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Erosion of gravel barriers and beaches

Leo C. Van Rijn and James Sutherland

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EROSION OF GRAVEL BARRIERS AND BEACHES

Leo C. Van Rijn[1] and James Sutherland[2]

[1] Deltares, Delft and Department of Physical Geography, University of Utrecht, The Netherlands.
[2] HR Wallingford, Howbery Park, Wallingford, Oxfordshire, OX10 8BA, U.K.

Abstract

The erosional behaviour of gravel type beaches and barriers has been studied using a parametric and a process-based model. Both models have been applied to large-scale laboratory experiments and to a field case at Pevensey Bay, UK. Both models show suprisingly good agreement for realistic wave conditions. The process-based model has been used to compose a graph of gravel beach erosion as a function of storm surge level and gravel size.

1 Introduction

Beaches consisting of gravel or shingle (2 to 64 mm), pebbles and cobbles (64 to 256 mm) are generally known as *coarse clastic beaches.*. Clasts are individual grains within coarse populations. Subgroups are pebbles, cobbles and boulders, which are clasts larger than 256 mm. A typical gravel/shingle beach can be seen as a layer of gravel material sloping up against a cliff. A gravel barrier can be seen as a dike of gravel material; swash-aligned barriers or longshore drift-aligned barriers are distinguished. Typical profiles are shown in Figure 1.

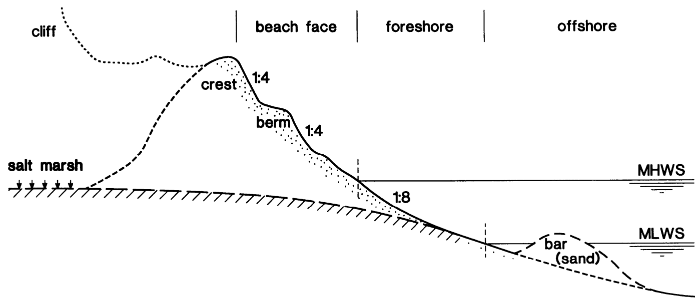

Figure 1 Typical cross-shore profile of gravel beach slop

Coarse clastic beaches can be found in many (formerly glaciated) mid- and high-latitude parts of the world (England, Iceland, Canada, etc.). Artificial gravel (marble) type beaches are also found along Mediterranean beaches eroded by wave attack (Figure 2 Bottom).

Generally, the upper beach consists of gravel/shingle material, while the lower beach or foreshore consists of sandy material, see Figure 2 Top-right. Some of these beaches have a large proportion of sand intermixed with gravel, especially in the foreshore zone just beneath the mean water line (see Figure 2 Top-right). In regimes with dominating gravel populations, the sand becomes a subsidiary interstitial component. In regimes with a relatively large tidal range the back beach may consist of gravel ridges fronted by a low-tide terrace of sand (exposed at low tide). These types of beaches have less appeal for recreational activities, but they are rather efficient (high dissipation of energy through high permeability) for coastal protection.

Figure 2 Timber groynes at beach of Eastbourne, East Sussex, UK (top) andmarble beaches at Carrara coast, Italy (bottom)

Gravel on beaches is moved almost exclusively by wave action (asymmetric wave motion); tidal or other currents are not effective in moving gravel/shingle material. The coarse particles move up the beach to the run-up limit by strong bores (uprush) and swash motions. The particles move down the beach close to the line of the steepest beach slope by the backwash (less strong due to percolation) plus gravity, resulting in a saw-tooth movement. Waves of long periods on steep beaches can produce peak swash velocities up to 3 m/s. The alongshore transport path of individual clasts (20 to 40 mm) may be as large as 1,000 m per day during periods with storm waves, based on tracer studies. Reviews of swash processes are given by: Van Rijn (2009b), Elfrink and Baldock (2002) and Butt and Russell (2000).

Gravel particles in shoaling and breaking waves generally move as bed load towards the beach during low wave conditions. As the near-bed peak orbital velocity in the onshore direction is greater than the offshore-directed value, the particles will experience a net onshore-directed movement during each wave cycle.

1.1 Laboratory and field data

Various small-scale and large-scale laboratory tests are available for analysis. A detailed physical model programme has been conducted in a random wave flume at HR Wallingford (Powell, 1990). A total of 181 detailed flume tests were undertaken at a scale of 1:17. A range of particle sizes and gradings from typical UK shingle beaches were represented by four distinct mixes of crushed anthracite. Various experiments on the behaviour of gravel and shingle slopes under wave attack have been performed by Deltares/Delft Hydraulics (1989) in the large-scale Deltaflume (length of 200 m, width of 5 m and depth of 7 m). Two gravel sizes have been used (d_{50}= 0.0048 m and d_{50}= 0.021 m). The initial beach slope was 1 to 5 (plane sloping beach) in all (nine) experiments. Various experiments on the behaviour of shingle slopes under irregular wave attack have been performed in the large-scale GWK flume in Hannover, Germany (López et al., 2006). The shingle material has d_{50} of approximately 20 mm. The initial slope of the beach was about 1 to 8. In June and July 2008 large-scale experiments on gravel barriers (11 mm) have been performed in the Deltaflume of Deltares (Buscombe, Williams and Masselink, 2008) by a consortium of researchers led by the University of Plymouth, UK. The sea water level was varied as a function of time to simulate tidal variations. The lagoon water level was also varied. High sea water levels were used to study the occurrence of wave overtopping and overwashing. The water level was gradually increased until barrier destruction occurred.

Field data have been reported by various authors. Barrier recession rates up to 4 m per year have been observed (Nicholls and Webber, 1988) at Hurst beach, Christchurch Bay, England. Most of the recession did occur during autumn and winter months. Bradbury and Powell (1990) give an example of barrier rollover and lowering at Hurst Spit, England.

The Pevensey Coastal Defence (Sutherland and Thomas, 2010) uses a schematized erosion profile due to a storm with a return interval of 400 years to evaluate the strength of the 9 km long shingle barrier along the coast of Pevensey Bay (East Sussex, English Channel coast of southern England) under storm conditions. The estimated erosion area based on extrapolation of observed erosion volumes is approximately 100 m^3/m. The highest waves arrive predominantly from the south-west with significant offshore wave heights up to 6 m. High water levels during major storms vary in the range of 3.5 to 4.5 m above MSL. The shingle barrier along the Pevensey Bay coast consists of a mixture of sand (smaller than 2 mm), gravel (2 to 60 mm) and cobbles (greater than 60 mm). This barrier can be overtopped by large waves, may leak or roll-back landward and ultimately may breach. Temporary flooding events did occur at Pevensey in 1926, 1935, 1965 and 1999.

1.2 Model simulation of gravel barrier erosion

Available models

Two types of models have been used: the parametric SHINGLE model of HR Wallingford (Powell, 1990) and the process-based CROSMOR-model of University of Utrecht (Van Rijn 2006, Van Rijn et al. 2003, 2007).

The parametric SHINGLE model (based on empirical scale model results) allows the user to predict changes of shingle beach profiles based on input conditions of sea state, water level, existing profile, sediment size and the underlying stratum. The profile shape and its location against an initial datum can be predicted and confidence limits for the predictions determined. This capability can be used to predict potential erosion of existing shingle beaches or to predict the performance of shingle renourishment schemes.

The CROSMOR2008-model is an updated version of the CROSMOR2004-model (Van Rijn, 2006, 2007). The model has been extensively validated by Van Rijn et al. (2003). The detailed swash processes in the swash zone are not explicitly modelled but are represented in a schematized way by introducing an effective onshore-directed swash velocity ($U_{sw,on}$) in a small zone just seaward of the last grid point, see Figure 3. It is assumed that the peak onshore-directed component of the swash velocity is much larger than the peak offsshore-directed swash velocity close to the shore. The swash velocity is added to the other cross-shore components of the near-bed velocity (orbital velocity including asymmetry, streaming) and then combined with the longshore near-bed velocity. The resulting instantaneous velocity is used to determine the instantaneous bed-shear stress and then the instantaneous bed-load transport (within the wave cycle).

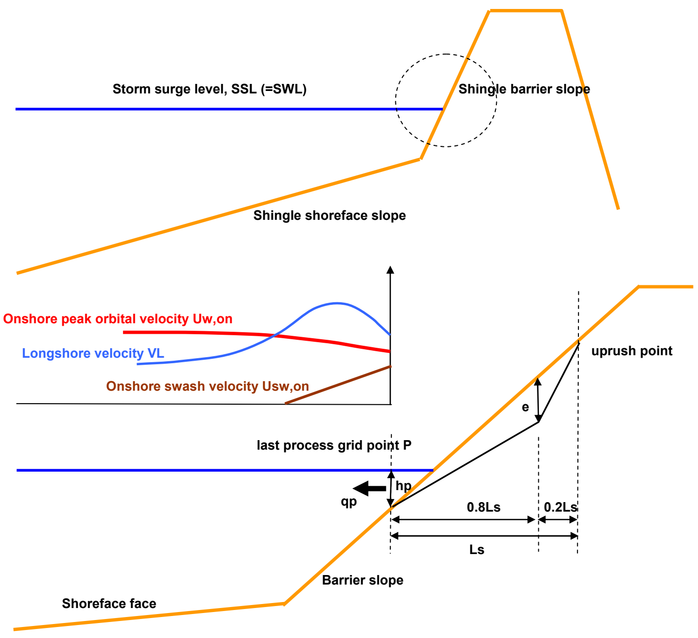

Figure 3

Figure 3 Schematization of swash erosion zone for shingle barrier

The sediment transport of the CROSMOR2008-model is based on the TRANSPOR2004 formulations (Van Rijn, 2006, 2007). The sediment 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.

Bed level changes seaward of the last grid point are described by:

$$\rho_s(1-p)\partial z_b/\partial t + \partial(q_t)/\partial x = 0 \qquad (1)$$

with: z_b= bed level to datum, q_t= q_b + q_s= volumetric total load (bed load plus suspended load) transport, ρ_s= sediment density, p= porosity factor.

In discrete notation: $$\Delta z_{b,x,t} = -[(q_{t,})_{x-\Delta x} - (q_t)_{x+\Delta x}]\,[\Delta t/(2\,\Delta x(1-p)\rho_s] \qquad (2)$$

with: Δt= time step, Δx= space step, $\Delta z_{b,i,x,t}$ = bed level change at time t (positive for decreasing transport in positive x-direction, yielding deposition). The new bed level at time t is obtained by applying an explicit Lax-Wendorf scheme.

Deposition and erosion in the swash zone between the waterline and the uprush point (landward of the last gridpoint) is a typical morphological feature of wave attack on a steep slope and is represented in a schematized way by using a subgrid model. The length of the swash zone is determined as the distance between the last grid point and the uprush point. In the case of steep shingle slopes the run-up level can be determined by an empirical expression: R_s= $(H_{s,o}\ L_{s,o})^{0.5}$ with a maximum value of 5 m above the mean water level. The maximum run-up is set to 5 m because the run-up along a steep, permeable shingle slope with percolation effects will be significantly smaller than along a rigid, smooth slope.

The total deposition or erosion area (A_D or A_E) over the length of the swash zone is herein defined as: $A_D=q_p\ \Delta t/((1-p)\rho_s)$ with: q_p= cross-shore transport computed at last grid point P at the toe of swash zone, Δt= time step, p= porosity factor of bed material, ρ_s= sediment density. The deposition (or erosion) profile in the swash zone is assumed to have a triangular shape, see Figure 3**.** The maximum deposition or erosion (e) can then be determined from the area A_D. The cross-shore transport on steep shingle slopes is onshore directed during low wave conditions due to the dominant effect of the velocity asymmetry and the percolation of fluid through the porous bed surface. The cross-shore transport is offshore-directed during storm conditions.

Deltaflume experiments

Tests 1, 2 and 9 of the Deltaflume experiments in 1989 have been used to verify the CROSMOR-model for gravel slopes. Since the CROSMOR-model is a model for individual waves; the wave height distribution is assumed to be represented by a Rayleigh-type distribution schematized into 6 wave classes. Based on the computed parameters in each grid point for each wave class, the statistical parameters are computed in each grid point. The limiting water depth is set to 0.5 m (water depth in last grid point). Based on this value (including the computed wave-induced set-up), the model determines by interpolation the number of grid points (x= 0 is offshore boundary, x= L is most landward computational grid point). The effective bed roughness is set to a fixed value of $k_s = 2d_{50}$.

In all runs the sediment transport is dominated by bed load transport processes. Low-frequency surf beat motion is taken into account based on a semi-empirical approach.

Figures 4 and 5 show simulation results of Deltaflume Test 1 and 2 after 260 minutes for shingle with d_{50}= 0.021 m based on the process-based CROSMOR-model and the parametric SHINGLE-model. Qualitatively the results of the CROSMOR-model are in reasonable agreement with the measured values. A swash bar of the right order of size is generated above the waterline in both experiments, but the computed swash bars are too smooth whereas the measured swash bars have a distinct triangular shape. The computed erosion zone is somewhat too large. The computed swash bar of Test 1 is much too small (only 0.05 m high) if the swash velocity is neglected (c_{sw}= 0). The computed run-up level has been varied in Test 2 to evaluate the effect on the computed bed profile. A relatively high run-up level results in a lower bar with a larger length. The swash bar produced by the parametric SHINGLE-model is too small for Test 1 and 2.

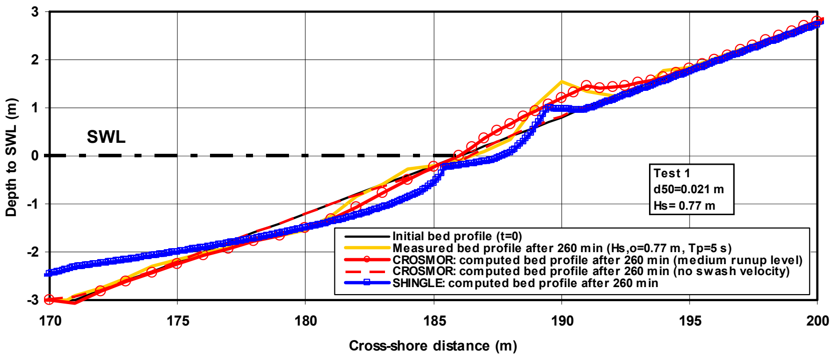

Figure 4 Simulation of Deltaflume Test 1 ($H_{s,o}$=0.77 m; d_{50}=0.021 m)

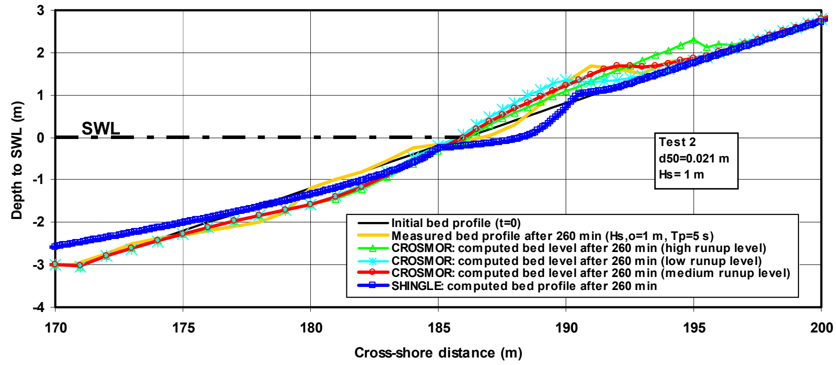

Figure 5 Simulation of Deltaflume Test 2 ($H_{s,o}$=1 m; d_{50}=0.021 m)

Figure 6 shows simulation results of Deltaflume Test 9 after 380 minutes for gravel with d_{50}= 0.0048 m. This test shows the presence of a relatively large swash bar further away from the water line and a relatively large erosion zone between the -3 m and -1 m depths. The simulation results of the CROSMOR-model also show a swash bar but at a much lower level on the gravel slope. The computed erosion zone is much too small. Since the swash bar area is of the right order of magnitude and the computed erosion area is much too small, the gravel is coming from the entrance section of the model, which is not correct. The swash bar produced by the SHINGLE-model is of the right order of magnitude, but the location of the computed swash bar is too low.

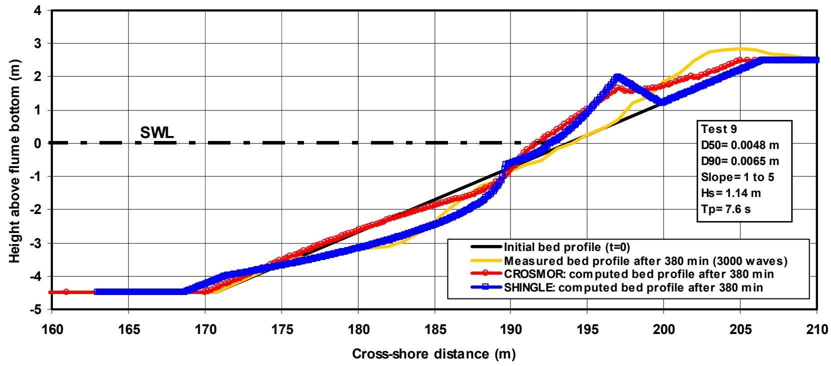

Figure 6 Simulation of Deltaflume Test 9 ($H_{s,o}$=1.14 m; d_{50}=0.0048 m)

Pevensey shingle barrier

To demonstrate the applicability of the model for prototype gravel barriers, the CROSMOR2008-model and the SHINGLE-model have been applied to the 9 km long shingle barrier at Pevensey Bay, East Sussex, UK. The tidal data are taken from Admiralty Tide Tables 2009 for Station Eastbourne, UK. 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, Van Rijn, 2009). Furthermore, the swash velocities near the water line and the streaming near the bed have been neglected (c_{sw}= 0, c_{LH} = 0).

Various storm cases are considered. Three cases (A,B,C) represent an event with a return interval of 1 to 400 years and one case (D) represent an extreme event with a return interval of 10000 years. These cases based on statistical analysis of joint data of maximum water levels and maximum wave heights, are given in Table 1. The offshore wave incidence angle is arbitrarily set to 30^{o} to include wave-driven longshore velocities.

Table 1 Storm wave cases

Case	Max. water level (m)	Storm setup (m)	Tidal range (m)	Offshore wave height Hs,o (m)	Offshore wave period T_m T_p (s)
A (1 to 400 years	3.5	1.0	5	6.0	8.7 11
B (1 to 400 years	4.0	1.5	5	5.0	8.0 10
C (1 to 400 years	4.5	2.0	5	3.0	7.0 8
D (1 to 10000 years)	4.5	2.0	5	5.5	8.3 10.5

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^o are shown in Figure 7. 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^o) 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 8 shows the barrier profile changes according to the CROSMOR-model for the 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 9 shows the bed profile changes based on the process-based CROSMOR-model and the parametric SHINGLE-model of HR Wallingford for Case A. The CROSMOR-model has been used with and without onshore-directed swash velocities near the water line. Runs without swash velocities produce the largest erosion values. The CROSMOR-model results without swash velocities and the SHINGLE-model results show rather good agreement for Case A (with the largest offshore wave height) with exception of the crest zone, where the SHINGLE-model predicts a relatively large build-up of the crest. The computed new crest level based on the SHINGLE-model is about 4.5 m above the HW level (about 2.5$H_{s,toe}$), which is rather large. The maximum crest level in the large-scale wave flume experiments was about 1.5 to 2$H_{s,toe}$ above the HW level. The erosion area between the mean sea level and the crest computed by both models is almost equal.

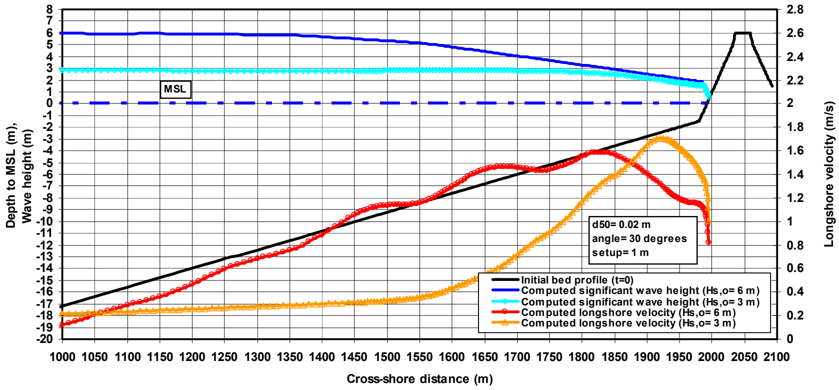

Figure 7 Bed profile, wave height, longshore velocity for offshore wave height of $H_{s,o}$= 3 and 6 m; setup= 1 m; offshore wave incidence angle= 30^o

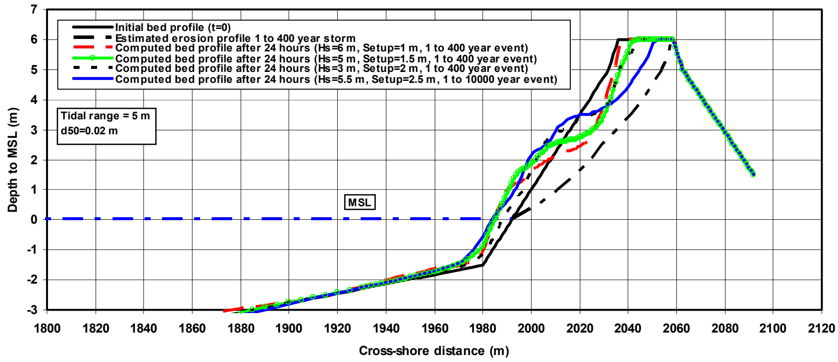

Figure 8 Computed bed level changes of Crosmor-model (Cases A to D)

The agreement between both models is less good for smaller wave heights (Cases B, C and D). The erosion in the upper zone computed by the SHINGLE-model for Case B and D is between that of both CROSMOR runs. The SHINGLE-model also predicts erosion at the toe of the barrier for Case B and D. The build-up of the crest predicted by the SHINGLE-model is quite large (about 4 m above the HW level) for Case D. The SHINGLE-model only predicts erosion in the lower beach zone for Case C. The eroded shingle is pushed up the barrier.

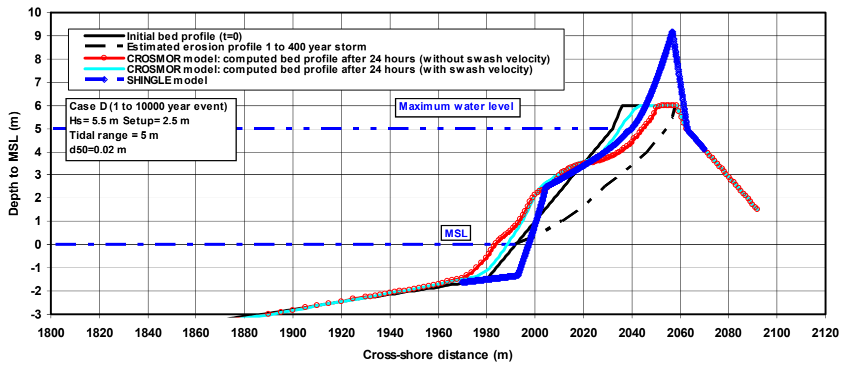

Figure 9 Computed bed profile changes of CROSMOR and SHINGLE (Case A)

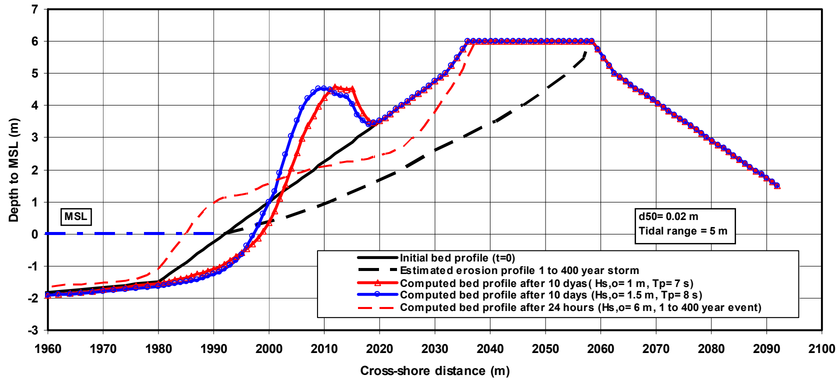

Figure 10 Accretion of shingle barrier during low wave conditions

Figure 10 shows the accretion of the shingle barrier after 10 days of low wave conditions ($H_{s,o}$ in the range of 1 to 1.5 m). The computed total accretion area at the upper beach is about 25 m^3/m (after 10 days) for shingle of 0.02 m (sef = 1, c_{LH} = 0.3, c_{SW} = 0.3). The shingle is pushed up the slope of the barrier by wave run-up processes which are somewhat stronger for higher waves. A similar pattern of bar formation has been observed at the Italian Carrara coast (Figure 2 Bottom-right). It will take some weeks with low waves for the shingle barrier to recover from the erosion (about 25 m^3/m) after a major storm event, assuming that sufficient shingle material is available in the foreshore zone. However, often the shingle material is carried away in longshore direction (passing around the short groynes, if present) during a major storm event. The shingle material may also be (partly) washed over the crest of the barrier during a major storm event.

Storm erosion of gravel barriers/beaches

The CROSMOR2008-model has been used to compute the erosion volume (in m^3/m) after 24 hours due to storm events for a range of conditions. It is assumed that a high storm surge level (SSL) corresponds to a high offshore wave height.

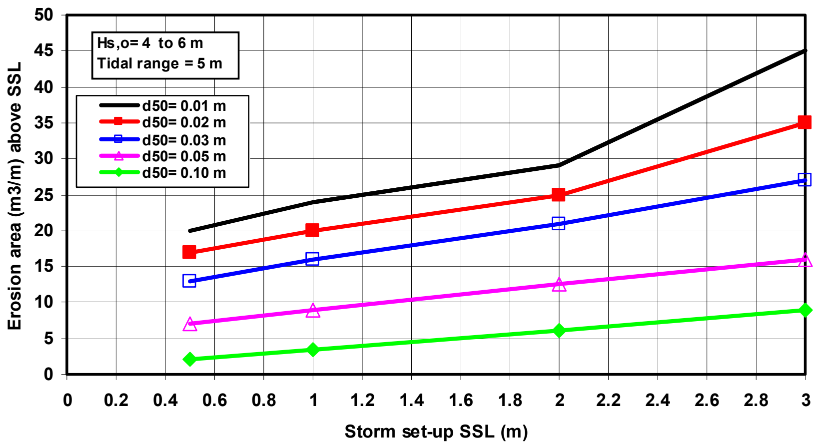

Figure 11 Erosion area (after 24 hours) as function of storm set-up and shingle/cobble size

Three storm events are considered: set-up = 0.5 m and $H_{s,o}$ = 4 m (T_p = 9 s); set-up = 1 m and $H_{s,o}$ = 4.5 m (T_p = 9.5 s), set-up= 2 m and $H_{s,o}$= 5 m (T_p = 10 s) and set-up = 3 m and $H_{s,o}$ = 6 m (T_p = 11 s). The incidence angle is 30^o to shore normal. Shingle size is between 0.01 to 0.1 m. Tidal range is 5 m. To obtain a conservative estimate of erosion, the undertow near the beach is increased by 50% and the sediment pick-up in the swash zone has been increased (sef = 2). The swash velocities and the streaming velocity near the bed have not been taken into account (c_{SW} = 0, c_{LH}= 0). The results are shown in Figure 11. The erosion area (in m^3/m) increases with increasing set-up and decreasing sediment size. The largest erosion area above the storm surge level is about 45 m^3/m for SSL = 3 m and $d_{50} = 0.01$ m. The smallest erosion area (about 5 to 10 m^3/m) occurs for a cobble barrier.

2 Conclusion

Two models (process-based CROSMOR2008-model and parametric SHINGLE-model) have been used to simulate gravel barrier erosion under high wave conditions (storm events). Test results of the Deltaflume and GWK experiments have been used to calibrate the CROSMOR-model for gravel and shingle slopes. Qualitatively the results are in reasonable agreement with the measured values. A swash bar of the right order of magnitude is generated above the waterline in both experiments, but the computed swash bars are too smooth whereas the measured swash bars have a distinct triangular shape and are positioned at a higher level on the slope. Simular results are obtained for the other large-scale laboratory tests.

To demonstrate the applicability of the process-based CROSMOR-model for prototype shingle barriers, the model has been applied to a real field case (Pevensey Bay, UK) and a schematized field case. The SHINGLE model of HRWallingford has also been applied to the field case of Pevensey Bay. Various storm cases are considered representing events with a returrn interval of 1 to 400 years and an extreme event with a return interval of 10000 years. The CROSMOR-model results and the SHINGLE-model results show rather good agreement of computed erosion values for the storm case with the largest offshore wave height of 6 m.

Erosion of sandy dune coasts due to storm events is a major problem at many sites. Under extreme storm conditions the erosion volume due to a severe storm with a duration of 5 to 6 hours is of the order of 100 to 300 m^3/m and shoreline recession values are of the order of 10 to 30 m. These values can be significantly reduced by using a protection layer of shingle or cobbles on the sandy dune face. The maximum horizontal recession for a dune protected by a layer of shingle is of the order of 2 m. When cobbles (of about 0.1 m) are used, the erosion will be minimum. Using a safety factor of 2, the minimum layer thickness of shingle to protect a sandy subsoil should be of the order of 2 to 3 m.

3 Acknowledgements

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