advection-diffusion Eq. (1) and in the sediment transport Eq. (4) by replacing the Eulerian velocities u^E and v^E by $(u^E + u_a \sin \theta_m)$ and $(v^E + u_a \cos \theta_m)$, respectively, where θ_m is the angle of wave incidence with respect to the cross-shore direction.

2.3. Previous Attempts to Reduce the Erosion Rates Over-predicted by XBeach

In order to overcome the overestimation of the erosion rates by XBeach under significant overtopping rates and high flow velocities, which are common under extreme storm surge conditions, previous studies such as those by De Vet (2014 [20]), McCall et al. (2010 [27]) and Terlouw (2013 [32]) suggested two artificial limiters as well as three physically-based limiters for the sediment transport (Fig. 1).

Fig. 1. Artificial and physically-based sediment transport limiters to reduce the erosion rates over-predicted by XBeach.

Artificial sediment transport limiters

Effective Shields parameter θ : it is restricted by a threshold value θ_{sf} at which sediment sheet flow occurs. In this case, the stirring velocity (Eq. 9) is restricted to the velocity U_{sf} that corresponds to a Shields parameter value at which sheet flow of sediments occurs (usually at $\theta_{sf} \geq 1$). The latter is possible through setting θ equals θ_{sf} (i.e. equals 1.0) in Eq. (10). As a result, this limiter sets an upper boundary for the difference between the stirring and the critical velocities in Eq. 6 and 7 because it forces the sediment stirring velocity calculated from Eq. (9) not to exceed U_{sf} . Consequently, this limiter sets an upper boundary limit for the equilibrium concentration C_{eq} as a source term in Eq. (1) and thereby reduces the depth average sediment concentration, sediment transport calculated by Eq. (4) and bed changes calculated by Eq. (3).

Sediment concentration: This limiter, suggested by Terlouw [32] and De Vet [20], is the restriction of the maximum sediment concentration C_{max} in Eq. (5) to a user-defined value instead of the current value $C_{max} = 0.1 \text{ m}^3/\text{m}^3$, e.g. the maximum sediment concentration suggested by De Vet [20] is $C_{max} = 0.3 \text{ m}^3/\text{m}^3$. As a result, this limiter also sets an upper boundary for the equilibrium sediment transport because it forces to select the input user-defined sediment concentration if the calculated one exceeds the user-defined value.

Both artificial limiters showed unexpected side effects and in some cases led to significantly worse predictions than without any limiters [20,32]. Moreover, they are rather artificial without any solid physical basis. Therefore, the overestimation of the erosion rates by XBeach means that it is due to either unknown or excluded physical processes that need to be explored and added to the model.

Consequently, other studies attempted to improve the XBeach prediction capability by including new physical processes such as the hindered erosion due to soil dilatancy [20,33,34], the effect of high sediment concentrations on fall velocity [20] and the bed slope effect on incipient sediment motion [20,68].

Physically-based sediment transport limiters

Hindered erosion by soil dilatancy: Van Rhee [33,34] reported that the erosion induced by high flow velocities might be affected by the bed soil dilatancy, which hinders the sediment pick up and reduces the sediment transport. As a result, Van Rhee derived a formula that amplifies the value of the critical Shields parameter θ_{cr} based on the soil properties. The latter increases the critical Shields parameter in Eq. (11) and therefore the critical velocity required to initiate the sediment particle motion increase. As a result, the difference between the stirring and the critical velocities in Eq. 6 and 7 reduces which reduces erosion rates accordingly. By other words, dilatancy causes an extra inward hydraulic gradient on the soil particles that increases the resistance of the soil mass to erosion. Although dilatancy is proven to be one of the most important erosion hindering processes, implementing the dilatancy effect to XBeach does not sufficiently reduce the erosion to achieve proper results [20,35].

Effect of high sediment concentrations on fall velocity: High sediment concentrations reduce the settling velocity w_s [69] and hence decrease sediment response to deposition/entrainment which at the end reduces sediment transport. De Vet [20] implemented to XBeach the effect of high sediment concentrations on fall velocity and reported that this effect might slightly reduce the erosion rates over-predicted under extreme flow conditions.

Bed slope effect on incipient sediment motion: the critical Shields stress for initial sediment motion increases with bed slope, which indicates that particles of the same size are more stable on steeper slopes [68,70,71]. As an attempt to reduce the erosion rates in XBeach, the latter fact is recently implemented in XBeach through reducing the sediment discharges (Eq. 4) calculated based on the common Shields curve [20,51]. However, considering the bed slope effect does not sufficiently reduce the erosion to achieve proper results [20,35].

The result of the previous studies as well as the outcomes of the numerical reproduction of large wave flume tests (GWK tests) motivated toward proposing new physically-based alternatives as a contribution to solve the aforementioned over-prediction problem.

3. Numerical Reproduction of Large-Scale Model on Dune Erosion Tests in GWK

XBeach is applied to reproduce seventeen large-scale model tests data on dune erosion performed on 2013 by Leichtweiß-Institute (LWI) in the Großer Wellenkanal (GWK) of the Coastal Research Centre (FZK), a joint institution of both universities Hannover and Braunschweig. The purpose of these GWK tests was to understand the erosion processes induced by the impact of storm waves on the front face of the sand dune of the coast of the Wangerooge Island, Germany under extreme storm surge events. However, in this study, the GWK-data are exclusively used to assess the performance of XBeach.

3.1. Physical Model Setup in GWK

The seventeen tests were performed on five cross-shore profiles (Fig. 2) in GWK, which is 310 m long, 5 m wide and 7 m deep. Each profile consists of three main zones: erodible foreshore (green part), the revetment zone (magenta part) and the dune extension (yellow part).

The erodible foreshore has the same levels and dimensions in the five profiles. However, the revetment zone and the dune extension differ. The revetment zone in profiles 1 and 2 has the same dimensions and levels, while for profiles 3, 4 and 5 the same dimensions and levels prevail, but different from those in profiles 1 and 2. For all profiles, the dune extension with a frontal slope 1:1 is built behind the revetment crest. The dune front starts at level 5.17 m for profile 1 (level 0.00 represents the flume bottom). In profile 2, the dune extension also starts at level 5.17 m, but the dune front has an offset of 3.33 m behind the revetment crest (Fig. 2). The dune extension for profiles 3, 4 and 5 starts at level 4.21 m and has no dune offset for profile 3, but both profiles 4 and 5 have offsets of 3.33 m and 6.66 m, respectively.

The tests aimed at understanding the effect of different dune offset on the erosion of the dune extension zone. Moreover, they aimed at testing the effect of different overtopping rates on the dune erosion, where variable overtopping rates are estimated using the empirical model EurOtop [72] based on varying the water depth in the flume. In summary, the GWK tests are performed considering the test conditions in Table 1 whilst the dune erosion is measured using a 3D scanner after each individual laboratory test.

Fig. 2. Profiles of the performed tests in GWK (all levels and dimensions are in meters and level 0.00 represents the flume bottom): left panel shows profiles 1 and 2 while right panel shows profiles 3, 4 and 5; differences among profiles in both panels, separately, lies in the offset of the dune front at the dune extension zone.

Table 1: Test conditions for dune erosion in GWK and their counterpart names in XBeach

GWK Test Nr.	Profile type	Overtopping rate	SWL	Wave type	Time	XBeach test Nr.	
[-]	[-]	q_{nom} [l/(s·m)]	[m]	[-]	[s]	[-]	
Test 01	Profile 1	1.92	3.66	Wave spectrum from GWK wave maker (irregular waves with significant wave height of 1.1 m and peak period of 6.6 s)	5740	XB1	
Test 02	Profile 1	<1.92	3.40		6702	XB2	
Test 03	Profile 1	1.92	3.66		5740	XB1	
Test 04	Profile 1	16.35	4.00		6705	XB3	
Test 05	Profile 1	192	4.50		2151	6492	XB4
Test 06					2156		
Test 07					2185		
Test 08	Profile 2	16.35	4.00		6632	XB5	
Test 09	Profile 3	423	4.00		2328	6979	XB6
Test 10					2316		
Test 11					2335		
Test 12	Profile 4	423	4.00		2311	6393	XB7
Test 13					1782		
Test 14					2300		
Test 15	Profile 5	423	4.00		2309	8908	XB8
Test 16					2308		
Test 17					2291		

In Table 1, Test 01, for instance, is performed on profile 1. The still water level (SWL) in the flume is 3.66 m, the planned overtopping rate is 1.92 l/s/m, the test duration is 5740 s and the generated waves have a significant wave height of 1.1 m with a peak period of 6.6 s (wave maker at distance 68 m from the toe of the erodible foreshore). These waves are measured by different wave gauges (see [73] or [74] for further details). However, the readings of the installed wave gauge above the toe of the erodible foreshore (Fig. 2) is used only to feed XBeach by the offshore wave boundary condition after performing the wave reflection analysis by the software L-davis (a reflection analysis software developed by LWI). As shown in Table 1, some GWK tests (e.g. Test 05 to Test 07) were performed on the same profile without reshaping the dune front to its initial state. Therefore, these tests are also performed in a single

XBeach test. In the last column of Table 1, the XBeach test number corresponding to the GWK test(s) is specified, e.g. GWK test 01 corresponds to XBeach test XB1 while GWK tests 05 to 07 correspond to the single XBeach test XB4. Moreover, GWK Tests 01 and 03 have the same conditions and are therefore reproduced only by single XBeach test, which is XB1.

3.2. Numerical Model Setup

Though the cross-shore profiles in the GWK tests can be simulated in XBeach as 1D profiles, the GWK tests are reproduced as 2DH models in order to reproduce the same laboratory conditions. Therefore, five topographies are generated, where each topography is defined by x-file, y-file and bed-file that contain the x, y and z-coordinates of the calculation meshes. As a result, grid resolution of 0.2 m is applied to each numerical test. Another five files are generated to specify the hard bed (revetment) zone for each profile. In addition, the same laboratory conditions and boundaries as in GWK are reproduced in XBeach. Table 2 summarises the main parameters used in the parameter file of XBeach in order to reproduce the same conditions and boundaries of the physical models in GWK.

Table 2: Main parameter settings and boundary conditions for the reproduction of GWK tests in XBeach

Parameter	Value	Note
D50	0.16 mm	Median grain size
D90	0.275 mm	D ₉₀ grain size
front	1	Absorbing-generating and weakly-reflective front side as 2D inflow boundary
left	1	Side wall as impermeable lateral boundary
right	1	Side wall as impermeable lateral boundary
back	2	Absorbing-generating and weakly-reflective back side as 2D outflow boundary
instat	6	Formatted variance-density spectrum file with records from wave gauge in Fig 2 after performing the reflection analysis is used as offshore wave boundary.
lateralwave	Neumann	Neumann boundary is a lateral wave boundary on both lateral sides of the model
cf	0.00062	Nondimensional friction coefficient (based on D ₅₀).
form	2	Sediment transport is calculated according to Van Thiel-Van Rijn formulation
morfac	10	Factor in Exner equation (Eq. 3) to accelerate calculations of morphological evolution
tstop	Table 1	Simulation time in XBeach = time duration of GWK test(s)

The sediment sizes are obtained from the distribution curve of the sand used in the flume. Based on the median grain size D_{50} , the bed friction coefficient c_f is calculated. Because the flume has sidewalls that prevent lateral water flow, lateral boundaries are selected as impermeable walls. Moreover, the SWL values in Table 1 are assigned to each XBeach test. Furthermore, the measured waves at the wave gauge shown in Fig. 2 is assigned to XBeach after performing the reflection analysis (i.e. incident wave spectrum) using a formatted variance-density spectrum file, defined in XBeach using instat = 6 or vardens. The other XBeach hydrodynamic and morphodynamic parameters are set at default values.

3.3. Outcomes of the Numerical Simulations

The numerical models of GWK tests are run using a compiled version of XBeach (revision no 4812). The results, in general, showed a relatively reasonable prediction capability for both the scour behind the revetment and the frontal dune erosion. Moreover, they showed that XBeach is capable of simulating the transition zones between hard (the revetment zone) and mobile beds (the dune extension). However, results also showed a better prediction performance with lower overtopping rates (e.g. Test XB3 in Fig 4) than with higher overtopping rates (e.g. Test XB8 in Fig 5).

According to Pender and Karunaratna [75], Sutherland et al. [76] and Van Rijn et al. [77], the accuracy of the post-storm profiles is usually assessed using the Brier Skill Score (BSS) and the root-mean-square error (RMSE). The BSS and the RMSE measures for comparing observed and simulated post-storm profiles are given as follows:

$$BSS = 1 - \frac{\langle (z_p - z_m)^2 \rangle}{\langle (z_b - z_m)^2 \rangle} \quad \& \quad RMSE = \sqrt{\langle (z_p - z_m)^2 \rangle} \quad (13)$$

The angled brackets in Eq. (13) indicate the average of N readings of the bed levels along the cross-shore profile, z_p is the predicted profile from XBeach, z_m is the observed post-storm profile and z_b is

the initial (pre-storm) profile. BSS is in fact a common statistical indicator for the prediction skill relative to a baseline prediction, which is used in almost all morphological studies by XBeach. However, it has some drawbacks. For instance, it cannot account for the migration direction (e.g. for a bar) because it just evaluates whether the computed bed level at specific time is closer to the actually observed bed level at the same time than the initial bed level or not. Moreover, BSS can be extremely sensitive to small changes for low values of the denominator [76,78], which results in large negative scores for the case of model overprediction. Therefore, to avoid such drawbacks, Othman et al. [78] reported that the BSS can be used in absence of other skill scores that produce only positive numbers. For the same reason, Van Rijn et al. [77] suggested the use of further statistical indicators with the BSS in order to partially overcome the BSS drawbacks. Given that the BSS is a common indicator in most XBeach studies, the RMSE is used as a further statistical indicator as recommended by Van Rijn et al. [77]. According to Van Rijn [77], the model performance classification is as follows: BSS < 0 bad fit, BSS = 0 - 0.3 poor fit, BSS = 0.3 - 0.6 reasonable/fair fit, BSS = 0.6 - 0.8 good fit and BSS = 0.8 - 1.0 excellent/best fit. To get an idea of how different predicted profiles can be for different BSS ranges, Fig. 3 provides an example for the BSS ranges for an exemplary profile. The profiles (initial, predicted and observed) are considered linear and BSS values are calculated based on readings of the bed levels at the profiles ends (i.e. $N = 2$), where one of the ends is considered non-movable. Thus, Fig. 3 shows that the highest the BSS value is the nearest the predicted profile to the observed profile and the more far from the initial profile. In addition, Fig 3 shows that the range of excellent BSS is quite wide, so that a BSS for a predicted profile with 0.9 can be quite far from the observed profile even though it is within the range of excellent/best fit. In fact, similar ranges cannot be provided for the RMSE but lower values of RMSE indicate better fit.

Fig. 3: Example of BSS ranges for an exemplary linear profile of one movable end; BSS values and ranges are calculated with considering $N = 2$.

When applying both accuracy indicators in Eq. (13) to the test XB3 (Fig. 4), which is characterised by lower overtopping rate $q_{nom} = 16.35 \text{ l/(s-m)}$, BSS value of 0.915 (excellent fit) and RMSE value of 0.30 m are obtained for the cross-shore distance x ($76 \text{ m} \leq x \leq 83 \text{ m}$). However, for the test XB8 (Fig. 5), which is characterised by higher overtopping rate $q_{nom} = 423 \text{ l/(s-m)}$, the BSS decreases to 0.68 (good fit) and the RMSE increases to 0.38 m ($70 \text{ m} \leq x \leq 85 \text{ m}$). These values confirm that the prediction capability of XBeach decreases with higher overtopping rates, which are mainly accompanied by higher flow velocity as often reported in the literature (e.g. [20,29,33,34]). As shown in Fig. 6, this conclusion is verified by comparing the observed and the modelled crest recession for all 17 GWK tests in Table 1.

Fig. 4. Measured (GWK test 04) and calculated (XBeach test XB3) cross-shore profiles after 6705 s of wave action with a lower overtopping rate $q_{nom} = 16.35 \text{ l/(s.m)}$; The BSS and RMSE are calculated for the cross-shore distance x ($76 \text{ m} \leq x \leq 83 \text{ m}$) showing BSS = 0.915 and RMSE = 0.30 m.

Fig. 5. Measured (GWK tests 15-17) and calculated (XBeach test XB8) cross-shore profiles after 8908 s of wave action with a higher overtopping rate $q_{nom} = 423 \text{ l/(s.m)}$; The BSS and RMSE are calculated for the cross-shore distance x ($70 \text{ m} \leq x \leq 85 \text{ m}$) showing BSS = 0.68 and RMSE = 0.38 m.

The simulations showed that both the crest recession (Fig. 6) and scour size behind the revetment crest are overestimated by 73 % in average. However, the mismatch between the observed and calculated profiles vanishes with the lower overtopping rates (e.g. GWK test 01-03) and also with increasing the distance of the dune offset from the revetment crest (e.g. GWK test 08). With increasing the overtopping rate from 1.92 l/(s.m) in GWK tests 01-03 to 16.35 l/(s.m) in GWK test 04, the overestimation value increases. Therefore, the overestimation is more likely under the higher

overtopping rates and less dune offset from the revetment crest (e.g. GWK test 09-11). By increasing the dune offset and fixing the overtopping rate (e.g. GWK test 12-17), the overestimation decreases. Such overestimated results represent the motivation for proposing, implementing and testing two further physically-based improvements associated with sediment stirring in XBeach as described below.

Fig. 6. Modelled vs observed crest recessions for the GWK tests 01-17 with different overtopping rates ($q_{nom} = 1.92 - 423 \text{ l/(s.m)}$) and different dune offsets (0 – 6.66 m).

4. Proposed XBeach Improvements

In order to improve the prediction capability of morphological evolution by XBeach, processes associated with sediment stirring (e.g. wave nonlinearity effect and onset of sediment motion) are investigated and better represented in an improved version of XBeach so that over-predicted erosion rates are reduced based on physical means. The latter was possible through proposing two new model modifications: the first is related to the effect of bed (beach) slope on wave nonlinearity and thus on sediment transport while the second is related to grain-stabilization effect on onset of sediment motion as respectively described in the following two subsections.

4.1. Bed Slope Effect on Wave Nonlinearity and Sediment Transport

The XBeach calibrations by Vousdoulas et al. [53] and Nederhoff [66] showed the model sensitivity to the flow velocity u_a induced by the wave nonlinearity (Eq. 12) or more precisely to the related calibration factor $facua$ (γ_{ua}), which is defined by Van Thiel de Vries [26] as a coefficient related to the phase shift between the intra-wave sediment concentration and flow. Van Thiel de Vries [26] suggested a value of 0.1, which is the default for this parameter in XBeach. However, Vousdoulas [53] and Nederhoff [66] reported that beaches of steeper slopes require higher values of $facua$ in order to improve the prediction capability of XBeach. Moreover, Bugajny et al. [65] reported that $facua$ is the most important parameter in XBeach, because it is the only parameter which affect the results most. Increasing this parameter results in fact in higher velocity u_a and subsequently in more onshore sediment transport that counteracts the offshore sediment transport induced by wave rundown. Therefore, increasing the calibration factor γ_{ua} would result in a significant reduction of the erosion which is generally over-predicted by the current XBeach code.

Both Vousdoulas [53] and Nederhoff [66] reported that a relation might exist between the seaward slope steepness and the calibration parameter $facua$, which needs to be implemented in XBeach. However, neither Vousdoulas [53] nor Nederhoff [66] attempted to determine this relation. For this purpose, a sensitivity analysis would be needed for the calibration parameter γ_{ua} using XBeach by

varying the slope steepness and comparing the observed erosion with the erosion predicted by XBeach. As a result, the value of γ_{au} providing the best fit between the observed and the calculated erosion is selected. Instead of performing such a sensitivity analysis, a mathematical relationship between γ_{au} and the beach slope steepness could be developed and implemented in XBeach based on data from previous literature. The implementation of such a relationship would allow the XBeach user to insert only the value of the average slope steepness in order to get the most appropriate γ_{au} -value for the simulations.

A relation between the average seaward slope steepness (measured **perpendicularly to shoreline** from the closure depth to the foredune (rise over run)) and *facua* (γ_{au}) is proposed in this paper based on the results of the sensitivity analyses performed by Bugajny et al. [65], Carrion Aretxabala [79], De Vet et al. [35], Nederhoff [66], Splinter and Palmsten [67] and Vousdoukas et al. [53]. Carrion Aretxabala [79] applied XBeach to simulate the morphological impact of the Sinterklaas storm on a dune system at Het Zwin (at the borders between The Netherlands and Belgium) where the average slope steepness is 7.85 %. As a result, the best fit of the modelled and observed profiles is achieved for $\gamma_{au}=0.15$. Nederhoff [66] applied XBeach to simulate the effect of hard structures on dune erosion and overwash, where the average slope steepness is 9.33%; the best fit is achieved for $\gamma_{au} = 0.25$. Moreover, Bugajny [65] reported that for a beach with slope steepness of 15.79% , a *facua* $\gamma_{au} = 0.5$ and for a beach with slope steepness of 14.39%, a *facua* $\gamma_{au} = 0.4$ would be required to achieve the best fit. On the other hand, Splinter and Palmsten [67] used XBeach to simulate the dune erosion of a beach with an average slope steepness of 11.80% and reported that the best fit is achieved for $\gamma_{au} = 0.3$. Vousdoukas [53] found, during the calibration of the XBeach model, that $\gamma_{au} = 0.1$ provides the best fit with a 6% slope steepness and $\gamma_{au} = 0.3$ provides the best fit with a 8% slope steepness. Furthermore, De Vet [20] and De Vet et al. [35] reported that the skewness and asymmetry calibration parameter γ_{au} plays an important role in reducing the overestimation of the erosion volumes of the Fire Island model by XBeach. Therefore, they reported that $\gamma_{au} = 0.2$ enhances the prediction capability of the model for an averaged seaward slope steepness of 6.5 %. Table 3 summarizes the data related to the average seaward slope steepness and the corresponding γ_{au} -values as extracted from the aforementioned studies. Moreover, either the BSS or the RMSE are also provided as accuracy indicators for the best fit.

Table 3: Summary of data from previous studies for the relation between the average slope steepness and *facua* parameter, including accuracy indicators for best fit.

Study	Average seaward slope steepness $S_s = \tan \beta$ **	γ_{au} -value for best fit	Accuracy indicator
Carrion Aretxabala (2015[79])	7.85%	0.15	RMSE = 3.80 m
Nederhoff (2014 [66])	9.33%	0.25	BSS = 0.97
Bugajny et al. (2013 [65])	15.79%	0.5	BSS = 0.82
	14.39%	0.4	BSS = 0.54
Splinter and Palmsten (2012 [67])	11.80%	0.3	RMSE = 1.07 m
Vousdoukas et al. (2012 [53])	6.00%	0.1	BSS = 0.40
	8.00%	0.3	BSS = 0.72
De Vet (2014 [20]) and De Vet et al. (2015 [35])	6.50%	0.2	BSS = 0.64

** The average seaward slope steepness ($S_s = \tan \beta$, β represents the average seaward slope angle) of a beach or a coastal barrier is measured **perpendicularly to shoreline** from the closure depth to maximum run-up point (no overwash case) or to the crest of the coastal barrier in case of wave overtopping (overwash case).

In order to obtain the relation between *facua* γ_{au} and the average seaward slope steepness ($S_s = \tan \beta$), the collected data from the previous studies in Table 3 are plotted in Fig 7. The data showed a strong correlation ($R = 0.89$) when fitted a power function $\gamma_{ua} = aS_s^b$. As a result, the relation between *facua* γ_{ua} [-] and average slope steepness s_s [-] can be expressed as:

$$\gamma_{ua} = 5.93s_s^{1.35} \quad (14)$$

Fig. 7. Relation between the average seaward slope steepness and *facua* parameter

Eq. (14), called hereafter “*facua-asabeta*” relationship, indicates that the beach slope has a direct control on the onshore sediment transport induced by the waves skewness and asymmetry. More precisely, Eq. (14) proves that the phase shift between the intra-wave sediment concentration and flow, represented by the factor *facua*, is significantly affected by the beach slope steepness. Higher phase shift values are expected with steeper slopes as obvious from Fig 7. As a result, instead of using the *facua* value $\gamma_{ua} = 0.1$ as proposed by Van Thiel de Vries [26], γ_{ua} should be determined as a function of the beach slope steepness as proposed in Eq. (14). Therefore, by considering the latter equation, the skewness and asymmetry discretization model in XBeach, as proposed in Eq. (12), can be modified as follows:

$$u_a = 5.93s_s^{1.35} \cdot u_{rms} (S_k - A_s) \quad (15)$$

Eq. (14) is implemented in XBeach, after defining a new model parameter called *asabeta* that indicates to the Average Slope Angle beta (β) (see [21] for the details), where the slope is measured perpendicularly to shoreline from the closure depth to maximum run-up point (when no overwash is expected) or to the crest of the coastal barrier in case of wave overtopping is expected (overwash case). This model improvement allows the XBeach user to insert the value of the average slope angle using the new parameter *asabeta*, so that adequate γ_{ua} -value is simply calculated by the model according to Eq. (14). Consequently, proper value for u_a is calculated according to Eq. (15), which, as discussed in Section 2.2, affects both the sediment entrainment/deposition in Eq. (1) and total transport loads in Eq. (4).

4.2. Grain-Stabilization Effect on the Inception of Sediment Motion

Some studies (e.g. [24,27,32,38,75]) attempted to overcome the over-prediction of the erosion volumes by XBeach, especially under overwash and breaching conditions by avoiding the sediment sheet flow regime. For this purpose, an upper limit for the sediment stirring flow velocity $U_{stirring}$ was fixed by setting the effective Shields parameter $\theta \approx 1.0$ in Eq. (10) since $\theta = 1.0$ represents the minimum value for incipient sheet flow conditions. Despite the success of this approach to reduce the over-predicted erosion and deposition sediment volumes, it fails to provide a physically-based justification for setting an upper limit to the stirring flow velocity. More recent studies (e.g. De Vet [20]) attempted to overcome this problem by increasing the critical Shields parameter θ_{cr} in Eq. (11), rather than setting the limit $\theta \approx 1.0$ for the effective Shields parameter in Eq. (10). The increase of θ_{cr} increases the critical stirring velocity U_{cr} and thereby reduces the difference between $U_{stirring}$ and U_{cr} in Eq. (6) and/or Eq. (7), which finally means less sediment transport and reduction of the overestimated erosion by XBeach. Therefore, De Vet [20] attempted to increase θ_{cr} by implementing in XBeach the

dilatancy effect proposed by [33,34]. However, the validation cases in the study of De Vet [20] did not show the same behaviour when sediment transport is hindered by the latter effect. In some cases, the erosion was so strongly reduced which means that the calculated value underestimates the actually observed value (e.g. the case of Zwin Dam, see section 5.2), while in other cases the reduced erosion was not sufficiently reduced to achieve a good agreement with the observed erosion (e.g. the Santa Rose Island case, see section 5.3). These and further similar cases showed the necessity to introduce a more appropriate approach for the reduction of the erosion by explicitly considering the grain-stabilization effect of the sediment particles on incipient motion.

The experiments of Shields [54], which build the basis of the Shields approach for the calculations of U_{cr} in XBeach, were carried out with steady uniform flow over a plane bed of uniform sediment with small grain-interlocking. Therefore, it is believed that the classical Shields concept for the inception of motion does not account for the grain-interlocking effect and the soil texture effect, which are more pronounced for natural mixed-grain-size sediment beds than for uniform sediment beds. Therefore, a large part of the erosion overestimation by XBeach is assumed to be attributed to the underestimation of the critical Shields parameter for incipient sediment motion. By other words, there are unaccounted forces in the formulation of the critical Shields parameter such as the uprooting force (to overcome the sediment interlocking). In fact, this assumption is in line (i) with the studies of Morris [80] and Özer et al [81], which reported that the soil erodibility depends on the soil texture as well as on the compaction energy and (ii) with study of Reid and Frostick [82] who reported, based on field measurements, that the shear stress required to initiate particle motion in natural sediment beds is considerably in excess of that predicted by Shields curve. For instance, the values of the effective shear stress measured by Reid and Frostick [82] for incipient bedload were consistently higher, ranging from 1.2 to 6.2 times the common critical shear stress values predicted by Shields curve. Reid and Frostick [82] attributed this excess to the mutual interference of bed sediment particles. Therefore, the latter studies [80–82] ensure that the critical shear stress calculated based on Shields curve is underestimated as long as sediments are mutually interlocked and stabilized under compaction/consolidation effect. This might result in the over-predicted erosion rates by XBeach.

As other sources for grain-stabilizations, Nowell et al. [83] reported that marine fine-grained sands are mainly stabilised by biological effects that result in increasing the critical shear stresses for incipient sediment motion by 2 to 3 times the values predicted by the Shields curve. The latter is generally valid for sediment of particle Reynolds number $Re_* \leq 1$ (Re_* is the ratio of inertial forces on sediment particle to viscous forces: $Re_* = u_* D / \nu$ where u_* is shear velocity, D is the grain diameter and ν is the kinematic viscosity). Other studies (e.g. [84,85]) reported that marine sand in general might be connected by chemical (salty) connections that increase the shear stresses required to initiate the sediment motion. Therefore, the overestimation of the erosion rates by XBeach can partially be attributed to the omission of the effects of diverse grain-stabilizing effects such as the interlocking, biological and/or chemical (salty) stabilizations on critical bed shear stresses for incipient sediment motion.

Despite the lack of processes based models to determine the aforementioned grain-stabilizing effects, the additional shear stress required to account for these effects can be implemented to XBeach by amplifying the critical stirring velocity U_{cr} using an amplification factor γ_{pi} as follows

$$U_{cr}^2 = \frac{\theta_c \cdot (s-1) \cdot g \cdot D_{50}}{c_f} \rightarrow U_{cr_{pi}}^2 = \frac{\theta_{c_{pi}} \cdot (s-1) \cdot g \cdot D_{50}}{c_f} \rightarrow U_{cr_{pi}} = \gamma_{pi} U_{cr} \rightarrow \theta_{c_{pi}} = \gamma_{pi}^2 \theta_c \quad (16)$$

where γ_{pi} is a new calibration FACTor for the grain-stabilization and Particle Interaction effect (hereafter called “*facpi*”) that considers the increase of the shear stresses required to initiate the sediment motion in case of interlocking, biological and/or salty connections among the sediment particles, $\theta_{c_{pi}}$ is an amplified critical Shields parameter that takes the grain-stabilization into account, $U_{cr_{pi}}$ is the depth averaged critical velocity after considering the grain-stabilization effect. Therefore, the proposed modification affects Eqs. 6 and 7 that are rewritten below accordingly using Eqs. 17 and 18, respectively.

$$C_{eq,b} = \frac{A_{sb}}{d} (U_{stirring} - \gamma_{pi} \cdot U_{cr})^{2.4} \& C_{eq,s} = \frac{A_{ss}}{d} (U_{stirring} - \gamma_{pi} \cdot U_{cr})^{2.4} \quad (17)$$

$$C_{eq,b} = \frac{A_{sb}}{d} (U_{stirring} - \gamma_{pi} \cdot U_{cr})^{1.5} \& C_{eq,s} = \frac{A_{ss}}{d} (U_{stirring} - \gamma_{pi} \cdot U_{cr})^{2.4} \quad (18)$$

These modifications are implemented in XBeach by defining a default value for $\gamma_{pi} = 1.0$ for the cases without any grain-stabilization effect. However, values larger than 1.0 will definitely reduce the mismatch between $U_{stirring}$ and U_{cr} so that equilibrium concentrations C_{eq} in Eq. 17 and 18 are reduced, which reduces the depth averaged concentrations calculated by Eq. (1) and hence the total sediment load calculated by Eq. (4). As a result, over-predicted erosion rates can be reduced. Because the value of γ_{pi} depends on the mutual interference of bed sediment particles, compaction state, concentration of soluble salts and the biological state at the considered study site, model calibrations are crucial in order to estimate real value for γ_{pi} as discussed in the following validation cases.

5. Validation Cases

In order to test the effect of the proposed XBeach improvements in the simulation results by XBeach, three testing cases are selected:

- GWK tests for dune erosion from the seaward side, which are exclusively discussed in Section 3,
- Zwin dam test for dune breaching [15,20,86,87], and
- Santa Rosa case for barrier island erosion and overwash under a hurricane event [20,27,28].

The selection of these test cases is based on:

- (i) the (partial) availability of pre- and post-storm data to validate the model with the new extensions,
- (ii) selecting all possible regimes for testing the proposed model improvements; thereby a dune erosion case (first case), a breaching case (second case) and a barrier island overwash and inundation case (third case) are selected, and
- (iii) selecting the cases that did not show the same behaviour with the previous attempts for overcoming the erosion over-prediction by XBeach.

As an example for the latter selection criterion, Zwin dam breach width was significantly underestimated when the artificial limiter for effective Shields parameter is activated while recent physically-based improvements by De Vet [20] were unable to satisfactorily solve the problem. In contrast, overestimation of erosion and overwash volumes at Santa Rosa Island was possible to be overcome through only the latter artificial limiter whilst no physically-based limiter was able to even improve the prediction capability of XBeach for this case. The details of these three cases are discussed in the following subsections.

5.1. Large-Scale Dune Erosion Tests in GWK

The XBeach dune erosion test XB8 corresponding to GWK tests 15-17 (see Table 1) represents an adequate means for examining the effect of the proposed improvements on the performance of XBeach. These tests are selected as being representative of the 17 GWK tests since they have the maximum dune offset. In addition, they are subject to the highest overtopping rates and thus represent more extreme and complex conditions. Consequently, seven new runs for XB8 are performed with the improved model by considering the same model parameters and boundary conditions as in Table 2. In the first run, only the dilatancy effect of De Vet [20] is considered, while in the second run, the γ_{pi} - γ_{pi} relationship is applied for an average slope angle of 4.41° (obtained from the GWK model dimensions) and the grain-stabilization effect is not considered by simply setting $\gamma_{pi} = 1.0$. In the third run, both dilatancy effect and average slope angle effect are considered besides setting $\gamma_{pi} = 1.0$. In the fourth run, an increase of the critical stirring velocity (U_{cr}) by 10% (due to grain-stabilization) is applied besides considering the average slope angle effect. Similarly, in runs 5 - 7, besides the average slope angle effect, an increase of critical stirring velocity by 20%, 50% and 100% is respectively considered, which are attributed to the grain-stabilization effect. The increases of the critical stirring velocity in runs 4 - 7 were possible by setting $\gamma_{pi} = 1.1, 1.2, 1.5$ and 2.0 , respectively. The results are shown in Fig. 8.

Fig. 8. Results of testing the modified XBeach code by considering dune erosion test XB8 in Table 1 (zoom in the dune extension zone)

The non-modified XBeach model results in over-prediction of the erosion volume with a BSS = 0.68 as previously shown in Fig 5. When the dilatancy effect is activated, the BSS increases to 0.92. However, the dilatancy activation results in underestimation of the frontal dune erosion and overestimation of the scour behind the revetment crest as clearly shown in Fig 9.

Fig. 9. Results of testing the modified XBeach code, including a comparison between the profile of best fit and the dilatancy profile.

The best fit (BSS = 0.983) is achieved without activating the grain-stabilization or the dilatancy effects, simply by assigning the correct value of the skewness and asymmetry calibration parameter $facua$ using the average slope angle β ($asabeta = 4.41^\circ$). When activating both the dilatancy effect and the $facua$ - $asabeta$ relationship, the erosion volume is significantly underestimated that results in a reduction of the BSS value to 0.715. Small values for the grain-stabilization effect ($facpi = 1.1$ and 1.2) beside assigning the correct value of $facua$ results in a considerable over-prediction of the erosion

volumes. This is because increasing the critical Shields parameter using the parameter $facpi$ hinders both onshore directed sediment transport, induced by wave skewness and asymmetry effects, and offshore directed sediment transport **induced by wave rundown effect**. With higher $facpi$ values, the overestimation decreases until an underestimation is reached with $facpi$ of 2.0 since the increase of the critical Shields parameter hinders the sediment transport onshore and offshore and could, therefore, result in **underestimated** erosion rates.

Based on the previous analysis, it can be said that the overestimation problem in XB8 was neither due to the grain-stabilization nor to the dilatancy effects, but simply because of the non-assignment of a proper value for the parameter $facua$. Once a proper $facua$ value is assigned using the new implementation of the $facua$ - $asabeta$ relationship, the overestimation problem is overcome **with a BSS = 0.983**, thus highlighting the massive importance of including the $facua$ - $asabeta$ relationship to XBeach. The latter score can be improved more by adopting other XBeach parameters such as $wetslp$, which is the critical avalanching slope under water (**i.e. wet avalanching slope**). However, $wetslp$ is not the topic of this paper, but **its effect** can be seen, for instance, in [53].

5.2. Zwin Dam Breaching Test

An artificial dam was constructed in the mouth of the tidal inlet of Zwin, which is located on the border between the Netherlands and Belgium. The test represents a full-scale dam breach experiment that was performed on the 7th of October 1994 to validate the breaching model BRES (**Dutch for breach**) of Visser (1998 [87]). Therefore, the test is well documented in Dutch by [86] and in English by [87]. The dam was constructed with a crest height of 3.3 m + NAP (**Dutch datum; NAP is the Dutch abbreviation form of MSL**), a crest width of 8 m, a landward slope of 1:3, a seaward slope of 1:1.6 and alongshore length of 250 m. Though the breach width at the narrowest point was the main measure, this test was used in the study of Roelvink et al. [15] to validate XBeach **capability to simulate** barrier breaching **under water overtopping**. Moreover, it was also used by De Vet [20] to validate his implementation of the dilatancy effect to XBeach.

In order to initiate a breach under overtopping conditions, an initial breach (pilot channel, see Fig 10) was enforced at the middle of the dam having a depth of 0.8 m, a bottom width of 1 m and side slopes of 1:1.6. This pilot channel was enforced to ensure that the breach is initiated at this location. The level of the surrounding seabed was about 0.7 m + NAP, while the mean tidal prism of the Zwin was about 350,000 m³. The experiment was performed under calm conditions and the water elevation was the main driver for the breaching process, i.e. there was no wave action. The breach developed to a width of 41 m within one hour after the breach initiation, **where eroded sediment from the landward side of the dam under overtopping effect is deposited inside the tidal inlet**. During that time, the evolution of the breach width is measured.

Fig. 10. Zwin dam breach initiation, after Bakker et al. (1994 [86])

The Zwin dam test was reproduced in XBeach by [15,31], using a non-uniform grid with grid sizes gradually varying from 0.5 m near the breach to approximately 50 m far away from it. The median

grain diameter D_{50} of the bed material was set to 0.3 mm in accordance with the prototype test conditions for the artificial dam. The results showed that XBeach overestimates the final breach width [31]. As a first attempt to overcome this over-prediction, De Vet [20] limited the critical Shields parameter at 1.0 to prevent sediment sheet flow conditions. However, this artificial limiter led to significantly worse predictions than without any limiter (Fig 11). As a result, De Vet [20] applied in a further attempt the dilatancy effect, which resulted in an underestimation of the breach width by 20%.

In order to examine the effect of the grain-stabilization effect, the Zwin dam model is run again six times using the improved model with considering the same model parameters of De Vet [20]. Because the test was performed under calm conditions, no waves are considered in the numerical model and therefore the new implementation of the skewness and asymmetry calibration parameter $facua$ ($facua$ - $asabeta$ relationship) cannot be examined using this test. Out of the six new runs, the first run considered no model modifications (default settings) while in the second run, the erosion is hindered by activating the dilatancy effect. Runs from 3 - 6 considered (without the dilatancy) $facpi$ $\gamma_{pi} = 1.1, 1.3, 1.5$ and 2.0 , respectively. As a result, the computed breach widths obtained from the numerical runs are compared to the observed breach widths as shown in Fig 11.

Fig. 11. Zwin Dam Breaching Test: Temporal evolution of breach width for the different runs.

Consistent with De Vet [20], the first run (Base run), with default settings, showed that XBeach overestimates the final breach width by 5.9%. Moreover, applying the dilatancy effect in the second run resulted in an underestimation of the breach width by 5.7%. Using $facpi$ of 1.1, results showed no enhancement of the prediction capability of XBeach as compared to the base run. However, the best fit between the final breach widths is achieved when the grain-stabilization parameter $facpi$ is set $\gamma_{pi} = 1.3$. This is also shown for run 04 by the determination of the correlation coefficient R^2 and the RMSE between the observed and the computed breach widths (Table 4). From the outcomes of runs 5 and 6, the higher values of $facpi$ result in a reduction of breach widths as showed in Fig 11.

The lowest R^2 and the highest RMSE between the observed and computed breach widths are achieved in run 7 using the artificial limiter followed by run 2 with the dilatancy effect activated. However, the highest R^2 and the minimum RMSE are achieved for run 4 with $\gamma_{pi} = 1.3$ that ensures the grain-stabilization role in hindering the erosion when setting a proper value for $facpi$. By other words, the overestimation of erosion volumes can be attributed to the underestimation of the critical stirring velocity U_{cr} by 23% or, more specifically, to the underestimation of the critical Shields parameter by 40.83% ($U_{cr} = f(\sqrt{\theta_{cr}})$). This underestimation can be interpreted by the non-consideration of the grain-stabilization effects in the Shields curve as discussed in Section 4.2.

Table 4. Statistical indicators for the simulation of Zwin breach width evolution using different values for $facpi$

Run	description	R^2	RMSE (m)
1	default settings (Base run)	0.9880	2.77
2	dilatancy activated	0.9776	5.84
3	$facpi = 1.1$	0.9881	2.77
4	$facpi = 1.3$	0.9897	2.73
5	$facpi = 1.5$	0.9866	3.59
6	$facpi = 2.0$	0.9821	5.14
7	Artificial limiter ($\theta = 1$)	0.8587	21.95

Applying the grain-stabilization effect using a proper value for $facpi$ has allowed to predict the final breach width better than the default settings and even better than the recent development by De Vet [20], which proves that the parameter $facpi$ was indeed capable of hindering the erosion. The too slow reaction of the model at the beginning with most of the cases means that there is some of the many user defined parameters of XBeach (e.g. the critical slopes for avalanching) that need to be calibrated in order to achieve better results at the beginning.

5.3. Overwash and Erosion of Santa Rosa Island

The studies of McCall (2008 [28]) and McCall et al. (2010 [27]) were the first studies for the validation of XBeach using a 2DH case for the dune erosion and overwash at Santa Rosa Island, Florida, the USA under Hurricane Ivan. A barely vegetated section of the barrier island (Fig 12) is chosen that showed a significant morphological response to the storm, which was recorded using a high-quality pre- and post-storm LIDAR (Light Detection and Ranging) data. According to McCall et al. [27], Hurricane Ivan was the largest of five hurricanes that struck the US coast in 2004, resulting in massive damages. Santa Rosa Island is a wave-dominated, narrow barrier island between the Gulf of Mexico and the Santa Rosa Sound on the Northern Florida Panhandle. The island is aligned approximately east-west with a length of almost 85 km. The width of the island varies between 150 m and 1000 m. The westernmost tip of the island is approximately 50 km from the location of landfall of Hurricane Ivan. During Hurricane Ivan, large stretches of Santa Rosa Island were overwashed (washover deposit depths up to 1.5 m and with multiple breaches on the westernmost tip).

Fig. 12. Aerial photo for the study site in Santa Rosa Island before (a) and after (b) Hurricane Ivan, after McCall, (2008 [28])

McCall [27,28] applied XBeach to a two kilometre stretch of Santa Rosa Island between Pensacola Beach and Navarre Beach, which is part of the Gulf Islands National Seashore (Fig 12). These studies were based on two LIDAR surveys of the study area that were carried out before and after Hurricane Ivan (Fig 13).

A grid spacing in the longshore direction is set to 20 m whilst the grid spacing varies from 2 m across the barrier island to 29 m on the offshore boundary. The wave and tide boundary conditions are hindcasted as shown in Fig 14 based on the available data from two tidal stations. As a result, the offshore wave boundary is described by JONSWAP shaped spectra. The significant wave height H_s varies from 2.5 m to 7.0 m, where the temporal evolution of H_s is centred symmetrically around the peak of the surge. The peak wave period at the offshore boundary decreases during the storm from 20.0

s to 10.0 s to reflect the frequency dispersion of the hurricane-generated waves in the wave hindcasting model [27]. Moreover, the surge level both offshore and behind Santa Rosa Island varies in the simulation from high astronomical tide 0.30 m + MSL to 1.75 m + MSL.

Fig. 13. LIDAR-measured bed elevation of the study site: (a) before Hurricane Ivan, (b) after Hurricane Ivan and (c) erosion–deposition plot based on the pre- and post-storm LIDAR data [27].

Fig. 14. Model boundary conditions: (a) significant wave height, (b) peak wave period on the offshore boundary, (c) surge level on the offshore and (d) surge level on the bay side [27].

The duration of the storm is set to 36 h, based on the duration of high energy wave conditions. For the first 6 h of the simulation by McCall et al. [27], the barrier island is supposed to be in the collision/impact regime since the surge level and wave heights are low and the level due to combined surge and wave run-up does not exceed the crest of the foredunes. Therefore, the morphological response is limited primarily to the erosion of the foredunes and deposition in the nearshore area. The second stage of the simulation running from approximately 6th to 12th hour, where the regime is overwash. During this time the surge level and wave height increase, leading to run up and overwash. Further erosion of the foredunes takes place, including small amounts of erosion and deposition on the bay side of the barrier. From the 12th hour onwards, the combined surge level and wave setup exceed

the height of the foredunes, which are already reduced in height by the preceding phases of the storm, leading to inundation overwash, which results in significant erosion from both seaside dune front and foredune and thereby deposition in the bay side.

McCall [27,28] reported that XBeach over-predicts the erosion and washover volumes for Santa Rosa by 200% that results in BSS = - 2.69. They, however, were able to overcome this over-prediction by limiting artificially the effective Shields parameter to 1.0, leading to BSS = 0.74. De Vet [20] arrived at the same conclusion, but he attempted to overcome the overestimation of the erosion volumes by implementing the dilatancy effect, rather than the artificial limiter. However, the results showed that the erosion and overwash volumes are still overestimated with BSS = - 4.64, which means that the overestimation of the erosion volumes remains even after the recent implementation of De Vet's improvements. In order to overcome the erosion overestimation considering the proposed improvements in Section 4, the Santa Rosa model is run again ten times using the same model parameters and boundary conditions of McCall et al. [27]. The first run is performed without hindering the erosion (i.e. using the default settings of XBeach). As a result, the outcomes showed significant overestimation of the erosion and overwash volumes as reported by [20,27,28], resulting in bad BSS = - 2.69 and high RMSE = 1.43 m as reported in Table 5.

Table 5. BSS and RMSE for the different attempts to overcome the erosion overestimation at Santa Rosa Island

Run	Properties	BSS	RMSE (m)
1	default settings	-2.690	1.430
2	asabeta = 1.15 + facpi = 1	-3.480	1.550
3	asabeta = 1.15 + facpi = 5	-2.258	1.324
4	asabeta = 1.15 + facpi = 5.8	0.695	0.405
5	asabeta = 1.15 + facpi = 6.0	0.719	0.389
6	asabeta = 1.15 + facpi = 6.2	0.759	0.360
7	asabeta = 1.15 + facpi = 6.5	0.781	0.343
8	asabeta = 1.15 + facpi = 7.0	0.799	0.329
9	asabeta = 1.15 + facpi = 7.1	0.827	0.305
10	asabeta = 1.15 + facpi = 7.2	0.816	0.314

In order to examine the effect of the *facua*-*asabeta* relationship, the average seaward slope angle *asabeta* is estimated to 1.15° and assigned to XBeach in the second run while *facpi* is set at 1.0. Since *asabeta* = 1.15° results in a *facua* value of 0.03 rather than 0.1 in the first run, less onshore sediment transport under nonlinear waves on the seaside is simulated, thus resulting in a higher overestimation for the erosion volumes. As a result, the BSS decreases to - 3.48 and the RMSE increases to 1.55 m. In runs 3 – 10 (see Table 5), *asabeta* is fixed at 1.15° while the value of *facpi* is increased until the highest BSS (0.827) and the lowest RMSE (0.305 m) are achieved for *facpi* = 7.1 in Run 9 (see Fig 15 for the outcomes of this run). Higher values for *facpi* result in lower BSS and higher RMSE because the erosion is hindered more than required.

By comparing Fig 13 (for the observations) with Fig 15 (for the predictions), one notice that the overestimation of the erosion is overcome by setting a proper value of the grain-stabilization effect using the new parameter *facpi*. The cross-shore cross-sections shown in Fig 16 provides a clearer visualisation of the enhancement of the prediction performance of the model with the new improvements. Fig 16 shows also the significant overestimation of the erosion and overwash volumes by the non-improved XBeach. When using the improved model, the prediction capability increases by assigning proper values for the new parameters *asabeta* and *facpi*.

The very high value for *facpi* (7.1) means that the critical stirring velocity U_{cr} is amplified 7.1 times, which means that the critical Shields parameter is amplified 50.41 times the value estimated by the common Shields curve. The need for the latter amplifications arises from the characteristics of the study area, which contains both longshore road and big parking zone (Fig 12). In addition to the hindered erosion by the asphaltic layer in the latter zones, the compaction of soil beneath these zones, which is common in such zones and leads to more particle interlocking and grain-stabilization, also hinders the sediment transport. Therefore, the hindered erosion by such grain-stabilization should be defined to XBeach, as proposed, in order to simulate proper erosion and overwash volumes.

Fig. 15. Plan view of Pre- and post-storm simulated topographies as well as the simulated erosion and depositions zones when facpi of 7.1 is used to represent the grain-stabilization effect (run 09).

Fig. 16. Comparison between final profiles by non-modified and improved XBeach model as well as observed profiles at different cross-shore cross-sections: (a) at 500 m longshore, (b) at 1000 m longshore, (c) at 1500 m longshore and (d) at 1800 m longshore.

Now it is possible to understand why McCall et al. [27] were able to fit measured and simulated profiles by using an artificial limiter that fixes the effective Shields parameter to a threshold value $\theta_{max} = 1.0$ that cannot be exceeded even if the flow velocity is higher. Indeed, they were not preventing the sheet flow, but they were limiting the difference between the effective and the critical Shields parameter ($\theta - \theta_c$) by fixing the effective one θ at 1.0. The same difference can indeed be achieved physically by increasing the critical Shields parameter θ_c . The latter approach is in fact more appropriate because it can be interpreted by the grain-stabilization effect while the artificial limiter suggested by McCall et al. [27] is more difficult to interpret.

6. Discussion

The performance of the improved XBeach model was examined for three selected cases. The selection was mainly based on the need to separately test the effect of the proposed improvements on (i) frontal dune erosion (section 5.1), (ii) sand barrier breaching (section 5.2) and (iii) barrier overwash and erosion (section 5.3). It is noticed that for all test cases, the default XBeach settings result in significant overestimates of the erosion rates. However, this overestimation is overcome using the proposed improvements using two new parameters: *asabeta* and *facpi*. Table 6 summarises the values of *asabeta* and *facpi* which are used to achieve the best fit between the measured and simulated profiles.

Table 6. Summary of values of the new parameters that were used to overcome the overestimation of erosion and overwash volumes by the modified XBeach.

Test case	<i>asabeta</i> (deg)	<i>facua</i> (-)	<i>facpi</i> (-)	D_{50} (mm)	Soil state
Frontal Dune erosion (GWK tests)	4.41°	0.188	1.0	0.16	Disturbed soil with low compaction density (artificial dune)
Sand barrier breaching (Zwin dam)	Not applicable	Not applicable	1.3	0.3	Disturbed soil low compaction density (artificial dam)
Barrier overwash and erosion (Santa Rosa Island)	1.15°	0.03	7.1	0.2	Undisturbed soil with high compaction density (natural barrier)

Table 6 represents rather a rough guide for recommended *facpi* values. The values of *asabeta* and the associated *facua* values are excluded from this guide because they can respectively be calculated based on the bathymetry change and the proposed *facua-asabeta* relationship. Therefore, there is no need to activate the sediment grain-stabilization effect in the case of artificial dunes/barriers made of sand with finer grain sizes ($D_{50} = 0.16$ mm) like for the GWK tests, where the soil is loose and the grain stabilization effects are negligibly small. For sand with coarser sediment sizes with $D_{50} = 0.3$ mm, *facpi* can be set to 1.3 as long as the soil compaction density is low like for the Zwin dam case. In the case of a natural coastal barrier, e.g. Santa Rosa Island, the grain stabilisation effects might be determinant thus significantly affecting the results. In fact, the soil is undisturbed, possibly consolidated resulting in strongly interlocked sediments. In such cases, *facpi* in the order of 7.1 should be used with a median grain size in the order of 0.2 mm. The need for the parameter *facpi* arose from the fact that the Shields curve is no longer applicable in the case of undisturbed and highly compacted soils.

It is also important to stress the importance of using the proposed *facua-asabeta* relationship (see Section 4.1) to properly simulate the effect of wave skewness and asymmetry on sediment transport because, under all the flow regimes of Sallenger [2], XBeach is very sensitive to the parameter *facua* and the *facua-asabeta* relationship enables to estimate the proper *facua* value. As a result, net onshore sediment transport under wave skewness and asymmetry effect can properly be simulated which might solve the overestimation problem.

This study has shown that the soil resistance (grain-stabilization effect) represents one of the main factors governing coastal erosion, coastal morphology and breach development. Higher values for *facpi* indicate indeed higher soil resistance due to grain-stabilization. In this study, a uniform grain-stabilization parameter is applied to reduce the over-predicted erosion by XBeach by using a unique value for *facpi* in each model run. However, a spatially varying grain-stabilization could be implemented to XBeach in future studies for barrier breaching by defining spatially variable *facpi* that can be assigned to the model as a matrix containing different *facpi* for each cell of the computational

grid. In this way, the spatially varying soil resistance could be considered to improve the prediction capability of breach location and dimensions. As a result, the interaction between soil resistance, longshore variability of hydraulic loads and longshore variability of barrier topography can be considered in future XBeach simulations as recently suggested in Elsayed and Oumeraci (2016 [22]).

7. Summary and Concluding Remarks

In this paper, the recent studies and the reasons for the overestimation of the erosion and overwash volumes by the XBeach model under storm wave conditions are explored. It is found that the main two reasons for the overestimation are:

- (i) the effect of wave nonlinearity on sediment transport or more specifically the calibration factor *facua* for the time-averaged flow due to wave skewness and asymmetry,
- (ii) the considerable excess of the actual shear stress required to initiate the sediment particle motion in the case of interlocked and compacted soils as compared to that predicted by Shields curve.

The first contribution of this study to overcome over-predicted erosion volumes by XBeach is related to reason (i) mentioned above and consists of developing a new relationship (called “*facua-asabeta*”) between the calibration factor *facua* for the net flow due to wave skewness/asymmetry and the seaward average slope angle β measured perpendicularly to shoreline from the closure depth to the foredune. The new relationship is derived from the results of sensitivity analyses to the *facua* factor recently carried out in previous studies. The relationship between *facua* and β is implemented in XBeach and tested for dune erosion and for barrier island erosion/overwash. The simulation results showed that the *facua-asabeta* relationship is crucial for assigning an appropriate value for the factor *facua* to the model by just providing input parameter β . The second contribution is related to reason (ii) mentioned above and consists of amplifying the critical shear stress required for incipient sediment motion according to Shields curve using an amplification factor γ_{pi} (called “*facpi*”). γ_{pi} accounts for grain-stabilization effects (e.g. interlocking, chemical and biological stabilizations) as a measure for soil resistance to erosion. For the simulation of large-scale laboratory tests with a median grain size $D_{50} = 0.16$ mm using XBeach, it was found that $\gamma_{pi} = 1.0$ (no amplification of the Shields curve based critical shear stresses is required). However, γ_{pi} can be increased to 1.3 for a soil has a median grain size of 0.3 mm. The increase of γ_{pi} from 1.0 to 1.3 could be interpreted by the interlocking effect, which is higher for coarser than for finer sediment. For the real case study of Santa Rosa Island, where the soil is undisturbed and consolidated, *facpi* had to be increased to $\gamma_{pi} = 7.1$ in order to achieve a good agreement of the calculated erosion and overwash volumes with the observed values.

It is common in morphological evolution models in coastal environments (e.g. XBeach) and in fluvial environments (e.g. [88]) that loose soil beds and hard beds (e.g. revetments and rocky beds) are simulated. Sediment entrainment from loose soil beds is often based on Shields curve while, in hard beds, sediment entrainment has to be deactivated. However, there no modelling tool that can yet calculate sediment entrainment from bed types between compacted to highly compacted beds. This study attempted to do that through an increase of the critical Shields parameter using the proposed parameter *facpi* to account for grain-stabilization effect. Future research could, therefore, attempt to establish physically-based predictors for the calibration factor *facpi* based on soil properties e.g. median grain size and void ratio (as a measure of the compaction degree of sandy soils).

Despite the success in overcoming the overestimation of the erosion/overwash rates and volumes using the two proposed improvements of the XBeach model, the following recommendations for future improvements are provided:

- (i) The proposed *facua-asabete* relationship is obtained based only on eight data points obtained from previous studies based on different conditions. Therefore, this relationship needs to be further tested and possibly improved in the light of new experiments.
- (ii) Instead of simply using the recommended values for the parameter γ_{pi} (*facpi*) in Table 6, it might be preferable to build Shields-like curves that account for the grain-stabilization effect. These curves could be determined from the results of systematic simulations for several real

907 study cases. The relation between the median grain size, the compaction energy and the
 908 parameter *facpi* that provides the best fit between the measured and simulated profiles can
 909 finally be obtained.

910 Acknowledgements

911 This research is a part of the PhD study of the first author which is supported by the DAAD in the frame of the
 912 Exceed-Swindon Project at Technische Universität Braunschweig. This support is gratefully acknowledged. We
 913 gratefully thank the Lower Saxony Water Management, Coastal Protection und Nature Conservation Agency
 914 (NLWKN) as contracting authority for making the GWK model tests data for Wangerooge for this research project
 915 available. The pre- and post-storm LIDER data for the Santa Rosa Island are gratefully obtained from Mr. Robert
 916 McCall after the permission of the USGS.

917 List of Symbols

A_s	[-]	Wave asymmetry
A_{sb}	[-]	Bed load coefficient
A_{ss}	[-]	Suspended load coefficient
C	[m ³ .m ⁻³]	Depth averaged sediment concentration
C_{ch}	[m ^{0.5} s ⁻¹]	Chezy parameter
C_d	[-]	Drag coefficient in Soulsby-Van Rijn formulation
C_{eq}	[m ³ .m ⁻³]	Depth averaged equilibrium sediment concentration
c_f	[-]	Coefficient of bed roughness
C_{max}	[m ³ /m ³]	Maximum sediment concentration
D	[m]	Grain diameter
d	[m]	Local water depth
D_{50}	[m]	Median sediment particle size
D_h	[m ² . s ⁻¹]	Horizontal sediment diffusion coefficient
f_{mor}	[-]	Morphological acceleration factor
f_{ts}	[-]	Correction factor in the expression of the adaptation time scale
g	[m.s ⁻²]	Gravitational acceleration
H	[m]	Wave height
H_s	[m]	Significant wave height
n	[m ^{-1/3} .s]	Manning roughness coefficient
q_{nom}	[l/(s.m)]	Overtopping rate
q_s	[m ² .s ⁻¹]	Sediment transport rate per unit width
q_{sx}	[m ² . s ⁻¹]	Cross-shore sediment transport rate per unit width
q_{sy}	[m ² . s ⁻¹]	Longshore sediment transport rate per unit width
R	[-]	Correlation coefficient
Re^*	[-]	Particle Reynolds number
s	[-]	Relative density of sediment (ρ_s/ρ)
S_k	[-]	Wave skewness
S_s	[-]	Average seaward slope steepness
t	[s]	Time
T	[s]	Wave period
T_s	[s]	Adaptation time scale in advection-diffusion equation for sediment
u*	[m.s⁻¹]	Shear velocity
u_a	[m.s ⁻¹]	Mean flow component due to nonlinear waves
U_{cr}	[m.s ⁻¹]	Critical flow velocity for sediment entrainment
$U_{cr,c}$	[m.s ⁻¹]	Critical flow velocity for sediment entrainment due to currents only
$U_{cr,w}$	[m.s ⁻¹]	Critical flow velocity for sediment entrainment due to waves only
$U_{cr,pi}$	[m.s ⁻¹]	Depth averaged critical velocity considering the grain-stabilization effect
u^E	[m.s ⁻¹]	Eulerian flow velocity in cross-shore direction
u_{rms}	[m.s ⁻¹]	Root-mean-square velocity
$U_{stirring}$	[m.s ⁻¹]	Depth average stirring velocity
v^E	[m.s ⁻¹]	Eulerian flow velocity in longshore direction
z_b	[m]	Bed level elevation
z_b	[m]	Initial (pre-storm) bed level
z_m	[m]	Measured post-storm bed level
z_p	[m]	Predicted profile from X-Beach
z_s	[m]	Water surface elevation
α	[-]	Weighting factor
β	[deg]	Average seaward slope angle
γ_{pi}	[-]	Calibration factor for the grain-stabilization effect

γ_{ua}	[-]	Coefficient for time-averaged flow due to nonlinear waves (facua)
θ	[-]	Effective Shields parameter
θ_c	[-]	Critical Shields parameter for the initiation of motion
θ_m	[deg]	Mean angle of wave incidence
θ_{sf}	[-]	Threshold value for effective Shields parameter for sheet flow occurrence
ρ	[kg/m ³]	Mass density of water
ρ_s	[kg/m ³]	Mass density of sand
ν	[m ² /s]	Kinematic viscosity

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