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Modelling erosion of gravel/shingle beaches and barriers

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CONTENTS

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* and can be found in many mid- and high-latitude parts (formerly glaciated) of the world (England, Iceland, Canada, etc.). These areas extend from beyond the limits of the last major ice zones to the present day ice caps and include much of the coasts of northwestern Europe, eastern North America and the far north Pacific coast. Generally, these coastlines are intricate and irregular, characterised by headland and cliff formations. The variety of depositional forms include: gravel/shingle barriers and beaches, barrier spits, bay-head barriers and transverse lag shoals. 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 (migrating landwards through rollover by overwashing) or longshore drift-aligned barriers are distinguished. Typical profiles are shown in **Figure 1.1**.

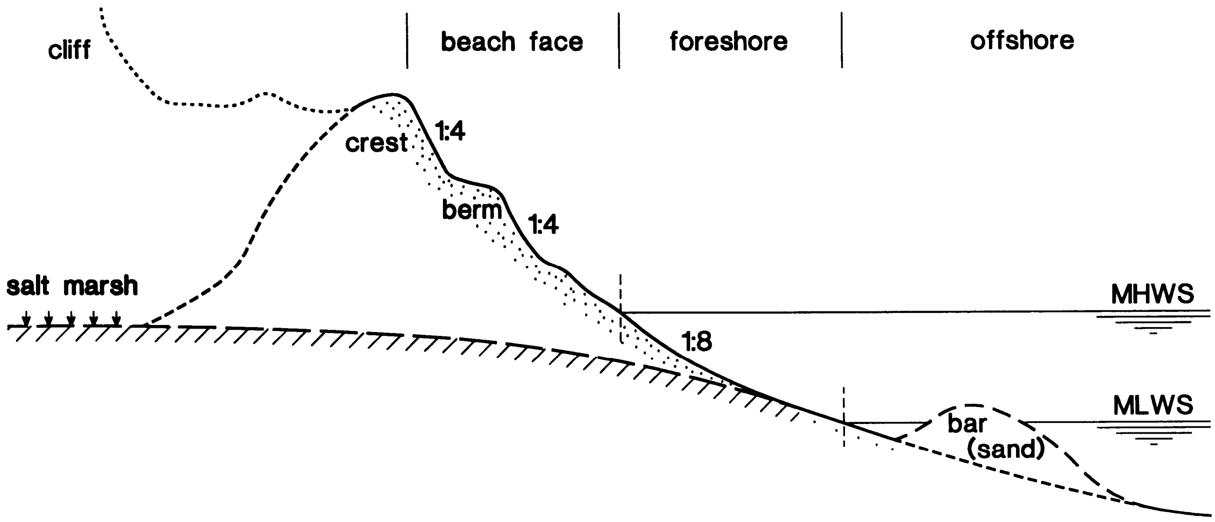

Figure 1.1 *Typical cross-shore profile of gravel beach slope*

Gravel beaches are also found along unconsolidated cliff-type coasts eroded by wave attack (like Mediterranean coasts) and along tectonic coasts where steep streams deliver coarse material to the shore. 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 **1.1right**). 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.

Gravel beaches are also known as shingle beaches or coarse clastic beaches. Clasts are individual grains within coarse populations. Subgroups are pebbles and cobbles (rounded clasts between 64 and 256 mm); boulders are clasts larger than 256 mm. The term shingle is most commonly identified with the coarse beaches of southern England.

Gravel, pebbles and cobbles may consist of quartzite or flint and chert (formed by silica-bearing organisms). The quartzites tend to be more discoid, whereas the flint and chert (splintering more easily) are more flaked or fractured through impact. Pebbles of flint and chert become more ellipsoid during the transport process. Generally, the wearing of gravel is an extremely slow process. Impacts between pebbles during transport result in fractures and in removal of small surface irregularities (attrition) by grinding processes. The beach acts as a grinding machine, slowly pulvarising the coarser clasts; the finest pulvarisation products are carried offshore resulting in a loss in terms of sediment budget. A review of coarse clastic beaches is given by **Carter and Orford (1993).**

The transport pathways of gravel/shingle have been studied extensively by tracer experiments. Most studies have been qualitative rather than quantitative because of low recovery rates (5% or even less). A wide range of tracers has been used: original beach material coated with dye or paint, labelled with radio-active isotopes, pebbles of a distinctive geological composition from other sites, artificial pebbles (aluminium, plastic filled with metal, etc.). These studies give information of the movement of pebbles under wave and current-induced forces.

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 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. To prevent the longitudinal spreading of coarse materials often small-scale timber groynes are used (see **Figure 1.2**).

Figure 1.2 *Timber groynes at beach of Eastbourne, East Sussex, UK*

Generally, the upper beach consists of gravel/shingle material, while the lower beach consists of sandy material, see **Figure 1.2 (right)**. 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. The finer grains may go into suspension as a result of the turbulence produced by the breaking waves and may be transported to the lower parts of the beach zone depending on the strength of the undertow.

2 Swash zone processes

Gravel/shingle transport mainly takes place in the swash zone. The swash zone is the zone which is intermittently wet and dry showing relatively large velocities during the uprush and backwash phases of the saw-tooth swash wave cycle due to bore propagation and bore collapse, often in combination with low-frequency oscillations which generally grow in amplitude towards the shoreline. It is a particularly complex zone of the nearshore where short and long waves, tides, sediments and groundwater flow (infiltration/percolation) all play an important role. Long waves are generated by the release of bound long waves in the surf zone due to the breaking of short waves and by cross-shore variations of the short wave breakpoints (surf beat). The role of percolation is especially important on steep, coarse-grained beaches leading to beach accumulation and steepening as a result of the diminished sediment carrying capacity of the reduced backwash volume of water and velocity, following percolation into the coarse-grained bed. These effects will lead to a landward bias (asymmetry) in swash transport depending on grain size. The swash zone is the most dynamic part of the nearshore zone of vital importance for the behaviour of gravel/shingle barriers.

Most field studies have been carried out on steep, sandy beaches with low waves (typical uprush and backwash durations of 3 to 7 s.). A key finding of these studies is that the uprush moves more sediment than the backwash under low wave conditions (offshore wave height of 1 to 2 m). Suspended sand concentrations and transport in the swash zone are an order of magnitude larger than those in the inner surf zone (concentrations up to 100 kg/m^3), (**Masselink et al., 2005**).

Reviews are given by: **Elfrink and Baldock (2002)** and **Butt and Russell (2000)**.

2.1 Run-up

When waves approach a coast, the majority of the wave energy is dissipated across the surf zone by wave breaking. However, a portion of that energy is converted into potential energy in the form of run-up on the foreshore of the beach (swash zone). Usually, the vertical wave run-up height above the still water level (SWL) is defined as the run-up level which is exceeded by 2% of the incident waves ($R_{2\%}$).

Run-up is caused by two different processes (see **Figure 2.1**):

- maximum set-up (h'), which is the maximum time-averaged water level elevation at the shoreline with respect to mean water level;
- swash oscillations (s_t), which are the time-varying vertical fluctuations about the temporal mean value (set-up water level); the run-up is approximately equal to $R= h' +0.5H_{swash}$ with $H_{swash}=2s_{max}$=swash height.

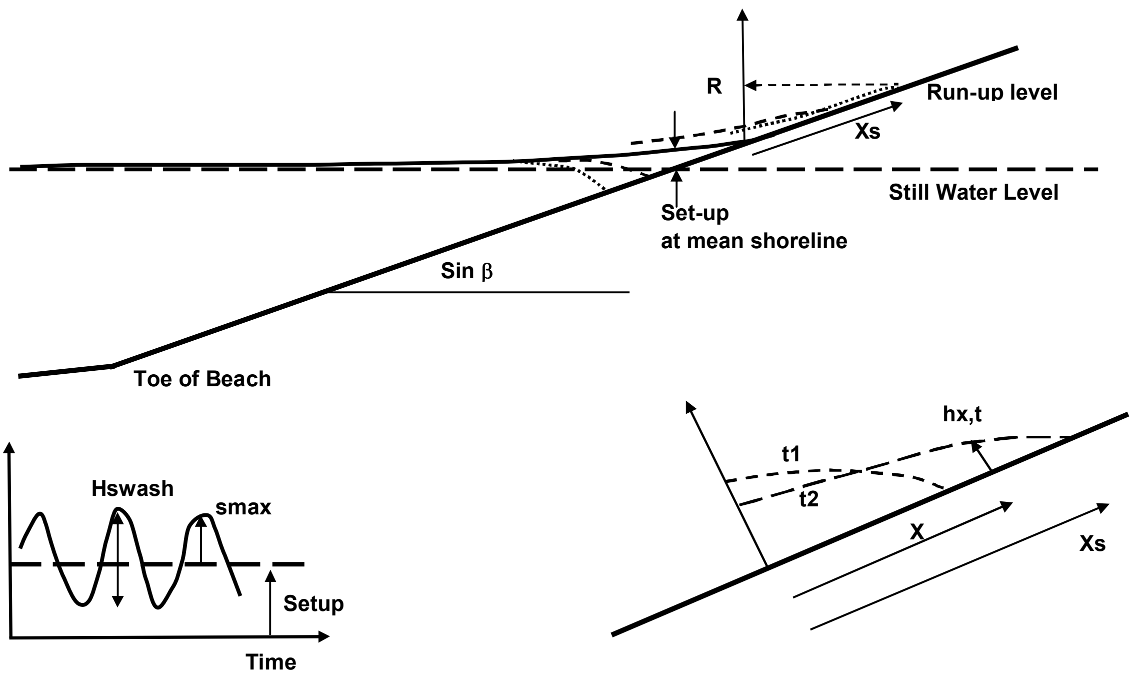

Figure 2.1 *Swash processes along beach*

Laboratory measurements with monochromatic waves on a plane beach have shown that the vertical swash height R increases with growing incident wave height until R reaches a threshold value. Any additional input of the incident wave energy is then dissipated by wave breaking in the surf zone and does not result in further growth of the vertical swash and run-up, i.e the swash is saturated, see **Figure 2.2**.

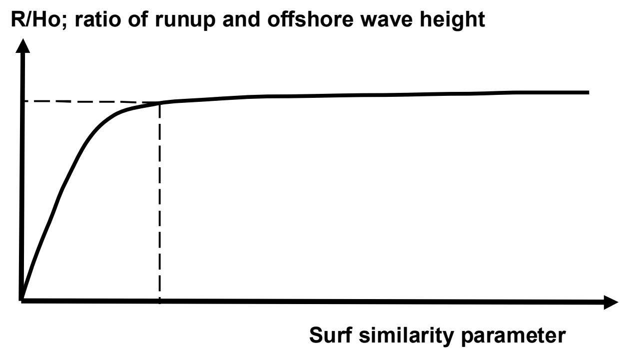

Figure 2.2 *Ratio of run-up and offshore wave height as function of surf similarity parameter*

Usually, the run-up height up to the threshold value is represented as:

$$R = \frac{\tan\beta}{(H_o/L_o)^{0.5}} \gamma H_o = \gamma \tan\beta \, [H_o L_o]^{0.5} \qquad (2.1)$$

$$R = \zeta_o \gamma H_o \qquad (2.2)$$

in which: R = run-up height measured vertically from the still water level (including wave set-up height) to the run-up point, H_o = wave height seaward of the swash zone, L_o= wave length in deep water and $\tan\beta$ =bottom slope, γ=proportionality coefficient.

The parameter $\zeta_o = \tan\beta(H_o/L_o)^{-0.5}$ is known as the surf similarity parameter. Low ζ_o-values (<0.3) typically indicate dissipative conditions (high breaking waves on flat beaches), while higher values (>1) indicate more reflective beaches (breaking waves on steep beaches). On dissipative beaches, infragravity energy (with periods between 20 and 200 s) tends to dominate the inner surf zone, especially the swash zone.

The run-up process can be modelled by considering the collapse of a bore at the shoreline involving the rapid conversion of potential energy to kinetic energy. Excluding infiltration and percolation, the momentum equation for a water column at the head of the swash lens can be expressed as (see **Figure 2.1**):

$$\rho(\partial h\bar{U}/\partial t) + \rho g\,(h)(\partial h/\partial x) + \tau_{bed} - \rho g\, h \sin\beta = 0 \qquad (2.3)$$

or

$$\partial\bar{U}/\partial t + g(\partial h/\partial x) + \tau_{bed}/(\rho h) - g \sin\beta = 0 \qquad (2.4)$$

with: x= coordinate along beach slope, h=local mean water depth, τ_{bed}=bed-shear stress, $\bar{U}$= depth-averaged velocity (long wave velocity) in swash lens. **Kobayashi and Wurjanto (1992)** have used a numerical long wave model to simulate the impact of bores on the sloping beach

A simplified approach can be obtained by assuming that the pressure gradient term and the bed-friction term are approximately equal (but acting in opposite direction: $g(\partial h/\partial x) \cong -\tau_{bed}/(\rho h)$). The net effect of both terms can be represented by a coefficient (α_1), as follows:

$$\partial\bar{U}/\partial t - g\,(1+\alpha_1) \sin\beta = 0 \qquad (2.5)$$

with: α_1=coefficient ($\alpha_1<0$ if pressure gradient is dominant; $\alpha_1>0$ if bed-shear stress term is dominant). This approach only describes the uprush and backwash above the mean water level (set-up level) due the breaking of the crest of the wave.

Equation (2.5) can be integrated, yielding:

$$\bar{U} = -[g\,(1+\alpha_1) \sin\beta]\, t + C \qquad (2.6)$$

Using: $\bar{U} = \bar{U}_o$ at t=0, it follows that: C= $\bar{U}_o$ and thus:

$$\bar{U} = dx_s/dt = \bar{U}_o - [g\,(1+\alpha_1) \sin\beta]\, t \qquad (2.7)$$

At the most landward end of the swash motion it follows that: $\bar{U} = 0$ at $t=T_{end}$, yielding:

$$T_{end} = \bar{U}_o/[g\,(1+\alpha_1) \sin\beta] \qquad (2.8)$$

with: $\bar{U}_o \cong (gh)^{0.5}$.

The swash period from the start of the uprush to the end of the backwash is:

$$T_{swash} = 2T_{end} = 2\,\bar{U}_o/[g\,(1+\alpha_1)\,\sin\beta] \tag{2.9}$$

Using $h \cong 0.5$ to 1 m, $\sin\beta \cong 0.05$ and $\alpha_1 = 0$, it follows that: $T_{end} = 4$ to 6 seconds and $T_{swash} = 8$ to 12 s.

Using: $H_b = \alpha_2 h$ = height of incoming bore and $\bar{U}_o = (gh)^{0.5} = [(1/\alpha_2)gH_b)]^{0.5}$, it follows that:

$$T_{swash} = 2[(1/\alpha_2)gH_b]^{0.5}/[g\,(1+\alpha_1)\,\sin\beta] \tag{2.10}$$

The swash period increases with increasing wave height (H_b) and decreasing slope ($\sin\beta$). The swash can behave as an individual oscillating motion along the beach as long as the period of the incident bores (T) is larger or equal to that of the swash (T_{swash}). The onset of interference occurs for $T = T_{swash}$, yielding:

$$H_{b,onset} = 0.25\,\alpha_2\,(1+\alpha_1)^2\,g\,(\sin\beta)^2\,T^2 \tag{2.11}$$

When the wave period (T) of the incident waves is smaller than that of the swash motion (T_{swash}), the new uprush interfers (overlaps) with the backwash of the previous wave resulting in two effects: (i) increased friction acting on the uprush of the new wave due to backwash of the previous one and (ii) more shoreward breaking of the new bore over the backwash of the previous one (larger water depths). The run-up level will be larger if the latter effect dominates and smaller if the former effect dominates. In the case of overlapping conditions the thickness (h) of the swash lens may grow somewhat due to increased friction and the velocity structure over the depth will show a seaward flow near the bottom due to the backwash of the previous wave and a landward flow near the surface due the uprush of the new incident wave.

Equation (2.7) can be integrated once again to find the swash excursion (x_s), yielding ($x_s = 0$ at $t = 0$):

$$x_s = [\bar{U}_o]\,t - [0.5\,g\,(1+\alpha_1)\,\sin\beta]\,t^2 \tag{2.12}$$

The vertical swash oscillation from the start of the uprusch to the end of the backwash is:

$$s = x_s \sin\beta = [\bar{U}_o \sin\beta]\,t - [0.5\,g\,(1+\alpha_1)\,\sin^2\beta]\,t^2 \tag{2.13}$$

which represents a parabolic swash oscillation in time.

Equation (2.13) can also be expressed as (see **Figure 2.3**):

$$s/s_{max} = 2t/T_{end} - t^2/(T_{end})^2 \tag{2.14}$$

with: s_{max}= maximum swash amplitude based on Equation (2.16).
The maximum value is (at $t=T_{end}$):

$$x_{s,max} = [\bar{U}_o]^2/[2\,g\,(1+\alpha_1)\,\sin\beta] \tag{2.15}$$

$$s_{max} = [\bar{U}_o]^2/[2\,g\,(1+\alpha_1)] \qquad (2.16)$$

The initial velocity at t= 0 can be approximated by the bore velocity, as follows:

$$\bar{U}_o = (gh)^{0.5} = [(1/\alpha_2)gH_b]^{0.5} \qquad (2.17)$$

with: $H_b=\alpha_2 h$= height of breaking bore at toe of beach, α_2= coefficient in range of 0.25 to 0.5.

This yields:

$$s_{max} = H_b/[2\,\alpha_2\,(1+\alpha_1)] \qquad (2.18)$$

Equation (2.18) shows that the maximum swash level of individual waves with T > T_{swash} is independent of slope and equal to 1 to $2H_b$ for α_1= 0 and α_2= 0.25 to 0.5 for a smooth, impermeable beach slope. The swash level will be lower for a permeable, gravel/shingle beach slope due to infiltration processes.

Using Equation (2.11), the s_{max}-value at the onset of wave-swash interference can also be expressed as:

$$s_{max} = 0.125\,(1+\alpha_1)\,g\,T^2\,(\sin\beta)^2 \qquad (2.19)$$

and is strongly related to the incident wave period and beach slope. The parameter s_{max} increases with increasing wave period and increasing slope.

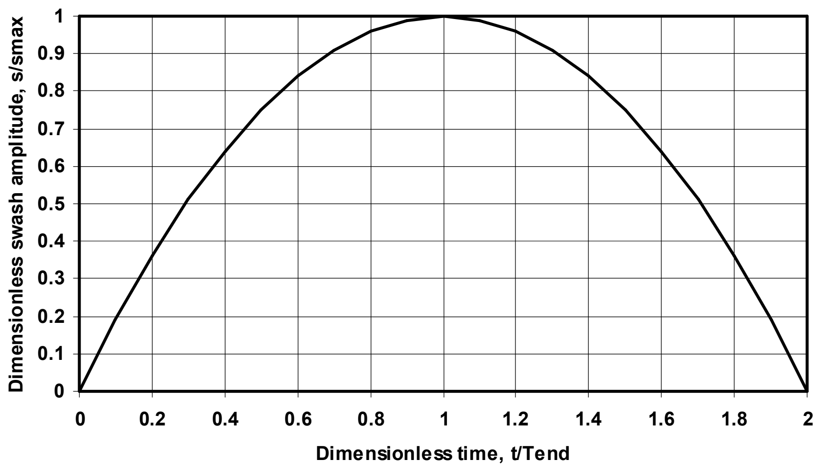

Figure 2.3 *Dimensionless swash amplitude as function of dimensionless time*

Baldock and Holmes (1999) have performed flume experiments with regular and random wave trains approaching a plane slope of 1 to 10. The bore height was measured at the intersection of the still water level and the beach slope (x= 0). The surf similarity parameter (ζ_o) based on deep water parameters is in the range of 0.4 to 1. The maximum run-up height above SWL of regular wave trains (no overlap between bores and swash) is found to be in excellent agreement with the theoretical value of Equations (2.18) and (2.19) for α_2= 0.25 and α_1= 0, suggesting that frictional effects over the beach slope are minimal. The bore height and the arrival time of the bore at x= 0 m are taken as the input values. The run-down below SWL is found to be negligble. The measured shoreline position can also be simulated quite well by using Equation (2.12). The measured shoreline velocity is very well simulated by Equation (2.7). Runs with saturated swash motion ($T < T_{swash}$) under regular wave trains (overlap between sequential incoming bores) show that the swash amplitude (variation) is reduced considerably. The maximum run-up distance and height are not so much affected. The simplified model of Equations (2.18) and (2.19) also yields reasonable results for random wave trains both with non-overlapping and overlapping conditions.

It is concluded that the shoreline motion of both non-overlapping and overlapping conditions for regular and random waves is largely driven by individual incident bores and does not exhibit a cumulative increase in additional harmonics due to swash-swash interaction. No accumulation of long wave energy has been observed.

If the surf zone is totally saturated with breaking waves, wave grouping is totally destroyed and then only free low-frequency waves will affect the swash motion.

Various field studies have shown the important contributions of the incident wave periods (T < 20 s) and the infragravity wave periods (T > 20 s) to the run-up height above SWL.

Stockdon et al. (2006) have analysed wave run-up data sets (with $0.1<\zeta_o<2.5$) of ten different field experiments (west and east coasts of USA, Terschelling coast of The Netherlands) based on video techniques. The swash amplitudes ($s_{max}= 0.5H_{swash}$) related to the incident wave band were in the range of 0.1 to 1.5 m and the swash heights related to the infragravity band were in the range of 0.2 to 1 m.
Their definitions are:

$$R_{2\%} = 1.1[h' + s_{max}] \quad (2.20)$$

h' = set-up value

$$s_{max} = 0.5[(H_{swash,hf})^2 + (H_{swash,ig})^2]^{0.5}$$

with: $R_{2\%}$= run-up height exceeded by 2% of the run-up values, $H_{swash,hf}$= vertical swash oscillation height of the high frequency incident waves and $H_{swash,ig}$= vertical swash oscillation height of the low frequency infragravity waves, s_{max}= maximum swash amplitude. Run-up statistics were defined as the measured maximum elevations of individual water level values above SWL, see **Figure 2.4.** The mean value of all individual values is defined to be equal to the mean set-up (h'). After subtraction of the set-up, the swash statistics can be computed. The swash oscillation height was computed from the spectrum as $H= 4(M_o)^{0.5}$, which represents the significant value similar to the calculation of significant wave height.

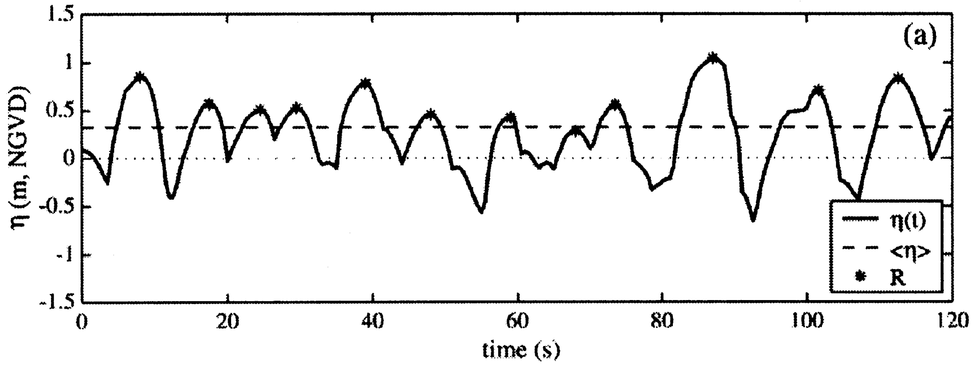

Figure 2.4 *Time series of measured water levels; <η>=set-up value (mean values of all water levels), R= run-up values (maxima of water levels); Stockdon et al. (2006)*

According to **Stockdon et al. (2006)**, the set-up values can be best parameterized by:

$$h' = 0.35\,(\sin\beta_f)\,[H_o\,L_o]^{0.5} \quad \text{for } \zeta_o > 0.3 \qquad (2.21)$$
$$h' = 0.016\,[H_o\,L_o]^{0.5} \quad \text{for } \zeta_o < 0.3 \text{ (dissipative beaches)} \qquad (2.22)$$

with: $\zeta_o = (\tan\beta_f)/(H_0/L_0)^{0.5}$, $\tan\beta_f$ = slope of foreshore (beach), H_o= significant wave height at deep water, L_o= significant wave length at deep water. Most set-up values are in the range of 0 to 1 m.

The inclusion of beach slope under dissipative conditions was found to result in lower correlation.

According to **Stockdon et al. (2006)**, the swash oscillation heights can be best parameterized by:

$$H_{swash,\,hf} = 0.75\,(\sin\beta_f)\,[H_o\,L_o]^{0.5} \quad \text{for all } \zeta_0 \qquad (2.23)$$
$$H_{swash,\,ig} = 0.06\,[H_o\,L_o]^{0.5} \quad \text{for } \zeta_0 < 1.25 \text{ (dissipative beaches)} \qquad (2.24)$$
$$H_{swash,\,ig} \cong 0 \quad \text{for } \zeta_0 > 1.25 \text{ (reflective beaches)} \qquad (2.25)$$

Most swash height values are in the range of 0 to 2 m. The largest value of the parameter $(\sin\beta_f)[H_o\,L_o]^{0.5}$ of their data set is about 30.

On dissipative beaches ($\zeta_0 < 0.3$; beach slope larger than 1 to 20 or $\sin\beta_f < 0.05$) the swash is dominated by the infragravity band for 90% of the data. Beach slope has not much influence on the infragravity–induced swash heights at dissipative beaches. The data do not support the inclusion of beach slope for dissipative beaches. On dissipative beaches, the magnitude of the infragravity-induced swash height grows with increasing H_o.

On reflective beaches (ζ_0>1.25) the swash is dominated by the incident band for 90% of the data. The application of H_{br} in stead of H_o was not found to improve the results. The application of the surf zone slope β_s in stead of the foreshore slope (beach) β_f was not found to improve the results.

Using the data of the dissipative beaches only (**Stockdon et al., 2006**), the run-up values above SWL can be computed by:

$$R_{2\%} = 0.043\ [H_o\, L_o]^{0.5} \qquad \text{for } \zeta_0 < 0.3 \text{ (dissipative beaches)} \qquad (2.26)$$

Using the data of the reflective beaches only (**Stockdon et al., 2006**), the run-up values above SWL can be computed by:

$$R_{2\%} = 0.75\ (\sin\beta_f)[H_o\, L_o]^{0.5} \qquad \text{for } \zeta_0 > 1.25 \text{ (reflective beaches)} \qquad (2.27)$$

Since run-up and swash are dependent on beach slope, beaches with longshore variation of beach slope will show considerable variation of wave run-up in alongshore direction. The data of the Duck site on the east coast of the USA showed that on days when the beach slope was longshore variable, the wave run-up of the incident band was also variable (up to 40% for highly three-dimensional conditions due to the presence of mega-cusps).

Van Gent (2001) has presented run-up data for steep slope structures such as dikes with shallow foreshores based on local parameters rather than on deep water parameters. Various types of foreshores were tested in a wave basin: foreshore of 1 to 100 with a dike slope of 1 to 4; foreshore of 1 to 100 with a dike slope of 1 to 2.5 and foreshore of 1 to 250 with a dike slope of 1 to 2.5. The test programme consisted of tests with single and double-peaked wave energy spectra, represented by a train of approximately 1,000 waves. The water level was varied to have different water depth values at the toe of the dike.

The experimental results for steep slope structures can be represented by (see also **Figure 2.5**):

$$R_{2\%}/H_{s,toe} = 2.3(\zeta)^{0.3} \qquad \text{for } 1 < \zeta < 30 \qquad (2.28)$$

with: $\zeta = \tan\beta/[(2\pi/g)H_{s,toe}/T^2_{m-1}]^{0.5}$ = surf similarity parameter based on the T_{m-1} wave period, $H_{s,toe}$=significant wave height at toe of the structure, T_{m-1} = wave period based on zero-th and first negative spectral moment of the incident waves at the toe of the structure (=0.7 to 1 T_p), T_p = wave period of peak of spectrum, β= slope angle of structure.

The run-up level $R_{2\%}$ varies roughly from $1H_{s,toe}$ to $5H_{s,toe}$ depending on the value of the surf similarity parameter. The influence of the wave energy spectrum can be accounted for by using the spectral wave period T_{m-1} of the incident waves at the toe of the structure.

Assuming: $H_{s,toe} = 0.25$ to $0.5\ H_{s,o}$, it follows that: $R_{2\%}/H_{s,o} \cong 0.7$ to $1.6(\zeta_o)^{0.3}$.

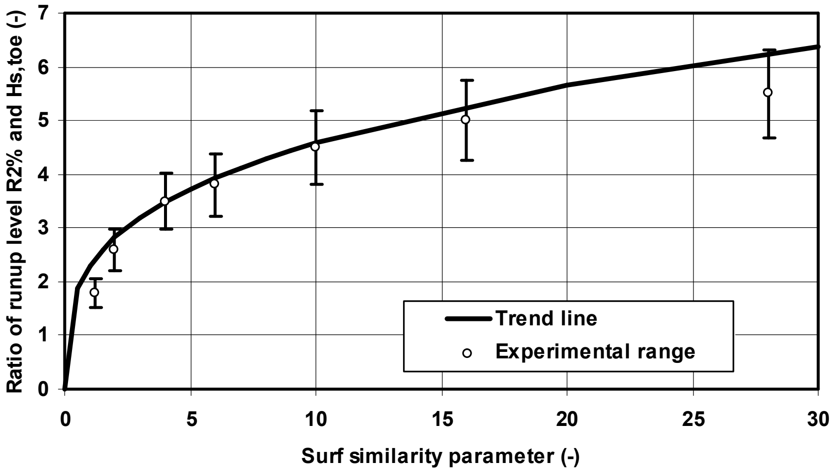

Figure 2.5 *Run-up level data as function of surf similarity parameter based on Van Gent (2001)*

During storm conditions with a significant offshore wave height of about 6 m (peak period of 11 s), the significant wave height at the toe of the gravel barrier is about 2 m (see **Figure 4.8**) resulting in a ζ_o-value of 2 to 3 and thus $R/H_{s,toe} \cong 2.5$ to 3 and $R \cong 5$ to 6 m above the mean water level. In practice, the run-up values will be significantly smaller along a coarse gravel/shingle barrier due to infiltration processes. In the large-scale wave flume tests (see **Section 3.1.2**) the maximum crest level of the swash bar generated by the swash run-up was about 1.5 to 2 m above the mean water level (about 2 $H_{s,toe}$).

2.2 Swash velocities and shear stresses

The transport of coarse sediments is most active in the swash zone of the beach face and is caused by wave uprush (decelerating flow) and backwash (accelerating flow). Both laboratory and field measurements over an impermeable bottom have shown that the swash of the incident waves on a steep beach is skewed and asymmetric (saw-tooth waves), i.e. the backwash is not simply the reverse of the uprush. Generally, onshore flow velocities during the uprush are larger but of shorter duration than the seaward velocities during the backwash. Maximum landward velocities occur at the start of the uprush, whereas maximum seaward velocities take place at the end of the backwash. The water depths that occur during the uprush are generally larger than those that occur during the backwash. These observed features are consistent with computational results of non-linear shallow water theory for swash behaviour following bore propagation and collapse over an impermeable bed (**Butt and Russell, 2000; Elfrink and Baldock, 2002**).

The dissimilarity in the hydrodynamics of the wave uprush and backwash is reflected in different modes of sediment transport. Turbulence-dominated suspended transport may be significant during the uprush phase whereas sheetflow type of bed load transport dominates during the backwash phase. During the uprush phase the sediment transport is a combination of sediments mobilised under and directly after bore collapse which are then advected landwards and of locally entrained sediments from the bed by developing boundary layer flow at the end op the uprush, whereas sediment transport during downrush mainly is related to locally entrained sediments. Measurements of sheet flow transport for half saw-tooth waves in a wave tunnel (**King, 1991**) indicate that the sediment transport under steep fronts (decelerating flow) is about twice as large as under steep rears (accelerating flow). **Nielsen (1992)** computed shear stresses under a saw-tooth wave and found that the landward peak shear stress was about twice as large as the seaward peak shear stress.

Swash motion over a steep permeable bed of coarse grains (gravel/shingle) is complicated by the presence of infiltration under wave uprush and exfiltration under wave downrush. Vertical flow through a porous bed can influence sediment motion in two ways: 1) seepage forces changing the effective weight of the surficial sediments and 2) the occurrence of boundary layer thinning (resulting in higher shear stresses) due to infiltration and thickening (smaller shear stresses) due to exfiltration. Generally, swash-related infiltration-exfiltration effects across a saturated beach face enhances the upslope transport of sediment transport (**Masselink and Hughes, 1998**) and reduces the downslope transport.

Research on swash velocities and shear stress in laboratory flumes and in the field have been done by **Cox et al. (2000), Cowen et al. (2003), Conley and Griffin (2004), Masselink et al. (2005), Pritchard and Hogg (2005), Masselink and Russell (2006)** and by **Barnes et al., (2009)**. Hereafter, the main research results are briefly summarized.

Cox et al. (2000) have performed swash zone measurements using a two-dimensional (horizontal and vertical) Laser Doppler velocitymeter in a laboratory flume with an impermeable beach slope of 1 to 10 covered by gravel particles of 6.2 mm under irregular waves with a peak period of 4 s. Bed shear stresses were determined from the logarithmic profile of the ensemble averaged velocity in the bottom boundary layer at many phases for each wave. Results of measurements in the inner surf zone (below the still water line) and in the swash zone (above the still water line) show a very asymmetric shear stress distribution (as function of time) in the surf zone and a typical saw-tooth distribution of shear stress in the swash zone. At both locations the ratio of the landward and seaward peak bed-shear stress is about 3, which is somewhat larger than the values shown in **Figure 2.6** based on the measurements **Conley and Griffin (2004)**. The landward peak bed-shear stress in the swash zone is much larger (factor 4) than the landward peak bed-shear stress in the surf zone. The peak bed-shear stress in the surf zone occurs at 0.1T and 0.85T, and at 0.05 T and 0.9T in the swash zone.

Cowen et al. (2003) have used a particle image velocimetry technique (PIV) to determine the vertically resolved two-dimensional measurements of the swash velocities and turbulence parameters in a laboratory wave flume with spilling and plunging waves. Their measurements indicate that the turbulent structure of the swash zone due to spilling waves is similar to that of plunging waves suggesting that the swash zone is driven by the turbulent bore that results from the wave breaking process and not by the mode of breaking. The uprush and downrush phases are not symmetric and are dominated by different turbulence processes. The uprush and early retreat phases are dominated by bore-advected and bore-generated turbulence that is considerably stronger than fully developed boundary layer turbulence. The temporal evolution of the post-bore uprush phases is analogous to decaying grid turbulence. During the last half of the retreat phase (downrush phase), the bore turbulence has decayed sufficiently and the wall boundary layer has grown sufficiently that boundary-generated turbulence becomes the dominant source of turbulent kinetic energy. The bed shear stress can also be estimated from the PIV-data and is found to has a strong phase dependence. The bed-shear stresses of the uprush phase are much larger than that of the downrush phase. Defining $f_{w,up}=\tau_{max,up}/(0.5\rho U_{max.up}^2)$ and $f_{w,down}=\tau_{max,down}/(0.5\rho U_{max.down}^2)$ with τ_{max}= maximum bed-shear stress and U_{max}= maximum fluid velocity, the friction factor of the uprush phase is about twice as large as that of the downrush phase in the swash zone.

Conley and Griffin (2004) have made direct measurements of bed stress under swash in the field (medium grained Barret Beach, Fire Island, New York) utilizing flush mounted hot film anemometry. Hot film sensors are thermal sensors that are maintained at a constant temperature which is higher than the ambient temperature. The energy required to maintain the temperature is related to the fluid velocity and fluid shear stress (calibration curve). In addition to the hot film package, a pressure sensor was deployed in water with a mean depth of 1.5 m approximately 10 m from the shoreline to provide information of the wave and tide conditions. A video camera was used to provide constant coverage of the hot film sensor with respect to the nearby beach face and water surface. **Figure 2.6** shows the measured dimensionless shear stress distibrution (skewed and asymmetric) based on about 100 discrete uprush events under calm wave conditions (significant wave height of 0.14 m at location of pressure sensor). This illustrates that the maximum stress exerted by the backwash flow is typically less than half of that exerted by the uprush. The duration of the backwash is, however, about 30% larger than that of the uprush. Assuming a quadratic relationship between shear stress and fluid velocity, the friction factor of the uprush phase is found to be considerably larger than that during the backwash phase in line with the findings of **Cowen et al. (2003)**.

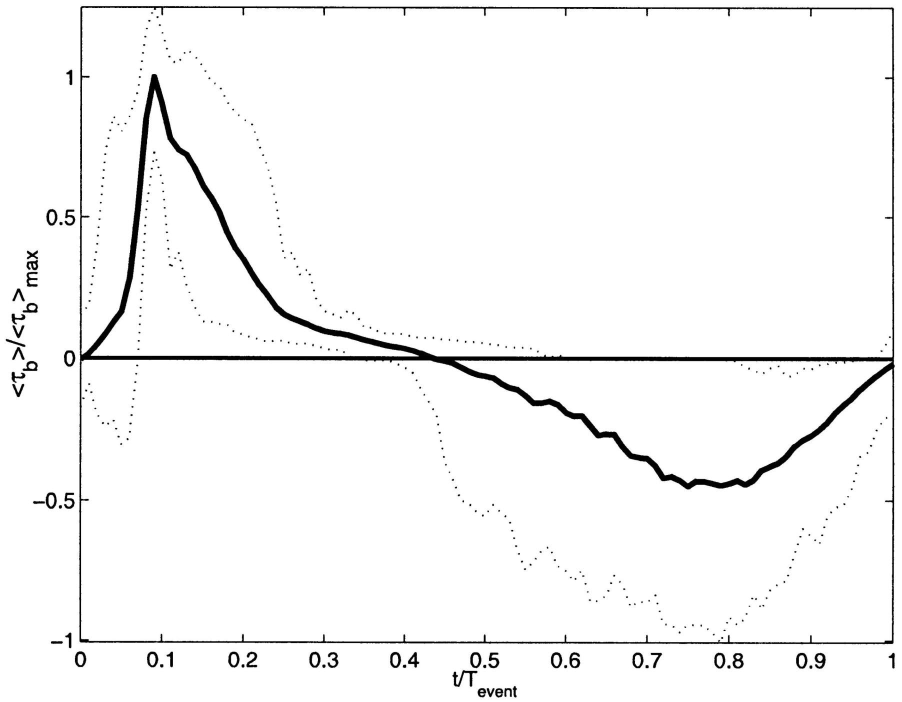

Figure 2.6 *Dimensionless bed shear stres distribution as function of time in swash zone; ensemble average of 100 individual swash events; Barret Beach, Fire Island, USA (Conley and Griffin, 2004)*

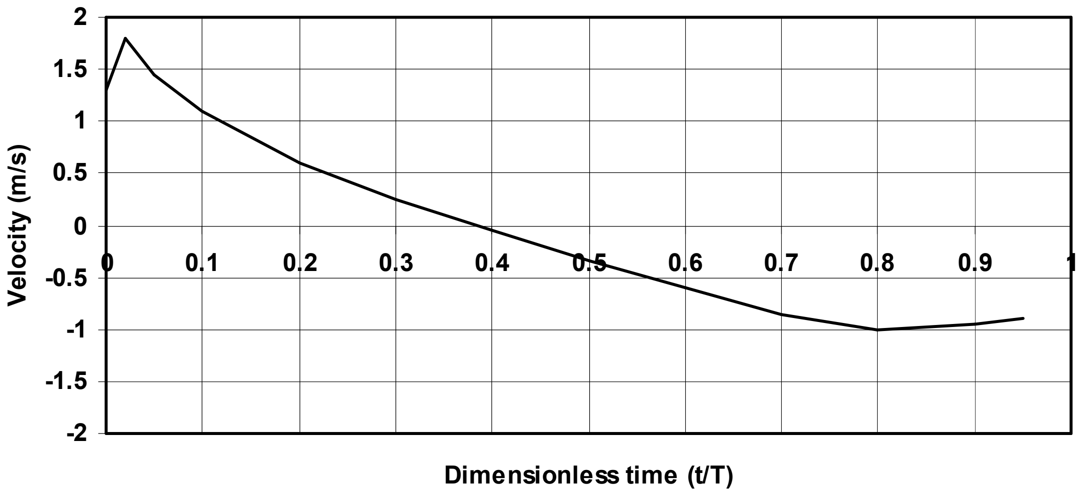

Figure 2.7 *Typical swash velocities as function of time above 0.28 mm sand bed; Perranporth Beach, Cornwall, UK (Masselink et al., 2005)*

Masselink et al. (2005) and **Masselink and Russell (2006)** have performed swash measurements in the high-tide swash zone of two macro-tidal beaches (fine sand 0.28 mm at mild sloping Perranporth Beach and coarse sand 0.55 mm at medium sloping Sennen Beach, Cornish coast, UK) using an array of mini electromagnetic current meters and optical backscatter sensors (OBS). The offshore wave height is in the range of 1 to 2 m (low wave conditions). The mean water depth at the transition between the surf zone with 100% inundation over time and the swash zone (intermittently wet and dry) is 0.25 m for Perranporth Beach and 0.4 m for Sennen Beach. The ratio of H_s/h at both transition points is of the order of 2. Their main findings show the presence of very energetic uprush and downrush velocities with values up to 2 m/s. **Figure 2.7** shows a typical (asymmetric and skewed) swash velocity distribution as a function of time. The vertical velocity gradient near the bed and the resulting bed-shear stress at the start of the uprush phase is significantly larger (factor 2) than that at the end of the backwash, see **Figure 2.6**. The time-averaged velocities are negative (offshore-directed) at both beaches and are of the order of -0.1 to -0.3 m/s during low wave conditions (offshore wave heights between 1 and 2 m). The near-bed suspended sand concentrations in the swash zone generally exceed 100 kg/m^3 at the start and the end of the backwash. During low wave conditions, the uprush induces a larger transport rate than the backwash indicating that the uprush is a more competent transporter of sediment than the backwash maintaining the beach (during low wave conditions).

Barnes et al. (2009) present direct measurements of bed-shear stress in the swash zone. The data were obtained using a shear plate with various types of roughness values (smooth to very rough). Numerical modelling was applied to calculate velocities and bed-shear stresses for the same tests. The measured bed-shear stresses and calculated velocities were used to back-calculate instantaneous local skin friction coefficients using the quadratic drag law. The data show rapid temporal variation of the bed-shear stress through the leading edge of the uprush, which is typically 2 to 4 times greater than the backwash shear stresses at corresponding low velocity. The data also indicate strong temporal variation in the skin friction coefficient, particularly in the backwash. Skin friction coefficients during the uprush are approximately twice those in the backwash at corresponding Reynolds number and cross-shore location.

According to **Ruessink and Van Rijn (2010)**, the skewness and asymmetry of the near-bed velocity in the inner surf and swash zone can to certain extent be represented by:

$$\tilde{U} = \hat{U}_1 \cos(\omega t) + \hat{U}_2 \cos(2\omega t - \beta) \qquad (2.29)$$

with: $\hat{U}_1$= amplitude of first harmonic, $\hat{U}_2$= amplitude of second harmonic, β= phase difference.

The skewness of this wave signal (velocity as a function of time) represents the wave asymmetry with respect to the onshore and offshore velocities (high and narrow peaks; wide and shallow troughs) and is defined as
$S_k = \langle U^3 \rangle/(\sigma_U)^3$ with $(\sigma_U)^2 = \langle U^2 \rangle = 0.5[(\hat{U}_1)^2 + (\hat{U}_2)^2)]$ and $\langle .. \rangle$ = time-averaging.

The asymmetry with respect to time within the wave cycle (forward leaning waves) is defined as the Hilbert transform of the velocity signal which can also be defined as the skewness of the derivative of the velocity signal: $A_s = -\langle(\omega\, d\tilde{U}/dt)^3\rangle/(\sigma_U)^3$. Symmetric waves (in time) yield a value of A_s= 0.

Using these definitions, it follows that:

$$S_k = \frac{0.75\,(\hat{U}_1)^2\,(\hat{U}_2)\cos(\beta)}{(\sigma_U)^3} \quad \text{and} \quad A_s = -\frac{0.75\,(\hat{U}_1)^2\,(\hat{U}_2)\sin(\beta)}{(\sigma_U)^3} \qquad (2.30)$$

$$\tan(\beta) = -A_s/S_k \quad \text{or} \quad \beta = -\operatorname{atan}(A_s/S_k) \qquad (2.31)$$

$$(\hat{U}_2)^3 - 2(\sigma_U)^2\,\hat{U}_2 + (4/3)\,(\sigma_U)^3\, S_k/\cos(\beta) = 0 \qquad (2.32)$$

$$(\hat{U}_1)^2 + (\hat{U}_2)^2 = 2(\sigma_U)^2 \qquad (2.33)$$

Equation (2.32) can be solved analytically using a complex function approach, yielding:

$$\hat{U}_2 = 2(P)^{0.5}\cos(\varphi) \qquad (2.34)$$

with: $P = 2(\sigma_U)^2/3$, $Q = (4/3)\,(\sigma_U)^3\, S_k/\cos(\beta)$ and $\varphi = 1/3[\arccos(-0.5Q\, P^{-1.5}) + n(2\pi)]$; n= 0, 1, 2 yields three roots; the smallest positive root is the solution. $\hat{U}_1$ follows from Equation (2.33).

Using linear wave theory, the standard deviation of the velocity is defined as: $\sigma_U = \pi H_{rms}/(1.41\ T \sinh(kh))$.

Based on the analysis of a large field data set of measured velocity time series, the parameters S_k, A_s and β are found to be:

S_k = B cos(β),
A_s = B sin(β),
β = –90 + 90 tanh(0.6373/$Ur^{0.5995}$),
B = 0.7939 $[1+\exp(K)]^{-1}$,
K = 2.8256[–0.6065 – $^{10}\log(U_r)$],
U_r = $0.75(0.5H_{Mo})\,k\,(kh)^{-3}$,
k = (2π/L)= wave number, h = water depth, H_{Mo}= 1.41 H_{rms}.

Almost perfect sinusoidal waves are present for Ursell numbers smaller than U_r < 0.01. Skewed waves are present for $0.01<U_r<0.1$. Skewed and asymmetric waves (bore type waves) are present for U_r> 0.1. The phase angle β (in degrees) increases to -90 degrees

Figure 2.8 shows the saw-tooth near-bed velocity time signal for h= 0.3 m, H_{rms}= 0.3 m, T= 7 s. The peak landward and seaward velocities are almost the same. The duration of the forward phase is about 3 s and that of the backward phase is 4 s. The peak landward velocity occurs at about 1.4 s after t= 0 and the peak seaward velocity at about 1.4 s before t_{end}.

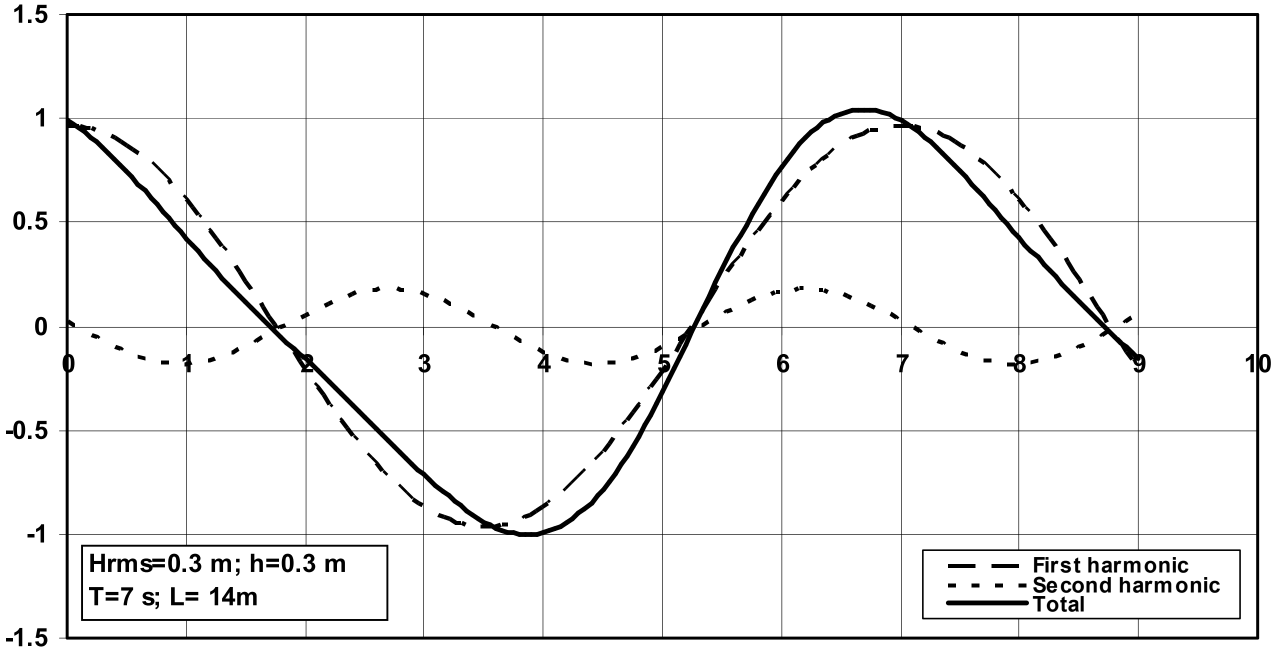

Figure 2.8 *Velocity time series for H_{rms}= 0.3 m, T= 7 s, h= 0.3 m*

3 Laboratory and field data of gravel/shingle transport and beach profile changes

3.1 Laboratory data

3.1.1 Small-scale laboratory tests

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, which provide the most satisfactory reproduction of natural beach permeability, sediment mobility threshold and onshore-offshore transport characteristics. Test conditions included 29 different wave conditions (based on JONSWAP spectra), four representative sediment mixes and several variations of effective beach thickness and core permeability values. All tests commenced with a standard beach face slope of 1:7 and a toe level in "deep water" seaward of the wave breaking point. Tests of "unlimited" beach thickness were run for a duration of 3000 waves, following a 3 hour profile compaction period using the largest waves calibrated for the study. Tests of restricted beach thickness (low permeability core at varying depths beneath the initial surface) were run for a duration of 1000 waves with no compaction period. Measurements were recorded of beach profile changes (at 500 wave intervals), wave run up exceedance and wave energy dissipation.

The factors tested by the flume study were: wave height (H_s), wave period (T_m), wave duration (N), beach material size (d_{50}), beach material grading (d_{85}/d_{15}) and effective thickness of beach material (D_B). Other factors of interest such as: foreshore level (D_w), water level (SWL), initial beach profile, wave spectrum shape and angle of wave attack were derived from other test results.

The test results show the following main features:

(a) The influence of wave height is most significant in the upper beach zone where an increase in height causes an increase in surf zone width (i.e. a flattening of the upper beach profile).

(b) The effect of wave period variations is apparent in the vertical dimensions of the profile; thus an increase in wave period will increase the crest elevation and lower the profile toe.

(c) Beach profiles react rapidly to changes in wave conditions. Tests show that 80% of the volumetric change occurred during the initial 500 waves of each test.

(d) The effective beach thickness appears to have greatest influence on horizontal regression of the beach above SWL. Exposure of the impermeable core and subsequent beach de-stabilisation generally occurs when the ratio D_B/d_{50} ratio is less than 30 (where D_B is the thickness of the mobile shingle layer measured normal to the initial beach slope and d_{50} is the median particle size).

(e) Beach particle size and grading appear to have some effect on beach profiles; however as only four sizes and two gradings were tested the observations cannot be considered conclusive. Smaller grain sizes appear to show more marked response to increases in wave steepness while broader grading ranges appear to result in higher crest levels.

(f) The foreshore level determines the location of the wave breaking zone. Waves breaking directly on a shingle beach will result in some form of step (swell waves) or bar and trough (storm waves) lower beach profile. Waves breaking seaward of the shingle will not develop either variation and will have reduced upper beach dimensions (crest elevation and run up distance).

(g) Variations in the steep initial beach slopes typical of shingle beaches are considered to have little effect on the ultimate beach profile, though they may affect the mode and duration of formation.

(h) Gradually varying water levels do not affect the shape of the slope of the beach profiles, but will determine the location of the crest profile on the beach face.

(i) The effect of varying the angle of wave attack on profile development data indicates that oblique wave action restricts the full development of at least part of the profile.

3.1.2 Large-scale laboratory tests

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, see **Table 3.1**). The initial beach slope was 1 to 5 (plane sloping beach) in all (nine) experiments. Irregular waves were generated (Pierson-Moskowitz spectrum). The basic data are given in **Table 3.1**.

The dynamic behaviour of beaches can be described by the parameter $H_{s,o}/((s-1)d_{50})$, as follows:

sand range (breaker bar formation):	$H_{,o}/[(s-1)d_{50}] > 200$
gravel range (swash bar formation):	$10 < H_{,o}/[(s-1)d_{50}] < 200$
cobble and stone range without slope deformation:	$H_{,o}/[(s-1)d_{50}] < 10$

The parameter $H_{s,o}/[(s-1)d_{50}]$ of the experiments in the Deltaflume is in the range of 50 to 80.

($H_{s,o}$= offshore significant wave height, T_p= peak wave period, s= relative density, d_{50}= sediment size).

Test	Beach slope	d_{10} (m)	d_{50} (m)	d_{90} (m)	SWL above flume bottom (m)	$H_{s,o}$ (m)	T_p (s)
1	1 to 5	0.014	0.021	0.029	3	0.77	5.0
2	1 to 5	0.014	0.021	0.029	3	1.0	5.0
3	1 to 5	0.014	0.021	0.029	4.5	1.5	5.5
4	1 to 5	0.0031	0.0048	0.0065	4.5	0.62	2.9
5	1 to 5	0.0031	0.0048	0.0065	4.5	1.24	4.5
6	1 to 5	0.0031	0.0048	0.0065	4.5	1.68	5.7
7	1 to 5	0.0031	0.0048	0.0065	4.5	1.28	4.5
8	1 to 5	0.0031	0.0048	0.0065	4.5	1.08	5.1
9	1 to 5	0.0031	0.0048	0.0065	4.5	1.14	7.6

Table 3.1 *Basic data of Delta flume experiments on gravel beaches (Deltares, 1989)*

The measured bed surface profiles of Tests 1, 2, 3, 4, 5, 6 and 9 are shown in **Figures 3.1** and **3.2**.

The most characteristic features are:

- formation of swash bar above SWL (up to 2.5 m) due to onshore transport; the swash bar extends vertically to about $2H_{s,o}$ above SWL indicating the effect of wave run-up; the swash bar size increases with increasing wave height and with increasing wave period (Test 9 with T_p= 7.6 s);
- formation of a small breaker bar extending to about $1H_{s,o}$ below SWL when relative fine gravel (d_{50}= 0.0048 m) is present for $H_{,o}/((s-1)d_{50})> 100$ (Tests 4, 5 and 6);
- generation of scour pit below SWL; the scour depth extends vertically to $3H_{s,o}$ below SWL;
- small zone (with height equal to $H_{s,o}$) direct above and beneath SWL showing almost no deformation;
- ripples with length scales of 1 to 3 m and height scales of 0.1 to 0.4 m at the lower part of the fine gravel slope (d_{50}= 0.0048 m) for $\psi > 10$ ($\psi=\hat{U}^2/[(s-1)gd_{50}]$, $\hat{U}$ = peak orbital velocity based on linear wave theory); the ripples are largest in Test 9 with relatively long waves (T_p= 7.6 s); the ripple lengths are roughly equal to $2\hat{A}$ to $2.5\hat{A}$ ($\hat{A}$ = peak orbital excursion).

The formation of the swash bar is strongly related to the wave uprush and downrush near the water line. The uprush is much stronger than the downrush due to the percolation of water through the porous gravel bed surface resulting in a relatively strong velocity asymmetry in the swash zone and hence net onshore transport of gravel particles. The maximum uprush velocity can be estimated by the bore velocity near the water line: $u_b= (gh)^{0.5}$ with h in the range of 0.1 to 0.2 m yielding u_b in the range of 1 to 3 m/s.

The vertical runup can be estimated by using $R_{2\%}= 2.3(\zeta)^{0.3}$ with $R_{2\%}$ =vertical runup above SWL exceeded by 2% of the values, $\zeta= \tan(\beta)/(2\pi H_{s,toe}/(gT_p^2))$ = surf similarity parameter, $\tan(\beta)$= gradient of gravel slope, T_p= wave period (**Van Gent, 2001**). The ζ-parameter is about 1 for the Deltaflume experiments resulting in runup values of about $2H_{s,toe}$ ($\cong 2H_{s,o}$), which is in reasonable agreement with observed values (vertical swash bar level of 2 to 2.5 $H_{s,o}$).

The swash bar area (A_s) is in the range of 2 to 5 m^2 after about 4 hours (duration of storm event). The swash bar area (A_s) can be made dimensionless by using the significant wave heigth ($H_{s,toe}$) at the toe of the gravel slope, which is about equal to the offshore wave height ($H_{s,o}$) in the Deltaflume experiments. The parameter $A_s/(H_{s,toe})^2$ is in the range of 2 to 4 after about 4 hours (duration of storm event).

The dimensionless breaker bar area $A_b/(H_{s,toe})^2$ also is in the range of 1 to 2 after about 4 hours.

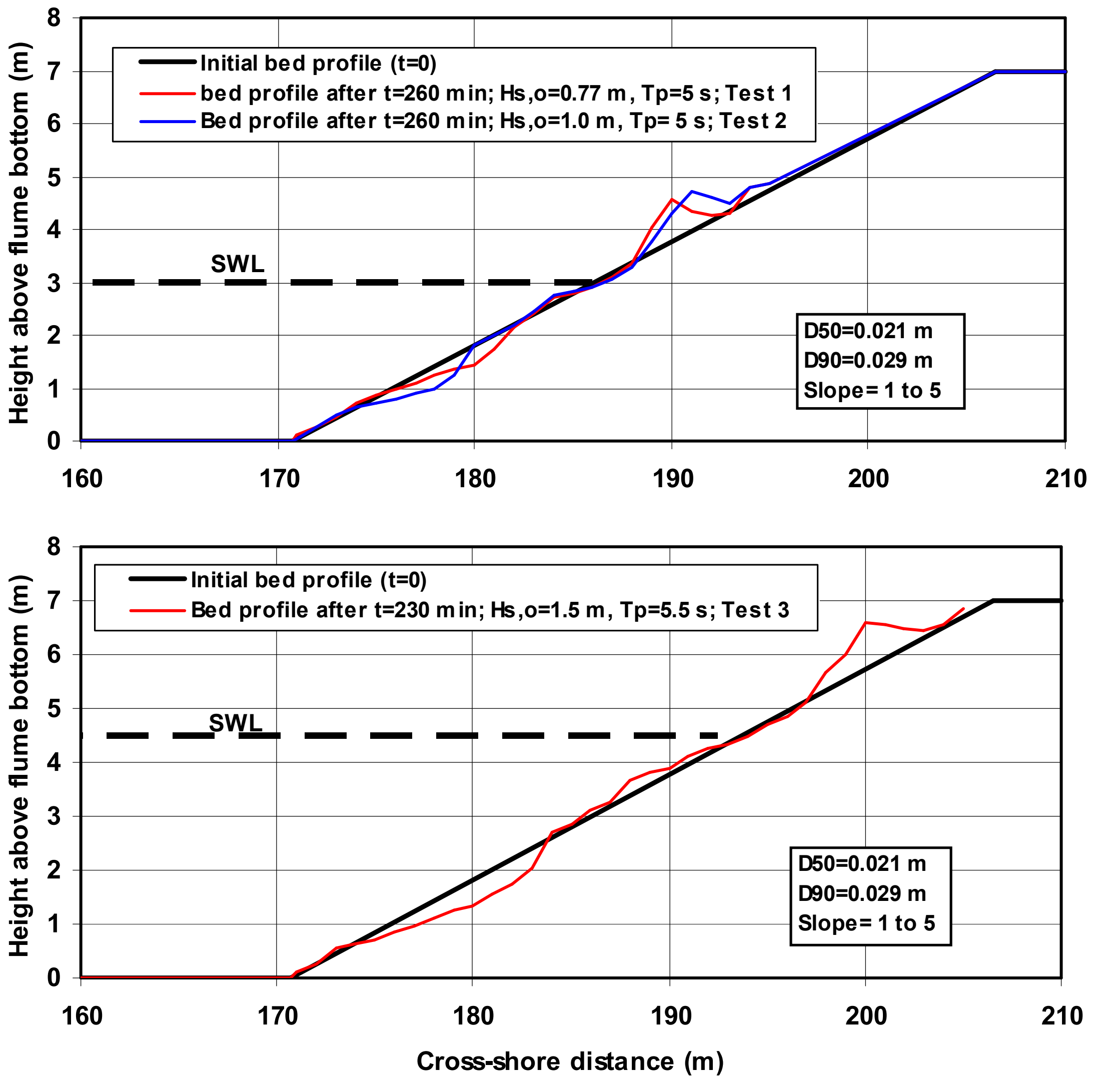

Figure 3.1 *Top: Tests 1 and 2 (d_{50}= 0.021 m; SWL=3 m); Bottom: Test 3 (d_{50}= 0.021 m; SWL= 4.5 m)*

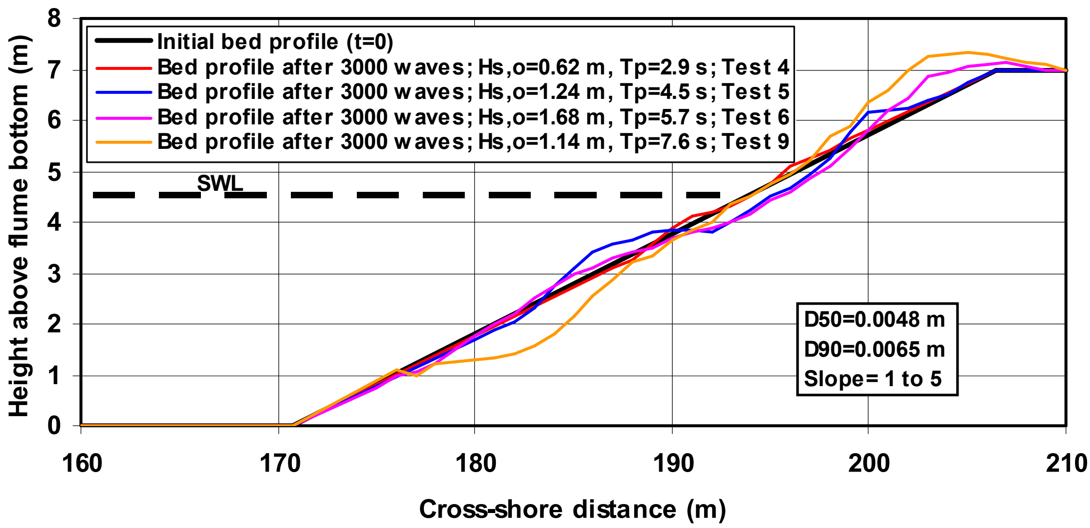

Figure 3.2 *Tests 4, 5, 6 and 9 (d_{50}= 0.0048 m; SWL= 4.5 m), Deltaflume, The Netherlands*

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 0.02 m. The initial slope of the beach is about 1 to 8. The basic data are given in **Table 3.2**.

The measured bed surface profiles of Tests 1 to 5 are shown in **Figure 3.3**. As can be observed, there is a gradual development of a triangular bar with a maximum height of about 2 m (about $2H_{s,toe}$) just beyond the still water line after 12000 waves (or 57100 s). The total accretion area is about 10 m^3/m. The transport of shingle passing the water line is about 10/57100= 0.0000175 m^2/s or about 15 m^3/m/day at an offshore wave height of about $H_{m,o}$= 1 m. This value fits well in the transport plot of **Figure 4.16**.

Test	Beach slope	d_{50} (m)	SWL above flume bottom (m)	$H_{m,o}$ (m)	T_p (s)	Number of waves
1	1 to 8	0.021	4.7	0.52	3.2	3000
2	1 to 8	0.021	4.7	0.91	4.1	2000
3	1 to 8	0.021	4.7	1.07	4.3	2000
4	1 to 8	0.021	4.7	0.95	5.1	3000
5	1 to 8	0.021	4.7	1.02	7.7	2000

Table 3.2 *Basic data of GWK flume experiments on shingle beaches*

In June and July 2008 large-scale experiments on gravel barriers (d_{50}= 0.011 m) 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 aim of these (BARDEX) experiments was to study the hydrodynamic and morphodynamic characteristics of a gravel barrier backed by a lagoon with a lower water level. 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.

The test results with a constant sea level show the formation of a typical swash bar landward of the still water level with a maximum crest level of about $2H_{s,toe}$ above the still water level (SWL). The test results with a time-varying sea level show the vertical build-up of the crest to a level equal to about $1.5H_{s,toe}$ above HWL (with HWL= maximum tidal water level).

The results of Test B3 with a constant water level on both sides of the barrier (no overtopping) are used in **Section 4.2.3** for comparison with computed results.

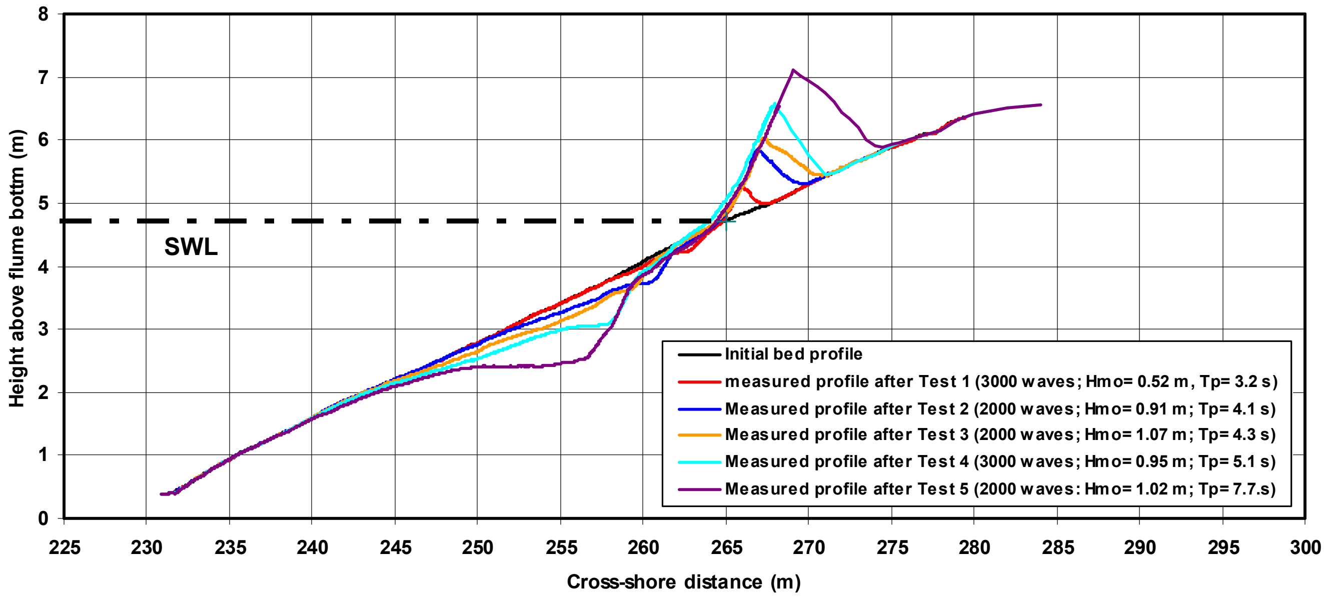

Figure 3.3 *Tests 1 to 5 (d_{50}= 0.02 m), GWK, Hannover, Germany*

3.2 Field data

3.2.1 Gravel transport

Chadwick (1989) used surface mounted traps to measure the alongshore gravel/shingle transport at the beach of Shoreham (West Sussex, England) during low and moderate wave conditions (wave heights between 0.3 and 0.8 m, periods between 2 and 4 s, wave angle between 20^o and 40^o). The beach consisted of shingle with a sand foot at about the low water mark (beach slope, $\tan\beta$= 0.10 to 0.12). Alongshore transport was measured on 18 occasions, together with concurrent measurement of wave height, angle and speed. The traps were positioned approximately on the waterline two hours before high water and the volumes of gravel/shingle collected were measured as the tide receded. Gravel/shingle transport occurred mainly in the swash zone. The transports rates measured by the trap were converted to swash-zone integrated transport rates (Q_t), based on an assumed cross-shore transport distribution (maximum at break point, dropping linearly to zero at both the lower and upper limit of the swash zone). The results roughly are ($H_{s,b}$= significant wave height at break point, α_b= 13^o to 20^o):

Q_t= 3 m^3/day for $H_{s,b}$= 0.3 m,
Q_t= 10 m^3/day for $H_{s,b}$= 0.35 m,

Q_t= 20 m^3/day for $H_{s,b}$= 0.4 m,
Q_t= 30 m^3/day for $H_{s,b}$= 0.7 m.

Nicholls and Wright (1991) performed tracer studies to determine longshore transport rates. Three experimental studies using aluminium tracers have been analysed. All three experiments were performed on shingle beaches (with nearshore zone of sand) at Hurst Castle spit, England. The beaches are exposed to wave action from the English channel and from the Atlantic Ocean. The tidal range is about 2 m. The beach and tracer characteristics (length of shortest and longest axis) are given in **Table 3.3**. The tracers were injected at low tide as a slug at one pebble depth.

Exp.	**Tracer size (mm)**	**Size orig. mat. (mm)**	**Duration of exp. (hrs)**	**Displace-ment of centroid (m)**	**Width of mobile layer (m)**	**Thickness of layer (m)**	**Longsh. transport (m^3/day)**	**Wave energy at br. line $(H_b)^{2.5}\sin 2\alpha$ ($m^{2.5}$)**
1) 9/5-25/5 1977	33-48	32	74	32	17-27	0.09-0.19	15-50	0.25
2) 20/2-11/3 1978	31-59	32	50	50	27-37	0.05-0.15	30-135	0.52
3) 8/3-27/4 1982	26-52	16	122	165	15-25	0.09-0.19	45-160	0.11-0.16

Table 3.3 *Data of gravel tracer study at Hurst Castle spit, England (Nicholls and Wright, 1991)*

In experiments 1 and 2 the tracer particles were placed on the upper foreshore, where the original material was of similar size. In experiment 3 the tracer material was split into three identical slugs injected at three sites across the foreshore from low water to high water. Cross-shore mixing of the tracer material with the original material was observed to occur rapidly (in hours) due to high wave energy in all three cases. About 60% to 70% of the tracer material was recovered. All three experiments were characterized by high energy conditions. During experiment 3 the wave height at the breakerline varied between 0.3 and 1.3 m (time-averaged value of about 0.6 m). The longshore transport rate was calculated as the product of the longshore displacement of the tracer centroid, the mean width of the mobile layer and the mean layer thickness per unit time. The wave energy was computed neglecting wave heights smaller than 0.5 m (threshold value for initiation of motion). The estimates of the longshore transport rates in experiment 3 may be relatively large, because the tracer size was relatively large resulting in a relatively large exposure to wave-induced forces.

3.2.2 Barrier erosion and migration

Barrier recession rates up to 4 m per year have been observed (**Nicholls and Webber, 1988**) at Hurst beach, Christchurch Bay, England (**Figure 3.4**). Most of the recession did occur during autumn and winter months. The barrier behaviour was influenced by saltmarsh deposits (peat, mud) exposed on the foreshore and rapidly eroded. This subsoil material leads to relatively rapid settlement beneath the weight of the gravel barrier reducing the crest level of the barrier. Differential settlement of the subsoil can lead to local depressions and increased overwashing.

Bradbury and Powell (1990) give an example of barrier rollover and lowering at Hurst Spit, England (**Figure 3.4**). The beach is characterised by coarse grain sizes of about 16 mm. The crest is generally between 2 and 4 m above HW, with a crest width of 3 to 10 m. Large-scale overwashing of the spit occurred in December 1989, resulting in crest lowering (about 1.5 m) along a beach length of 800 m. The beach was moved back by about 30 to 40 m in a single storm event, see **Figure 3.4**.

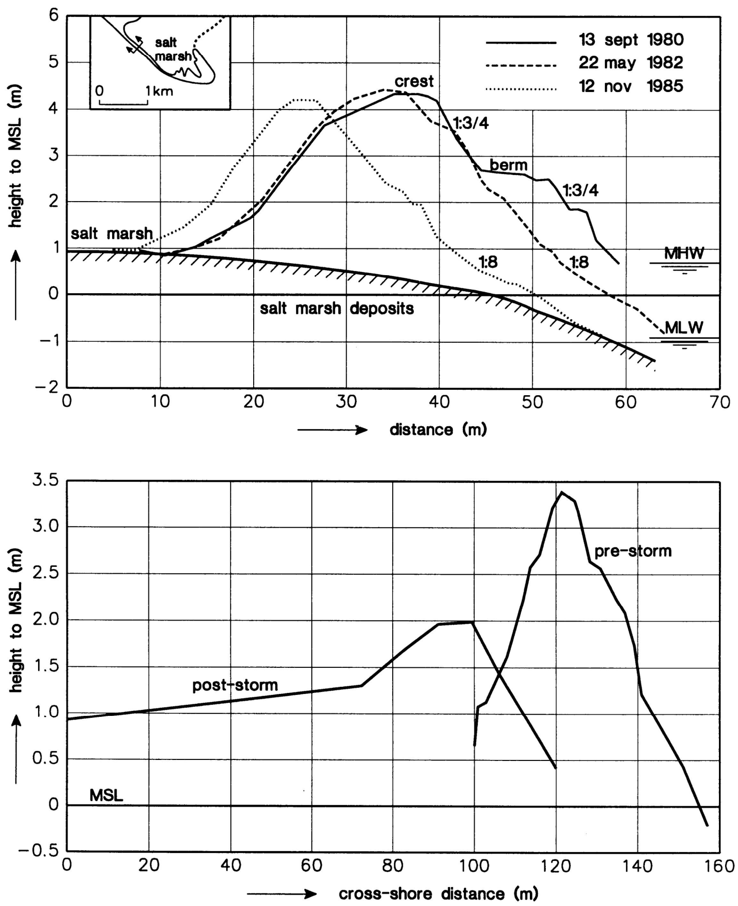

Figure 3.4 *Top:Hurst beach, Christchurch Bay, England (Nicholls and Webber, 1998. Bottom: Barrier response to storm event, Hurst Spit, South-England (Bradbury and Powell, 1990)*

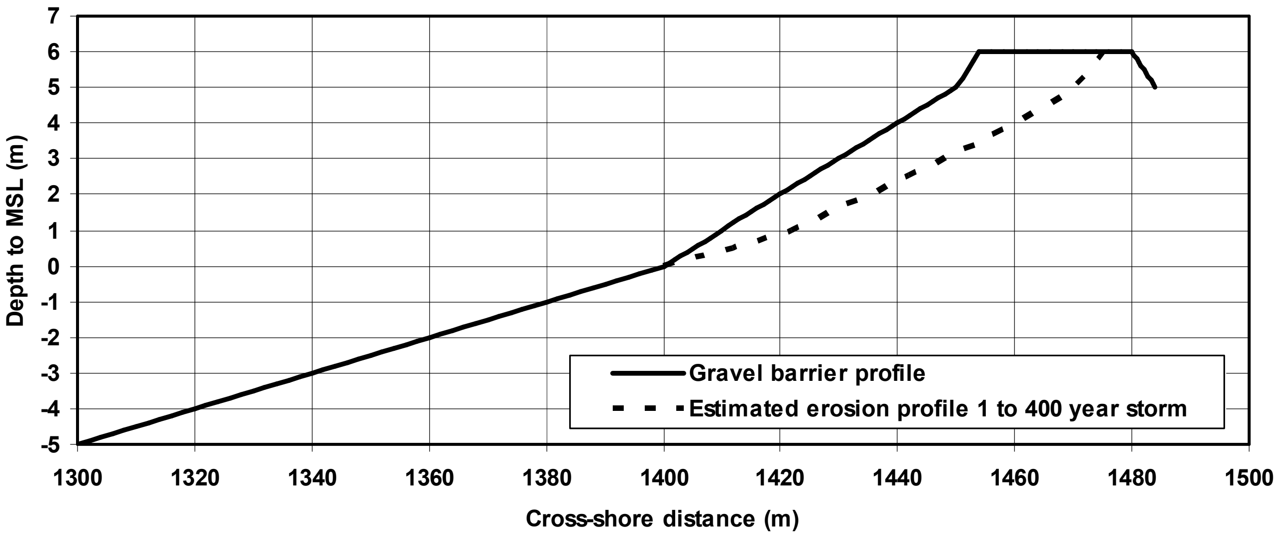

Figure 3.5 *Erosion profile of shingle barrier at Pevensey Bay, East Sussex, UK (www.pevensey-bay.co.uk)*

The **Pevensey Coastal Defence** (**Sutherland and Thomas, 2009**) uses a schematized erosion profile (as shown in **Figure 3.5**) 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.

3.2.3 Minimum barrier dimensions

A novel approach to determining the minimum required cross-sectional area to withstand gravel barrier breaching based on laboratory and field data has been developed by **Sutherland and Obhrai (2009)**. This approach uses the concept of barrier inertia (**Bradbury, 2000**) as a means of identifying the threshold of breaching of gravel barrier beaches. This is a non-dimensional measure of a barrier's ability to withstand breaching given by (see definition sketch in **Figure 3.6**):

$$B_i = R_c A_c/(H_{s,toe})^3 \qquad (3.1)$$

where: B_i = barrier inertia; R_c = barrier freeboard above mean water level (including tide and storm surge); A_c = cross-sectional area of barrier above mean water level; and $H_{s,toe}$ = significant wave height at the toe of the barrier.

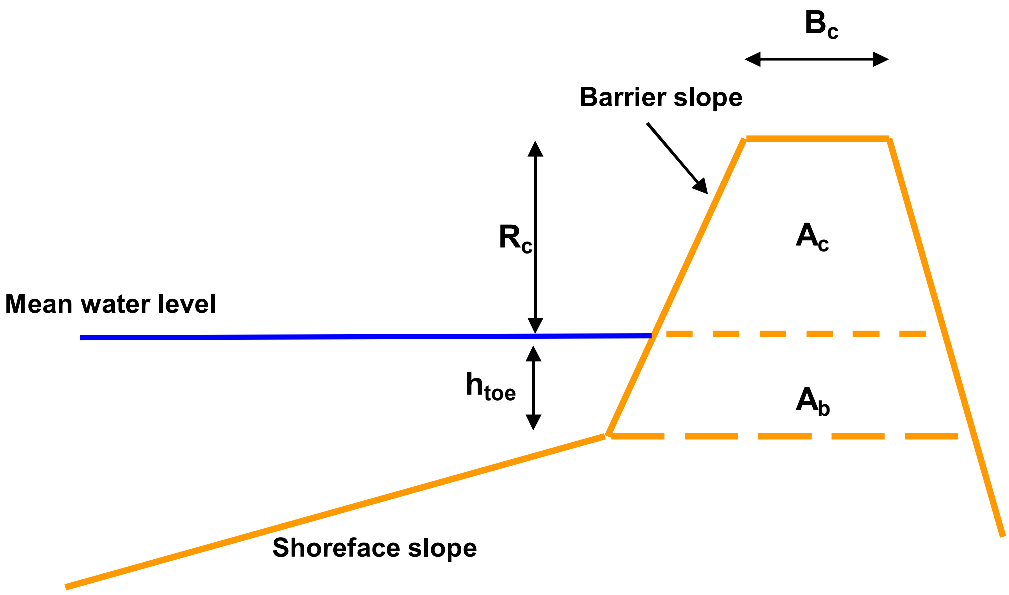

Figure 3.6 *Definition sketch of gravel barrier*

Bradbury (2000) developed an empirical framework to predict the threshold for breaching of shingle barrier beaches, based on extensive fieldwork (at Hurst Spit) and physical model data. **Obhrai et al. (2008)** extended the range of validity of this approach to lower and higher steepness waves. The **Obhrai et al. 2008** formula for the minimum required cross-sectional area (A_c) is:

$$A_c \geq (1/R_C)[-153.1\ (H_{s,toe}/L) + 10.9]\ (H_{s,toe})^3 \qquad (3.2)$$

Equation (3.2) can be roughly represented by: $A_c \geq (8/R_C)(H_{s,toe})^3$ (3.3)

Assuming: $H_{s,toe} \cong 0.5\ H_{s,o}$, it follows that: $A_c \geq (1/R_C)(H_{s,o})^3$ (3.4)

Equation (3.4) is shown in **Figure 3.5** for various values of R_c and $H_{s,o}$. The A_c-values are in the range of 15 to 250 m^2 for $H_{s,o}$ in the range of 4 to 8 m.

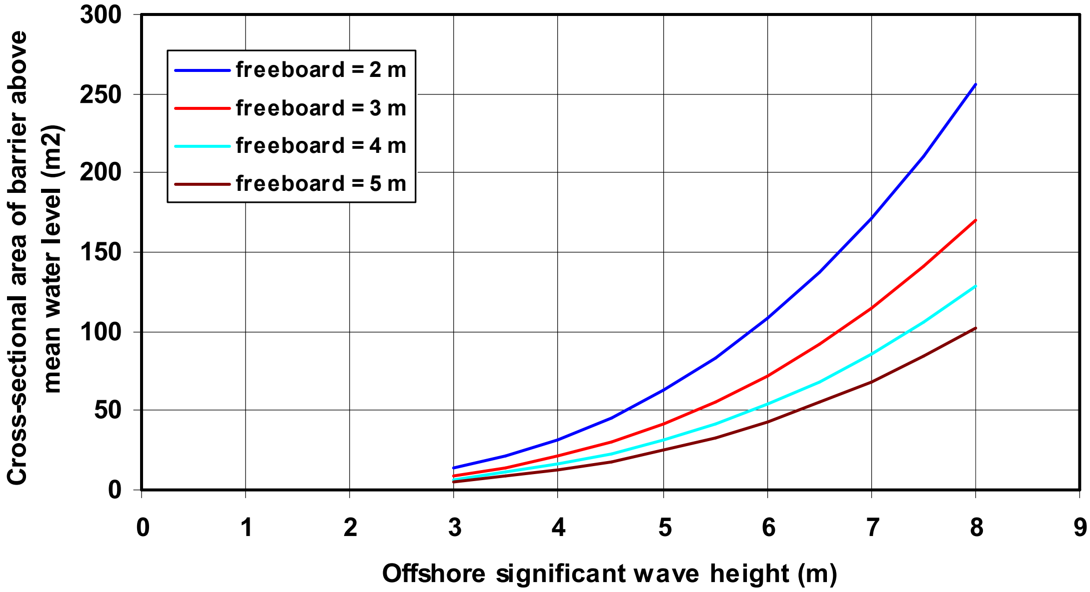

Figure 3.7 *Minimum cross-sectional area (A_c) of barrier above mean water level as function of freeboard and offshore significant wave height.*

4 Model simulation of gravel barrier erosion

4.1 Available models

4.1.1 Parametric SHINGLE model (HR Wallingford)

The beach profile prediction model SHINGLE was developed at HR Wallingford (**Powell, 1990**) as a coastal management tool. It is a parametric model which 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 data used to derive the basic algorithms for the SHINGLE-model were obtained from a physical model programme conducted in a random wave flume (see **Section 3.1.1**). The results have been validated against field trials at several UK locations.

The SHINGLE-model addresses two aspects of profile prediction: the predicted profile shape and the location of the predicted profile against an initial datum. These aspects are dealt with from a probabilistic rather than deterministic approach, requiring an understanding of the confidence limits that can be placed on the profile prediction.

The prediction process breaks the profile into three curves between the following limits:

1. beach crest and SWL;
2. SWL and top edge of the profile step;
3. top edge of the profile step and the lower limit of profile deformation.

These curves are characterised by a series of profile descriptors defining the position and elevation of each transition point. These profile descriptors are linked with non-dimensional groupings of the most influencial profile development variables to give three non-dimensional equations. The actual form of these equations was determined from regression analysis of the flume test data. Confidence limits were determined from the variation of the test results at any point along the predicted profiles.

The position of the predicted end profile relative to the initial profile assumes that beach material moves only in an onshore-offshore direction and that differential longshore transport is zero. The areas under the two curves are compared relative to a common datum and the predicted curve is shifted along SWL axis until the areas equate to provide the location of the predicted profile. If differential longshore transport is significant and a reasonable value can be assigned to the area of loss/gain, then a simple correction can be made to the predicted profile location.

The input data required are:

- initial profile, including foreshore,
- depth and slope of the underlying non-mobile stratum,

- beach particle size (d_{50}),
- effective beach thickness ratio (D_B/d_{50}),
- offshore wave height (H_s),
- offshore wave period (T_m),
- either ½ tidal cycle parameters of start and finish water levels plus stepped increment size or still water level (SWL),
- area change due to differential longshore transport if applicable,
- groyne and/or seawall cross-section.

4.1.2 Process-based CROSMOR2008 model

Hydrodynamics and sand transport

The CROSMOR2008-model is an updated version of the CROSMOR2004-model (**Van Rijn, 1997, 2006, 2007d**). The model has been extensively validated by **Van Rijn et al. (2003)**. The propagation and transformation of individual waves (wave by wave approach) along the cross-shore profile is described by a probabilistic model (**Van Rijn and Wijnberg, 1994, 1996**) solving the wave energy equation for each individual wave. The individual waves shoal until an empirical criterion for breaking is satisfied. The maximum wave height is given by $H_{max}= \gamma_{br}$ 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 (**Van Rijn et al., 2003; Van Rijn and Wijnberg, 1994, 1996**).

The application of a numerical cross-shore profile model to compute the erosion of the beach and duneface poses a fundamental problem which is related to the continuous decrease of the water depth to zero at the runup point on the beach face. The numerical modelling of the (highly non-linear) wave-related processes in the swash zone with decreasing water depths is extremely complicated and is in an early stage of development. In the CROSMOR-model the numerical solution method is applied up to a point (last grid point) just seaward of the downrush point, where the mean water depth is of the order of 0.2 to 0.5 m. The complicated wave mechanics in the swash zone (wet-dry zone) is not explicitly modelled, but taken into account in a schematized way (subgrid model). The limiting water depth of the last (process) grid point is set by the user of the model (input parameter; typical values of 0.2 to 0.5 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 the semi-empirical method of **Isobe and Horikawa (1982)** with modified coefficients (**Grasmeijer and Van Rijn, 1998**; **Grasmeijer, 2002**) or by the new method (see Section 2) proposed by **Ruessink and Van Rijn (2010)** based on input specifications. Near-bed streaming effects are modelled by semi-empirical expressions based on the work of **Davies and Villaret (1997, 1998, 1999)**. The streaming velocities at the edge of wave boundary layer may become negative for decreasing relative roughness values (A_w/k_w with A_w= peak wave excursion near bed; k_w= wave-related bed roughness value).

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 (**Svendsen, 1984; Dally and Osiecki, 1994**) is taken into account. The vertical distribution of the undertow velocity is modelled by schematizing the water depth into three layers with a logarithmic distribution in the lower two layers and a power distribution in the upper layer, yielding velocities which approach to zero at the water surface.

Low-frequency waves are generated in the surf zone due to spatial and temporal variation of the wave breaking point resulting in spatial and temporal variation of the wave-induced set-up creating low-frequency waves (surf beat). This involves a transfer of energy in the frequency domain: from the high frequency to the low frequency waves. The total velocity variance (total wave energy) consists of high-frequency and low-frequency contributions ($U^2_{rms}=U^2_{hf,rms}+U^2_{lf,rms}$). The low-frequency waves are represented by a semi-empirical expression based on analysis of Deltaflume experiments (**Van Rijn, 2008, 2009**).

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 4.1**. 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 peak onshore swash velocity is of the order of 1.5 to 2 m/s, see **Figure 2.7**. The swash velocity is added to the other cross-shore components of the near-bed velocity (orbital velocity, 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).

The sediment transport of the CROSMOR2008-model is based on the TRANSPOR2004 formulations (**Van Rijn, 2006, 2007a,b,c,d**). The effect of the local cross-shore bed slope on the transport rate is taken into account (see **Van Rijn, 1993, 2006**).

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.

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 (**Van Rijn, 1993, 2006, 2007**). 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, using the approach given by **Houwman and Ruessink (1996)**. The method is described by **Van Rijn (2006, 2007a,b,c,d)**. 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.

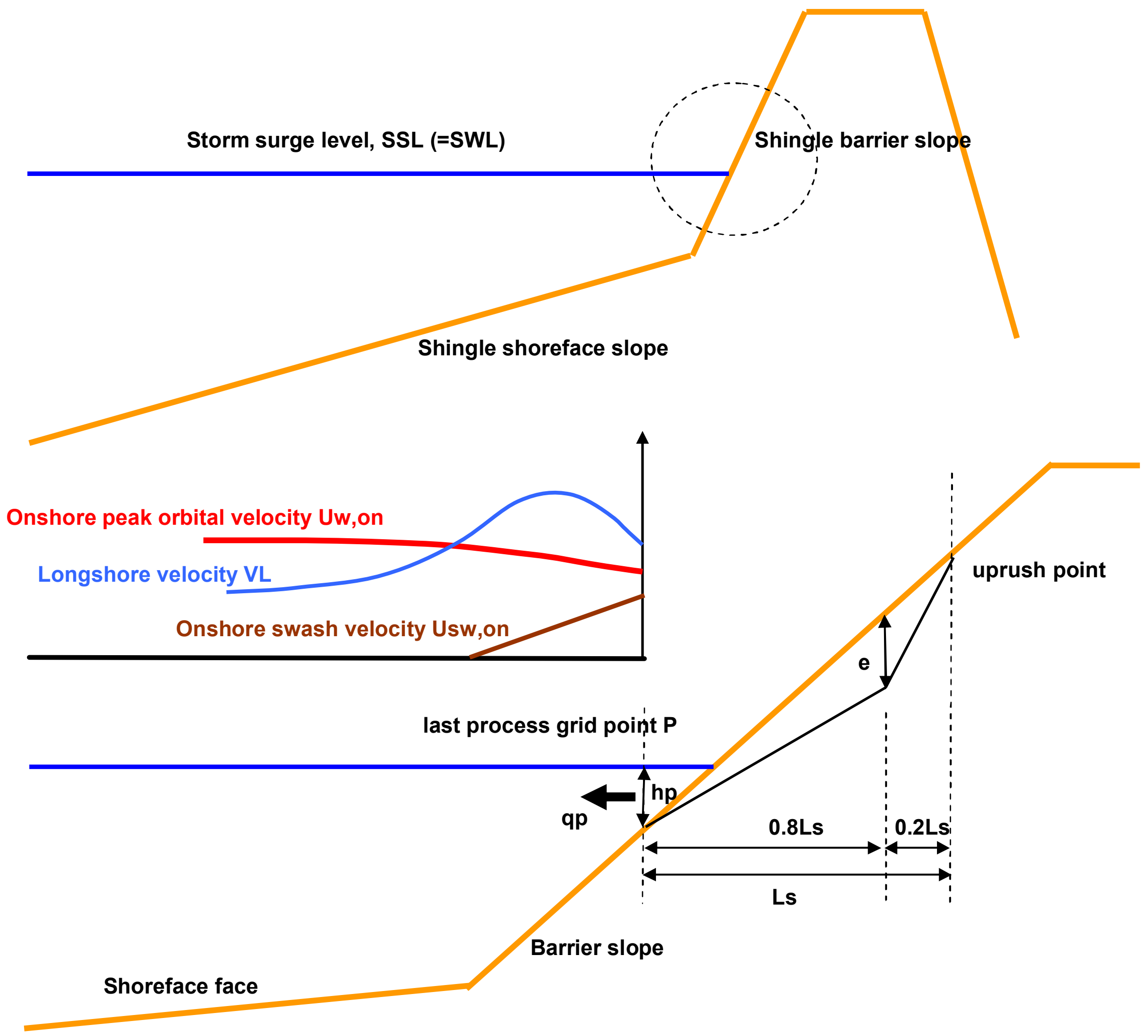

Figure 4.1 *Schematization of swash erosion zone for shingle barrier*

Bed level changes and deposition in swash zone

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

$$\rho_s(1\text{-}p)\partial z_b/\partial t + \partial(q_t)/\partial x = 0 \quad (4.1)$$

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:

$$\Delta z_{b,x,t}= -[(q_{t,})_{x-\Delta x} - (q_t)_{x+\Delta x}]\ [\Delta t/(2\ \Delta x(1\text{-}p)\rho_s] \quad (4.2)$$

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 $\Delta t/((1\text{-}p)\rho_s)$ 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 4.1.** 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.

4.2 Simulation results of gravel/shingle slopes

4.2.1 Deltaflume experiments (Deltares)

Tests 1, 2 and 9 of the Deltaflume experiments in 1989 (see **Table 3.1**) 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. The instantaneous orbital velocities are based on the method of **Ruessink and Van Rijn (2010)**. Low-frequency surf beat motion is taken into account based on a semi-empirical approach.

Figures 4.2 and **4.3** 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 much too small for Test 1 and 2.

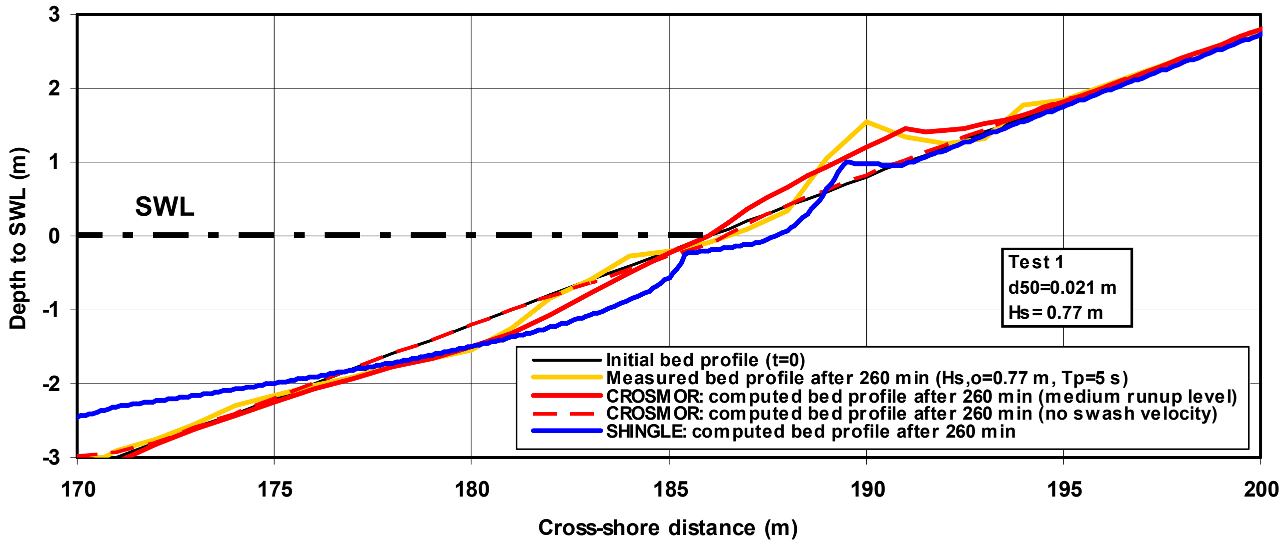

Figure 4.2 *Simulation of Deltaflume Test 1 ($H_{s,o}$=0.77 m; d_{50}=0.021 m)*

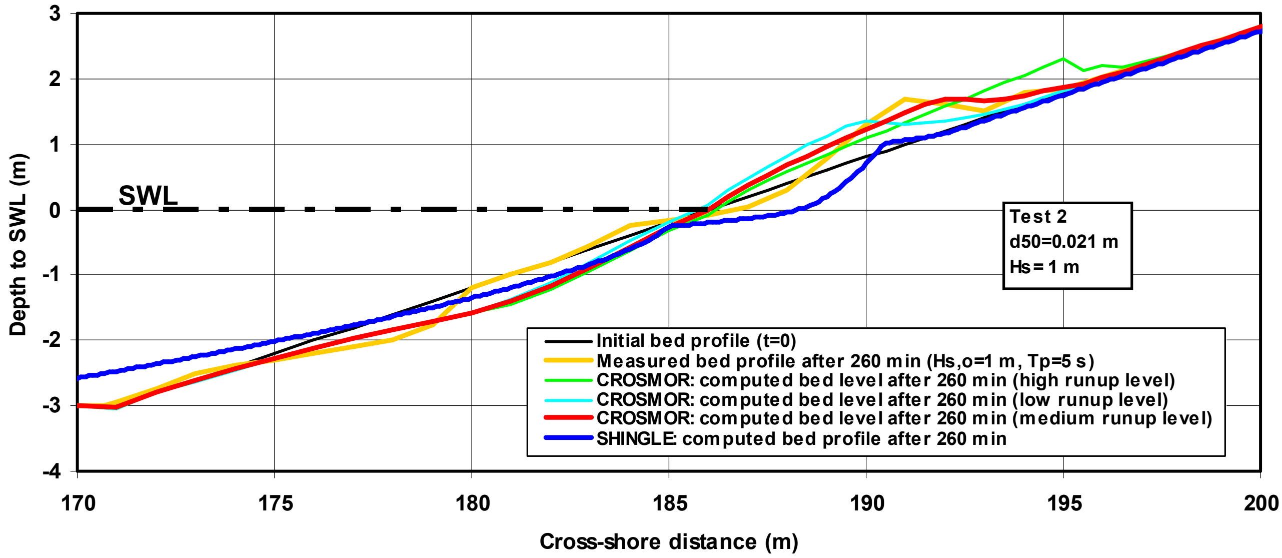

Figure 4.3 *Simulation of Deltaflume Test 2 ($H_{s,o}$=1 m; d_{50}=0.021 m)*

Figure 4.4 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.

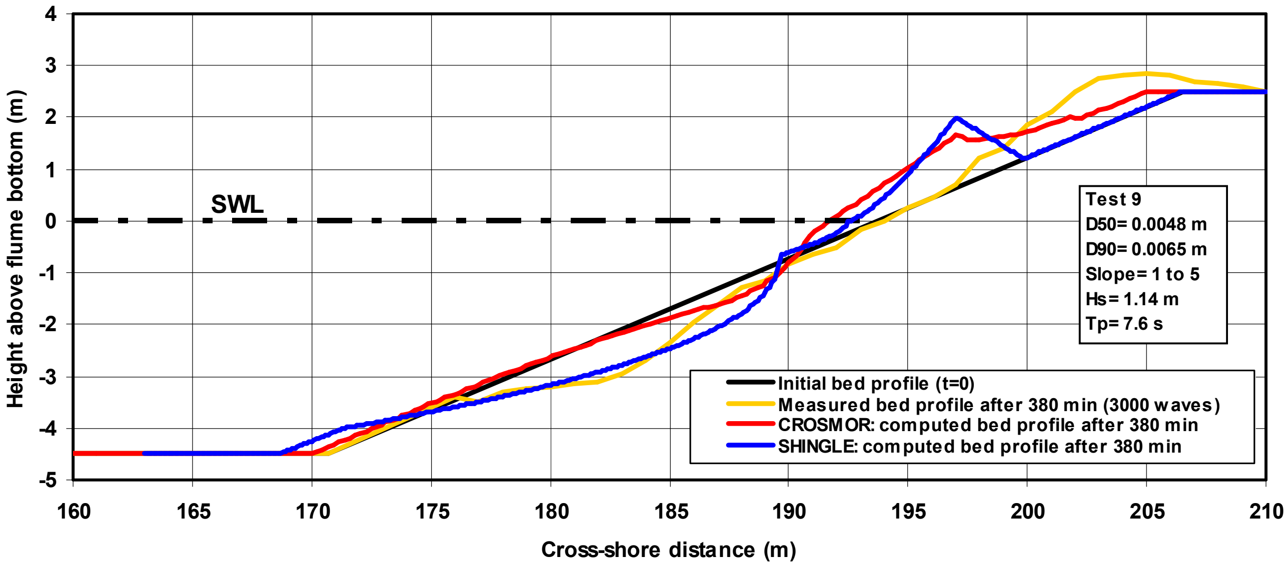

Figure 4.4 *Simulation of Deltaflume Test 9 ($H_{s,o}$=1.14 m; d_{50}=0.0048 m)*

4.2.2 GWK flume experiments

Tests 1 to 5 of the GWK flume experiments (see **Table 3.2**) have been used to verify the CROSMOR-model for gravel slopes.

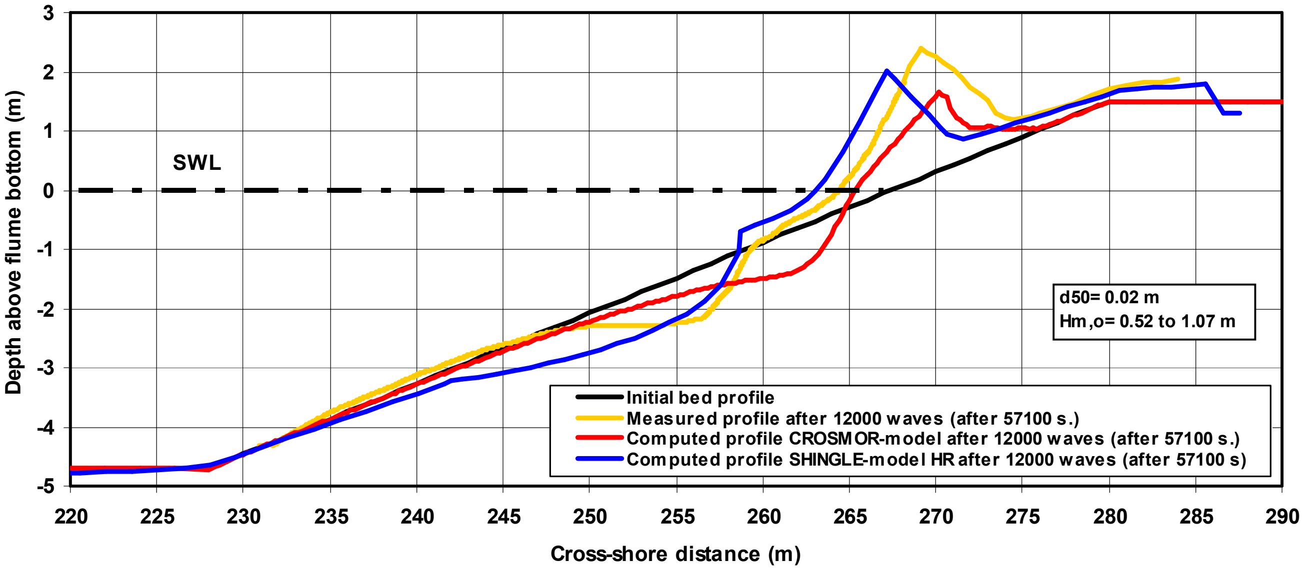

Figure 4.5 *Simulation of GWK Test 1 to 5 ($H_{m,o}$= 0.52 to 1.07 m; d_{50}=0.02 m)*

Figure 4.5 shows the simulation results of the GWK Tests 1 to 5 using both the parametric SHINGLE model of HR Wallingford and the process-based CROSMOR-model of Deltares. The computed swash bar area of the SHINGLE model is of the right order of magnitude, but its location on the profile is somewhat too low. The computed erosion volume below the still water line is much too large. The computed swash bar area of the CROSMOR-model is somewhat too small, but its position on the profile is rather good. The computed erosion zone is of the right order of magnitude, but its position on the profile is much too high.

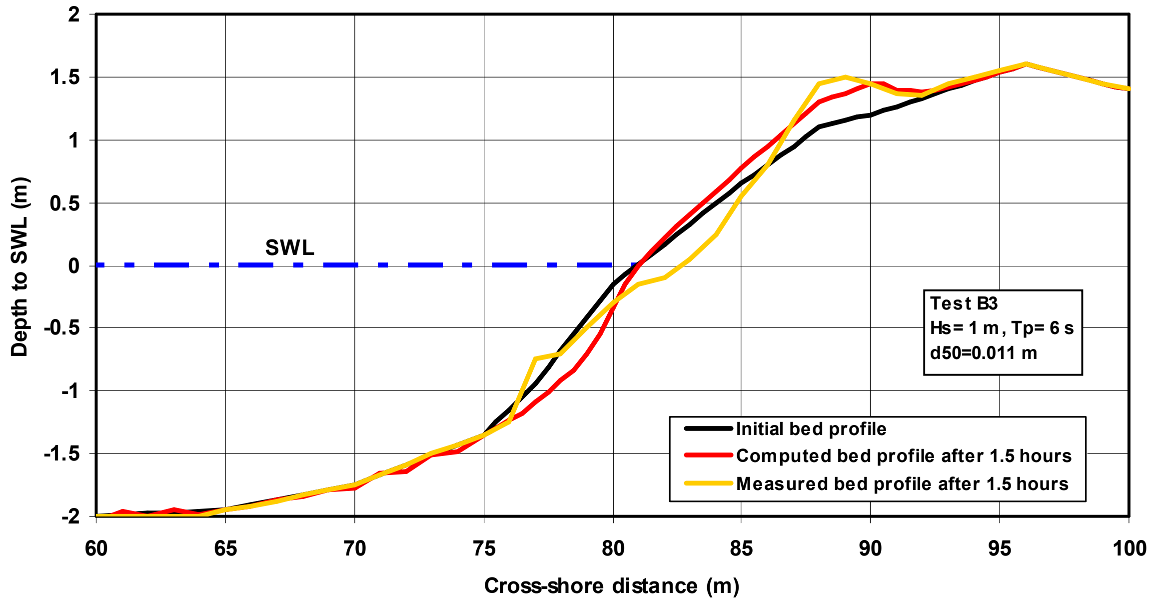

Figure 4.6 *Simulation of BARDEX Test B3 ($H_{m,o}$= 1.0 m; d_{50}= 0.011 m)*

4.2.3 BARDEX experiments Deltaflume

Test B3 of the BARDEX experiments (**Buscombe, Williams and Masselink, 2008**) has been used to verify the CROSMOR-model for gravel slopes.

Figure 4.6 shows the simulation results of the CROSMOR-model. The computed swash bar area is of the right order of magnitude (2 m^3/m). The computed erosion volume also is of the right order of magnitude, but its position on the profile (below SWL) is much too low.

The results of the SHINGLE model runs are not shown, because the model did not produce meaningful results for this case (computed beach slope everywhere seaward of the initial slope). The SHINGLE model uses an equilibrium slope concept, by which the model slope is forced away from the slope in the flume. The actual beach used in the flume tests may have been somewhat too steep for that size of shingle.

4.3 Gravel/shingle barrier applications

4.3.1 Pevensey shingle barrier

To demonstrate the applicability of the model for prototype gravel barriers, the CROSMOR2008-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 4.1** (see also **Sutherland and Thomas, 2009**). 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 (see **Figure 4.7**) 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 4.2**. The offshore wave incidence angle is arbitrarily set to 30° to include wave-driven longshore velocities.

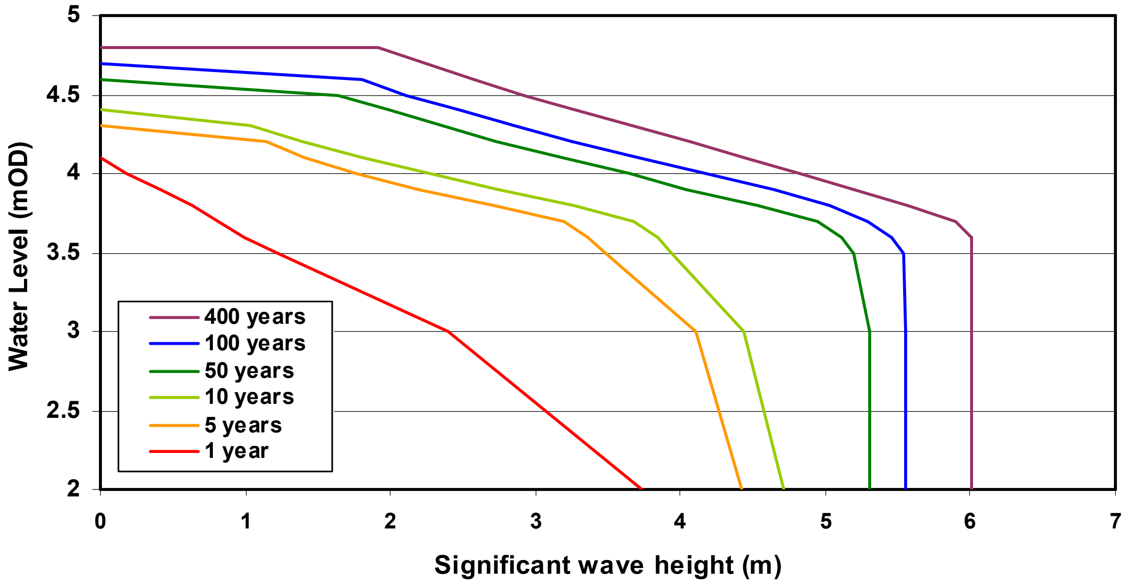

Figure 4.7 *Joint probability conditions for Pevensey Bay (sector 165° to 195°)*

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 4.1 *Data of shingle barrier at Pevensey Bay, UK*

Case	Maximum water level (m)	Storm setup (m)	Tidal range (m)	Offshore wave height Hs,o (m)	Offshore wave period T_m (s)	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

Table 4.2 *Storm wave cases*

The joint probability curves represent a standard shape for conditions where the wave height and surge are weakly correlated. At Pevensey Bay the largest surges would probably come from the south-west as would the largest offshore waves. However, Pevensey is sheltered by Beachy Head and the offshore bathymetry; so the largest waves in deep water are not the largest waves inshore. The most severe wave conditions are for waves from the south, which would generate a smaller surge. It is highly unlikely that the highest waves will come at the same time as the highest water levels. In fact, water levels and wave heights are almost completely uncorrelated. For the uncorrelated case a 400 year return interval occurs for any combination of wave height and water level return intervals that, when multiplied, give 400 years. An example of a joint return interval of 400 years is a 100 year return interval for the wave height and 4 year return interval for the water level.

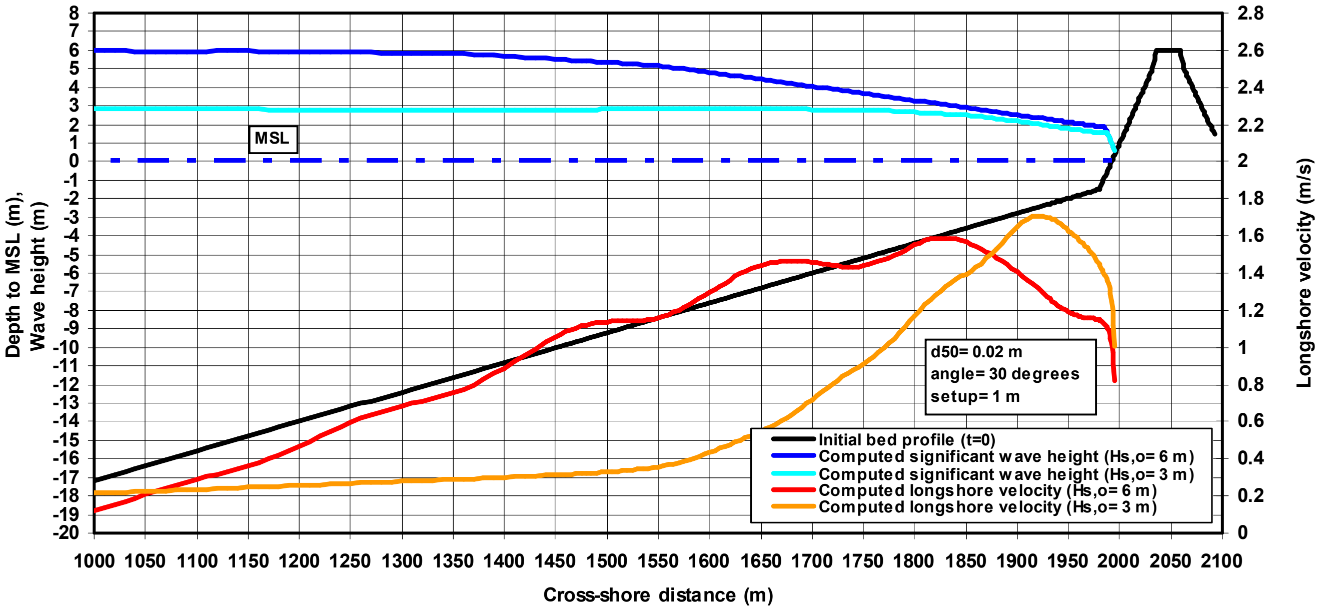

Figure 4.8 *Bed profile, wave height, longshore velocity for offshore wave height of $H_{s,o}$= 3 and 6 m; setup= 1 m; offshore wave incidence angle= 30^o for Pevensey Bay shingle barrier, UK*

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 4.8**. 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 4.9 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.

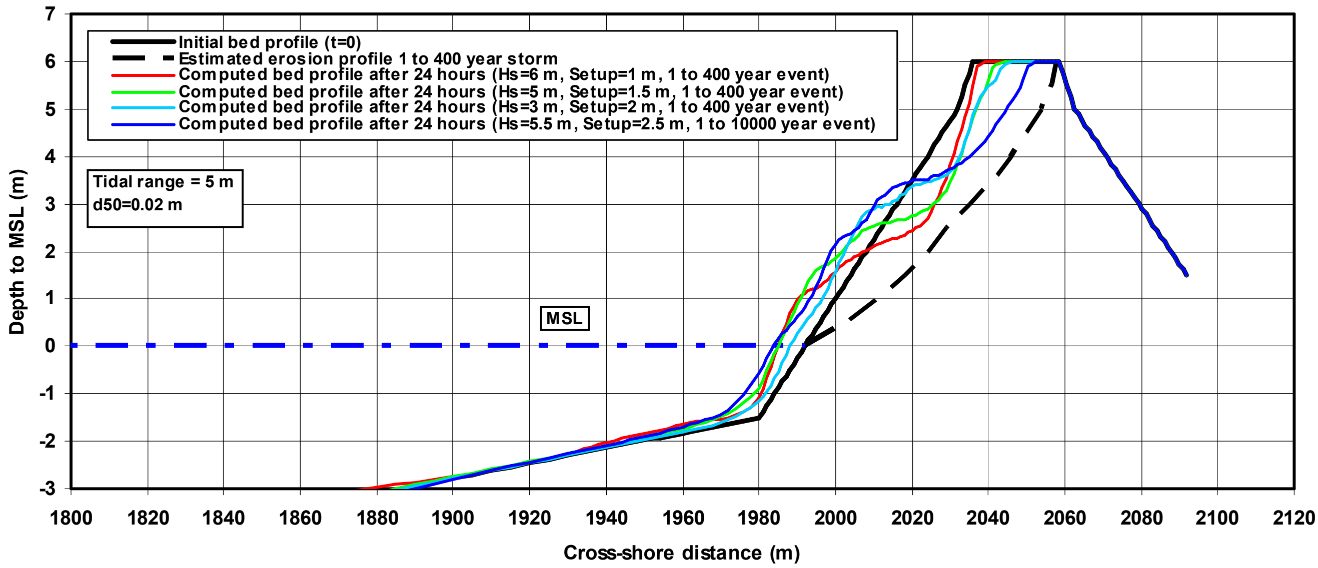

Figure 4.9 *Effect of storm surge level and storm intensity on the erosion of Pevensey Bay shingle barrier*

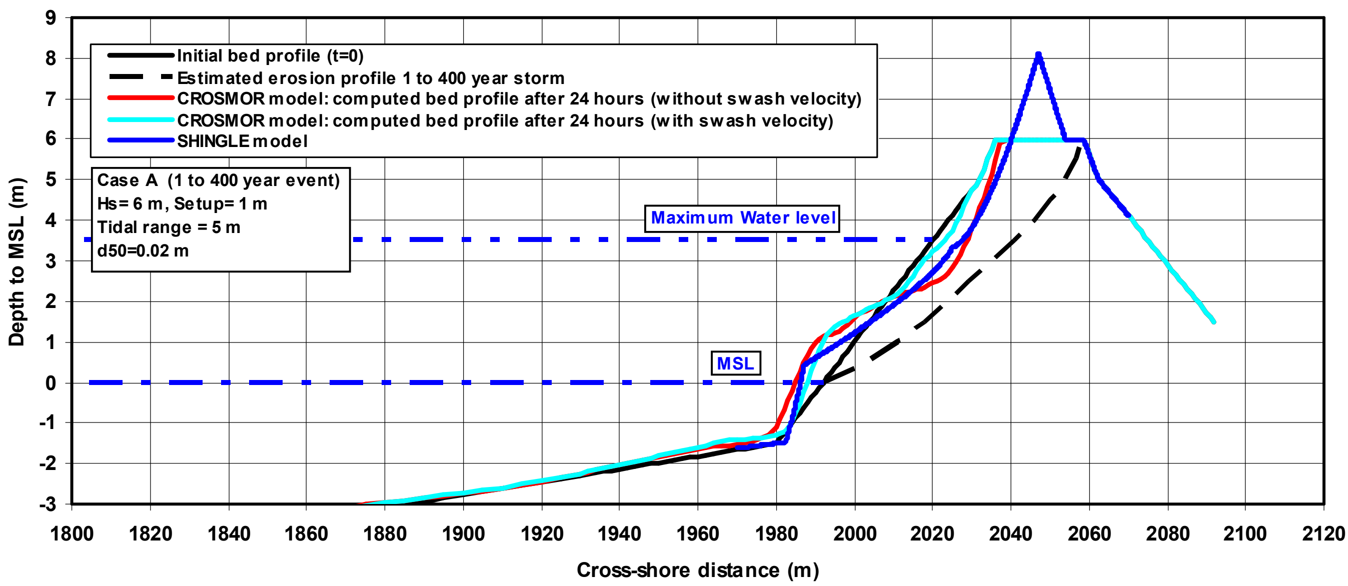

Figure 4.10A *Computed bed profile changes of CROSMOR and SHINGLE models for Case A, Pevensey Bay*

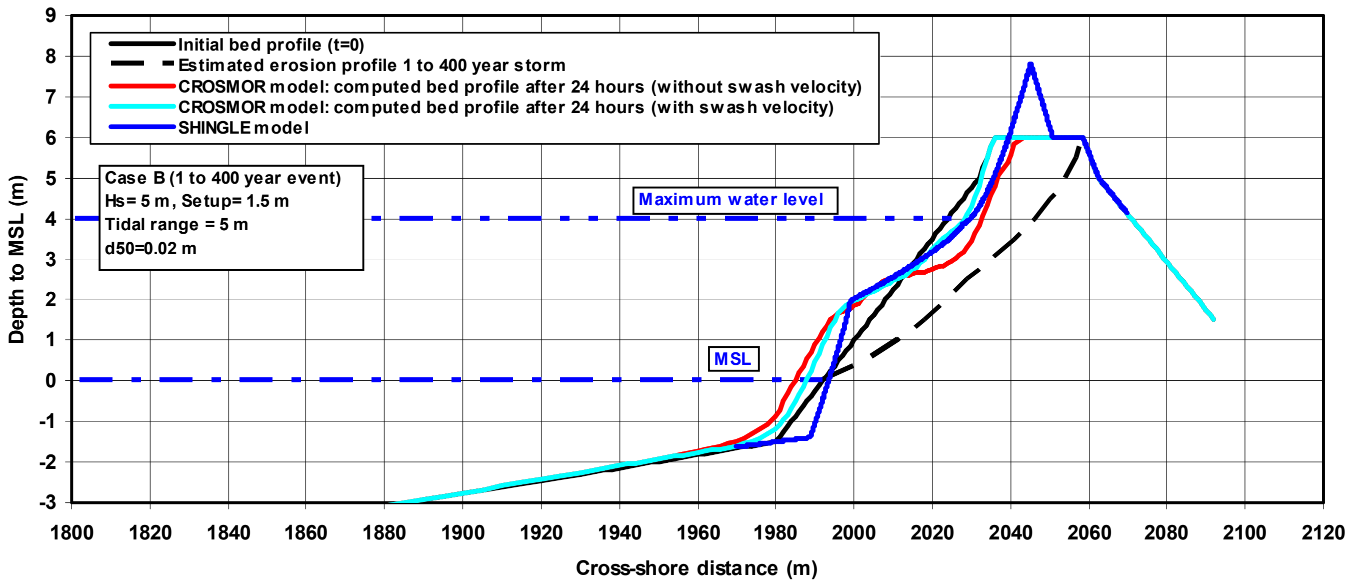

Figure 4.10B *Computed bed profile changes of CROSMOR and SHINGLE models for Case B, Pevensey Bay*

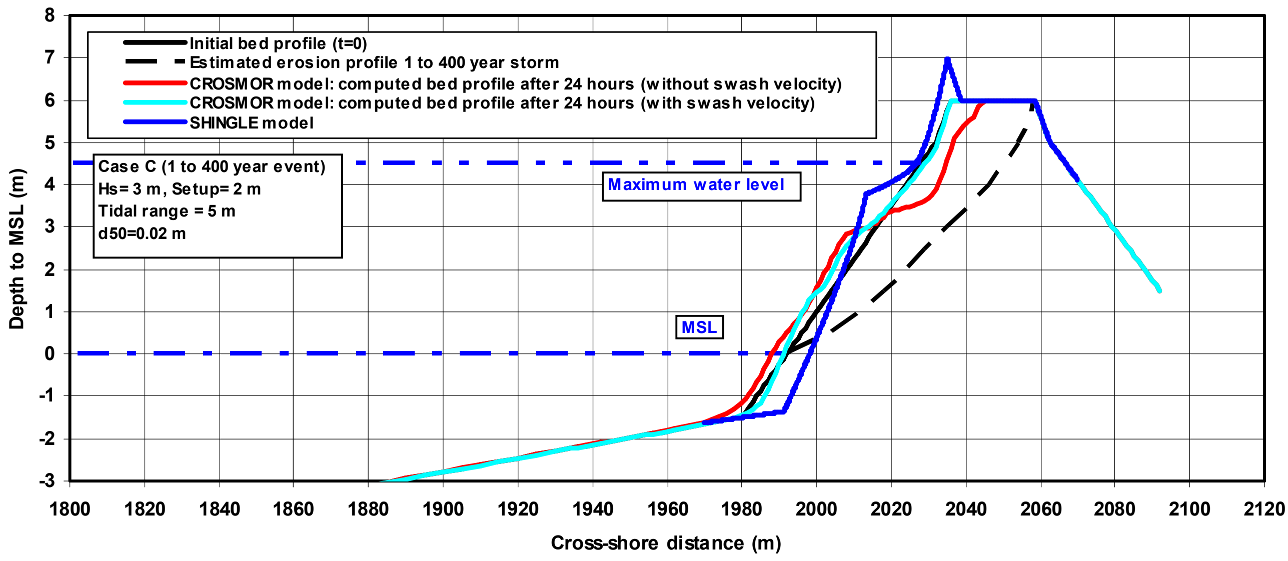

Figure 4.10C *Computed bed profile changes of CROSMOR and SHINGLE models for Case C, Pevensey Bay*

Figures 4.10A,B,C,D show the bed profile changes based on the process-based CROSMOR-model and the parametric SHINGLE-model of HR Wallingford for each case. 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 $2.5H_{s,toe}$), which is rather large compared to the results of the BARDEX-experiments. The maximum crest level in these experiments is about 1.5 to $2H_{s,toe}$ above the HW level. The erosion area between the mean sea level and the crest computed by both models is almost equal.

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.

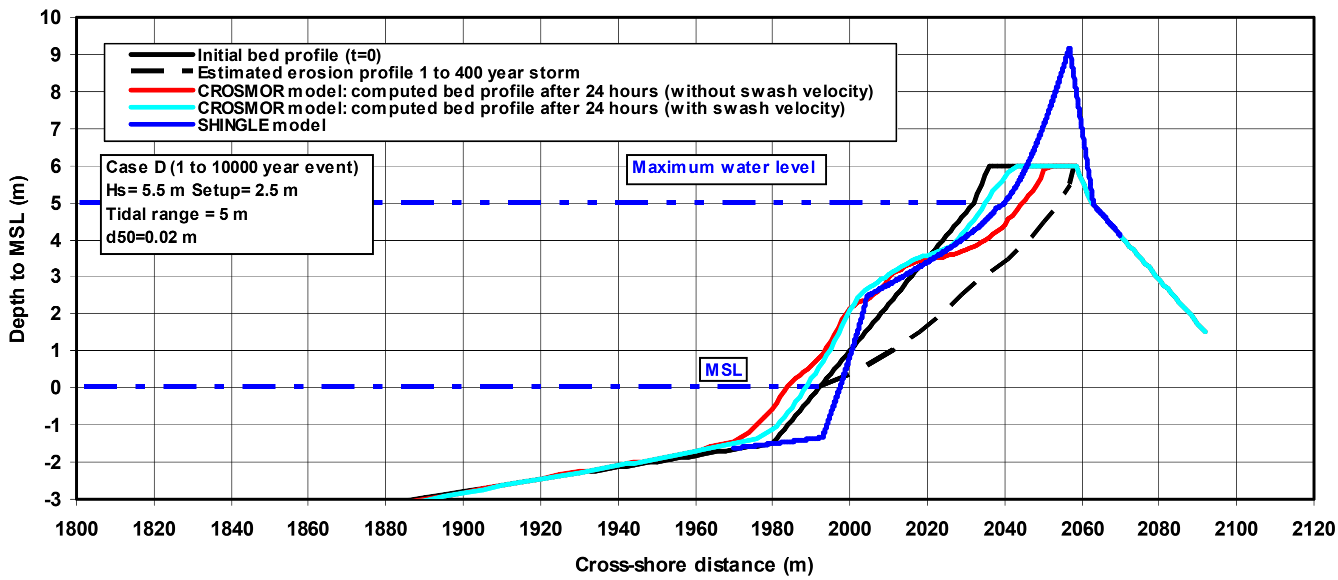

Figure 4.10D *Computed bed profile changes of CROSMOR and SHINGLE models for Case D, Pevensey Bay*

Figure 4.11 shows the effect of the tidal range (varied from 0 to 6 m) based on the CROSMOR-model. The total erosion area after 24 hours shows a marginal increase from about 20 m^3/m for a tidal range of 0 m tot about 25 m^3/m for a tidal range of 6 m. The erosion zone is located at a higher level along the profile for increasing tidal range. The horizontal crest recession also is largest (about 10 m) for the largest tidal range.

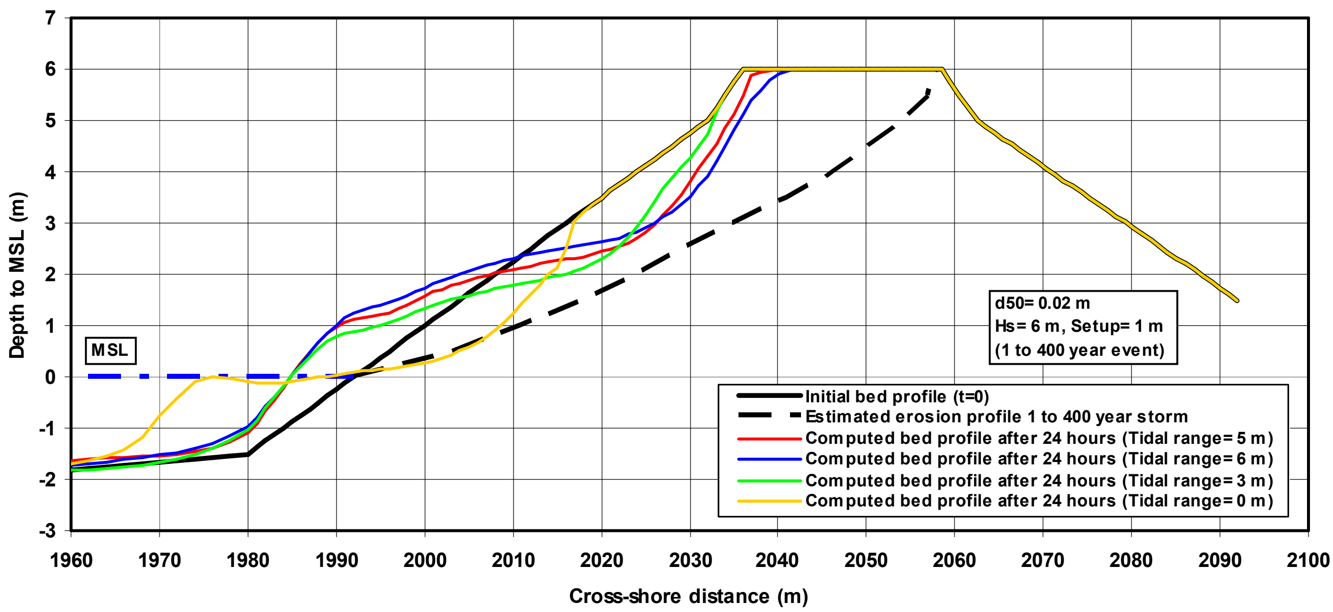

Figure 4.11 *Effect of tidal range on the erosion of Pevensey Bay shingle barrier, UK*

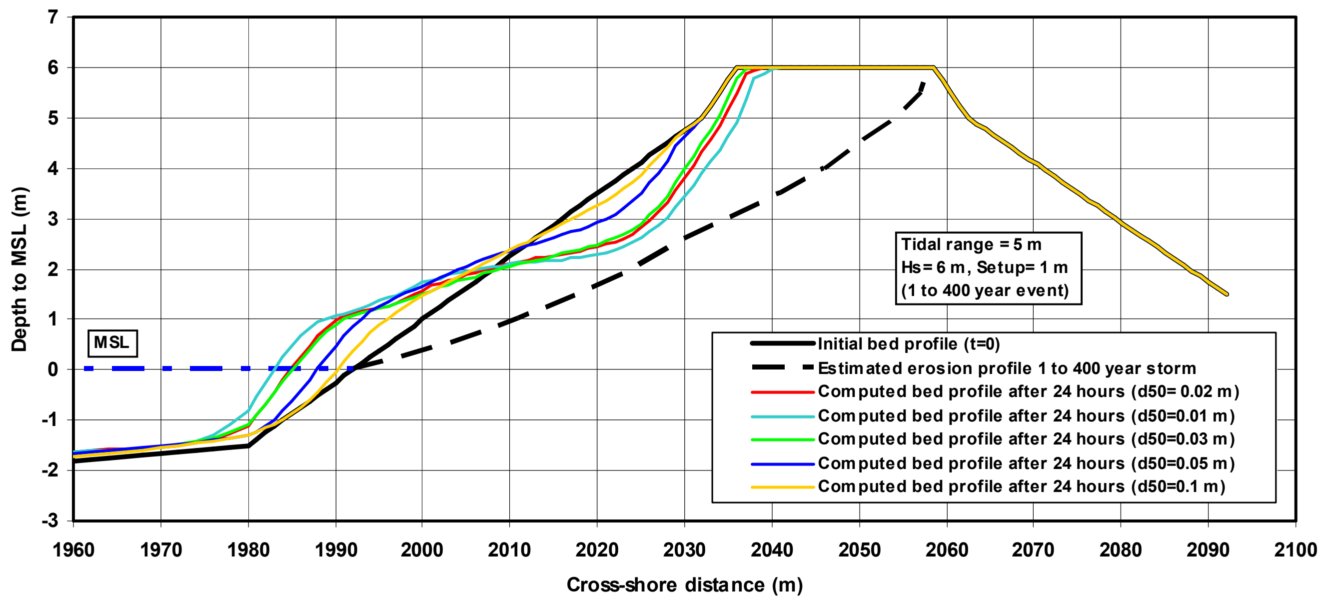

Figure 4.12 *Effect of shingle size on the erosion of Pevensey Bay shingle barrier, UK*

Figure 4.12 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).

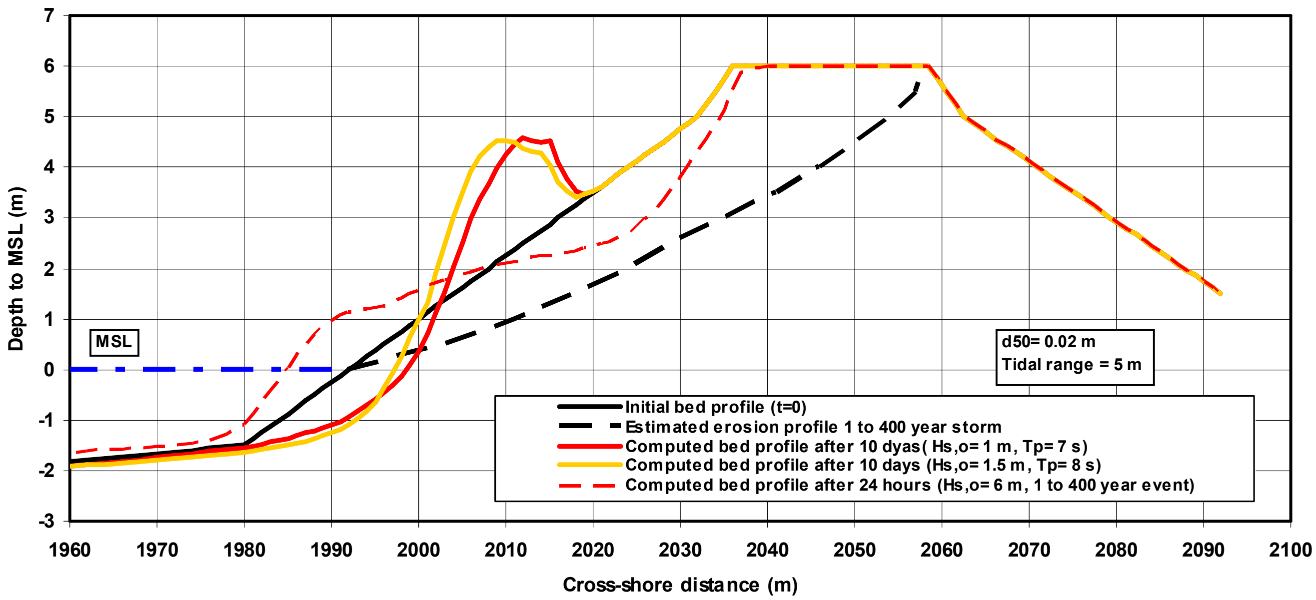

Figure 4.13 *Accretion of shingle barrier during low wave conditions at Pevensey Bay, UK*

Figure 4.13 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 to the slope of the barrier by wave run-up processes which are somewhat stronger for higher waves. It will take some weeks with low waves for the shingle barrier to recover from the erosion (about 25 m^3/m) during 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.

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. Three storm events are considered: set-up = 0.5 m with $H_{s,o}$ = 4 m (T_p = 9 s); set-up = 1 m with $H_{s,o}$ = 4.5 m (T_p = 9.5 s), set-up= 2 m with $H_{s,o}$= 5 m (T_p = 10 s) and set-up = 3 m with $H_{s,o}$ = 6 m (T_p = 11 s). The wave incidence angle is 30^o to the shore normal. The shingle size is varied in the range of 0.01 to 0.1 m. The tidal range is 5 m for all events. To obtain a conservative estimate of the erosion volume, the undertow velocities near the beach are 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 4.14**. 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. This plot can be used to get a first estimate of the erosion of gravel/shingle barriers.

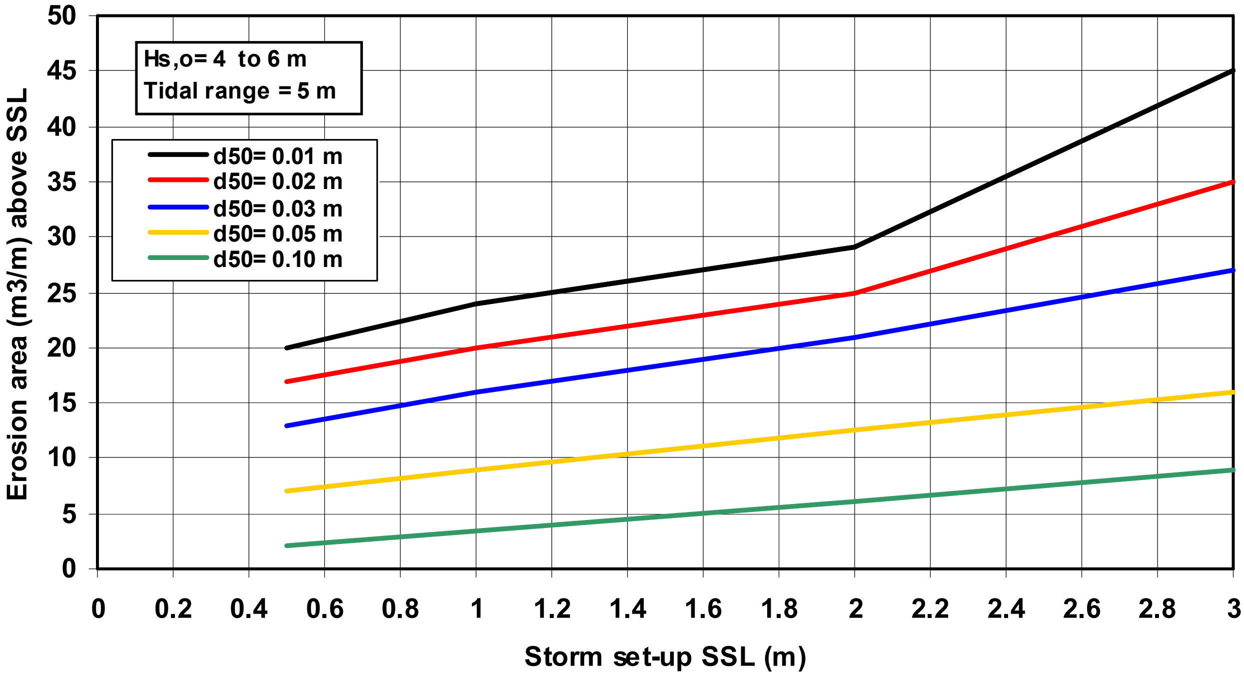

Figure 4.14 *Erosion area (after 24 hours) as function of storm set-up and shingle/cobble size*

4.3.2 Schematized high shingle barrier (no overwash)

The CROSMOR2008-model has also been applied to a schematized field case with a high shingle barrier consisting of d_{50}= 0.02 m and a beach slope of 1 to 8 with a crest level at 10 m above MSL (mean sea level). The shoreface is assumed to have a relatively steep slope of 1 to 20. The profile characteristics and boundary conditions used are given in **Table 4.3**. The tidal range is 4 m.

Parameter	Values
Bed profile	slope of 1 to 20 between -30 m and -3 m slope of 1 to 8 between -3 m and +10 m
Sediment d_{50} **d_{90}**	0.02 m 0.04 m
Bed roughness k_s	0.04 m
Horizontal mixing	0.1 m^2/s
Peak tidal water level	2 m (flood); -2 m (ebb)
Peak tidal velocity	0.6 m/s (flood); -0.6 m/s (ebb)
Offshore significant wave height $H_{s,o}$	2, 3, 4, 5, 6, 8 m (6 wave classes using Rayleigh distribution)
Peak wave period T_p	7, 8, 9,10, 12, 15 s
Storm surge level above MSL	0, 0.5, 1, 2, 3, 4 m

Table 4.3 *Data of field case of shingle barrier*

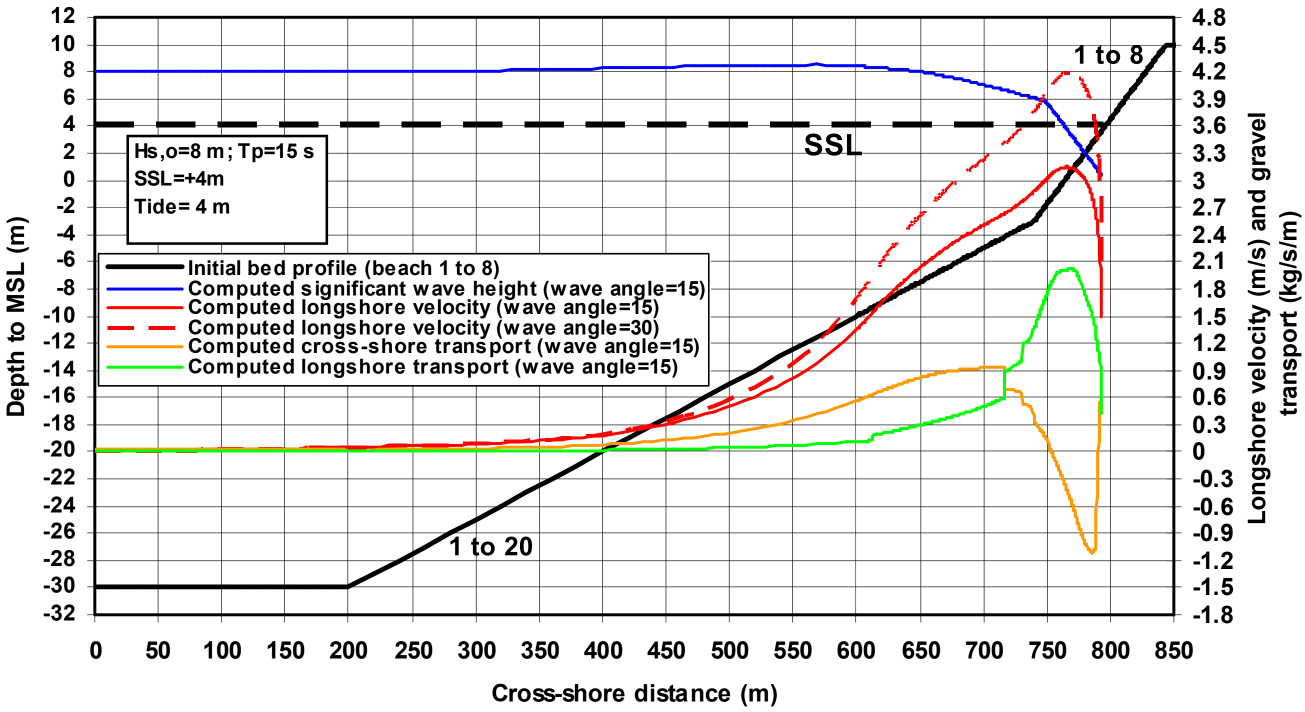

Figure 4.15 *Bed profile, wave height, longshore velocity and shingle transport for offshore wave height of $H_{s,o}$= 8 m and offshore wave incidence angles of 15° and 30°*

Figure 4.15 shows the cross-shore distribution of the significant wave height for superstorm conditions with an offshore wave height of $H_{s,o}$= 8 m (T_p = 15 s) and an offshore wave incidence angle of 0 (waves normal to the coast). The storm surge level is set to 4 m above mean sea level. The wave height is almost constant up to the depth contour of -7 m. Landward of this depth the wave height gradually decreases to a value of about 6 m at the toe of the beach (at x= 740 m). At the beach slope (1 to 8) the wave height strongly decreases from 6 m to about 1 m at the landward end of the gravel beach. Wave-induced longshore velocities are shown in **Figure 4.15** for offshore wave incidence angles of 15° and 30°. The longshore velocity increases strongly landward of the -15 m depth contour where wave breaking becomes important (5% wave breaking at -15 m, 40% breaking at -9 m, 70% breaking at -5 m; 100% breaking at -2 m). The longshore current velocity has a maximum value of 3 m/s for an offshore wave angle of 15° and 4 m/s for an angle of 30°, just landward of the toe of the beach slope.

Figure 4.15 also shows the cross-shore distribution of the computed cross-shore and longshore transport of shingle (mainly bed load transport) for an offshore wave incidence angle of 15°. The cross-shore transport shows onshore values at the shoreface up to the toe of the shingle beach (-3 m depth line) and at the most landward end of the beach (swash bar generation over length of a few meters). The cross-shore transport of shingle is seaward at the beach between x = 754 m and x = 791 m. The longshore transport increases strongly landward of -4 m line and is maximum at the location where the longshore velocity is maximum (around 0 m depth line).

Offshore significant wave height $H_{s,o}$ (m)	Peak wave period T_p (s)	Storm surge level above Mean Sea Level (m)	Computed Longshore transport (kg/s)	(m^3 incl.pores/day)
0.7	5	0	0.11	6
1	6	0	0.21	11
2	7	0	0.75	40
3	8	0.5	4.1	220
4	9	1	13	700
5	10	2	29	1565
6	12	3	56	3025
8	15	4	162	8750

Table 4.4 *Integrated longshore transport (d_{50}=0.02 m)*

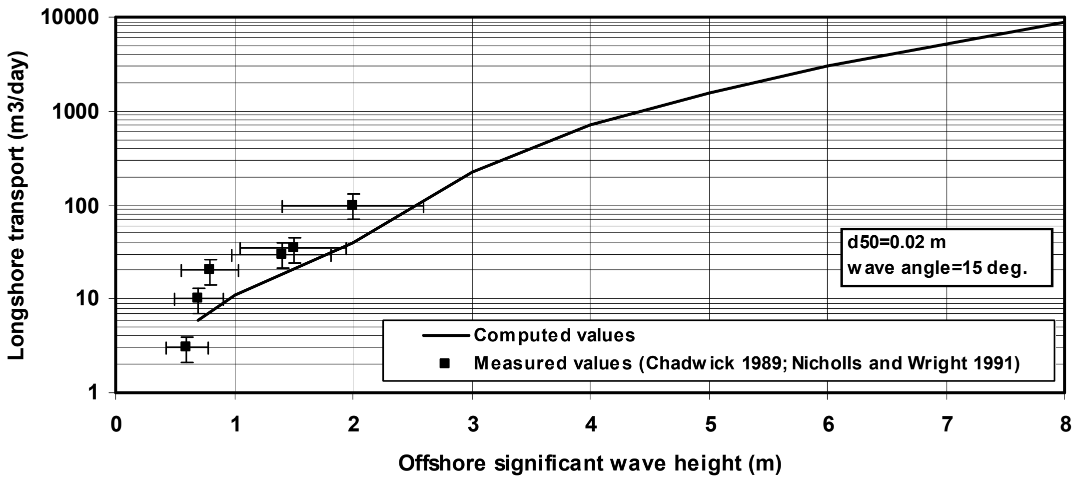

Figure 4.16 *Longshore transport as function of offshore wave height*

Figure 4.16 shows the computed values (in m^3/day) of the longshore transport integrated over the cross-shore profile (see also **Table 4.4**) for offshore wave heights between $H_{s,o}$= 0.7 and 8 m. The computed values vary roughly between 5 m^3/day and 9000 m^3/day (including pores). About 80% of the longshore transport occurs in the surf zone landward of the -6 m depth contour and about 70% landward of the -4 m depth line. Assuming a North Sea wave climate with waves from one main direction between 0 and 30 degrees (75 days H < 1 m; 50 days H= 1-2 m; 20 days H= 2-3 m and 5 days H > 3 m), the weighted total longshore transport over 150 days is about 20.000 to 30.000 m^3. Measured longshore transport rates based on the work of **Chadwick (1989)** and **Nicholls and Wright (1991)** are also shown in **Figure 4.16** assuming that the offshore significant wave height is twice the observed nearshore breaking wave height. The computed longshore transport rates (in m^3/day) roughly are a factor of 2 to 3 too small for low wave conditions. The measured values essentially represent the longshore transport of shingle/gravel in the swash zone (wave uprush and downrush zone). It should be realized that this zone is represented rather crudely using a subgrid model approach. The hydrodynamics in the swash zone are not modelled explicitly.

Morphological simulation results are shown in **Figures 4.17 to 4.19**.

Figure 4.17 shows the effect of the offshore wave angle on the cross-shore bed surface profile for a superstorm with $H_{s,o}$= 8 m. Bed level changes are relatively large for waves oblique to the shore (15° and 30°) due to the presence of relatively large wave-induced longshore velocities. A small swash bar is generated at the landward end of the beach (above +6 m line) and a breaker bar is generated at the toe of the beach (below the +1 m line). The total erosion volume around the water line (around storm surge level line) is about 20 to 25 m^3/m after 1 day. The total volume of the breaker bar is about 50 m^3/m after 1 day due to erosion of the beach (seaward transport) and erosion of the shoreface (landward transport).

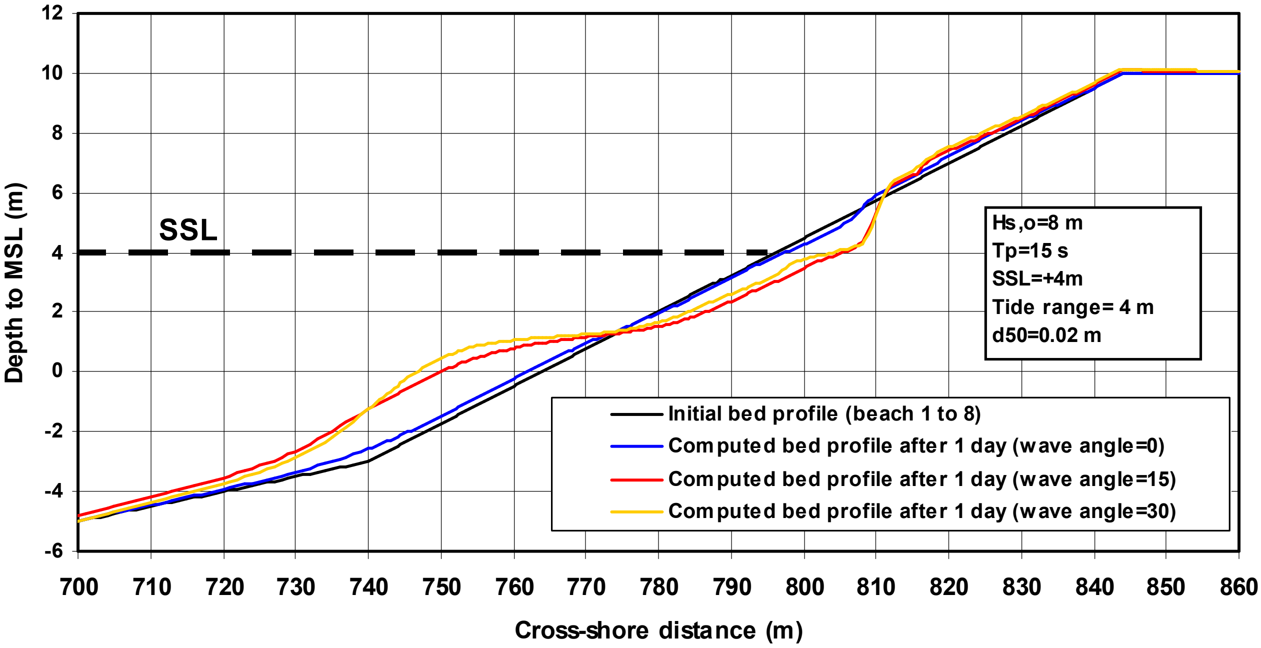

Figure 4.17 *Bed profile changes after 1 day for $H_{s,o}$= 8 m and wave incidence angles of 0, 15° and 30°*

Figure 4.18 shows the cross-shore bed surface profile after 1 day for offshore wave heights between 2 and 8 m. A relatively large swash bar with a volume of about 30 m^3/m is generated due to onshore transport. The shingle is eroded from a zone directly seaward of the water line. For increasing wave heights (and increasing storm surge levels) the swash bar is located at higher beach levels and reduces in size (about 10 m^3/m for $H_{s,o}$= 8 m). A breaker bar is generated for offshore wave heights larger than about 4 m. The maximum beach erosion under superstorm conditions is of the order of 25 to 30 m^3/m after 1 day. The crest level of the shingle slope should be at least 10 m to prevent overwashing of the crest under superstorm conditions.

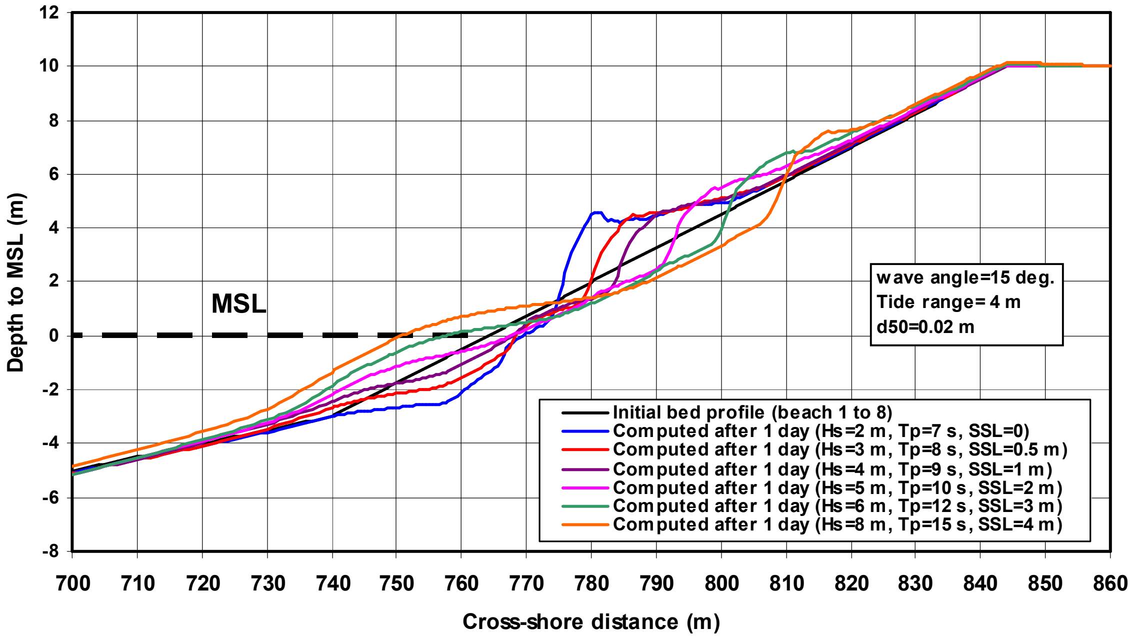

Figure 4.18 *Bed profile changes after 1 day for $H_{s,o}$= 2 to 8 m and wave incidene angle of 15°*

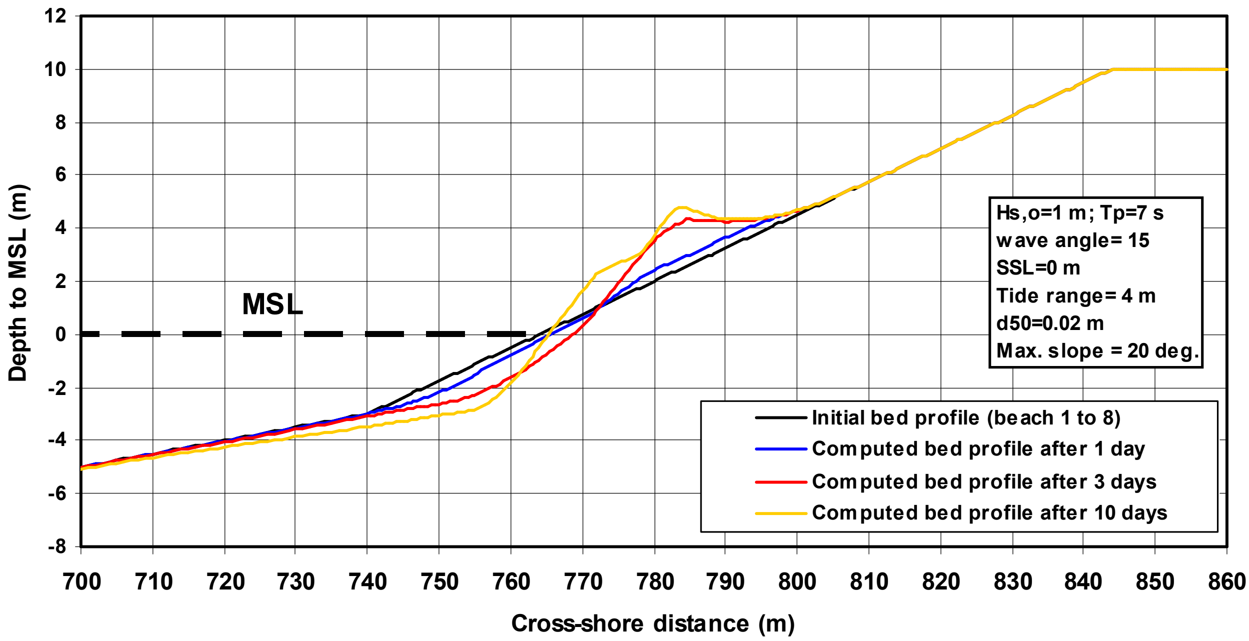

Figure 4.19 *Bed profile changes for $H_{s,o}$= 1 m and wave incidence angle of 15°*

Figure 4.19 shows bed profile changes after 1 to 10 days for low wave conditions ($H_{s,o}$= 1 m) with onshore transport (sef = 1, c_{LH} = 0.5, c_{SW} = 0.5). The maximum beach slope is set to 20°. Beach sliding will occur for slopes larger than this value. The model produces a distinct swash bar with a crest level at about 4 m. The swash bar volume increases in size as a function of time. The swash bar volume is about 7 m^3/m after 1 day and about 40 m^3/m after 10 days. The gravel is eroded from a zone landward of the -4 m depth line.

Parameter	Values
Bed profile	slope of 1 to 20 between -30 m and -3 m slope of 1 to 8 between -3 m and +4 m crest level at +4 m; crest width=3 m landward slope of 1 to 5; land surface at MSL
Sediment d_{50} **d_{90}**	0.02 m 0.04 m
Bed roughness k_s	0.04 m
Horizontal mixing	0.1 m^2/s
Peak tidal water level	0
Peak tide-induced and wind-induced velocity	1 m/s
Offshore significant wave height $H_{s,o}$	8 m (6 wave classes using Rayleigh distribution)
Peak wave period T_p	15 s
Storm surge level above MSL	4.5 and 5 m
Overwash discharge	0.5 to 2 m^2/s

Table 4.5 *Data of field case of shingle barrier with overwash*

4.3.3 Schematized low gravel barrier (with overwash)

Shingle barriers with a crest level at 3 to 6 m above MSL (or better HW level) are vulnerable to overwash which may easily result in landward migration (roll back) during storm conditions with surge levels of 2 to 3 m above MSL and a tidal range of 3 to 6 m (see **Figure 4.20**).

The CROSMOR-model has been used for exploring computations of crest erosion and landward migration of a low shingle barrier with crest level at 4 m above MSL (see **Figure 4.20**). The profile characteristics and boundary conditions are given in **Table 4.5**. Two cases are considered with a surge level at 4.5 m above MSL (water depth above crest = 0.5 m) and at 5 m above MSL (water depth above crest = 1 m). The overwash discharge has been varied in the range of 0.5 to 2 m^2/s by specifying a small velocity at the inflow boundary (x = 0) at a depth of 30 m below MSL. The fluid velocity above the crest is in the range of 0.5 to 2 m/s.

Figure 4.20 shows the computed significant wave height, the cross-shore and longshore velocities and the bed level changes after 1 day along the profile for a storm surge level of SSL = 4.5 m including tidal elevation (water depth above the crest = 0.5 m) and an overwash discharge of 1 m^2/s (velocity above crest = 2 m/s). The offshore wave incidence angle is set to 15^o. The computed significant wave height decreases from about 6 m at the toe of the beach to about 0.3 m (for h_{crest} = 0.5 m) above the crest and remains constant at the landward side of the barrier. The cross-shore velocity is seaward-directed at the beach due to the generation of the undertow and changes to a strong landward-directed overwash flow above the crest (maximum value of 2 m/s).

The longshore velocity has its peak value (about 2.5 m/s) just seaward of the crest and decreases above the crest due to frictional effects (strong reduction of water depth). The longshore current velocity landward of the crest is strongly influenced by the wave breaking process. When the wave parameters are averaged over 1 wave length (1L; default approach) the longshore velocity first increases and then decreases when the wave breaking process decays. When the averaging process is set at 0.1L, the longshore velocity strongly decreases landward of the crest. The former approach is not realistic but the latter approach also is questionable as (breaching and) overwash is a very local process violating the longshore uniformity approach of the CROSMOR2008-model. The flow velocity landward of the crest will be three-dimensional due to spreading of the flow over the land surface.

The computed bed level changes after 1 day (24 hours) show crest erosion of the order of 1.2 m and landward barrier migration of the order of 3 m. The barrier migration reduces for a smaller overwash of 0.5 m^2/s (v_{crest} = 1 m/s). The crest lowering by erosion is not much influenced by the overwash discharge and is about 1.2 m after 1 day.

A depositional bar with a height of about 1 m and a length of about 30 m is formed at the beach, see **Figures 4.20 and 4.21**. This latter bar is mainly caused by onshore transport of shingle enhanced by relatively large longshore velocities (2.5 m/s) under oblique wave attack. When the wave incidence angle is set to 0, the bar formation is much less pronounced, as shown in **Figure 4.21**. The crest erosion and migration also are smaller for waves normal to the coast.

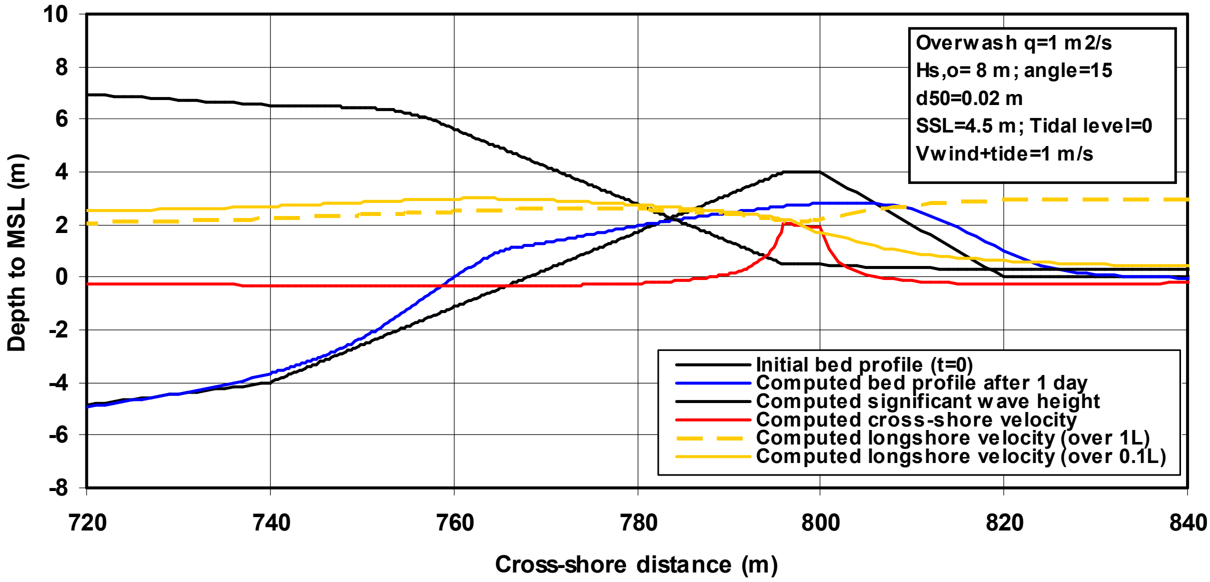

Figure 4.20 *Computed wave height, cross-shore and longshore velocity and bed level changes for storm surge level of 4.5 m (h_{crest}= 0.5 m) and overwash discharge 1 m^2/s (v_{crest}= 2 m/s)*

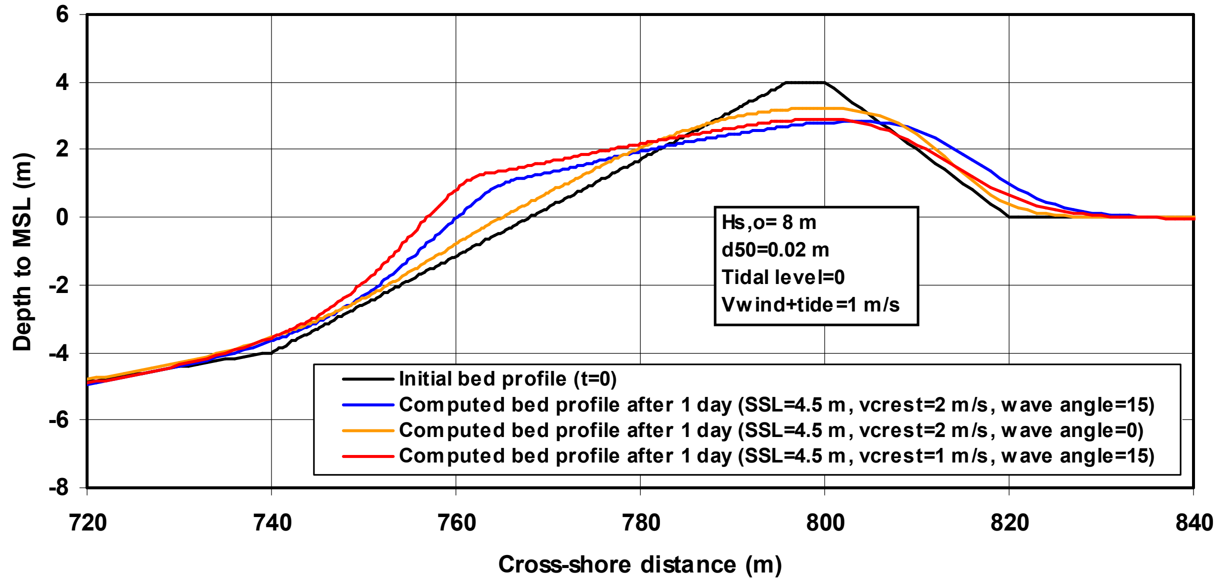

Figure 4.21 *Computed bed level changes for storm surge level of 4.5 m (h_{crest}= 0.5 m) and overwash discharge 0.5 and 1 m^2/s (v_{crest}= 1 to 2 m/s)*

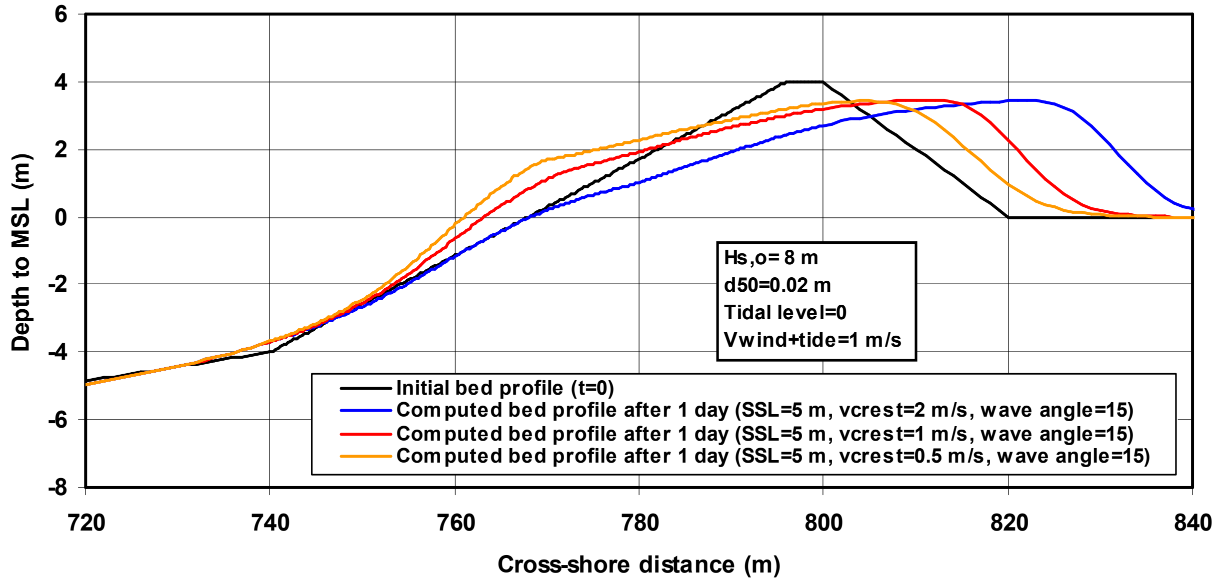

Figure 4.22 *Computed bed level changes for storm surge level of 5 m (h_{crest}= 1 m) and overwash discharge 0.5, 1 and 2 m^2/s (v_{crest}= 0.5 to 2 m/s)*

Figure 4.22 shows computed bed levels after 1 day for a storm surge level of 5 m (h_{crest} = 1 m) and overwash discharges of 0.5, 1 and 2 m^2/s (v_{crest} = 0.5 to 2 m/s). The crest erosion is about constant (about 0.6 m) for all three cases. The crest migration increases strongly with increasing flow velocity above the crest. Under extreme conditions the maximum crest migration is about 20 m/day. The probability of occurrence of a storm surge level of 5 m above mean sea level is of the order of 10^{-4} (once in 10,000 years). These high crest migration values are exceptional, but not irrealistic in comparison with the data of Hurst beach and Hurst spit (see **Figure 3.4**) where crest migration values of 20 m in one storm event and long term values of 10 m over a period of about 2 years have been observed. Measured vertical crest erosion values are in the range of 0.5 to 1.5 m at Hurst beach and spit.

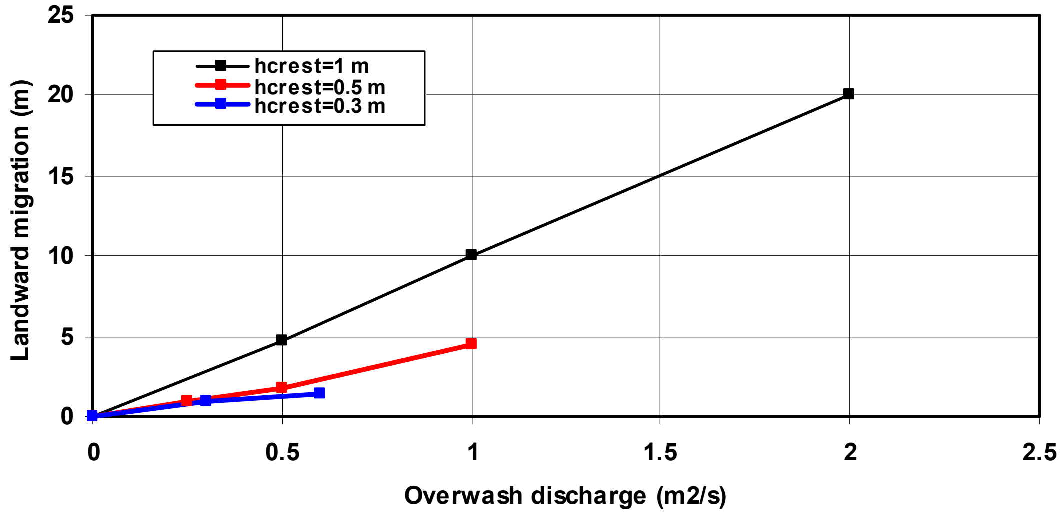

Figure 4.23 *Landward migration of gravel barrier as a function of overwash discharge and the initial water depth at crest during an extreme storm event (duration of 24 hours)*

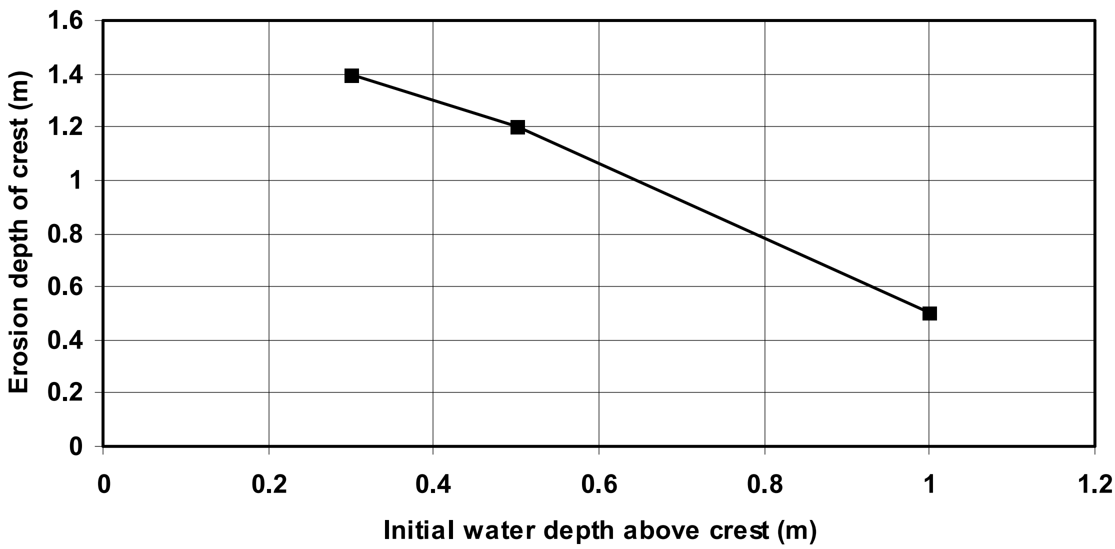

Figure 4.24 *Erosion of barrier crest as a function of the initial water depth at crest during an extreme storm event (duration of 24 hours)*

All computational results on crest erosion and barrier migration are summarized in **Figures 4.23** and **4.24**. **Figure 4.23** shows the barrier migration as a function of the overwash discharge and the initial water depth at the crest of 0.3, 0.5 and 1 m. The migration increases with increasing initial crest depth (and thus larger wave height) and with increasing overwash discharge. Extreme values are as large as 20 m/day.

Figure 4.24 shows the crest erosion as a function of the initial water depth at the crest. The crest erosion decreases from about 1.5 m to 0.5 m for a water depth at the crest increasing from 0.3 m to 1 m. The overwash discharge has not much effect on the erosion depth.

4.3.4 Coastal protection of sandy dunes using gravel/shingle material

The 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 (**Vellinga, 1986; Steetzel, 1993; Van Rijn, 2008**).

To show the reduction of coastal erosion using a protection layer of coarse shingle material on top of the sand surface, the CROSMOR-model has been applied to a typical cross-shore profile along the Dutch coast. The profile characteristics and boundary conditions are given in **Table 4.6**.

Figure 4.25 shows computed bed profiles after a storm duration of 5 hours for a case with a storm surge level at 5 m above MSL using sandy material (d_{50}= 0.000225 m; 0.225 mm) and shingle material (d_{50}= 0.02 m; 20 mm). The dune erosion is of the order of 170 m^3/m for the sandy case and only 15 m^3/m for the shingle case. The maximum horizontal recession for the shingle case is of the order of 2 m (corresponding to a value of about 0.7 m normal to the barrier slope of 1 to 3). The erosion values will be somewhat larger for oblique incoming waves due to the generation of longshore velocities. When cobbles ($\cong$ 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. The toe of the layer should extend to below the low water mark (-2 m below MSL) resulting in a total length of about 100 m assuming a beach slope of 1 to 8 between the -2 m and +3 m line and a barrier slope of 1 to 3. Hence, a total volume of about 200 to 300 m^3/m is required per m shoreline or 200,000 to 300,000 m^3 per km. This solution may be attractive at specific locations (near structures, harbours, inlets, etc.) where the erosion processes of the sandy dune system are excessively large resulting in relatively large maintenance costs (nourishments costs).

Parameter	Values	
Bed profile	slope of 1 to 180 between -30 m and -3 m slope of 1 to 70 between -3 m and 0 m slope of 1 to 20 between 0 m and +3 m slope of 1 to 3 between +3 m and +15 m	
Sediment **d_{50}** **d_{90}**	Sand 0.000225 m 0.000450 m	Shingle 0.02 m 0.04 m
Bed roughness k_s	0.001 m	0.04 m
Horizontal mixing	0.5 m^2/s	
Peak tidal water level	0 m	
Peal tidal velocity	0.5 m/s (flood); -0.5 m/s (ebb)	
Offshore significant wave height $H_{s,o}$	7.6 m (6 wave classes using Rayleigh distribution)	
Offshore wave incidence angle	0^o	
Peak wave period T_p	12 s	
Storm surge level above MSL	5 m	

Table 4.6 *Data of Dutch Reference Storm Case*

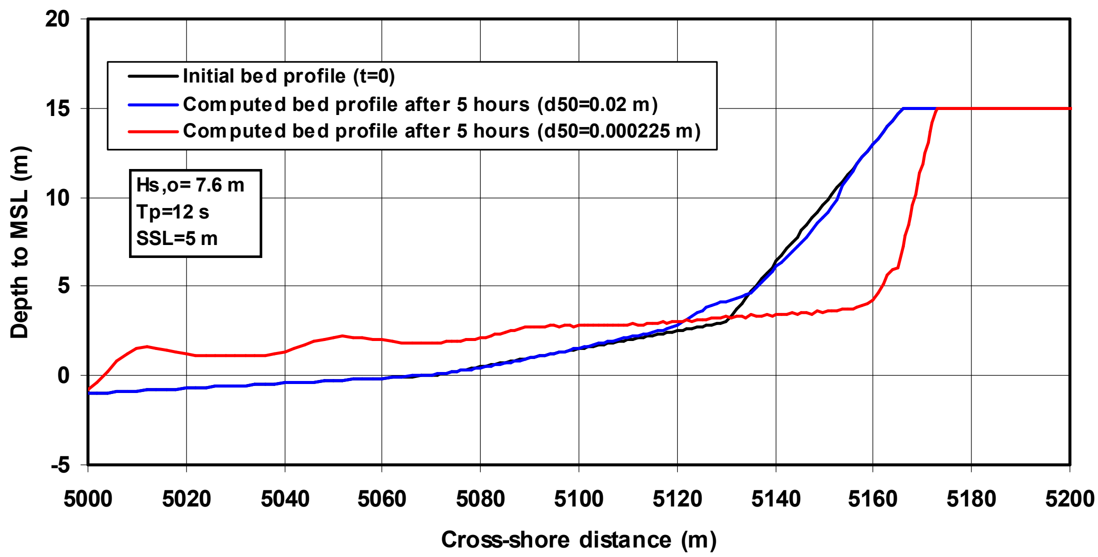

Figure 4.25 *Computed bed profiles after 5 hours for a storm event using d_{50}=0.000225 m and d_{50}=0.02 m for Dutch Reference Case*

5 Summary and conclusions

Beaches consisting of gravel (2 to 64 mm), pebbles and cobbles (64 to 256 mm) are generally known as *coarse clastic beaches* and can be found in many mid- and high-latitude parts (formerly glaciated) of the world (England, Iceland, Canada, etc.). Gravel beaches are also found along unconsolidated cliff-type coasts eroded by wave attack (Mediterranean coasts) and along tectonic coasts where steep streams deliver coarse material to the shore. Some of these beaches have a large proportion of sand intermixed with gravel, especially in the foreshore zone.

Gravel beaches are also known as shingle beaches or coarse clastic beaches. Clasts are individual grains within coarse populations. Subgroups are pebbles and cobbles (rounded clasts between 64 and 256 mm); boulders are clasts larger than 256 mm. The term shingle is most commonly identified with the coarse beaches of southern England.

Gravel on beaches is moved almost exclusively by wave action (asymmetric wave motion); tidal or other currents are not effective in moving gravel material.

The gravel particles move up the beach to the run-up limit by strong bores (uprush) and 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. Gravel particles in shoaling and breaking waves generally move as bed load. 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. The finer grains may go into suspension as a result of the turbulence produced by the breaking waves and may be transported offshore or inshore depending on the strength of the undertow.

Gravel transport mainly takes place in the swash zone. The swash zone is the zone which is intermittently wet and dry showing relatively large velocities during the uprush and backwash phases of the saw-tooth swash wave cycle due to bore propagation and bore collapse, often in combination with low-frequency oscillations which generally grow in amplitude towards the shoreline. It is a particularly complex zone of the nearshore where short and long waves, tides, sediments and groundwater flow (infiltration/percolation) all play an important role. The swash zone is the most dynamic part of the nearshore zone of vital importance for the behaviopur of the gravel/shingle barrier.

Wave-induced run-up is caused by two different processes: set-up, which is the maximum time-averaged water level elevation at the shoreline and swash oscillations, which are the time-varying vertical fluctuations about the temporal mean value (set-up water level). Wave run-up along steep sloping gravel barriers can be estimated using the experimental data of laboratory and field experiments.

Swash velocity measurements show that the swash related to the incident waves on steep beaches is skewed and asymmetric (saw-tooth waves), i.e. the backwash is not simply the reverse of the uprush. Generally, onshore flow velocities during the uprush are larger but of shorter duration than the seaward velocities during the backwash. Maximum landward velocities occur at the start of the uprush, whereas maximum seaward velocities take place at the end of the backwash. The water depths that occur during the uprush are generally larger than those that occur during the backwash.

The dissimilarity in the hydrodynamics of the wave uprush and backwash is reflected in different modes of sediment transport. Turbulence-dominated suspended transport may be significant during the uprush phase whereas sheetflow type of bed load transport dominates during the backwash phase. During the uprush phase the sediment transport is a combination of sediments mobilised under and directly after bore collapse which are then advected landwards and of locally entrained sediments from the bed by developing boundary layer flow at the end op the uprush, whereas sediment transport during downrush mainly is related to locally entrained sediments. Measurements of sheet flow transport for half saw-tooth waves in a wave tunnel indicate that the sediment transport under steep fronts (decelerating flow) is about twice as large as under steep rears (accelerating flow).

Swash motion over a steep permeable bed of coarse grains (gravel/shingle) is complicated by the presence of infiltration under wave uprush and exfiltration under wave downrush. Vertical flow through a porous bed can influence sediment motion in two ways: 1) seepage forces changing the effective weight of the surficial sediments and 2) the occurrence of boundary layer thinning (resulting in higher shear stresses) due to infiltration and thickening (smaller shear stresses) due to exfiltration. Generally, swash-related infiltration-exfiltration effects across a saturated beach face enhance the upslope transport of sediment transport.

Various experiments on the behaviour of gravel 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). Gravel and shingle material 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. The most characteristic features are: the formation of a swash bar above the still water level (SWL) due to onshore transport; the formation of a small breaker bar; the generation of a scour pit below SWL and the presence of a small transition zone direct above and beneath SWL with almost no deformation. Similar tests have been performed in the GWK (Hannover, Germany). The BARDEX-experiments in the Deltaflume of Deltares (2008) include time-varying water levels to simulate tidal variations and wave overtopping/overwashing to simulate crest erosion of the barrier.

Two models (process-based CROSMOR2008-model and parametric SHINGLE-model) have been used to simulate the cross-shore swash bar formation under low wave conditions and gravel barrier erosion under high wave conditions (storm events). The SHINGLE-model is a parametric profile model based on shape functions. The process-based CROSMOR-model describes the propagation and transformation of individual waves (wave by wave approach) along the cross-shore profile using a probabilistic approach by solving the wave energy equation for each individual wave. The detailed swash processes in the swash zone are not explicitly modelled but are represented in a schematized way by introducing a time-averaged effective swash velocity in a small zone just seaward of the last grid point. The swash velocity is of the order of 1 to 1.5 m/s. The deposition (or erosion) profile in the swash zone is assumed to have a triangular shape.

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. The SHINGLE-model predicts a relatively large build-up of the crest. The agreement of computed profiles for the other storm cases with smaller offshore wave heights is less good.

The CROSMOR-model has also been applied to a schematized field case with a relatively steep nearshore slope. Wave-induced longshore velocities are relatively large for oblique wave approach along the steep slope. The longshore current velocity has a maximum of 3 m/s for an offshore wave angle of 15^o and 4 m/s for an angle of 30^o, just landward of the toe of the beach slope. The longshore transport increases strongly in the nearshore zone and is maximum at the location where the longshore velocity is maximum (around 0 m depth line). The computed longshore transport rates vary roughly between 5 m^3/day and 9000 m^3/day (including pores) for offshore wave heights between 1 and 8 m. About 80% of the longshore transport occurs in the surf zone landward of the -6 m depth contour and about 70% landward of the -4 m depth line.

Morphological simulation results for superstorm conditions (offshore wave height of 8 m) show the generation of a small swash bar at the landward end of the beach (above +6 m line) and a breaker bar at the toe of the beach (below the +1 m line). The total erosion volume around the water line (=around storm surge level line) is about 20 to 25 m^3/m after 1 day. The total volume of the breaker bar is about 50 m^3/m after 1 day due to erosion of the beach (seaward transport) and erosion of the shoreface (landward transport). The crest level of the gravel slope should be larger than about 10 m to prevent overwashing of the crest under superstorm conditions.

Shingle barriers with a low crest level at 3 to 4 m above MSL are vulnerable to overwash which may easily result in landward migration (roll back) during storm conditions with surge levels of 3 to 4 m above MSL. The CROSMOR-model has been used for exploring computations of crest erosion and landward migration of a gravel barrier with a low crest level at 4 m above MSL. The overwash discharge has been varied in the range of 0.5 to 2 m^2/s by specifying a small velocity at the inflow boundary (x= 0) at a depth of 30 m below MSL. The fluid velocity above the crest is in the range of 0.5 to 2 m/s.

The longshore velocity has its peak value (about 2.5 m/s) just seaward of the crest and decreases above the crest due to frictional effects (strong reduction of water depth). The computed bed level changes after 1 day (24 hours) show crest erosion of the order of 1 m and landward barrier migration of the order of 3 m. The barrier migration reduces for a smaller overwash of 0.5 m^2/s (v_{crest}= 1 m/s). The crest lowering by erosion is not much influenced by the overwash discharge and is about 1 m after 1 day. A depositional bar is formed at the beach. This latter bar is mainly caused by onshore transport of gravel enhanced by relatively large longshore velocities (2.5 m/s) under oblique wave attack. When the wave incidence angle is set to 0, the bar formation is much less pronounced. The crest erosion and migration also are smaller for waves normal to the coast.

Under superstorm conditions (water level at 5 m above MSL) the crest erosion is up to 1 m and the maximum crest migration is about 20 m/day. Data of Hurst beach and Hurst spit show crest migration values of 20 m in one storm event and long term values of 10 m over a period of about 2 years. Measured vertical crest erosion values are in the range of 0.5 to 1.5 m at Hurst beach, which are of the same order of magnitude as the computed values.

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. The toe of the layer should extend to below the low water mark (-2 m below MSL) resulting in a total length of about 100 m assuming a beach slope of 1 to 8 between the -2 m and +3 m line and a barrier slope of 1 to 3. A total volume of about 200 to 300 m^3/m is required per m shoreline or 200,000 to 300,000 m^3 per km. This solution may be attractive at specific locations (near structures, harbours, inlets, etc.) where the erosion processes of the sandy dune system are excessively large resulting in relatively large maintenance costs (nourishments costs).

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