CROSMOR cross-shore modelling

1. Introduction

This document describes various applications of the CROSMOR-model, which can be used to compute the distribution of the bed level changes along a cross-shore coastal profile between the most landward dune zone and the most seaward offshore zone.
The CROSMOR-model is a numerical model (FORTRAN-code) which can compute the following parameters: wave height, wave asymmetry, undertow velocity, longshore current velocity, Longuet-Higgins streaming, bed load and suspended load transport and bed profile development over time along the cross-shore coastal profile in a situation with minor alongshore gradients (almost uniformity in alongshore direction).

Typical problems that can be solved using the CROMOR-model are:

1. Dune zone
   - storm erosion of dune front (exposed and sheltered sites);
   - erosion of high sand berms in front of hard structures (dikes);
   - storm erosion of gravel barriers;
   - overwash of dune crest;
2. Beach and surf zone
   - erosion of beach and surf zone nourishments;
   - sorting of beach sediment;
   - bar migration;
   - erosion and sedimentation of mining pits
   - toe scour near hard structures;
3. Offshore zone
   - erosion of sand mounds;
   - erosion/deposition of sand mining pits.

2. Description of CROSMOR-model

The CROSMOR2013-model can be used to compute the cross-shore profile distribution of wave heights, peak orbital velocities, undertow velocity, longshore currents, bed and suspended load transport and the bed profile development as function of time.

The CROSMOR2013-model is an updated version of the CROSMOR2004-model (Van Rijn, 1997, 2006, 2007d). The model has been extensively validated by Van Rijn (2008) and Van Rijn et al. (2003). All relevant literature is given in References.

The propagation and transformation of individual waves (wave by wave approach) along the cross-shore profile is described by a probabilistic model 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 $H_{max}=\gamma_{br}$ h with γ_{br}= 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 a 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. Laboratory and field data have been used to calibrate and to verify the model. Generally, the measured $H_{1/3}$-wave heights are reasonably well represented by the model in all zones from deep water to the shallow surf zone. The fraction of breaking waves is reasonably well represented by the model in the upsloping zones of the bottom profile. Verification of the model results with respect to wave-induced longshore current velocities has shown reasonably good results for barred and non-barred profiles.
A definition sketch of wave and current directions is given in **Figure 2.1**.

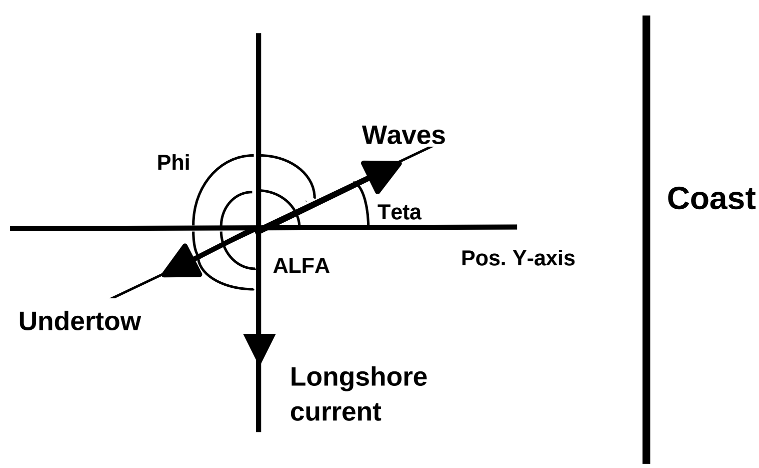


Figure 2.1 *Definition sketch*
Teta= angle between wave direction and positive y-axis
Phi= angle between wave and current direction
Alfa= angle between longshore current direction and positive y-axis
(v_{long}<0 m/s; alfa=270^o and v_{long}>0 m/s; alfa=90^o)

The complicated wave mechanics in the swash zone is not explicitly modeled, but taken into account in a schematized way. The limiting water depth of the last (process) 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 grid point by interpolation after each time step (variable number of grid points).

The cross-shore wave velocity asymmetry under shoaling and breaking waves is described by a semi-empirical method. Near-bed streaming effects are modelled by semi-empirical expressions. The velocity due to low-frequency waves in the swash zone is also taken into account by an empirical method.

The depth-averaged return current (u_r) under the wave trough of each individual wave (summation over wave classes) is derived from linear mass transport and the water depth (h_t) under the trough. The mass transport is given by 0.125 g H^2/C with C= (g h)$^{0.5}$ = phase velocity in shallow water. The contribution of the rollers of broken waves to the mass transport and to the generation of longshore currents is taken into account.

The sand transport of the CROSMOR2007-model is based on the TRANSPOR2004 sand transport formulations. The effect of the local cross-shore bed slope on the transport rate is taken into account.

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. The net (averaged over the wave period) total sediment transport is obtained as the sum of the net bed load (q_b) 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 using a formula-type of approach.

The net suspended load transport is obtained as the sum ($q_s= q_{s,c} + q_{s,w}$) of the current-related and the wave-related suspended transport components.
The current-related suspended load transport ($q_{s,c}$) is defined as the transport of sediment particles by the time-averaged (mean) current velocities (longshore currents, rip currents, undertow currents).
The wave-related suspended sediment transport ($q_{s,w}$) 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 ($q_{s,w}$) has been implemented in the model.

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 convection-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. The sediment composition can also be taken into account.

3. Dune zone cases

3.1 Erosion of sand dune due to extreme storms; exposed Holland coast, The Netherlands

The standard case of dune erosion for the Holland coast refers to extreme storm conditions as given in **Table 3.1.1**. The median sediment diameter along the Dutch coast is set to 225 μm (0.225 mm). The high storm surge level (SSL) of 5 m above mean sea level (MSL) is assumed to be constant over the storm duration of 5 hours during the peak of the storm. This equivalent duration of 5 hours yields approximately the same overall dune erosion volume as a complete storm cycle with growing and waning phases.

Parameter	Prototype conditions
Offshore wave height/period (m) (s)	7.6 (Pierson and Moskowitz spectrum); 12
Offshore water depth (m)	21 m
Storm surge level above MSL (m)	+5 m NAP during 5 hours (NAP is about 0.1 below MSL)
Median sediment diameter (μm)	225
Median fall velocity (m/s)	0.0267 (temperature= 10° C)
Cross-shore profile	a) dune height at +15 m NAP, b) dune face with slope of 1 to 3 down to a level of +3 m NAP, c) slope of 1 to 20 between +3m and 0 m NAP, d) slope of 1 to 70 between 0 and -3 m NAP, e) slope of 1 to 180 seaward of -3 m NAP line

Table 3.1.1 *Parameters of Dutch coastal profile (Reference Case Dune erosion)*

Figure 3.1.1 shows computed dune erosion profiles for different values of the storm surge level (SSL) above mean sea level (MSL). The storm duration is 5 hours with constant significant wave height of $H_{s,o}$=7.6 m. The dune erosion volume strongly increases with increasing storm surge level. The computed dune erosion is about 170 m^3/m for SSL=5 m above MSL.

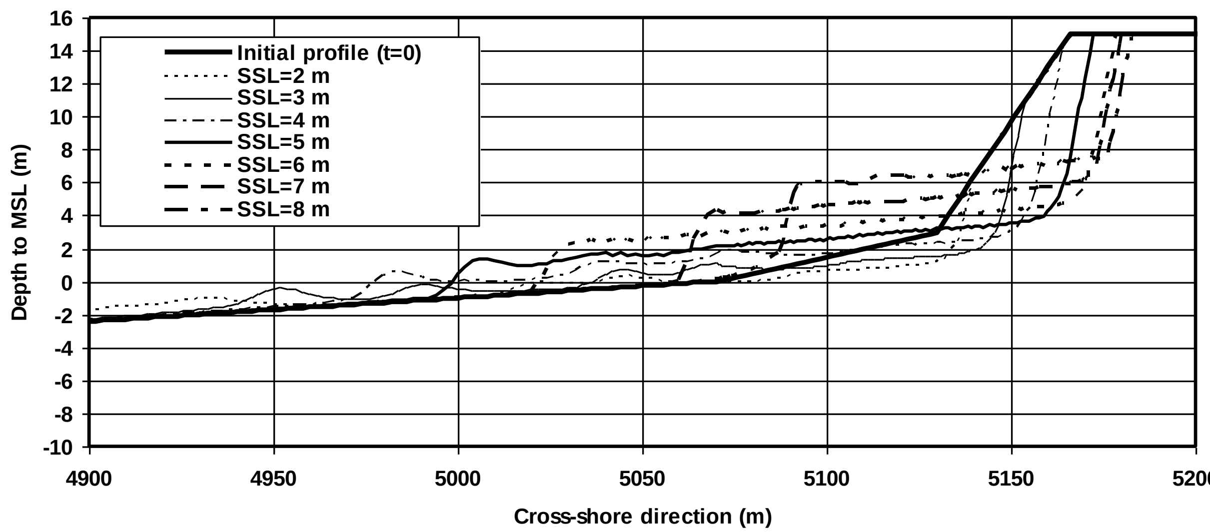


Figure 3.1.1 *Effect of storm surge level on computed bed profile after 5 hours for Reference Case*

3.2 Erosion of high berm in front of hard dike; sheltered lake coast Markermeer, The Netherlands

The old hard dike is reinforced with a sand berm in front of the (Houtrib) dike. The initial bed profile of sand (d_{50}=0.21 mm) has a slope of 1 to 32. Profile and boundary conditions are given in **Table 3.2.1**.
The design storm with return period of 10000 years has a trapezoidal shape in time with a total duration of 47 hours. The maximum significant wave height is 1.8 m. The maximum surge level is 1.92 m above NAP-datum.

PARAMETERS	**Values**
Water depth at deep water; waterlevel	4 m to NAP (NAP=0.1 m below mean sea level) 1,92 m above NAP (constant)
Significant wave height, period and wave incidence angle at deep water	t=0 hr: H_s=0 m, T_p=6.5 s, angle= 0°; 30° t=21 hr: H_s=1.8 m, T_p=6.5 s, angle= 0°; 30° t=27 hr: H_s=1.8 m, T_p=6.5 s, angle= 0°; 30° t=47 hr: H_s=0 m, T_p=6.5 s, angle= 0°; 30°
Bed slope; crest	1 to 32; 2 m above NAP
Boundary depth near beach	0.2 m
Grid size; total length	0.5 tot 1 m; 350 m
Number of wave classes per wave height	1 and 10
Wave asymmetry	Isobe-Horikawa
Coefficient Longuet-Higgins streaming; roller effect	0.5 (default=1); 0.5 (default=1)
Grain diameter sand d_{50}	0.21 mm
Coefficients sandtransportformulas	1 (default= 1)
Coefficient sandtransport wave asymmetry	0.2 (default= 1)
Coefficient sand entrainment beach zone	1 (default)
Coefficient undertow	1 (default)
Extra entrainment at dune front (sef)	1.5
Bed roughness	Automatisch
Temperature and salinity	10 degrees and 0 promille
Files	HDIJKS.inp; HDIJKG.inp

Table 3.2.1 *Input data of CROSMOR-model*

Figure 3.2.1 shows the bed profile after an extreme storm with duration of 47 hours.
The erosion volume is:

- 50 m^3/m above 1 m NAP for a wave incidence angle of 30° and 60 m^3/m above 0 m NAP;
- 35 m^3/m above 1 m NAP for perpendicular waves (no longshore current and less transport capacity).

The recession at the berm crest is about 70 m. The erosion volume is slightly larger using a spectrum of waves (10 wave classes). The erosion is substantially smaller for perpendicular waves (no longshore current).
The beach steepens to 1 to 15. The erosion is relatively large because the waves can wash over the crest.

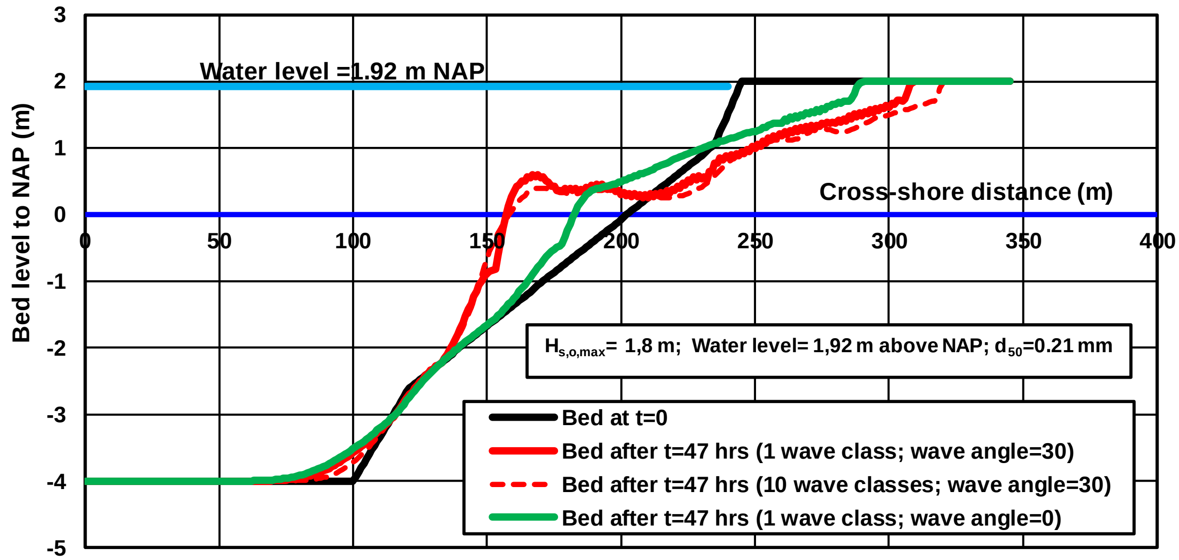


Figure 3.2.1 *Bed profile development after a storm of 1x 10.000 years*

3.3 Overwash over dune crest; Lagos coast, Nigeria

The CROSMOR-model has been used to compute the cross-shore morphological behaviour of the beach ridge during a storm event with overwash conditions. The wave height at the toe of the beach is set to H_s= 2.1 m with T_p=15 s and normal to the shore. The water level is assumed to vary between 2.2 and 2.8 m above mean sea level (MSL). The beach crest is at 2 m above MSL. The beach material is varied in the range of 0.2 to 0.55 mm. The input data are given in **Table 3.3.1**.

Figure 3.3.1 shows the computed bed levels after 1 and 2 days for beach material of 0.55, 0.3 and 0.2 mm. The beach crest is strongly eroded due to overwash processes during the storm event. The beach crest erosion is about 50 m^3/m after 1 day and d_{50}=0.55 mm. The erosion volume after 1 day is somewhat larger for finer sediment. Based on these results, it can be concluded that the breach crest can move landward over a cross-shore distance of 40 to 50 m due to overwash processes during a storm event.

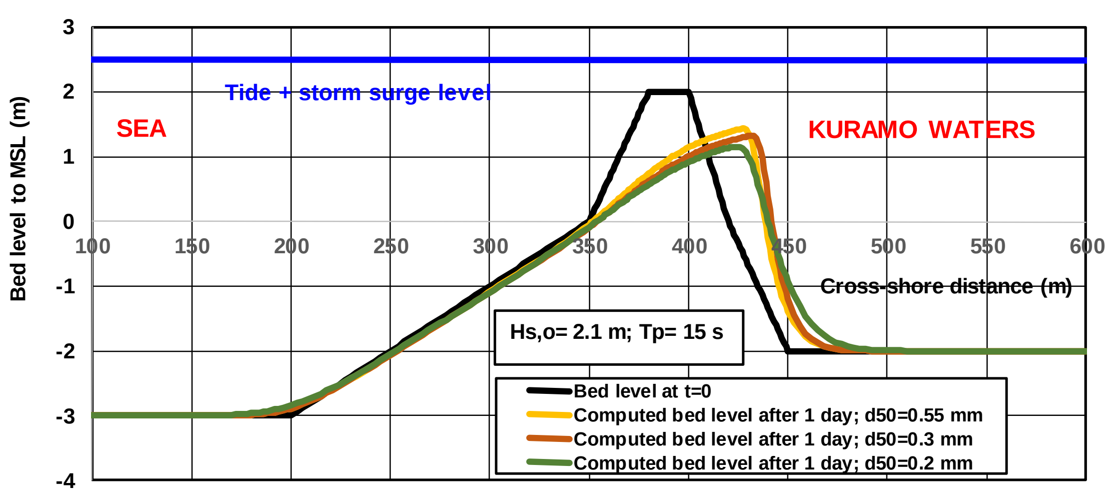


Figure 3.3.1 *Cross-shore beach profile changes during storm event; d_{50}=0.55, 0.3 and 0.2 mm; sef=1*

PARAMETERS	VALUES
Tidal conditions at x=0	Time(s) Cross-Current (m/s) Waterlevel (m)
	0 0.2 0 3600 0.2 0.15 10800 0.2 0.3 18000 0.2 0.15 21600 0.2 0 25200 0.2 -0.15 32400 0.2 -0.3 39600 0.2 -0.15 43200 0.2 0
Water depth at deep water	3 m
Storm surge level above MSL	2.5 m
Slope of sea bottom in the surf zone beteen -3 m and 0 m below MSL	1 to 50
Beach crest	2.0 m above MSL
Water depth at last grid point	0.1 m
Grid size and time step	1 to 3 m; 100 to 200 s
Total traject length	600 m
Significant wave heigth at x=0; wave period	2.1 m; 15 s
Wave direction at x=0 (to shore normal)	0 degrees
Number of wave classes per condition (to represent wave spectrum)	10
Wave asymmetry	Yes,based on Isobe-Horikawa
Coefficient Longuet-Higgins streaming roller effect	0.5 (default=1); 0.5 (default=1)
Sand grain size d_{50}	0.2 mm; 0.3 mm; 0.55 mm
Coefficients sand transportformulations	1 (default= 1)
Coefficient sand transport wave asymmetry	0 (default= 1)
Coefficient return flow (undertow)	1 (default)
Coefficient additional stirring of sand in beach-dune zone (sef)	1 (default; no effect) and 1.5
Bed roughness	Automatic
Temperature and salinity	20 degrees and 30 promille
Files	Nigerb.inp

Table 3.3.1 *Input data of CROSMOR-model*

3.4 Erosion of gravel barrier/dune due to extreme storms; exposed Pevensey Bay, UK

The CROSMOR-model has been applied to the 9 km long shingle barrier at Pevensey Bay, East Sussex, UK. The profile characteristics and boundary conditions are given in **Table 3.4.1**.
To obtain a very conservative estimate of the erosion volume along the profile, the seaward-directed undertow velocities have been increased by 50% and the erosion rate in the swash zone has been increased (sef = 2). Furthermore, the swash velocities near the water line and the streaming near the bed have been neglected (c_{sw}= 0, c_{LH} = 0).

Parameters	Values
Bed profile	slope of 1 to 62.5 between -30 m and -1.5 m (to MSL) slope of 1 to 8 between -1.5 m and +5 m slope of 1 to 4 between +5 m and +6 m crest width of 22.5 m
Sediment d_{50} d_{90}	0.01 to 0.1 m 0.04 m
Bed roughness k_s	0.01 to 0.1 m
Horizontal mixing	0.1 m^2/s
Peak tidal water level	Mean tidal range = 5.0 m OD (OD is approx. MSL) Spring tidal range = 6.7 m OD Neap tidal range = 3.7 m OD
Longshore peak tidal velocity (offshore)	0.5 m/s (flood); -0.5 m/s (ebb)
Offshore significant wave height $H_{s,o}$	1.5 to 6 m (6 wave classes using Rayleigh distribution for each case)
Peak wave period T_p	8 to 11 s
Wave incidence angle to coast normal	30°
Storm surge level above MSL	0 to 3 m

Table 3.4.1 *Data of shingle barrier at Pevensey Bay, UK*

Figure 3.4.1 shows the cross-shore distributions of the significant wave height and the longshore velocity during storm conditions with an offshore wave height of 6 and 3 m (T_p= 11 and 8 s), storm set-up value of 1 m and an offshore wave incidence angle of 30°. The tidal elevation is zero in this plot. During major storm conditions with $H_{s,o}$= 6 m, the wave height is almost constant up to the depth contour of -10 m. Landward of this depth the wave height gradually decreases to a value of about 2 m at the toe of the barrier (at x = 1980 m). During minor storm conditions with $H_{s,o}$= 3 m, the wave height remains constant to a depth of about 4 m. The wave height at the toe of the barrier is about 1.8 m. The longshore velocity increases strongly landward of the -10 m depth contour where wave breaking becomes important (larger than 5% wave breaking). The longshore current velocity has a maximum value of about 1.6 m/s for $H_{s,o}$= 6 m and about 1.7 m/s for $H_{s,o}$= 3 m (offshore wave angle of 30°) just landward of the toe of the beach slope. These relatively large longshore velocities in combination with the cross-shore velocities can easily erode and transport gravel/shingle particles of 0.02 m.

Figure 3.4.2 shows the barrier profile changes according to the CROSMOR-model for four storm cases at Pevensey Bay. The computed erosion area after 24 hours is largest (about 25 m^3/m) for the largest offshore wave height of 6 m, which occurs for a storm setup of about 1 m. An offshore wave height of 3 m in combination with a setup of 2 m leads to an erosion area of about 20 m^3/m. The maximum computed recession at the crest is of the order of 5 m. The 1 to 10000 year-storm event yields an erosion area of about 30 m^3/m and a maximum crest recession of about 15 m. In all cases the computed erosion profile is seaward of the enveloppe erosion profile (erosion area of about 100 m^3/m) as used by the Pevensey Coastal Defence for the 1 to 400 year storm case.

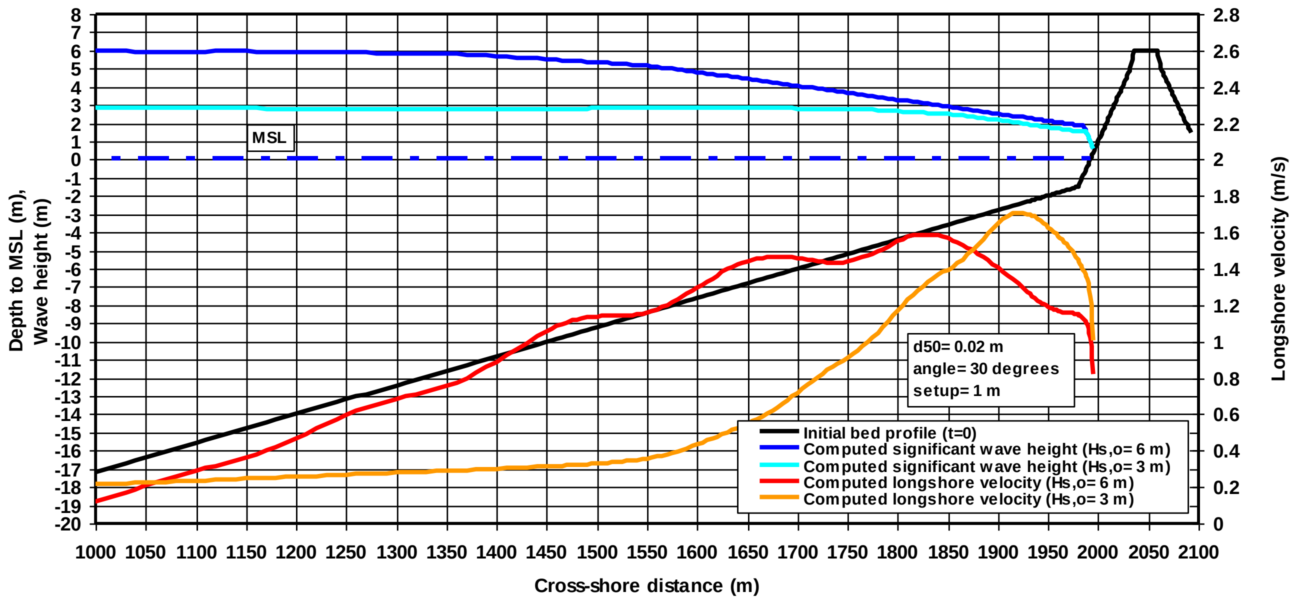


Figure 3.4.1 *Bed profile, wave height, longshore velocity for offshore wave height of $H_{s,o}$= 3 and 6 m; setup= 1 m; offshore wave incidence angle= 30° for Pevensey Bay shingle barrier, UK*

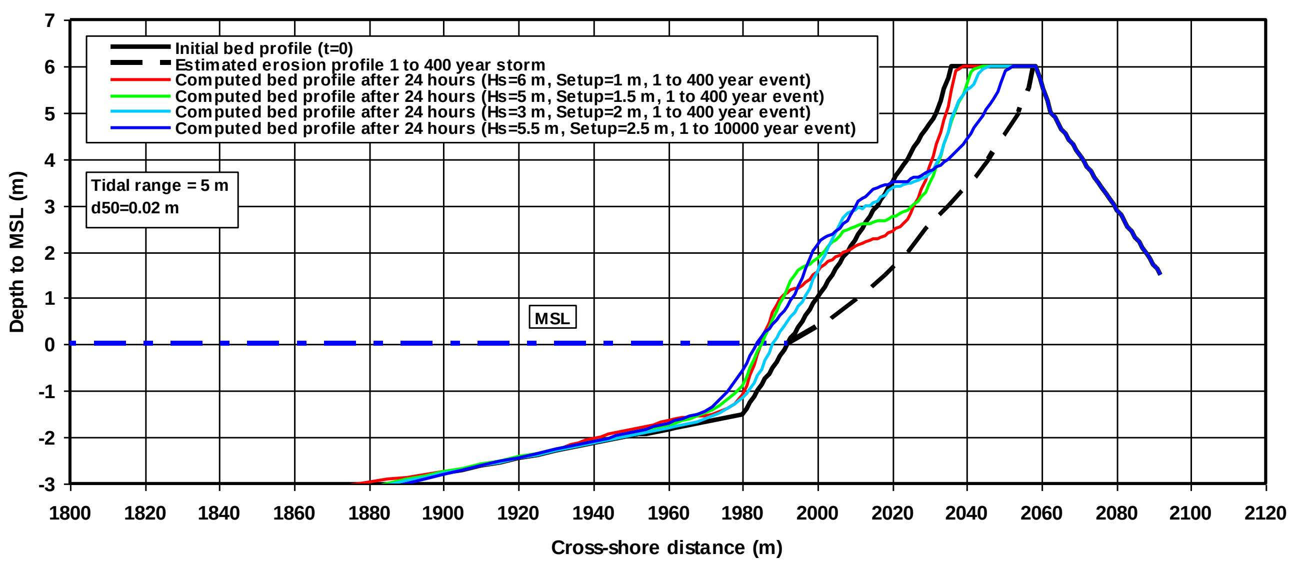


Figure 3.4.2 *Effect of storm surge level and storm intensity on the erosion of Pevensey Bay shingle barrier*

Figure 3.4.3 shows the effect of the shingle size (varied in the range of 0.01 to 0.10 m) on the erosion of the shingle barrier at Pevensey Bay, UK. The computed total erosion area after 24 hours based on the CROSMOR-model is about 30 m^3/m for shingle size of 0.01 m and about 20 m^3/m for shingle size of 0.03 m. The erosion area reduces greatly to about 3 m^3/m when cobbles of 0.1 m are present. General cobble movement will occur at a (Shields) shear stress of about 85 N/m^2. Close to the shore the maximum orbital velocity is of the order of U_{max}= 2 m/s giving a bed-shear stress of about $\tau_{max}=0.5\ \rho\ f_w\ (U_{max})^2 \cong 100\ N/m^2$ (using f_w= 0.05).

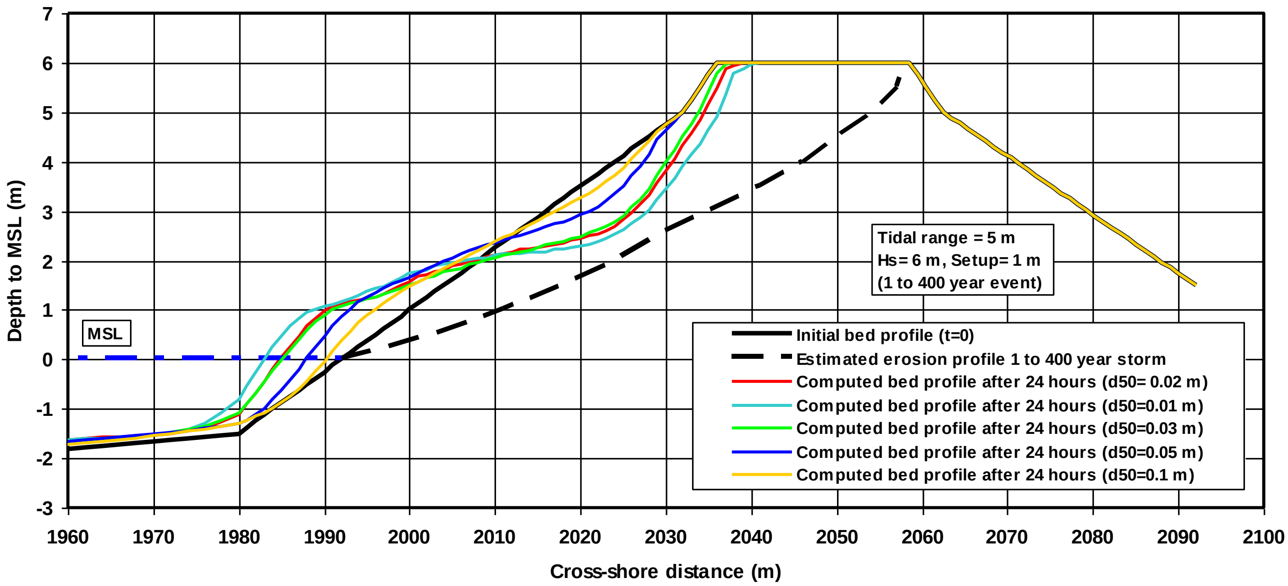


Figure 3.4.3 *Effect of shingle size on the erosion of Pevensey Bay shingle barrier, UK*

4. Beach and surf zone cases

4.1 Beach stability under daily waves; sheltered coast Lake Markermeer, The Netherlands

The CROSMOR-model which includes both onshore- and offshore-directed transport processes, has been used to get information of the beach deformation during daily wave conditions.
A small beach with an alongshore length of about 450 m was made in the summer period of 2014. The beach was made of sand with d_{50}-value of 0.265 mm (mean value of many samples). Measured wave heights at a depth of 2.5 m are up to H_s= 1.2 m, mostly from west to south-west. The lake level varies between -0.2 m and -0.4 m NAP. Water level setup is up to 0.4 m. Wind-induced circulation currents to the east may be as large as 0.4 m/s during stormy periods. The measured wave climate is schematized to 6 wave classes for model input.
Figure 4.1.1 shows the initial beach profile in the middle of the beach and computed bed profiles after 2 years for d_{50}= 0.25 and 0.3 mm. The measured erosion above -1 m NAP is about 20 m^3/m after 2 years. The computed erosion of about 10 m^3/m after 2 years is less than the measured value, but it is a fairly good result given the fact that some of the measured erosion is caused by longshore transport processes which are not taken into account by the model.

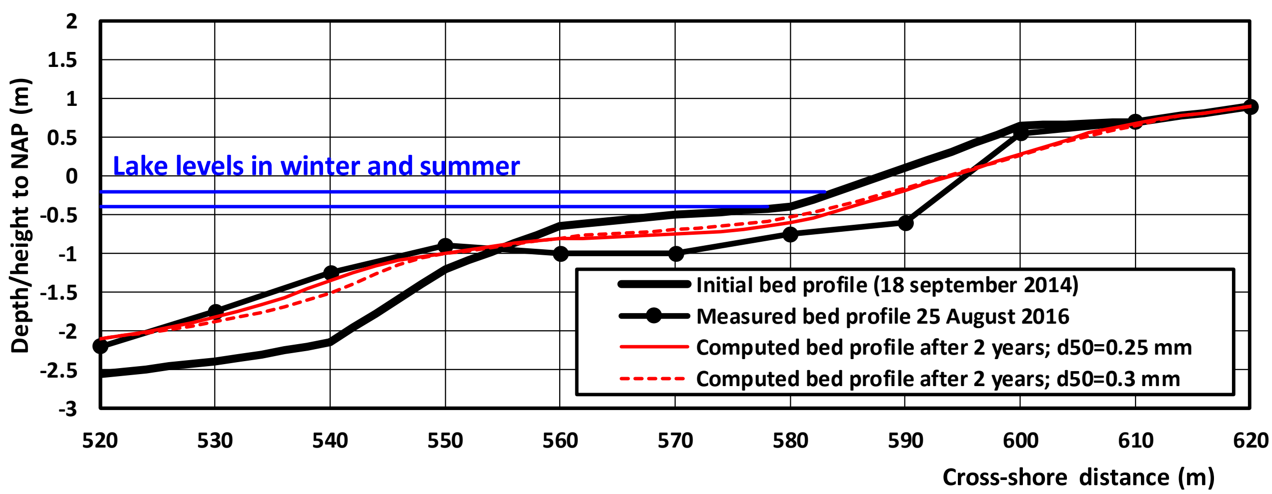


Figure 4.1.1 *Computed beach profiles for daily wave conditions; validation case CROSMOR-model*

4.2 Beach stability under daily waves; sheltered south-east coast of Texel island, The Netherlands

The CROSMOR-model has been applied to find out what is the most stable beach of a new sand dike/dune at the the south-east coast of Texel (bordering the Wadden Sea). A stable beach is close to equilibrium conditions with almost no erosion and maintenance for daily wave conditions. Daily waves at the project site are in the range of 0.3 and 1 m (periods of 3 to 5 s) with wave incidence angles of about 10°. The nearshore tidal current is set to 0.1 m/s. Many runs have been done for a range of beach slopes between 1 to 15 and 1 to 50 to study the most optimum beach slope minimizing the erosional losses on the seasonal time scale. A relatively steep beach of 1 to 15 is attractive because it gives a relatively small construction volume of sand. However, a steep beach slope leads to significant erosion of sand at the upper beach and accretion at the lower beach, ultemately creating a much milder beach slope. The best result (least erosion) was obtained for a beach slope consisting of a mild slope of 1 to 50 below NAP (about mean sea level) and a slope of 1 to 15/20 above NAP up to the dune toe level at +3 m NAP.
Figure 4.2.1 presents the computed bed profiles showing beach accretion due to onshore sand transport. The top layer of the beach consists of a relatively coarse wear layer (erosion buffer) layer with grain sizes of 0.4 to

0.5 mm extending to the -1.5 m NAP depth line. The sea bottom seaward of the -1.5 m depthline consists of fine sand with grain size of about 0.2 mm. This was simulated by running the CROSMOR-model over 3 years with two sand fractions, as follows:

- seaward of -1.5 m NAP: 90% sand of 0.2 mm and 10% sand of 0.5 mm;
- landward of -1.5 m NAP: 10% sand of 0.2 mm and 90% sand of 0.5 mm.

The CROSMOR-model was also run over 3 years with one sand fraction of 0.4 mm. Beach erosion is hardly present. The total beach accretion volume is of the order of 10 to 20 m^3/m/year coming from the lower beach zone. The shallow foreshore consisting of fine sand of 0.2 mm is slightly eroded and a minor part is deposited seaward.

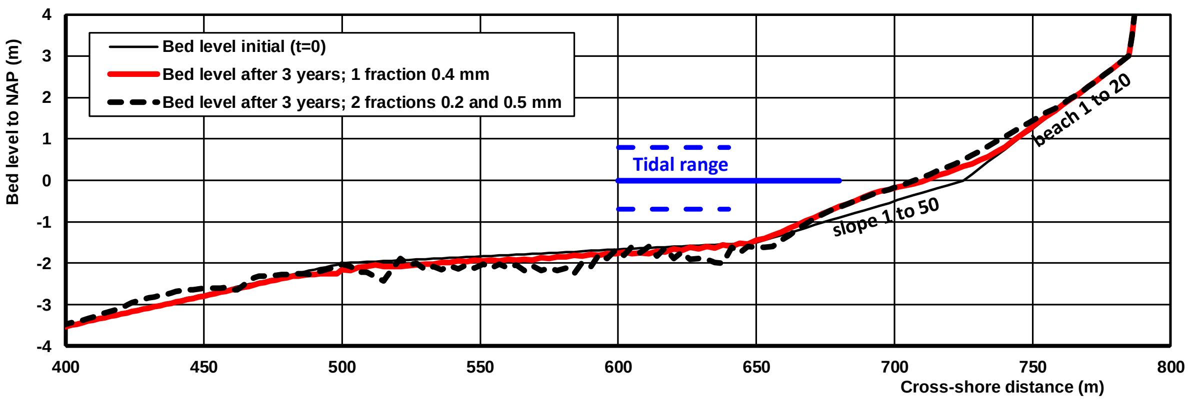


Figure 4.1.2 *Computed beach accretion for daily wave conditions; CROSMOR-model*

4.3 Erosion of beach nourishment for annual wave climate; Holland coast, The Netherlands

The section Petten to Camperduin of the Holland coast was protected by an asphalt-type of dike. Around 2010, it was decided to construct a sand beach-dune system in front of the dike. This coastal section is known as the 'Hondbossche and Pettemer' Sea dike with a length of about 6 km. The sea dike section protrudes into the sea over a distance of about 200 m with respect to the surrounding coastline.

The wave climate is a yearly-averaged wave climate based on observations in the period 1980-1988 and has 8 wave height classes ($H_{significant}$) between 1.5 and 3.2 m. A storm with a deep-water wave height of $H_{s,o}$= 6 m and a duration of 12 hours is added. This is an extreme wave height with a recurrence period of 100 years, which is applied 5 times over the computation period of 5 years. Two storm surge levels (including tide levels) of +3 and +4 m NAP have been used. The wave asymmetry of the near-bed velocities is computed by the method of Isobe-Horikawa. The vertical tide is between +1 and -0.8 m NAP. The maximum flood velocity in deep water is set to 0.6 m/s; the maximum ebb velocity is set to 0.5 m/s. Two median particle sizes have been used: d_{50}= 0.21 and 0.25 mm.

Figure 4.3.1 shows the initial bottom and the computed bed levels after 1, 2 and 5 years for d_{50} = 0.25 mm of Profile 21.230 km (final design).

In the first year a new sand bar with a width of about 150 m and a height of about 4 m is generated between x= 500 m en 700 m. The erosion at the upper beach is caused by the storm waves with a surge level of +4 m. Most of the erosion takes place at the lower beach.

Based on all CROSMOR-results, the average annual erosion volume during the initial phase of about 5 years after construction is estimated to be in the range of 55 to 85 m^3/m/year (70±15 m^3/m/year; uncertainty of about 20%). The sand material is eroded from the beach zone and deposited into the surf zone beyond the -3 m NAP-line. The maximum recession due to cross-shore transport processes after 5 years is estimated to be about 100±30 m.

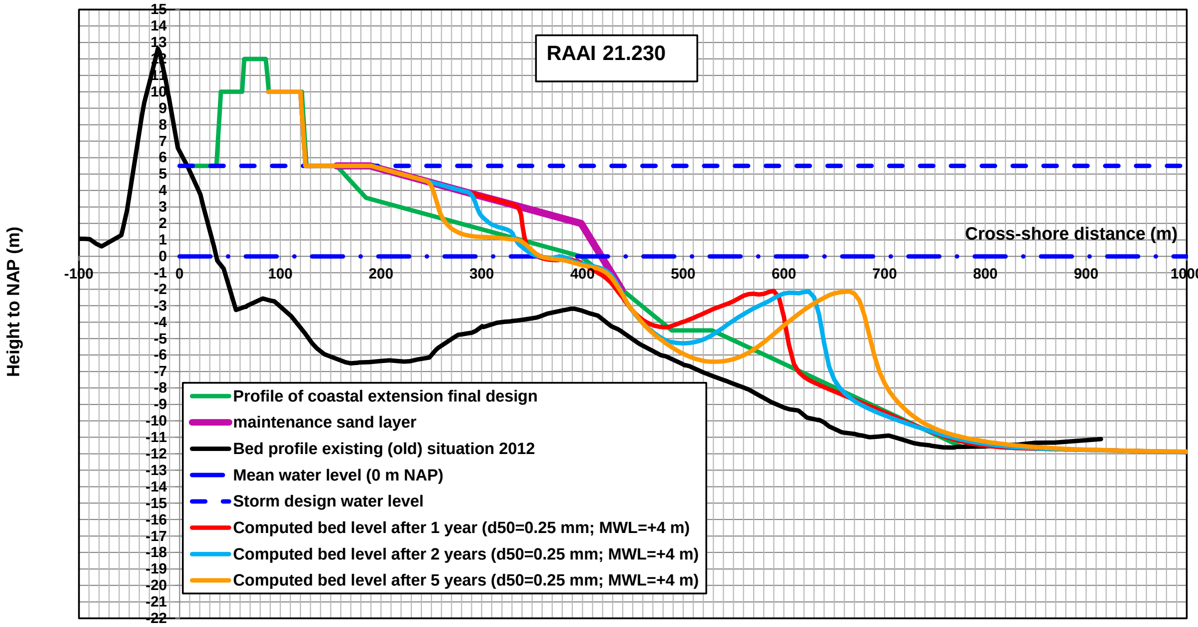


Figure 4.3.1 *Computed bed level of cross-shore profile 21.230 km; CROSMOR; d_{50}= 0.25 mm; maximum water level=+4 m.*

4.4 Sediment sorting in beach and surf zone; Katwijk beach, The Netherlands

Detailed information of cross-shore grain size variations at the Katwijk-site (about 20 km north of The Haque) along the North Sea coast of The Netherlands has been presented by Terwindt (1962). He studied grain size variations and the effect of a storm event on the cross-shore particle size distribution. The results of Terwindt (1962) for the location Katwijk are based on the analysis of samples collected in a summer period under different hydraulic conditions (fairweather and minor storm event; 1 to 2 weeks). The maximum significant wave height outside the surf zone was estimated to be about 3 m during summer storm conditions. The summer storm period has been modelled by assuming offshore significant wave heights between 1 and 3 m during 1 week (see input data below).
The observed cross-shore grain size variations (Figure 4.4.1) show the following features:
<u>*Before summer storm period*</u>

- relatively coarse material (d_{50} of about 300 μm) in the shallow swash zone near the water line;
- systematic fining of sediment material in seaward direction over the width of the surf zone; seaward of the outer breaker bar the d_{50} has reduced to a value of about 140 micron during periods with calm weather;
- fining of sediment from the swash zone (300 micron) to the dune top (220 micron);

<u>*After summer storm period*</u>

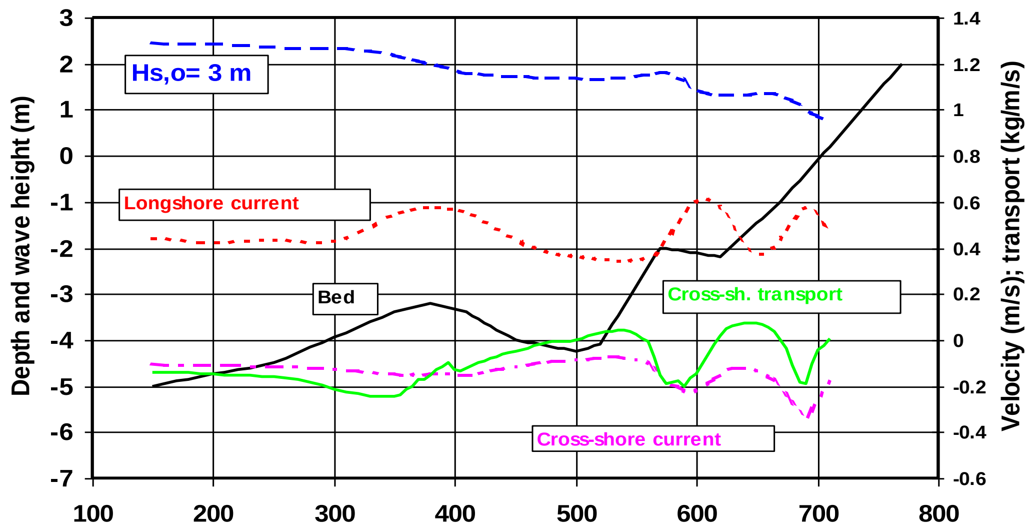
- the sediments in the outer surf zone are found to be somewhat coarser (10% to 20%; d_{50} is about 180 micron) after a summer storm period; the fraction 105-150 micron shows the greatest variations; during calm periods the fraction 105-150 micron is dominant (50% to 70%) in the bed material; after the storm period the contribution of the 105-150 fraction is reduced to about 20%; thus the finer material is washed out during conditions with higher waves and is most probably transported in suspension to deeper water where it is deposited.

The CROSMOR-model has been used to simulate the Katwijk-measurements. The input data are:

Significant wave height at depth=10 m		
t= 0 hrs	$H_{1/3,0}$	= 1 m
t= 24 hrs (1 day)	$H_{1/3,0}$	= 1 m
t= 36 hrs (1.5 days)	$H_{1/3,0}$	= 3 m
t= 60 hrs (2.5 days)	$H_{1/3,0}$	= 3 m
t= 72 hrs (3 days)	$H_{1/3,0}$	= 1 m
t= 168 hrs (7 days)	$H_{1/3,0}$	= 1 m
Water depth (x=0 m)	h_0	= 10 m
Wave incidence angle (x=0 m)	θ	= 10 degrees
Longshore tidal velocity (x=0 m)	v_0	= 0.6; -0.5 m/s
Tide levels (x=0 m)	Δh	= 0.8; -0.8 m
Peak period	T_p	= 8 s
Number of wave classes	NW	= 5
Fluid density	ρ_w	= 1030 kg/m^3
Bottom friction	$k_{s,w}$	= 0.02 m
Water temperature	Te	= 15 degrees C
Salinity	Sa	= 30 promille
Current-related bed roughness	$k_{s,c}$	= 0.02 m
Wave-related bed roughness	$k_{s,w}$	= 0.02 m
Multi fraction method (N=5)	d_{50}	= 0.14 to 0.28 mm (d_i = 0.1; 0.15; 0.2; 0.3; 0.4 mm)
Thickness of bed surface layer	δ	= 0.2 m
Factor high-freq. susp. transport	γ	= 0.2

Figure 4.4.1 shows the computed distribution of the d_{50} along the cross-shore profile after 7 days based on the multi fraction method. The initial particle size distribution (model input) is represented by a variation between 0.14 and 0.28 mm based on the trend of the measured d_{50}-values before the summer storm period.
The computed results are:

- during the storm event the computed suspended transport is dominant and offshore-directed in the swash zone (670 m < x < 720 m); on the landward flank of the inner bar (560 m < x < 620 m) and at the outer bar (x < 470 m), where the longshore and cross-shore currents are relatively large;
- bed-load transport is dominant and onshore-directed in the troughs landward of the inner and outer bars (470 m < x < 560 m; 620 m < x < 670 m);
- deposition can be observed in the swash zone (650 m<x< 690 m) where a swash bar is generated just seaward of the mean waterline (MSL-line); deposition also occurs around the crest of the inner bar (550 m < x < 580 m) and just landward of the outer bar (390 m < x < 420 m);
- erosion can be observed around the mean waterline (680 m < x < 720 m); in the trough landward of the inner bar (580 m < x < 650 m) and at the crest of the outer bar (330 m < x < 390 m);
- the changes in particle size (d_{50}) correspond strongly to the computed deposition and erosion patterns; coarsening of the bed surface does occur in the erosion zones where the finer particles are winnowed from the bed and are transported to the deposition zones resulting in fining of the bed surface; the beach shows a tendency for coarsening of the bed surface;
- the computed coarsening effects in the trough between the inner and outer bar are in reasonably good agreement with the observed pattern; the coarsening effect observed seaward of the outer bar is not represented by the model; the coarsening effect in the trough zone landward of the inner bar and the fining effect seaward of the mean waterline are in reasonable agreement with the observed patterns.



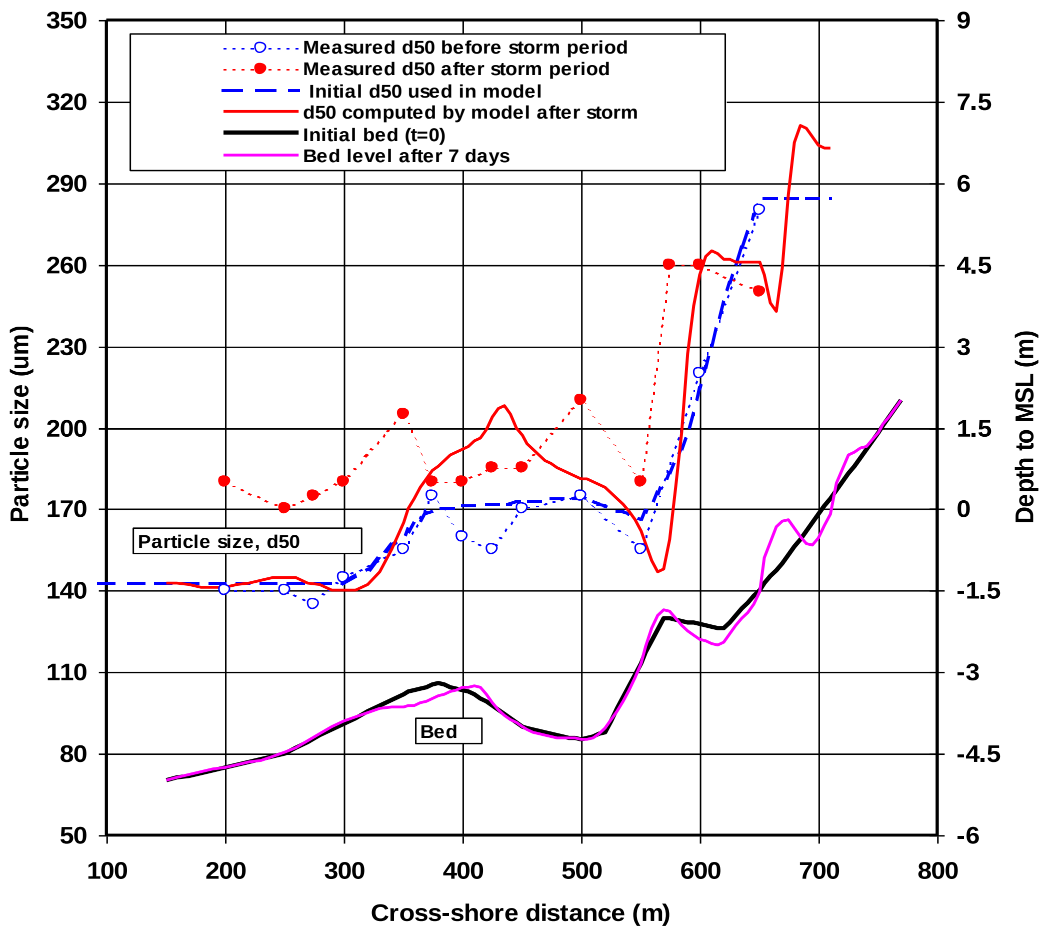


Figure 4.4.1 *Effect of storm event on cross-shore particle size distribution, Katwijk-site, The Netherlands*
Top: Wave heights and currents
Bottom: Particle size and bed profile

4.5 Sedimentation and erosion of mining pit for land reclamation; China

The City of Dong Ying (China) has planned the construction of a large-scale land reclamation site (approximately 4 x 14.5 km²), just south of the mouth of the Yellow River in China. The project site is characterised by a very shallow nearshore area (with a slope in the range between 1 to 1500 and 1 to 2000), meso tidal water level movement of the Bohai Sea, large sediment supply to the coastal zone originating from the Yellow River (although greatly reduced in recent years as a consequence of river training works upstream) and large amounts of very fine, non-cohesive sediments in the coastal zone. The sediment for the land reclamation site is taken from a nearshore borrow area (sediment extractionmining pit).
The CROSMOR-model has been run for a period of 6 months with waves (no waves in the other 6 months). The tide is schematized into 13 blocks of 1 hour with quasi-steady flow. The tidal range is 1 m; the peak tidal velocities are 0.5 m/s in the flood and in ebb directions.
The pit (borrow area) is situated between the -5 m and -2 m depth lines, the bottom of the pit is at -10 m, the seaward slide slope is 1 to 200 and the landward side slope is 1 to 125. The length of both side slopes is 1000 m. The initial (at t=0) bed profile of the pit is shown in **Figures 4.4.1**. One constant, representative wave height of $H_{s,o}$=2 m ($H_{rms,o}$=1.4 m) was used over a period of 6 months (183 days).
Runs with and without tidal filling velocities have been made. The tidal filling velocities are associated with the velocities required to generate the tidal water level changes (rising and falling of tide). These velocities may be quite large due to the large horizontal scale of the bed profile (20 km). Assuming a water level rise of 0.15 m per hour (3600 s), the tidal filling and emptying velocity at 2 km (2000 m) from the shore (water depth= 2 m) is: v=0.15x2000/(2x3600) = 0.042 m/s. This latter velocity is of the same order of magnitude as the offshore-directed undertow generated by the waves. It should be realized, however, that the modelling of the rising and falling of the tide in the cross-shore direction is not fully realistic. Generally, the tides are propagating parallel to the shore. Therefore, runs with and without this effect have been made.

Figure 4.5.1 shows the computed bed profiles after 1 year. The main results are:

- erosion occurs in the nearshore zone and is of the order of -1 m over a length of about 1500 m;
- deposition occurs on the landward side slope of the pit; in reality this deposition will be much more spread out due to the delayed settling of the fine sediments;
- erosion occurs at the seaward edge of the pit and is of the order of -1 m;
- erosion and deposition is about 50% less if the cohesive effects of the clay fraction (30% clay) is taken into account;
- tidal filling has not much effect on the erosion and deposition values.

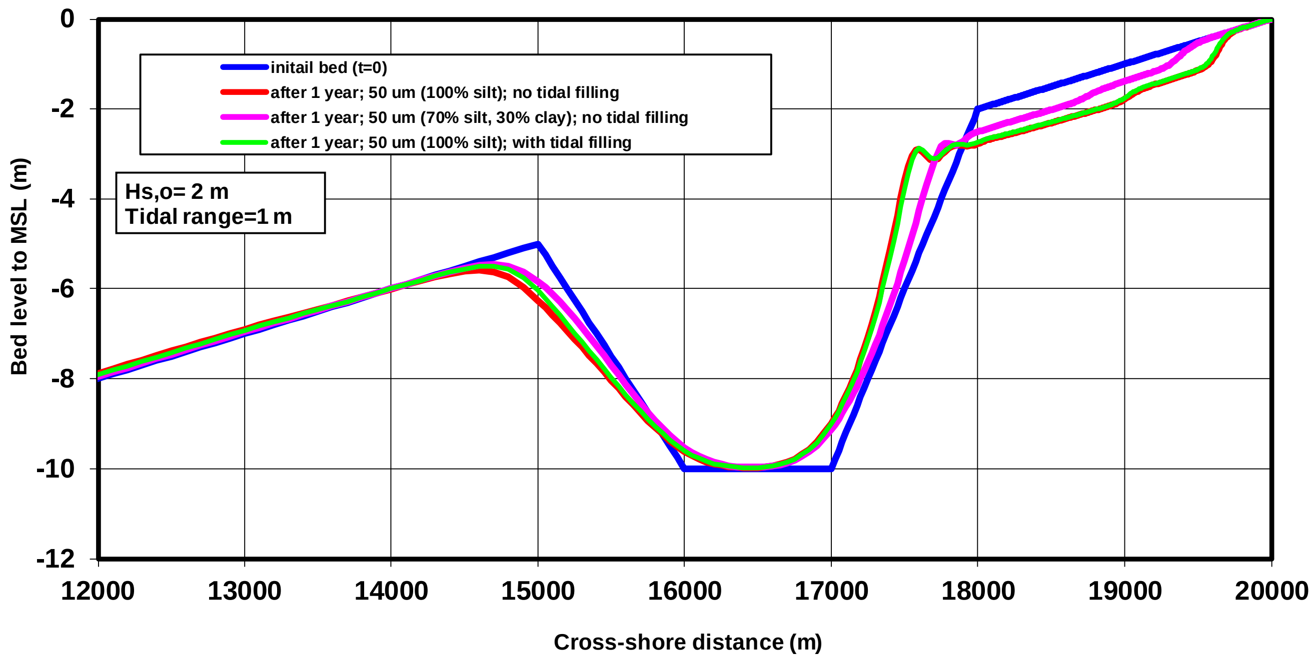


Figure 4.5.1 *Computed bed profiles after 1 year for nearshore mining pit; Dong Ying, China*

4.6 Onshore bar migration at beach; Duck beach , USA

The present data were obtained during the Duck94 field experiment conducted near Duck, North Carolina (USA) on a barrier island exposed to the Atlantic Ocean (Gallagher et al., 1998). The data period covered is 21 September to 20 October 1994.

The presence of a breaker bar can be observed around the cross-shore position of 200 m from the reference line. The bathymetry characteristics can be described as:

- 21 September: minor storm on 21 Sept., $H_{s,offshore}$= 2.5 m (maximum);
  the bar has a reasonably straight alignment between the main transect 945 m and the transect 1200 m; an oblique (crescentic) pattern can be observed between transects 700 m and 945 m;
- 21 September - 4 October: significant wave height $H_{s,offshore}$ between 1 and 1.5 m;
  the bar has moved offshore (about 25 m) between transects 700 and 800 m; the bar has moved onshore slightly between transects 875 and 975 m; bar alignment is almost straight;
- 4 October - 10 October: $H_{s,offshore}$ is maximum 1 m; bar movement is minor;
- 10 October - 14 October: $H_{s,offshore}$ between 1 and 1.5 m;
  bar has moved offshore (about 30 to 40 m); bar alignment is almost staight;
- 14 October - 16 October: $H_{s,offshore}$ is between 2 and 3 m (storm period);
  16 October - 20 October: $H_{s,offshore}$ is between 1.5 and 2 m;
  bar has moved offshore (about 50 m) between transects 875 and 975 m; bar has moved onshore near transects 700-850 m; bar has moved onshore near transects 1050-1200 m; bar alignment is crescentic.

During the period September 21-26, the bar crest around the main transect at 945 m moved onshore over a distance of about 15 m. Longshore variations of the bar are minor over a distance of 50 m on both sides of the main transect 945 m. The significant wave height at a depth of about 5 m varied between 0.5 and 2.5 m; a minor storm with a duration of about 12 hours occurred on September 21; most of the time (5 days) the wave height varied between 0.5 and 1 m.

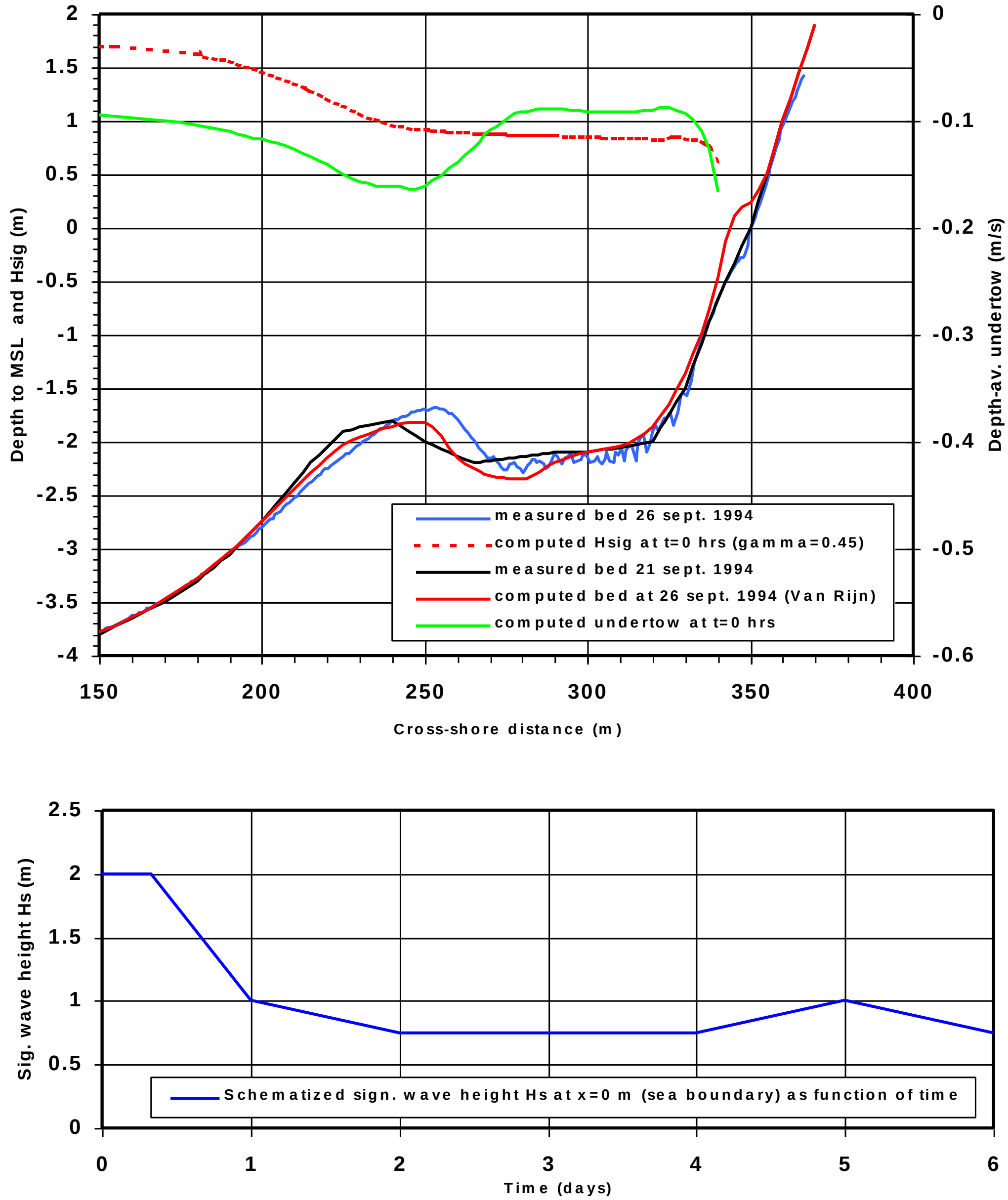


Figure 4.6.1 *Measured and computed bed levels for 21-26 September 1994 event and schematized offshore significant wave height*

The profile model CROSMOR has been applied to simulate the bed level changes in the main transect at 945m. Relatively large wave-induced ripples are assumed to be present in the trough zone, having a wave-related bed roughness value of 0.05 m. The bed material is represented by d_{50}= 0.15 mm and d_{90}= 0.3 mm.
Preliminary runs have been made to calibrate the transport factor (multiplication factor to standard sand transport) yielding 0.5. The wave breaking coefficient is taken as γ= 0.45. The sand transport model of Van Rijn has been used to compute the morphologic development of the cross-shore profile.

The computed results (**Figure 4.6.1**) show the following features:

- slight onshore migration of the bar, which is considerably smaller than the oberved values; the sand transport due to the wave asymmetry and the undertow creates a convergence point just landward of the bar crest resulting in onshore bar migration; the relatively large bed roughness in the trough zone is necessary to generate sufficiently large offshore-directed sand transport by the undertow in the trough zone;
- generation of beach bar (about 4 m^3/m) near the mean waterline, which is not observed in the field data.

5. Offshore zone cases

5.1 Shoreface nourishment; Holland coast

The CROSMOR-model has been used to compute bed level changes along a shoreface nourisment, see **Figure 5.1.1**.
The tidal range is 1.1 m. The peak tidal velocity is 0.5 m/s at deep water.
The significant wave heights are between 1.7 and 2.7 m with periods periods in the range of 6.5 to 7.5 s over a period of 100 days. The wave incidence angle= 0 (perpendicular waves). The grain size is 0.3 mm.
Figure 5.1.1 shows the bed profile development after 100 days. Minor erosion occurs at the seaward end of the nourishment zone due to onshore transport.

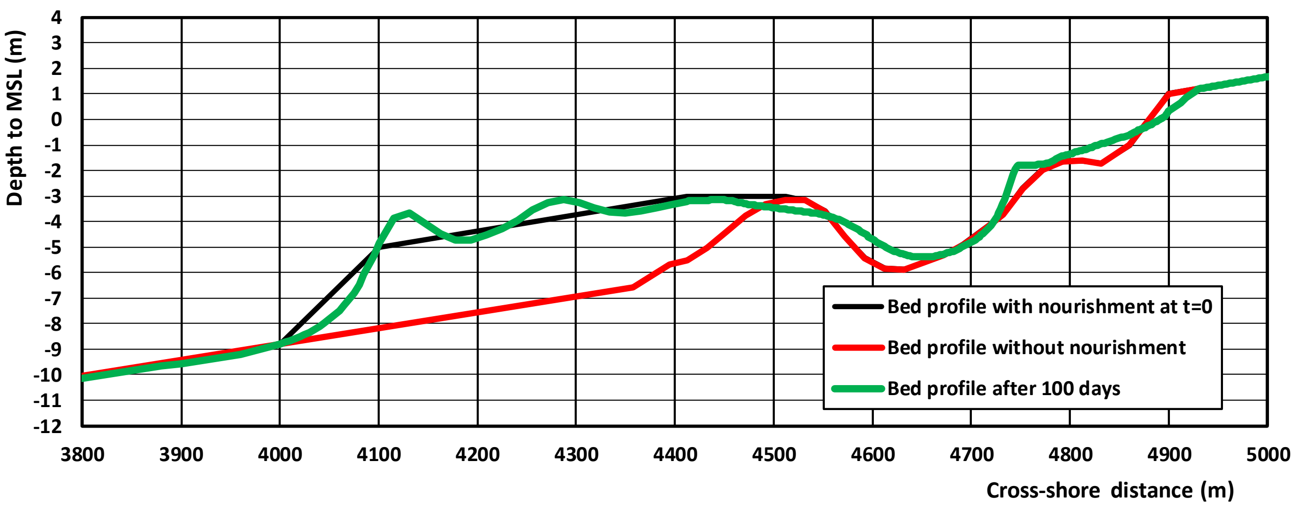


Figure 5.1.1 *Erosion of shoreface nourishment at barred coastal profile*

References

Dally, W.R. and Osiecki, D.A., 1994. The role of rollers in surf zone currents. 24th ICCE, Kobe, Japan
Davies, A.G. and Villaret, C., 1997. Oscillatory flow over rippled beds. In: J.N. Hunt (ed.), Gravity waves in water of finite depth: Chapter 6, p. 215-254. Advances in fluid mechanics, Computational Mechanics Publications
Davies, A.G. and Villaret, C., 1998. Wave-induced currents above rippled beds, p. 187-199. In: Physics of estuaries and coastal seas, edited by J. Dronkers and M. Scheffers, Balkema, Brookfield
Davies, A.G. and Villaret, C., 1999. Eulerian drift induced by progressive waves above rippled and very rough bed, p. 1465-1488. Journal of Geophysical Research, Vol. 104, No. C1
Gallagher, E.L., Elgar, S. and Guza, R.T., 1998. Observations of sand bar evolution on a natural beach. Journal of Geophysical Research, Vol. 103, N0. C2, p. 3203-3215
Grasmeijer, B.T., 2001. Sand transport and morphology in the surf zone of Egmond, The Netherlands. Doc. Thesis, Dep. of Physical Geography, University of Utrecht, The Netherlands (in preparation)
Grasmeijer, B.T. and Van Rijn, L.C., 1998. Breaker bar formation and migration. Proc. ICCE., Copenhagen, Denmark
Houwman, K.T. and Ruessink, B.G., 1996. Sediment transport in the vicinity of the shoreface nourishment of Terschelling. Dep. of Physical Geography. Univ. of Utrecht, The Netherlands
Isobe, M. and Horikawa, K., 1982. Study on water particle velocities of shoaling and breaking waves. Coastal Engineering in Japan, Vol. 25.
Svendsen, I.A., 1984. Mass flux and undertow in the surf zone. Coastal Engineering, Vol. 8, p. 347-365
Terwindt, J. J., 1962. Study of grain size variations at the coast of Katwijk, The Netherlands, Note K-324, Rijkswaterstaat, Deltadienst, Den Haag
Van Rijn, L.C., 1990, 2011. Principles of fluid flow and surface waves in rivers, estuaries and coastal seas. Aqua Publications, Nederland (www.aquapublications.nl)
Van Rijn, L.C., 1993, 2006. Principles of sediment transport in rivers, estuaries and coastal seas. Aqua Publications, Nederland (www.aquapublications.nl)
Van Rijn, L.C., 1997. Cross-shore sand transport and bed composition. Coastal Dynamics, Plymouth, England, p. 88-98
Van Rijn, L.C., 1998. The effect of sediment composition on cross-shore bed profiles. ICCE. Copenhagen, Denmark
Van Rijn, L.C., 2005/2012. Principles of sedimentation and erosion engineering in rivers, estuaries and coastal seas. www.aquapublications.nl
Van Rijn, L.C., 2006, 2012. Principles of sedimentation and erosion engineering in rivers, estuaries and coastal seas. Aqua Publications, The Netherlands (www.aquapublications.nl)
Van Rijn, L.C., 2007a. Unified view of sediment transport by currents and waves, I: Initiation of motion, bed roughness and bed-load transport. Journal of Hydraulic Engineering, ASCE, Vol. 133, No. 6, p. 649-667
Van Rijn, L.C., 2007b. Unified view of sediment transport by currents and waves, II: Suspended transport. Journal of Hydraulic Engineering, ASCE, Vol. 133, No. 6, p. 668-689
Van Rijn, L.C., 2007c. Unified view of sediment transport by currents and waves, III: Graded beds. Journal of Hydraulic Engineering, ASCE, Vol. 133, No. 7, p. 761-775
Van Rijn, L.C., 2007d. Unified view of sediment transport by currents and waves, IV: Application of morphodynamic model. Journal of Hydraulic Engineering, ASCE, Vol. 133, No. 7, p. 776-793
Van Rijn, L.C., 2008. Modelling of beach and dune erosion. Report Z4173/Z4230. Delft Hydraulics, Delft , The Netherlands
Van Rijn, L.C., 2009. Prediction of dune erosion due to storms. Coastal Engineering,56, p. 441-457

Van Rijn, L.C., 2010. Coastal erosion control based on the concept of sediment cells. EU-Conscience project. Deltares, 2010, Delft, The Netherlands.

Van Rijn, L.C. and Wijnberg, K.M., 1994. One-dimensional modelling of individual waves and wave-induced longshore currents in the surf zone. Report R 94-09, Department of Physical Geography, University of Utrecht.

Van Rijn, L.C. and Wijnberg, K.M., 1996. One-dimensional modelling of individual waves and wave-induced longshore currents in the surf zone. Coastal Engineering, Vol. 28, p. 121-145

Van Rijn, L.C., Walstra, D.J.R., Grasmeijer, B., Sutherland, J., Pan, S. and Sierra, J.P., 2003. The predictability of cross-shore bed evolution of sandy beaches at the time scale of storms and seasons using process-based profile models, p. 295-327. Coastal Engineering, 47