CHAPTER 114

A MODEL FOR CROSS-SHORE SEDIMENT TRANSPORT

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ABSTRACT

A model for cross-shore sediment transport due to random waves is described which adopts a vertically integrated transport description for sheetflow situations. The formulation of the transport as a function of the instantaneous velocity field is based on the approach of Bailard (1981). This approach assumes in essence that the instantaneous transport is proportional to some power of the instantaneous near-bottom velocity. Implementation of this transport description in a time-dependent model requires a formulation of the time-mean and some low order moments of the near-bottom velocity field. An initial formulation based on a monochromatic, second order Stokes wave representation is presented. The model is checked on the basis of both field and laboratory data. Some consequences for further study are indicated.

1. INTRODUCTION

Observation of sediment transport due to random waves on a two-dimensional beach indicates that one of the more important mechanisms under active surf conditions may be the transport of sediment by the time mean, seawards directed flow near the bottom induced by the breaking of waves. It was shown (Stive and Battjes, 1984) that this mechanism is so dominant that a vertically integrated model incorporating this mechanism alone describes the bottom variations in an active surf zone to a satisfactory, first approximation. Extension of this model with other transport mechanisms is a logical step towards a more complete cross-shore sediment transport model. Here some first suggestions are made to extend the model with transport due to the asymmetry of the wave motion, so that also a mechanism for onshore transport is included.

2. TRANSPORT FORMULATION

Since we are interested in a transport formulation which takes also the effects of wave asymmetry into account, it is essential to adopt a formulation describing the instantaneous transport. A simple approach would be to assume that the instantaneous sediment transport rate, q, is proportional to some power of the local relative velocity between the bed and the fluid outside the boundary layer. For example,

$$q(t) = A\, u(t)\, |u(t)|^n \tag{1}$$

where $u(t) = u_b \cos\omega t$ + higher harmonic terms, with u_b the orbital velocity amplitude just outside the boundary layer and ω the angular frequency.

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The latter approach has been elaborated consistently for surf zones on a plane sloping beach by Bailard (1981), who extended the work of Bailard and Inman (1981). Based on Bagnold's (1963) energetics concept these authors use as a starting point a description of the instantaneous sediment transport basically in the form of Equation (1), extended with the effect of a bottom slope. Bailard (1981) distinguishes between bedload transport in a granular-fluid shear layer of a thickness in the order of the wave boundary layer and suspended transport in a layer of greater thickness, typically in the order of several centimeters. For the bedload transport the power n as introduced by equation (1) is given as 2, while for the suspended transport it is given as 3. Here the formulation is reduced for application in the cross-shore direction which yields the instantaneous total load sediment transport equation (see also Bailard, 1982):

$$i(t) = i_B(t) + i_S(t) = \rho c_f \frac{\varepsilon_B}{\tan\phi} \left[|u(t)|^2 u(t) - \frac{\tan\beta}{\tan\phi} |u(t)|^3 \right] + \rho c_f \frac{\varepsilon_S}{w} \left[|u(t)|^3 u(t) - \frac{\varepsilon_S}{w} \tan\beta \, |u(t)|^5 \right] \tag{2}$$

where i is the total cross-shore immersed weight sediment transport rate (composed of the bedload transport rate, i_B, and the suspended load transport rate, i_S), ρ is the water density, c_f is the drag coefficient for the bed, tan β is the slope of the bed, ϕ is the internal angle of friction of the sediment, w is the sediment's fall velocity and ε_B and e_S are bedload and suspended load efficiencies, respectively. The efficiency factors ε_B and ε_S denote those (constant) fractions of the total power produced by the fluid motion which are expended in transporting. The immersed weight sediment transport rate is linked to the volumetric transport rate q by

$$q = \frac{i}{(\rho_s - \rho) g N} \tag{3}$$

where ρ_S is the sediment density, g the gravitational acceleration and N the local volume concentration of solids.

The above sediment transport formulation uses vertically integrated equations. As a consequence, the sediment transports are assumed to respond to the near bottom water velocity in an instantaneous, quasi-steady manner. This assumption is probably valid for most natural beaches with prevailing sheet flow conditions and incident wave periods in excess of 5 seconds approximately.

Another uncertainty in the transport formulation concerns the use of bedload and suspended load efficiency factors. Although constant values have been found adequate for certain types of flow, the estimation of their optimal values, and their possible variations with the type of flow considered leaves at least some quantitative uncertainty.

3. WAVE INDUCED CROSS-SHORE FLOWS

Given the variation of the cross-shore velocity field the mean cross-shore sediment transport rate may in principle be calculated from the time averaged Equation (2):

$$\langle i \rangle = \rho\, c_f \frac{\varepsilon_B}{\tan\phi} [\langle |u|^2 u\rangle - \frac{\tan\beta}{\tan\phi} \langle |u|^3\rangle +$$

$$+ \rho\, c_f \frac{\varepsilon_S}{w} [\langle |u|^3 u\rangle - \frac{\varepsilon_S}{w} \tan\beta \langle |u|^5\rangle] \tag{4}$$

The total velocity u is decomposed into a near-bottom, mean (overbar) and an oscillatory (tilde) flow component,

$$u = \bar{u} + \tilde{u} \tag{5}$$

Conceptual simplifications follow by assuming that the oscillatory velocity is due to a single plane wave of frequency ω and some small non-linear harmonics:

$$\tilde{u} = u_m \cos \omega t + u_{2m} \cos 2\omega t + \ldots \tag{6}$$

in which $u_m > u_{2m} > \ldots$

Using Equations (5) and (6) in Equation (4) yields:

$$\langle i \rangle = \rho c_f u_m^3 \frac{\varepsilon_B}{\tan\phi} [\psi_1 + \frac{3}{2}\delta_u - \frac{\tan\beta}{\tan\phi}(u3)^*] +$$

$$+ \rho c_f u_m^4 \frac{\varepsilon_S}{w} [\psi_2 + \delta_u (u3)^* - \frac{u_m}{w} \varepsilon_S \tan\beta (u5)^*] \tag{7}$$

in which the relative current strength, δ_u, is

$$\delta_u = \bar{u}/u_m \tag{8}$$

and ψ_1 and ψ_2, represent non-dimensional velocity moments, defined as

$$\psi 1 = \langle \tilde{u}^3 \rangle / u_m^3 \tag{9a}$$

$$\psi 2 = \langle |u|^3 \tilde{u} \rangle / u_m^4 \tag{9a}$$

The even velocity moments $(u3)^*$ and $(u5)^*$ are defined as:

$$(u3)^* = \langle |u|^3 \rangle / u_m^3 \tag{10a}$$

$$(u5)^* = \langle |u|^5 \rangle / u_m^5 \tag{10b}$$

Retaining first order in the relative current strength and odd moments only three velocity moments may be simplified further, i.e.

$$u_m^4 \psi_2 \approx \langle |\tilde{u}|^3 \tilde{u} \rangle + 3\bar{u} \langle |\tilde{u}|^3 \rangle \tag{11}$$

and $$u_m^3 (u3)^* \approx \langle |\tilde{u}|^3 \rangle + 3\bar{u} \langle |\tilde{u}| \tilde{u} \rangle \tag{12a}$$

$$u_m^5 (u5)^* \approx \langle |\tilde{u}|^5 \rangle + 5\bar{u} \langle |\tilde{u}|^3 \tilde{u} \rangle \tag{12b}$$

Inspection of the above expressions indicates that we need to evaluate the mean return flow and the oscillatory velocity moments.

Mean return flow

It has been hypothesized by Dyhr-Nielsen and Sørensen (1970) that the seaward directed returnflow or undertow in the surf zone -which compensates for the shoreward directed mass flux above wave trough level- is driven by the imbalance between the vertically non-uniform wave momentum flux on the one hand and the vertically uniform pressure gradient on the other hand. Quantitative evaluations of these ideas leading to models for the circulation have been presented by e.g. Dally (1980), Svendsen (1984), and more recently by Stive and Wind (1986). These cross-shore flow models are all based on a periodic wave formulation. To apply these models in the case of random waves Stive and Battjes (1984) have shown that satisfactory results are obtained, by simply applying the periodic formulation to that fraction of the waves that are breaking.
An evaluation applying this idea to Stive and Wind's model and a generalization to the case of quasi-3D current models has recently been made by De Vriend and Stive (1987). The formulation for the purely 2DV (two-dimensionally vertical) case has been adopted here. It is described below leaving out the rather large number of analytic expressions for which the reader is referred to the paper.

In order to derive the wave-mean cross-shore current, the water column is divided into three layers, viz. a surface layer above the wave trough level, a middle layer and a bottom layer. Stive and Wind (1986) propose to consider only the area below the wave trough level, and to take account of the surface layer effects via an effective shear stress at the trough level, compensating for the momentum decay above it, and via the condition that the net undertow must compensate for the mass flux in the surface layer. This means that the surface layer model is reduced to the formulation of the effective shear stress and the mass flux. The bottom layer velocity due to non-breaking waves is assumed to be similar to Longuet-Higgins' (1953) "conduction solution" for progressive waves. For breaking waves, Stive and Wind (1986) show that assuming a zero bottom shear stress leads to acceptable predictions of the current outside the wave boundary layer. This means that for this part of the current the bottom layer as such can be left out of consideration. Thus the problem has been reduced to solving the velocity in the middle layer from the horizontal momentum balance, both for the case of breaking and of non-breaking waves. In both cases the prescribed shear stress at the wave trough level provides an upper boundary condition, whereas the lower boundary condition follows from the zero shear stress approximation (breaking waves) or from matching with with bottom layer solution (non-breaking waves). The integral condition of continuity can be used to determine the remaining unknown constant (the bottom shear stress for non-breaking waves, the mean return current velocity for breaking waves).

In De Vriend and Stive (1986) these middle-layer solutions are considered in detail, for non-breaking waves, for breaking waves and for a random breaking wave field, respectively. In the latter case part of the waves are breaking, say a fraction $\tilde{Q}_b$ ($0 < \tilde{Q}_b < 1$). If mutual interaction between the waves is left out of consideration, this means that the secondary current velocities for the breaking and the non-breaking waves have to be added with appropriate weight factors.

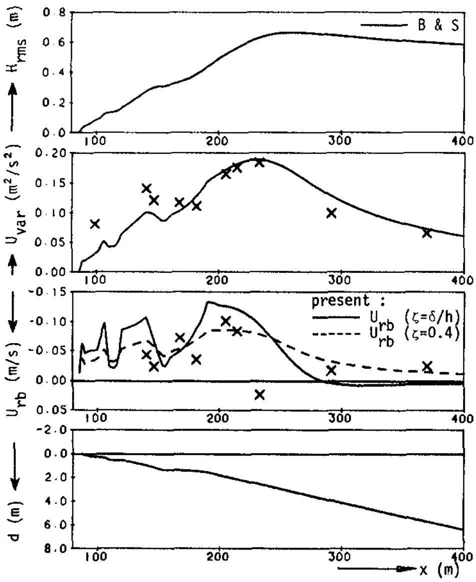

Fig. 1 Wave-induced undertow: comparison with NSTS measurements Nov. 20, 1978 at Torrey Pines beach (after Guza & Thornton, 1985); profile of bottom elevation below MSL (d), cross-shore undertow velocity (U_{rb}), variance of horizontal orbital velocity (U_{var}), and rms wave height (H_{rms}) versus distance normal to shore (x). Data points: measured values. Curves: computation with the models, see legend; B & S is Battjes and Stive (1985), present is present model, where U_{rb} ($\zeta = \delta/h$) is the undertow on top of the bottom boundary layer and U_{rb} ($\zeta = 0.4$) is that at 0.4 times the water depth.

To conclude this section a comparison is made with a field data and laboratory data. The field data are from the NSTS campaign at Torrey Pines Beach (Guza and Thornton, 1985). The wave energy decay prediction model (Battjes and Stive, 1985) was calibrated ($\alpha = 5.0$ and $\gamma = 0.5$) on the basis of the measured orbital velocity variance in the frequency range $0.05 < f < 0.5$ Hz. The velocity measurements were conducted at elevations of 0.4 m to 1.0 m above the bed. Taking account of the inherent inaccuracy of the measured, mean cross-shore velocities, the agreement is good, for the magnitude as well as for the cross-shore distribution of the velocity.

The laboratory data concern a more complicated depth profile (with two bars) under random wave attack. The wave energy decay prediction is again after the above mentioned model, but now according to the suggested parameterization The velocity measurements were conducted at an elevation of 0.05 m above the bed. Assuming that the accuracy of the measured, mean cross-shore velocities is relatively high the agreement in this case is less good. It is expected that the observed discrepancy is mostly due to the difficulty in predicting the breaking wave fraction on a barred beach (see the comparison between measurements and computations of $\tilde{\sigma}_b$).

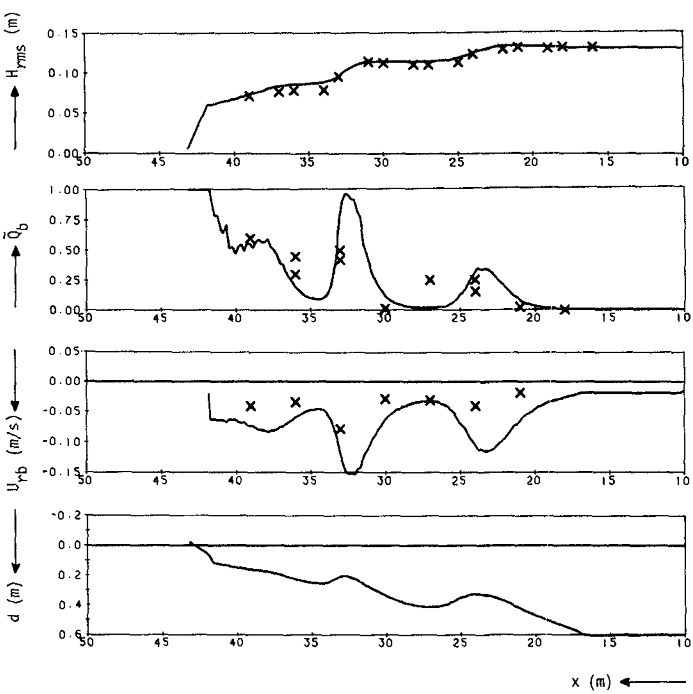

Fig. 2 Wave-induced undertow: comparison with flume measurements: profile of bottom elevation below MWL(d), cross-shore undertow velocity (U_{rb}), fraction of breaking waves ($\tilde{Q}_b$) and rms wave heights (H_{rms}) versus distance normal to shore (x).
Data points: measured values. Curves: computation with the model, where U_{rb} is the undertow at 0.05 m above the bottom.

Oscillatory velocity moments

From the evaluation of the present transport formulation it appears that the following low order oscillatory velocity moments are of importance:

- the four lowest even moments $\langle\tilde{u}^2\rangle$, $\langle|\tilde{u}|^3\rangle$, $\langle\tilde{u}^4\rangle$, $\langle|\tilde{u}|^5\rangle$, which are non zero for symmetric velocities,
- the two lowest odd moments $\langle\tilde{u}^3\rangle$, $\langle|\tilde{u}|^3\,\tilde{u}\rangle$, which are zero for symmetric velocities.

The latter moments are the most difficult to estimate: they are nonzero only for nonlinear waves such as actually occur nearshore. The shoreward velocities in such waves are typically stronger and of shorter duration than the offshore flows, leading to nonzero values for the odd moments.

A theoretical evaluation of the even moments for a random, linear sea (Gaussian model) is given by Guza and Thornton (1985). The theoretical moments are compared to field observations from the NSTS study. Their results indicate that even moments do not critically depend on cross-shore velocity asymmetry. This is due to the fact that also for symmetric velocities these terms are nonzero. At the present stage we will therefore rely on the Gaussian estimates for the even moments.

The odd moments are zero for a symmetric velocity field, but can be nonzero for asymmetric (nonlinear) motions. A relevant nonlinear property is the asymmetry of the wave surface about the horizontal axis. For non-breaking waves this asymmetry may to a first approximation well be predicted on the basis of a horizontal bottom, nonlinear wave theory, assuming that due to gradual bottom variations the waves locally behave as on a horizontal bottom (see Flick et al, 1981). However, in the horizontal bottom, nonlinear wave theories the phases of the harmonics are locked to zero and there is no vertical wave profile asymmetry possible. This asymmetry about the vertical plane is an essential property of the sawtooth shaped breaking waves in the surf zone. These theories are deficient in this respect and thus unsuitable for calculations of odd velocity moments which depend critically on phase. To illustrate this we calculate the two lowest order odd moments assuming that the velocity fluctuation is described by a second order approximation with a locked but nonzero phase between the two components:

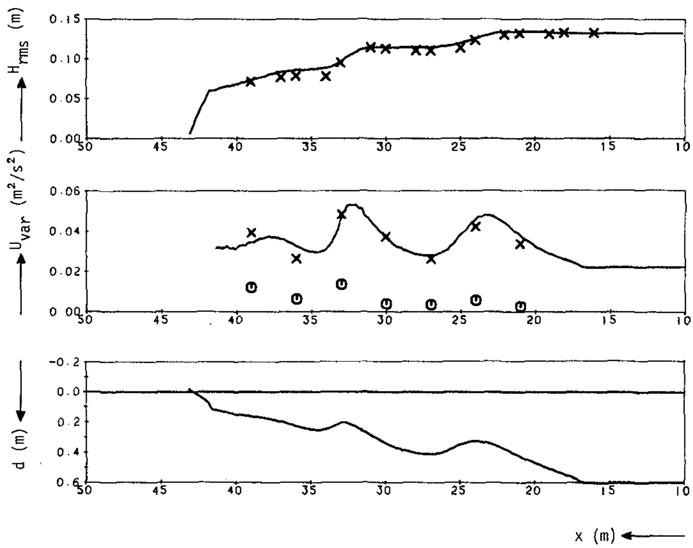

Fig. 3 Wave-induced near-bed velocity variance: comparison with flume measurements; profile of bottom elevation below MWL(d), oscillatory horizontal velocity variance (U_{var}) and rms wave height (H_{rms}) versus distance to shore (x).
Data points: measured values, the circles indicate U_{var} in the frequency range 0-0.25 Hz. Curves: computations with the model.

$$\tilde{u} = u_m \cos \omega t + u_{2m} \cos (2\omega t + \phi_2) \tag{14}$$

in which $u_m > u_{2m}$. After some algebraic manipulation it may be show that to lowest order the two odd velocity moments are given by:

$$\langle \tilde{u}^3 \rangle = \frac{3}{4} u_m^2 \, u_{2m} \cos \phi_2 \tag{15a}$$

$$\langle |\tilde{u}|^3 \tilde{u} \rangle = \frac{12}{5\pi} u_m^3 \, u_{2m} \cos \phi_2 \tag{15b}$$

An interesting perspective now arises when we combine these results with the following observations. In the inner surf zone where the breaking waves are quasi-steady the relative phase of the second harmonic increases smoothly toward the asymptotic value (see Flick et al, 1981):

$$\phi_2 \rightarrow \pi/2 \tag{16}$$

Thus, according to Eq. 15a, 15b, the odd velocity moments for breaking waves vanish ultimately.

At this point we may specify our wave decay model which predicts linear and nonlinear properties necessary to derive the velocity moments. As a starting point Battjes and Stive's (1985) wave decay prediction model is adopted to yield the cross-shore variation of the variance of the wave elevation. Given the wave variance, linear theory may be applied to provide the near-bottom velocity variance and thus the even velocity moments based on the Gaussian model. The odd velocity moments are estimated from the nonbreaking fraction of waves only, assuming that the contribution of the breaking waves is negligible in view of the above conclusions. To provide results from this model we use the second order Stokes expansion with

$$\tilde{u} = u_m \cos \omega_p t + \frac{3}{4} \frac{u_m^2}{c} \sinh^{-2}(k_p h) \cos 2\omega_p t \tag{17}$$

and choose $u_m = u_{rms}$ from the consideration that the monochromatic representation of the random wave field should have to same variance.

We conclude with a comparison between the above theory and the same laboratory measurements used earlier in the undertow comparison. Firstly, the velocity variance prediction may be checked from Fig. 3. It appears that the linear, Gaussian estimate is quite accurate. Secondly, the prediction of the remaining low order velocity moments may be checked from Fig. 4. It is noted that all moments are normalized by the variance, that the even moments are based on the linear, Gaussian estimate and that the odd moments are based on the nonlinear, monochromatic estimate. Here it appears that the prediction of the lowest order moments are reasonable given all assumptions made but that the discrepancies increase with increasing order of the moments as could be expected.

4. COMPUTATION OF TRANSPORT AND BOTTOM CHANGES

In the present model the local mean, volumetric cross-shore sediment transport rate, $\langle q \rangle$, is calculated according to the following expressions, where use has been made of expressions (4) and (8)...(12):

$$\langle q \rangle = B_{as} \langle q_{as} \rangle + B_{un} \langle q_{un} \rangle - B_{sl} \langle q_{sl} \rangle \tag{18a}$$

$$\langle q_{as} \rangle = F_B \, \psi_1 + F_S \, \psi_2 \tag{18b}$$

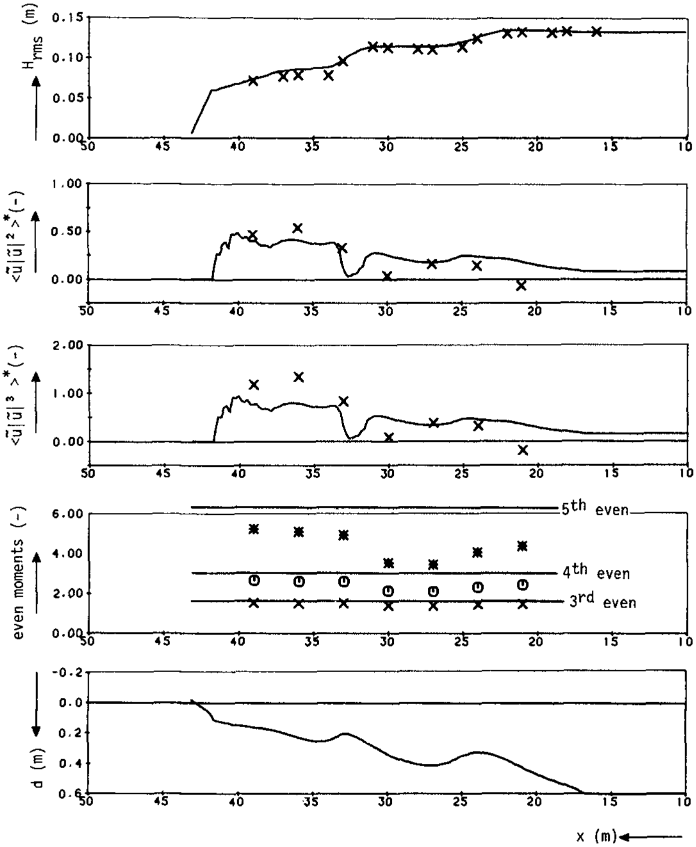

Fig. 4 Wave-induced near-bed velocity moments: comparison with flume measurements; profile of bottom elevation below MWL (d), normalized odd and even moments and rms wave height (H_{rms}) versus distance to shore (x).
Data points: measured values. Curves: computations with the model, where the even moments are according to the linear, Gaussian model.

$$\langle q_{un}\rangle = F_B \frac{3}{2}\delta_u + F_S\, 4\delta_u\, (u3)^* \tag{18c}$$

$$\langle q_{sl}\rangle = F_B \frac{\tan\beta}{\tan\phi} (u3)^* + F_S \frac{u_m}{w} \varepsilon_S \tan\beta\, (u5)^* \tag{18d}$$

$$F_B = \frac{c_f u_{rms}^3 \varepsilon_B}{\Delta g N \tan\phi} \tag{18e}$$

$$F_S = \frac{c_f u_{rms}^4 \varepsilon_S}{\Delta\, g\, N\, w} \tag{18f}$$

Here c_f is the drag coefficient equal to $\frac{1}{2}f_w$ with f_w the friction factor as defined in Stive and Battjes (1984) and B_{as}, B_{un} and B_{sl} are proportionality constants which should be O(1) if the description is right. The free parameters in the above expressions are ε_B and ε_S which for cross-shore transport are given by Bailard (1982) on the basis of field observations as 0.10 and 0.02 respectively. These values are adopted here.

The cross-shore variation of the local, mean sediment transport may now be calculated with the above expressions (18a...f) given the results of the wave height decay and kinematics model. Through application of the mass balance for the sediment (of which the properties are assumed constant) the bottom changes may be calculated. This procedure may be repeated for the new beach profile. In the numerical evaluation a second order Runge-Kutta algorithm is used in the wave decay model and a modified Lax scheme in the bottom change calculations. As a boundary condition on the waterline the present formulation yields $\langle q \rangle = 0$.

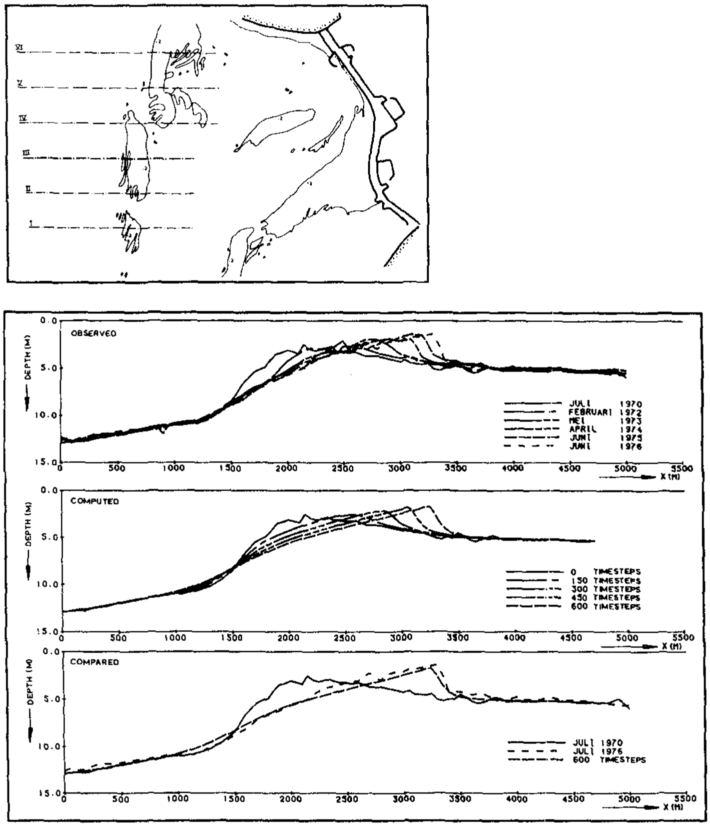

Fig. 5 Profile evolution of the underwater delta in the former mouth of the Grevelingen estuary. Top: situation in 1976. Bottom: results of model hindcast for section III.

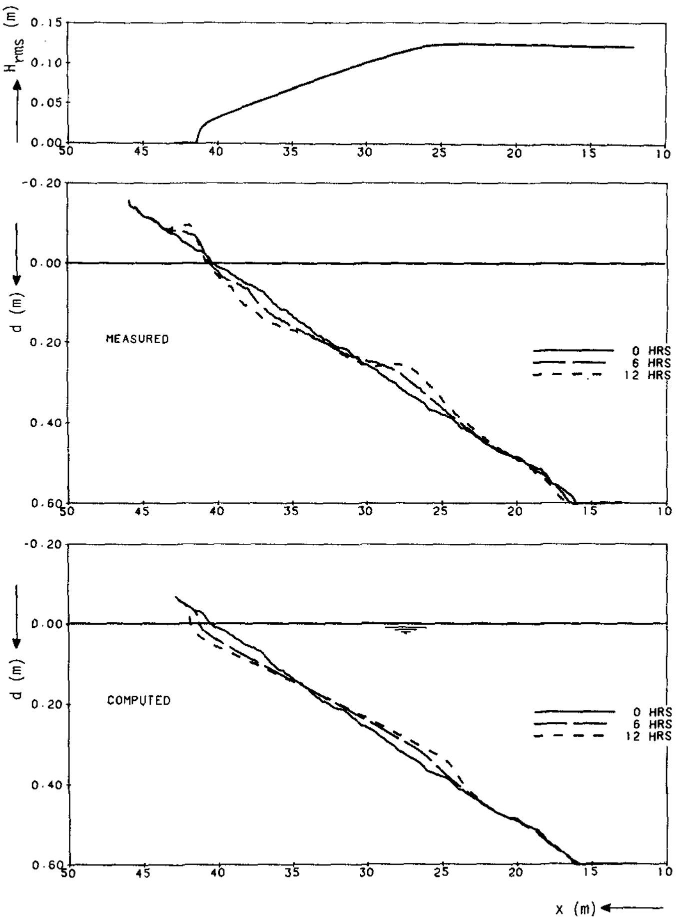

Fig. 6 Deformation of initially plane beach due to random waves: comparison with flume measurements; profiles of bottom elevation below MWL (d) and rms wave height (H_{rms}) versus distance to shore (x).

5. MODEL VERIFICATION

For the purpose of verification of the present model, a comparison was made with available field data, and laboratory measurements were carried out. The field data set concerns observed bar formation and deformation in an estuary region in the South of the Netherlands, the socalled Voordelta, which occurred after closure of one of the Southern Dutch estuaries. The profile deformation in cross-shore direction is appreciable (see Figure 5). The comparison between the hindcast results and the measurements is satisfactory, despite the fact that the wave climate and hydraulic conditions were schematized to one value for the incident wave characteristics and a fixed waterlevel. The proportionality constants B_{as}, B_{un} and B_{sl} were set at 1.0. Some characteristic parameters of this case are collected in Table 1 below.

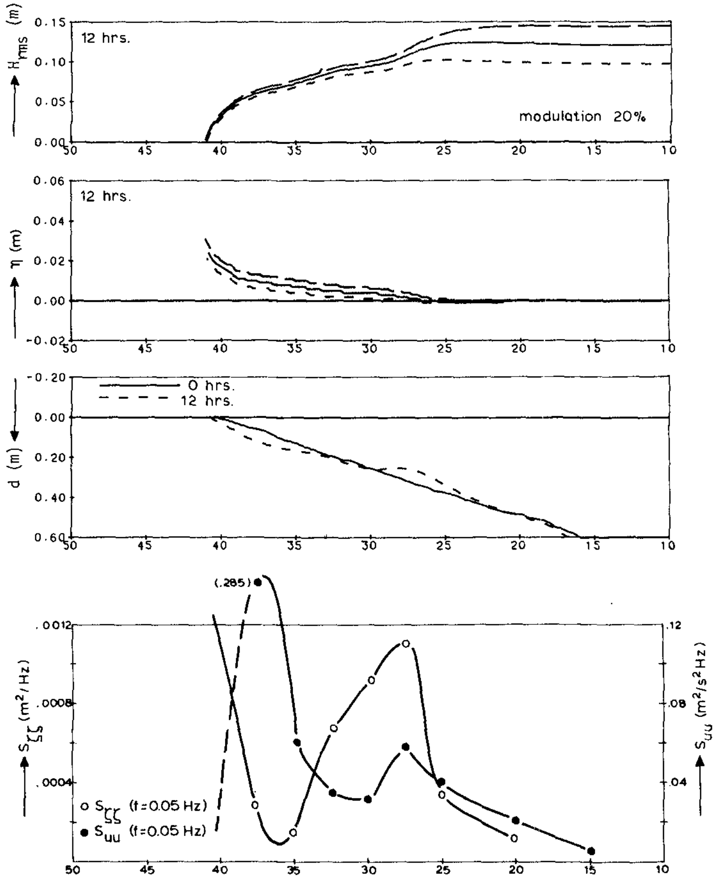

Fig. 7 Low frequency energy on the initially plane beach: computed wave height and set-up modulation and measuerd, resonant 1/2 long wave mode between surf zone's outer region and the shore.

The laboratory measurement programme involved the collection of surface elevation, near-bottom velocity and profile deformation data on two beach profiles -one initially plane and the other barred- under random wave attack (see Table 1 for some characteristic parameters). The velocity data on the barred beach were used in the earlier presented comparisons with the theoretical wave-induced cross-shore flows. The resulting beach profile deformations - restricted to the plane beach case due to space limitations - are compared with the model predictions here.

Fig. 6 gives the measured and computed profile deformation after 6 hrs and 12 hrs on the initially plane beach. It appears that the main erosion and sedimentation patterns agree, also quantitatively (the proportionality constants were all set on 2.0). Here we ignore the small sedimentation in the swash zone above the waterline, a process which is not included in the present model. An important discrepancy, however, is found in the horizontal and vertical development of the sedimentation in the surf zone's outer region. The measurements indicate a more pronounced development of an offshore bar, eventually with a negative shoreside slope. Similar indications of the absence of these developments in the model are found by the results for the profile deformations of the initially single-barred beach. The present model is unable to predict the development of the bar's shore side.

case	profile	grain diameter (μm)	$H_{rms,incident}$ (m)	f_p (Hz)
field	deltaic bar	225	1.50	0.17
laboratory	plane	90	.123	0.50
laboratory	single bar	90	.081/.133	0.50/0.50

Table 1 Characteristic parameters field and laboratory cases

Analysis of the discrepancies has resulted in strong indications that resonant surf beat modes may be the cause of the shoreside bar developments. Some results in support of this are given furtheron. First, we discuss possible surf beat sources. An inventory of the literature on this topic yields two "natural" sources (1, 2) and two "laboratory" sources (3, 4), viz.

(1) the release of or forcing by group-bound long waves due to wave breaking (Longuet-Higgins and Stewart, 1964);
(2) forcing by the time variation of the initial breaking region due to wave-grouping (Symonds and Bowen, 1984);
(3) wave paddle reflection of flume bound long waves;
(4) parasitic, wave paddle generated free long waves.

The third of these sources can be eliminated in the present measurements since the method of active wave absorption was adopted (Kostense, 1984). Discrimination between the other sources is very difficult: in all cases amplitudes and frequencies are closely related and proportional to the wave group characteristics. Whichever source is responsible, the following observation is of importance. On the initially plane beach a resonant ½ wave surf beat mode develops (see Fig. 7) between the water line and the outer surf zone, at which latter position the undertow deposits sand. The interesting, yet not well understood fact is that this mode is resonant. This is not expected by any theory, but the results in Fig. 7 for the modulated wave height indicate that both first two mentioned surfbeat sources can explain a forcing. Also, for the barred beach similar -although more complicated- observations were made.
Obviously, this is a relevant mechanism which should be included in a model. It is noted though that compared to a natural situation the relevance of this development may be somewhat overemphasized, i.e. with the timescale being approximately 4 the above storm wave conditions with fixed water level would have to be strictly constant for two days.

6. DISCUSSION AND CONCLUSION

In this paper a first suggestion is made to extend the earlier formulated model for offshore sediment transport due to undertow (Stive and Battjes, 1984) with the effects due to horizontal asymmetry in the wave motion. To arrive at these results it was necessary to model some low order odd moments of the near-bottom velocity field. An initial formulation based on a monochromatic, second order Stokes wave representation is shown to give a reasonable, first approximation to the odd velocity moments, but obviously the formulation needs improvement.

The odd velocity moments were readily used in the transport formulation of Bailard (1981). This concerns a vertically integrated description of the sediment transport in sheetflow conditions, which assumes that the instantaneous transport is proportional with some power of the instantaneous near-bottom velocity. The validity of this approach for natural surf zones needs further investigation.

A check was made of the ability of the model to predict beach profile deformations. Reasonable agreement was found in the general patterns. However, it appears that in an erosive situation only the onset of the bar formation in the outer surf zone is indicated by the model, whereas subsequent bar growth is not well predicted. The growth is believed to be associated with resonance of particular long wave modes forced by wave grouping effects. This phenomenon needs further investigation, before it can be included in a dynamic cross-shore sediment transport model.

It is finally noted that analysis of this model for several realistic cases has resulted in the following conclusions. There are two main contributions to the sediment transport, viz. that induced by the undertow yielding offshore transport and that induced by the wave-asymmetry yielding onshore transport. In low-frequency or swell conditions the latter dominates and in high-frequency or storm conditions the former dominates. These conclusions coincide with the common suggestions that low-frequency waves build up a coast and that high-frequency waves erode it.

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REFERENCES

Bagnold, R.A. (1963). "Mechanics of marine sedimentation", The Sea: Ideas and observations, vol. 3, New York, Interscience Publishers, 1963, pp. 507-526.

Bailard, J.A. (1981). An energetics total load sediment transport model for a plane sloping beach. Journal of Geophysical Research, vol. 86, no. C11, 1981, pp. 10938-10954.

Bailard, J.A. (1982). Modeling on-offshore sediment transport in the surf zone. Proc. 18th Int. Conf. Coastal Eng., 1982, pp. 1419-1438.

Bailard, J.A. and D.L. Inman (1981). An energetics bedload transport model for a plane sloping beach; local transport. Journal of Geophysical Research, vol. 86, no. C3, 1981, pp. 2035-2043.
Battjes, J.A. and M.J.F. Stive (1985). Calibration and verification of a dissipation model for random breaking waves. Journal of Geophysical Research, Vol. 90, no. C5, 1985, pp. 9159-9167.
Dally, W.R. (1980). A numerical model for beach profile evolution. M.Sc. Thesis, University of Delaware, 1980.
De Vriend, H.J. and M.J.F. Stive (1987). Quasi-3D modelling of nearshore currents. To be published in Coastal Engineering, 1987.
Dyhr-Nielsen, M. and T. Sørensen (1970). Some sand transport phenomena on coasts with bars. Proc. 12th Int. Conf. Coastal Eng., 1970, pp. 855-865.
Flick, R.E., R.T. Guza and D.L. Inman (1981). Elevation and velocity measurements of laboratory shoaling waves. Journal of Geophysical Research, vol. 86, no. C5, 1981, pp. 4149-4160.
Guza, R.T. and E.B. Thornton (1985). Velocity moments in nearshore Journal of Waterway, Port, Coastal and Ocean Engineering, Vol. 111, No. 2, 1985, pp. 235-256.
Kostense, J.K. (1984). Measurements of surf beat and set-down beneath wave groups. Proc. 19th Int. Conf. Coastal Eng., 1984, pp. 724-740.
Longuet-Higgins, M.S., (1953). Mass transport in water waves. Phil. Trans. Royal Soc., A254, 1953, p.p. 535-581.
Longuet-Higgins, M.S. and R.W. Stewart (1964). Radiation stresses in water waves; a physical discussion, with applications. Deep-Sea Res., Vol. 11, 1964, pp. 529-562.
Stive, M.J.F. and J.A. **Battjes (1984).** A model for offshore sediment transport. Proc. 19th Int. Conf. Coastal Eng., 1984, pp. 1420-1436.
Stive, M.J.F., and H.G. Wind (1986). Cross-shore mean flow in the surf zone. Coastal Engineering, 10, 1986, pp. 325-340
Svendsen, I.A. (1984). Mass flux and undertow in a surf zone. Coastal Engineering, 8, 1984, pp. 303-329.
Symonds, G. and A.J. Bowen (1984). Interactions of nearshore bars with incoming wave groups. Journal of Geophysical Research, Vol. 89, No. C2, 1984, pp. 1953-1959.