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An Investigation on Beach Profile Changes in Front of Seawalls

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ABSTRACT

Although there exist advanced models which predict beach profile for natural beaches, the behavior of the beaches in front of seawalls still suffers from the lack of appropriate theoretical models and sufficient measured data. In this paper, following the results obtained from the measurements, a beach profile evolution model is developed, using the measured probability distribution of the near-bed horizontal velocities as input, to predict the short-term bed level changes in the vicinity of a partially reflective seawall. The present model introduces a new approach in which based on integrating the probability density functions of the near-bed horizontal velocities, the sediment displacements and consequently bed level changes are calculated in front of a partially reflective structure. The results obtained from the model in comparison with the experiments are promising and encouraging for further developments of the proposed model.

1. Introduction

Wave-induced hydrodynamics within the near-shore region and the subsequent evolution of beaches under wave attack are important elements governing the stability of the coastal zone. Based on surf zone hydrodynamics several efforts have been made to estimate the sediment transport and beach profile evolution during storm conditions. Recently, it has been found that the dominate processes responsible for cross-shore sediment transport in the coastal zone can be categorized into three major parts: onshore transport due to asymmetric orbital velocities, offshore transport due to mean return flow or undertow, and down slope sediment transport due to the component of the gravity force acting on individual grains [1,2]. Previous researches into wave kinematics and erosion processes in front of coastal structures have concentrated on two main areas, namely wave reflection and toe scour, but the effect of seawalls on beaches and on coastal processes has not been well documented [3]. Therefore, there was seen to be a need for additional research into the effects of seawalls on coastal processes to obtain the quantitative data on seawall performance. A number of researches have shown that the beach change near seawalls, both in magnitude and temporal variation, is similar to that on beaches without seawalls, if a sediment supply exists [4]. The physical processes of the seawalls and beach interaction, however, must be well understood in order to assess the relative performance of seawalls and alternative shore protection methods [5]. Clearly, investigations of these processes demands measured data for an arbitrary beach profile which can be acquired by performing laboratory-scale experiments in both two and three dimensions. Kraus (1988) extensively reviewed a number of experimental and theoretical works regarding the effects of seawalls on coastal processes, showing that there are still many unknown areas concerning beach-structure interaction which need to be investigated [4]. Lashteh Neshaei et al (2009) showed that a seawall located onshore can have significant effects on coastal regime and sediment and sediment transport inside the surf-zone and finally presented an analytical model to predict the beach profile evolution in the vicinity of such reflective structures [6]. The major goal of the present study is to compensate for the lack of information describing the effects of reflective structures on coastal processes, particularly cross-shore sediment transport and short-term beach profile evolution in response to storm events. In the present study, first, the conceptual elements in developing the semi-empirical model are described in detail; following which some of the limitations associated with the present model are introduced and the possible ways of improving the model are briefly discussed.

2. Structure of the Model

2.1 Wave transformation

In order to determine the wave-induced velocity field and beach profile evolution inside the surf zone, laboratory experiments were performed in a large two-dimensional wave channel at Imperial College Hydraulics Laboratory [6]. Figure 1 shows the experimental setup in order to measure the water surface elevations hydrodynamics of the surf zone in front of a partially reflective seawall. The overall dimensions of the tank were 1.00 m wide, 1.2 m deep and 26 m long. A plane beach profile with a constant slope of 1:20 was built at the end of the tank. Waves were generated at one end of the tank by a wave generator controlled by an electro-hydraulic system and the water particle velocities were measured in the surf zone using a two-component Electro-magnetic current meter. A permeable seawall was built at the end of the beach. The exact distance of seawall from the shoreline was 0.6m resulting in 0.100m water depth in front of the structure. A sampling rate of 25Hz with a record length of 6 minutes was selected to provide the suitable conditions. Table 1 summarizes three different wave spectra (S1, S2 and S3).

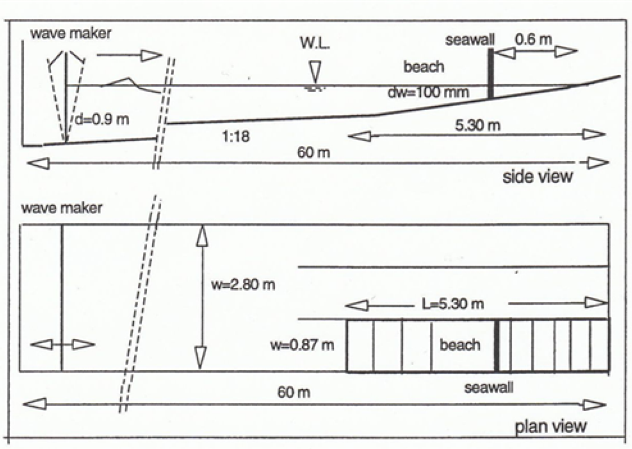


Figure 1. Illustration of the experimental arrangement for partially reflective seawall.

Table1. Descriptions of spectra[6].

Spectrum	H_s (m)	T_z (s)	L_0 (m)	S_0
S1	0.074	1.24	2.40	0.031
S2	0.088	1.46	3.33	0.027
S3	0.095	1.67	4.35	0.022

H_s= Significant wave height
T_z= Zero-crossing period
L_0= Deep water wave length
S_0= Deep water wave steepness

To estimate the incident and reflected wave spectra and the reflection coefficient from the structure, a “three wave gauges method” presented by Gaillard et al (1980) was used. For this purpose, it is necessary to mount three capacitance wave gauges at certain locations seaward of the beach in deep water [7]. The main assumption here is that the irregular waves can be described as a linear superposition of an infinite number of discrete components each with their own frequency, amplitude and phase. In order to achieve this, the co-existing waves at three known positions in the channels seaward of the beach, must be measured. Fourier analysis of these signals and calculation of auto and cross spectra, provides a method to resolve the partial standing wave in front of structure into incident and reflected waves. The method is explained in detail in Gaillard et al (1980) and is based on the following equations:

$$S_I(f) = \frac{\bar{S} - \bar{C} + \bar{Q}}{2S_\alpha} \qquad (1)$$

$$S_R(f) = \frac{\bar{S} - \bar{C} - \bar{Q}}{2S_\alpha} \qquad (2)$$

$$K_r(f) = \sqrt{\frac{S_R(f)}{S_I(f)}} \qquad (3)$$

Where:

$\bar{S} = S_1 + S_2 + S_3$

$\bar{C} = C_{21}\cos\theta_{21} + C_{31}\cos\theta_{31} + C_{32}\cos\theta_{32}$

$\bar{Q} = Q_{21}\sin\theta_{21} + Q_{31}\sin\theta_{31} + Q_{32}\sin\theta_{32}$

$S_\alpha = \sin^2\theta_{21} + \sin^2\theta_{31} + \sin^2\theta_{32}$

$\theta_{ij} = k(x_i + x_j)$

k = wave number
x = coordinate of the wave gauge with respect to the shoreline
S = Auto spectrum
C = Co spectrum
Q = Quad spectrum
$S_I(f)$ = spectral density of incident wave
$S_R(f)$ = spectral density of reflected wave
$K_r(f)$ = reflection coefficient
And subscripts i, j, 1, 2, 3, refer to gauge numbers.

2.2 Hydrodynamics

According to linear wave theory, for regular standing waves, water particles oscillate in a horizontal plane beneath the node and in a vertical plane beneath the anti-node. The extreme values of horizontal and vertical velocity therefore occur under the nodes and antinodes of the water surface profile respectively. Based on linear wave theory, it is possible to estimate the water particle velocities in front of a partially reflective structure for a spectrum of non-breaking waves [8]. In a real situation, however, the nonlinearity and random nature of incident waves as well as the breaking phenomena in front of structures combine to create a complicated velocity field. Therefore, it seems that the application of linear theory in such cases does not provide a sufficiently accurate estimation of the velocity field, leading to the conclusion that there is a strong need to improve existing theoretical methods by taking wave nonlinearity and breaking processes into account.

Hughes (1992) presented a simple linear equation for estimating water velocity parameters beneath reflected, two dimensional, irregular, non-breaking waves as a function of the incident wave spectrum, water depth and spatial location. The surface elevation, $\eta(x,t)$, of a linear, partially reflected, unidirectional wave on a flat bottom can be expressed as [8]:

$$\eta(x,t) = a\cos(kx - \sigma t - \varepsilon) + ak_r \cos(kx + \sigma t + \theta + \varepsilon) \qquad (4)$$

Where:

a = amplitude of the incident wave

k = wave number ($2\pi/L$, where L is the wave length)

x = horizontal position

σ = angular wave frequency ($2\pi/T$, where T is the wave period)

t = time

k_r = reflection coefficient

θ = phase shift due to reflection

ε = initial phase angle of the incident wave

In this equation the first term represents the incident wave propagating in the positive x direction, and the second term represents the partially reflected wave propagating in the negative x direction. The velocity potential function associated with equation 4 is given by:

$$\varphi(x,z,t) = \frac{-ag}{\sigma}\frac{\cosh[k(h+z)]}{\cosh(kh)} .\sin(kx - \sigma t - \varepsilon) + \frac{k_r ag}{\sigma}\frac{\cosh[k(h+z)]}{\cosh(kh)} .\sin(kx + \sigma t + \theta + \varepsilon) \qquad (5)$$

Where:

g = gravitational acceleration

h = water depth

z = vertical coordinate, positive upward with z=0 at the still water level and z=-h at the bottom

Therefore, in the time domain, the horizontal component of fluid velocity can be calculated as:

$$u(x,z,t) = -\frac{\partial\varphi(x,z,t)}{\partial x} = \frac{agk}{\sigma}\frac{\cosh[k(h+z)]}{\cosh(kh)} .[\cos(kx - \sigma t - \varepsilon) - k_r \cos(kx + \sigma t + \theta + \varepsilon)] \qquad (6)$$

An approximation to the horizontal fluid velocity under random waves is then found by linear superposition of many components as:

$$u(x,z,t) = \sum_{n=1}^{\infty}\frac{a_n g k_n}{\sigma_n}\frac{\cosh[k_n(h+z)]}{\cosh(k_n h)} .[\cos(k_n x - \sigma_n t - \varepsilon_n) - k_m \cos(k_n x + \sigma_n t + \theta_n + \varepsilon_n)] \qquad (7)$$

Where the subscript n refers to individual components. The time series of horizontal velocity, expressed by equation 7, is the result of linear superposition of many independent sinusoidal components with random phases, ε_n, uniformly distributed between zero and 2π. Hence, based on the Central Limit Theorem, the velocity, $u(t)$, at a fixed point in space has a Gaussian probability density function with a zero mean and a standard deviation σ_u. The procedure for calculation of the probability density functions of measured data across the profile is mainly based on counting the number of relative frequencies of the given set of data with a number of class intervals, d_u. Given a time series of near-bed velocity $u(t)$ at a location in the surf zone, the probability density function (PDF) of $u(t)$, denoted by $p(u)$, can be obtained from the following equation [9]:

$$p(u_i) = prob(u_i - \frac{du}{2} < u \le u_i + \frac{du}{2}) \qquad (8)$$

In the next step, using the mean and standard deviation values for each point across the profile, the correspond Gaussian distribution, $p_G(u)$, can be calculated according to the equation:

$$p_G(u) = \frac{1}{\sqrt{2\pi}\sigma}\exp[-0.5(\frac{u-\bar{u}}{\sigma})^2] \qquad (9)$$

Where:

$\bar{u}$ = the mean value of the horizontal velocity

σ = standard deviation

The following equations were used to calculate the statistical parameters associated with the probability density functions from the time series of velocity and surface elevation data:

$$\bar{u} = \sum_{i=1}^{N}\frac{u_i}{N} \qquad (10)$$

$$\sigma = \sqrt{\sum_{i=1}^{N}\frac{(u_i - \bar{u})^2}{N}} \qquad (11)$$

$$\mu_j = \sum_{i=1}^{N}\frac{(u_i - \bar{u})^j}{N} \qquad (12)$$

$$\text{Skewness} = \frac{\mu_3}{\sigma^3} \qquad (13)$$

$$\text{Kurtosis} = \frac{\mu_4}{\mu_2^{\ 2}} \tag{14}$$

Where:
N = number of velocity data point
μ_j = jth moment of the velocity PDF
and subscript i denotes the individual velocity data.

In summary, the horizontal particle velocities for linear, random, non-breaking, standing waves can be described by a Gaussian probability density function with a zero mean and standard deviation σ_u, for which a typical shape is given in Figure 2. Since in practical cases there is clearly a significant variation about the mean velocity to determine the sediment motion may not be accurate enough to estimate beach profile evolution. It can be assumed that the sediment in the top layer of the bed moves at the fluid velocity when the near-bed velocity exceeds a certain value, termed the threshold velocity (u_{cr}).

By adapting the concept of a threshold velocity for the initiation of sediment motion, it is possible to estimate the percentage of time for which the velocity of the flow is greater than a critical value, represented by the hatched areas under the PDF in Figure 2. Clearly, assuming linear wave theory and Gaussion distribution with zero mean for the near-bed velocity results in zero net sediment movement in the surf zone because the areas under the PDF on both positive and negative sides will be equal. On the other hand, probability density functions of the measured near-bed velocities obtained from the experiments clearly show the skewness of the real PDF due to nonlinearity and breaking effects in front of the reflective structure. It is therefore clear that using the Gaussian distribution as a driving function for sediment movement can not lead to an estimation of the sediment transport rate and consequently the beach profile change in the surf zone. To overcome this problem, it is necessary to use a modified probability density function for the near-bed velocity in front of the reflective structure to present the real condition in the surf zone. A typical skewed PDF for the near-bed velocity is shown in Figure 3.

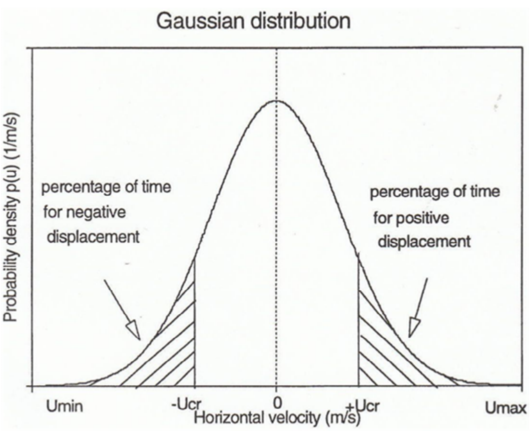


Figure 2. Gaussian PDF for the horizontal velocity in front of seawall.

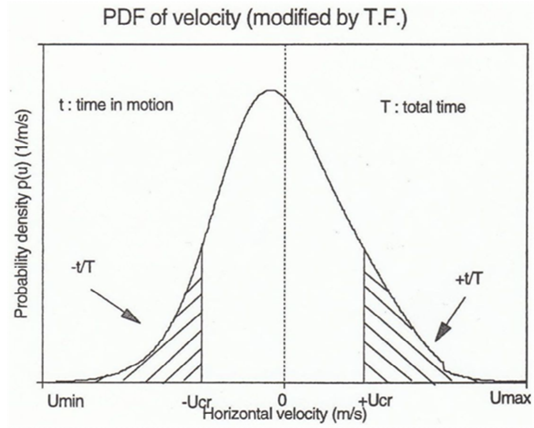


Figure 3. Skewed PDF for the horizontal velocity in front of seawall.

Using such probability density functions as a driving function for sediment motion enables the net sediment transport to be estimated by calculating the area under the positive and negative sides of PDF.

The essential step in describing the hydrodynamic part of the present model is to estimate the modified probability density functions as accurately as possible across the profile. For this purpose, an analysis of measured data and comparison with the Gaussian distribution were performed. As a first step, the probability density functions of horizontal velocity at all measured points, 5 mm above the bed, were calculated using statistical methods. It was assumed that the horizontal velocity at 5 mm above the bed is representative of the flow-field influencing sediment motion. By dividing the measured PDF by a Gaussian PDF with the same mean and standard deviation it is possible to derive a "Transfer Function" defined as :

$$T.F. = \frac{(PDF)_{measured}}{(PDF)_{Gaussion}} \tag{15}$$

Interestingly, the transfer function is found to be relatively independent of the wave conditions within the range of $\pm 2.5\sigma_u$ (where σ_u is the standard deviation of horizontal velocity at 5 mm above the bed) but it depends on the water depth and also the location of the point relative to the structure. It has to be noted that the calculation of transfer function is valid within the range of $\pm 2.5\sigma_u$ as outside this range, because of the small values for probability density functions, the potential error in calculating the T.F. will be high. Considering the fact that the area under the PDF in the tails of the curve can not be significant in terms of sediment motion, the selection of the $\pm 2.5\sigma_u$ range as a limit for calculation of T.F. is felt to be reasonable. Based on the transfer function derived from the experimental data it is possible to calculate the modified PDF for each measured point as follows for use as a driving function for sediment motion across the beach in the surf zone:

$$(PDF)_{\text{modified}} = (PDF)_{Gaussian} \times T.F. \quad (16)$$

2.3 Sediment transport

A simple sheet flow model is derived to calculate the net sediment transport across the profile in front of a partially reflective structure based on integrating the probability density functions of the near-bed velocities. One of the most important feature of the model is the incorporation of a threshold condition for the incipient grain motion in the calculation of sediment displacements.

In oscillatory flow there is no generally accepted relationship for initiation of motion on a plane bed [10]. One of the most popular equations is that of Komar and Miller (1974) which is used in the present model and reads as [11]:

$$\frac{u_{cr}^2}{(s-1)gd_{50}} = 0.21\left(\frac{2A_{cr}}{d_{50}}\right)^{0.5} \qquad d_{50} < 0.0005m \quad (17)$$

$$\frac{u_{cr}^2}{(s-1)gd_{50}} = 1.45\left(\frac{2A_{cr}}{d_{50}}\right)^{0.25}: \quad d_{50} \geq 0.0005m \quad (18)$$

$$A_{cr} = \frac{H}{2\sinh kh} \quad (19)$$

In which:

u_{cr} = critical peak value of orbital velocity near the bed

A_{cr} = critical peak value of orbital excursion near the bed

D_{50} = the median grain diameter of bed material

s = specific gravity ($\frac{\rho_s}{\rho}$ where and ρ are sediment and fluid density respectively)

h = water depth

H = wave height

k = wave number ($2\pi/L$, where L is the wave length)

g = acceleration of gravity

It is assumed that the sediment in the top layer of the bed moves at the fluid velocity when the threshold velocity for initiation of sediment motion (u_{cr}) is exceeded. One the grain layer is in motion, it will continue to move in the same direction even when $u < u_{cr}$ and it is likely that the mobile grains will decelerate and stop around the point of reversal of the fluid velocity (Figure 4). As a first approximation, this deceleration effect can be ignored and therefore the sediment is assumed to stop moving when $u < u_{cr}$. It has to be noted that at this stage the above assumption is used because of its simplicity in the calculation of the net sediment movements. However, more evidence is required to verify this assumption. Referring to Figure 4, the negative and positive displacements of the sediment can be derived as:

$$neg.\ dis. = \int_{t_1}^{t_2} u\,dt \ \text{ and } pos.\ dis. = \int_{t_3}^{t_4} u\,dt \quad (20)$$

Where:

$$u(t_1) = u(t_2) = -u_{cr}$$
$$u(t_3) = u(t_4) = +u_{cr}$$

Using this method and adding the successive displacements for the positive and negative components of velocity time series separately, it is possible to calculate the net sediment displacement for a given location during a period of time (T) as:

$$net\ dis. = \sum_{t=0}^{T}(pos.dis.) - \sum_{t=0}^{T}(neg.dis.) \quad (21)$$

In practical situations, however, using the time series of velocity in order to calculate the net sediment displacement is not very convenient. Hence, by transferring the problem to the probability domain, an alternative approach based on the PDF of the velocity time series is introduced to avoid the difficulties associated with time series calculations. This approach can be used for prediction of the gross and net sediment transport rate in the surf zone across the beach. As discussed by Lashteh Neshaei (1998), it is possible to estimate the PDF of the near-bed horizontal velocity based on simulating a Gaussian probability density function and modifying it using a transfer function [12].

According to the definition of the probability density function of horizontal velocity, the shaded area under the PDF in both positive and negative sectors, Figure 3, represent the percentage of the time in which the velocity of sediment motion exceeds the critical value (u_{cr}). It is assumed that the sediments move with the same velocity as the water particles near the bed. Therefore, the duration of the time which is spent by sediment in motion in positive and negative directions can be estimated, respectively, as follows:

$$t_{pos} = T\int_{u_{cr}}^{u_{\max}} p(u)du \quad \text{and} \quad t_{neg} = T\int_{u_{\min}}^{-u_{cr}} p(u)du \quad (22)$$

Where T is the total period of the time in which the calculation of sediment displacement is made (duration of storm).

According to the basic kinematic equation of motion for a particle, the displacement of a particle in motion can be derived simply as the product of its velocity and the duration of the time in which the particle is in motion. Therefore, the positive and negative displacements can be given, respectively, as:

$$pos.dis. = \int_{u_{cr}}^{u_{max}} up(u)du$$

and (23)

$$neg.dis. = \int_{u_{min}}^{-u_{cr}} up(u)du$$

The product *up* (*u*) is the first moment of the probability density function of the sediment velocity at the top of the mobile layer. Figure 5 shows a typical shape of the function *up*(*u*). As can be seen in this figure, for a Gaussian PDF with a zero mean, since the positive and negative displacements are equal for a given u_{cr}, the net displacement will be zero and no net sediment transport will occur in the surf zone. In reality, because of the skewness of the PDF due to nonlinearity and breaking effects in shallow water, the net sediment displacement for each particular location in the surf zone can be calculated as:

$$net\ dis. = pos.\ dis. - neg.\ dis. = \int_{u_{cr}}^{u_{max}} up(u)du - \int_{u_{min}}^{-u_{cr}} up(u)du \qquad (24)$$

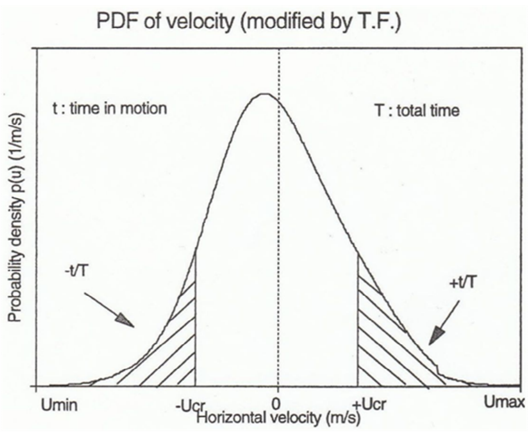


Figure 3. Skewed PDF for the horizontal velocity in front of seawall.

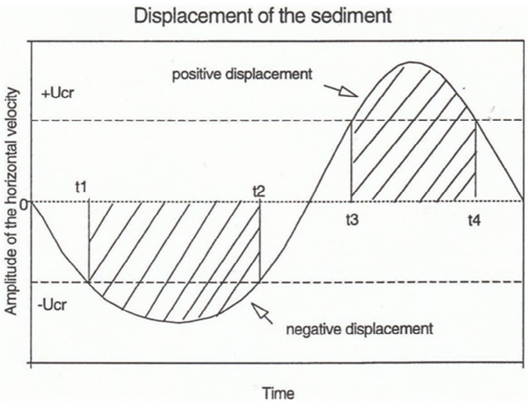


Figure 4. Calculation of sediment displacements based on the time series of horizontal velocity.

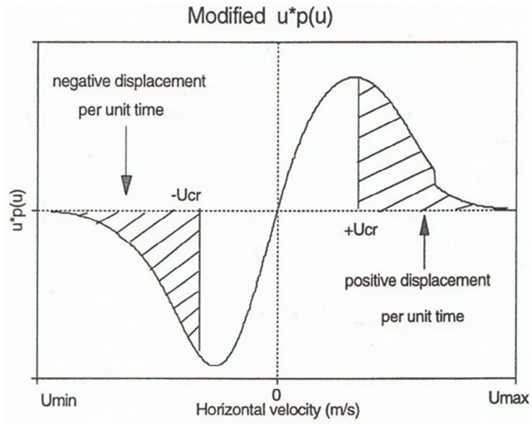


Figure 5. Calculation of sediment displacements based on the probability density function of the horizontal velocity.

2.4 Beach profile evolution

The final feature of the proposed model is to calculate the bed level changes and consequently beach profile evolution in front of a partially reflective structure. As described earlier, the positive and negative displacements for a sediment particle per unit time can be calculated by integrating the *up*(*u*) curve using equation 23. By discretizing the profile into a number of segments, as shown in Figure 6, it is possible to assign a positive and/or a negative displacement to each section. The next step is to displace each section of the profile into a new position determined by its displacement magnitude. Keeping in mind that the displacements calculated by equation 23 are per unit time, it is necessary to repeat this process for each section for the time in which sediments are in motion.

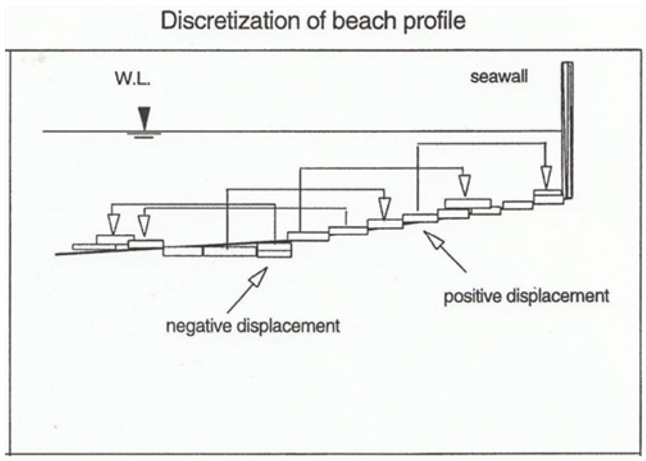


Figure 6. Discretization of beach profile into a number of elements.

This time can conveniently be calculated from the area under the probability density function of the horizontal velocity for each point across the profile using a numerical integration according to equation 22. By multiplying the time of motion by the volume of each section per unit width of the profile section and transferring the result into the proper location (determined by the magnitude of displacement), i.e. adding the accumulated area, it is possible to calculate the rate of change in bed level across the profile and the final shape of the beach in front of the reflective structure after a given period of time. This assures that the net displacement of sediment from its initial position is sufficiently small so that the ambient velocity field does not change significantly over that displacement. The important query here is how deep the sediments can be eroded in the vertical plane, i.e. what is the exact volume per unit width of each section which is displaced across the profile? To implement this concept into the model, an intrusion depth, defined as the depth to which sediment moves – at the ambient velocity for sheet flow condition is used to evaluate the vertical length scale of eroded sediments across the profile. According to Nilson (1992) and following the assumption made by Bagnold (1956) for sheet flow condition, the vertical scale of the bed load distribution is defined by [13,14]:

$$L_B = \frac{1}{c_{\max}} \int_0^\infty c_B(z)dz \quad (25)$$

In which:

$C_{\max}$ = sediment concentration at the top of immobile bed

$C_B(z)$ = bed load sediment concentration

which can be estimated using Bagnold's assumptions by:

$$L_B = k_B(\theta' - \theta_c)d \quad (26)$$

Where:

d = sediment grain diameter

θ' = skin friction or effective Shields' parameter (defined by Eq. (27))

θ_C = critical Shields' parameter (0.03-0.05 for practical cases)

K_B = a constant (suggested to be 2.5 by Bagnold)

The non-dimensional Shields' parameter corresponding to the effective bed shear stress is defined by:

$$\theta' = \frac{\tau'}{\rho(s-1)gd} \quad (27)$$

Where:

τ' = the effective bed shear stress

ρ = fluid density

Hence, the area of each section can be calculated by the product of L_B and the length of each section. Finally, by moving this volume per unit width to a new location determined by the magnitude of net displacement and considering the time which is spent by each section in motion, it is possible to calculate the profile development across the beach. Examples of profile evolution results predicted by the present model and their comparison with those found in the experimental measurements are given in Figure 7. Clearly, the accuracy of the model is still to be improved. Nevertheless, these results illustrate the feasibility of the new approach used in modeling the beach profile evolution in front of a partially reflective structure. Figure 8 shows the net displacements calculated over a number of points across the profile for different wave conditions used in the experiments. Figure 7 indicates the comparison of measured bed level changes in the present model and the approaches proposed by Bagnold (1956) and Wilson (1987) for both fine and coarse and particular wave conditions indicating good compatibility [14,15].

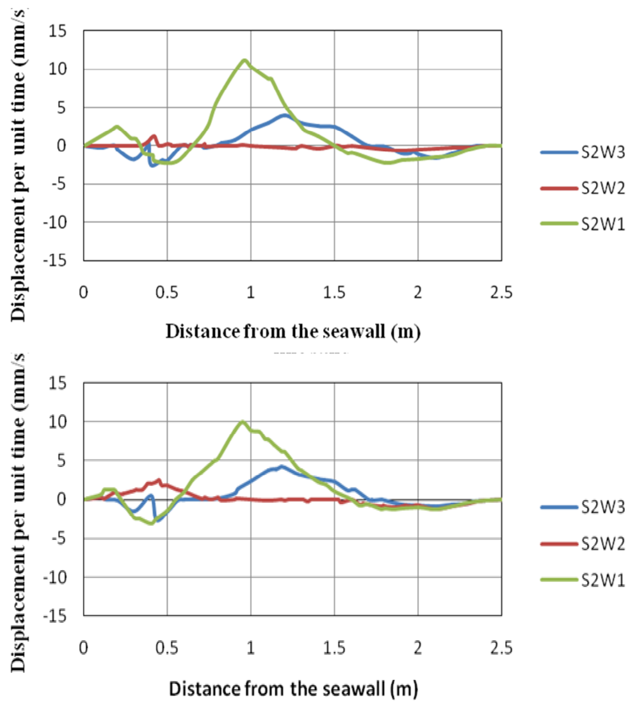
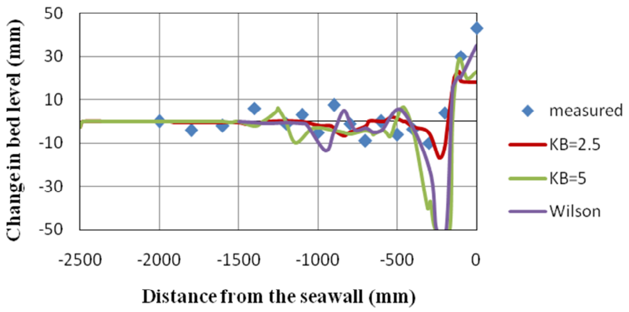


Figure 7. Calculated net sediment displacements across the profile for different wave conditions used in the experiments. W1, W2 and W3 represent 0.05, 0.1 and 0.15 m water depths at the wall, respectively.



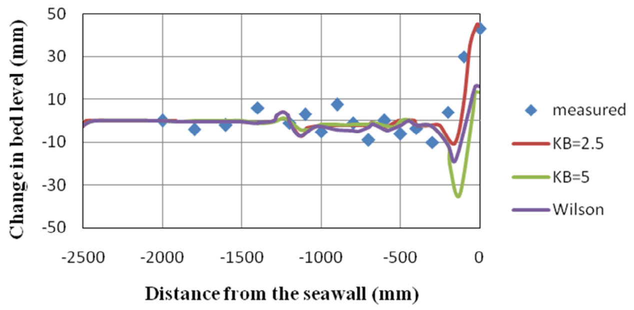


Figure 8. Comparison of measured bed level changes in the presented approach and the approaches proposed by Bagnold (1956) and Wilson (1987) for both fine and coarse sediments and particular wave conditions [14,15]

3. Discussions

It is important important to emphasize that since the model uses measured velocity data as an input to calculate the sediment displacement, it is a semi-empirical model. To develop a comprehensive numerical model, theoretical work is required to enable the velocity probability density functions to be obtained directly from the surface elevation time series or from a defined offshore wave spectrum. Clearly, investigations of such processes can not be achieved without considering the nonlinear properties of standing waves and the breaking procedure in front of partially reflective structures.

Concerning relative velocity of sediment-water (two phase flow), as a first attempt in the proposed model, these velocities are taken to be equal. However, in the modified model, the corrected ratio of sediment/water velocity will be taken into account according the criteria presented by Asano (1990) [16]. It is to be mentioned that, at present, the incorporation of this ratio into the model has not changed the results drastically. It seems that this coefficient is a function of bed form and can improve the predictive results of the proposed model. It is hoped that by applying a semi-empirical equation for this ratio into the presented model, the results are much improved.

The model presented here can be considered as an initial sedimentation model in terms of cross-shore mechanisms which goes only once through the sequence of constituent models. In other words, the hydrodynamic and sediment transport computation is based on the assumption of an invariant bed topography and only the rate of the sedimentation or erosion for that topography is computed at every location. Therefore, the model is difficult to interpret in terms of longer-term morphological evolution. However, the model results can be used in comparative studies, for example to predict the effects of different designs of a reflective structure and to assess the structure's impact on coastal processes.

One of the main steps towards improving the model results is to use updated hydrodynamics and bed elevations after a certain time step in order to increase the level of accuracy in the predictions. To do this, it is necessary to perform further experimental studies in which the modified hydrodynamics and the associated beach profiles are measured after each time step. The results of such experiments would be of great benefit in calibrating and developing the model towards predicting the longer-term behavior of beaches in the vicinity of partially reflective structures.

Figure 8 shows the net displacements calculated over a number of points across the profile for different wave conditions used in the experiments. A numerical integration method was applied to calculate the area under the *up(u)* curve, which corresponds to the sediment displacement per unit time. As can be seen in this figure, in most cases, inside the surf zone the positive displacements (towards the shoreline) are dominate, resulting in onshore transport and building a berm profile in front of the seawall. These results are in agreement with the experimental observations.

Another important parameter to determine the volume of sediments in motion is the constant parameter k_B in equation 26. The value of k_B has been reported to be between 2.5 and 10 by different researchers. For example, the suggested value for k_B is 2.5 as mentioned by Nielsen (1992) and 10 as reported by Asano (1990) [13,16]. Figure 7 shows the sensitivity of the predicted results to the selected values of k_B. As can be seen, the results obtained from the model are quite sensitive to the value of k_B because it determines the depth of eroded material across the beach. At present, there is no satisfactory work in the literature concerning the concept of intrusion depth. Therefore, further investigations on the depth of eroded material in sheet flow conditions are required.

Finally, the present model is only able to deal with the short-term two dimensional cross-shore circulation in the surf zone. Therefore, in order to expand the range of applicability of the model, its development towards a three dimensional capability which could include the effects of a partially reflective structure on alongshore sediment transport is recommended.

4. References

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