CROSS-SHORE TRANSPORT DURING STORM SURGES

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Abstract

In order to predict the amount of dune erosion due to a storm surge, a model for cross-shore transport during storm surges has been developed. During a storm with intensive wave breaking, suspended sediment transport is predominant. In the model the nett local transport is computed from the time averaged sediment concentration and the time averaged (secundary) flow profiles. Both profiles are related to the local hydraulic conditions. Finally the computed erosion profile development and the amount of dune erosion are well predicted by the model.

1. Introduction and background

On behalf of the Technical Advisory Committee on Water Retaining Structures (TAW) of the Ministry of Transport and Public Works in the Netherlands, a mathematical model for beach and dune profile changes due to storm surges is being developed. The model, which also predicts the beach profile changes in front of dune revetments (Steetzel, 1987), is to be used as a tool to check the safety of the narrow stretch of sandy beaches and dunes which protect the Dutch population against the sea. It should be used in addition to (or perhaps replace) the model as described by Vellinga (1986).

This paper focusses primarily on the computation of the nett cross-shore transport as incorporated in this model. This transport is based on both time averaged sediment concentrations and time averaged secundary currents.

2. Approach

In order to develop a cross-shore transport model series of investigations in wave flumes have been carried out.
These model tests were conducted on different scales reaching from small scale (n_d = 30) to full scale (n_d = 1).
Most investiations concern unprotected dunes, whereas there are only a few tests with dune revetments and three tests with partly protected dunes. Most of those tests were carried out in the large Delta Flume of the DELFT HYDRAULICS.

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The amount of information gathered during those tests varies from only initial and final profile to a (complete) profile development, supplemented with data on wave heights, secundary currents and sediment concentrations. The latter were measured using a transverse suction method (Bosman and Steetzel, 1986).
Tests with a lot of data are used for (cross-shore transport) model formulation and calibration. In case of only initial and final profiles the test is used for verification.

The different phases of the dynamical model are obvious. On the initial bottom profile the momentary offshore hydraulic conditions determine the local wave heights by using the wave height decay model ENDEC (Battjes and Janssen, 1978).
From this the relevant parameters (e.g. cross-shore secundary current and sediment concentrations) are computed resulting in a local cross-shore transport. This step is illustrated in more detail in the next chapter. Finally the bottom changes are computed through application of the mass balance equation of the sediment. The new bottom profile is computed and the procedure is, for a next time step, started all over again.

3. Model formulation

3.1 Introduction

The fomulation of the cross-shore transport is split up into several different aspects, respectively the principle of cross-shore transport computation in both the general and this specific case, the sediment concentrations, the secundary currents and the nett transport computation. Finally some justification of the presented cross-shore transport computation is given.

3.2 Nett cross-shore transport

In general nett cross-shore transport should be computed from the well-known equation:

$$S_x(x) = \frac{1}{nT} \int_{t=0}^{n.T} \int_{z=-d(x)}^{\eta(x,t)} u_x(x,z,t).C(x,z,t).dz.dt \qquad [kg/m/s]$$

in which:

S_x	the nett transport (x-component)	[kg/s]
T	the wave period	[s]
u_x	the velocity (x-component)	[m/s]
C	the sediment concentration	[kg/m³]
z	the level above the mean water level	[m]
η	the instantaneous water level	[m]

In case of random breaking waves a proper description of time (t) and place (x,z) variation of both velocity and concentration is not available.

In order to evaluate transport contributions, the transport at a certain level above the bottom can be written as a mean (time averaged) product of velocity and concentration.
If both parameters are split up into a mean (overbar) and a fluctuating part (accent), the nett transport can be shown to be composed

of a mean contribution of mean velocity times mean concentration and a, more difficult, correlation component:

$$\left.\begin{array}{l} u = \overline{u} + u' \\ C = \overline{C} + C' \end{array}\right\} \quad \overline{S} = \overline{uC} = \overline{u}\cdot\overline{C} + \overline{u'C'}$$

This correlation component $\overline{u'C'}$ is mainly originated from the asymmetry of the waves.

During a storm surge the transports due to intensive breaking of waves and the local beach slope are dominant above other contributions. Relative to non-breaking waves, breaking waves result in a dramatic increase in turbulence level, and therefor lift sediment into suspension over the whole depth.
The concentration fluctuations well above the bed are relatively small and hardly correlated with the velocity fluctuations (DELFT HYDRAULICS, 1989). The contribution of the correlation component will therefor be relatively small. This means that the nett transport can be computed from the mean velocities and the mean sediment concentrations.

3.3 Sediment concentration distribution

The time averaged sediment concentration (the overbar is omitted) can be written as a product of a reference concentration (at the bed level) C_o and a distribution function $f_c(z)$:

$$C(z) = C_o \cdot f_c(z) \qquad [kg/m^3]$$

Using the one-dimensional stationary convection-diffusion equation to describe the distribution, the vertical profile of the time mean sediment concentration depends on the vertical distribution of the diffusion (mixing) coefficient $\varepsilon(z)$, according to:

$$f_c(z) = \exp\left[-w_s \cdot \int_o^z \frac{dz}{\varepsilon(z)}\right] \qquad [-]$$

in which w_s stands for the (level independent) sediment fall velocity.

In order to describe the diffusion coefficient distribution $\varepsilon(z)$, a great number of measured sediment concentrations were analysed. From each of the 68 used data sets (DELFT HYDRAULICS, 1987) the $\varepsilon(z)$-distribution was computed from the concentration profile using:

$$\varepsilon(z) = -w_s \cdot [\partial(\ln C(z))/\partial z]^{-1} \qquad [m^2/s]$$

The amount of mixing increases with increasing level above the bottom, which is in accordance with results of investigations carried out by other researchers (e.g. v.d. Graaff, 1988; Ras and Amesz, 1989). As a final result for the description of the diffusion profile, $\varepsilon(z)$ is modelled as a linear function of the level above the bed, according to:

$$\varepsilon(z) = \varepsilon_o + \mu\cdot z \qquad [m^2/s]$$

in which:
ε_o the reference mixing coefficient [m²/s]
μ the vertical mixing gradient [m/s]

Both ε_o and μ are functions of the local hydraulic conditions. This linear relationship is also suggested by Songvisessomja et al (1988), who compared different $\varepsilon(z)$-relation, by analysing data from Nielsen (Nielsen, 1984).

The vertical distribution of the mean sediment concentration is now simply described by:

$$C(z) = C_o \cdot \left[1 + \frac{\mu \cdot z}{\varepsilon_o}\right]^{(-w_s/\mu)} \qquad [\mathrm{kg/m^3}]$$

in which:
C_o the reference concentration [kg/m³]
w_s the fall velocity of the bed material [m/s]

An example of the procedure is presented in Figure 1. The relevant steps are:
(i) From the measured concentration (□ -symbols the $\varepsilon(z)$-values are estimated (x-symbols);
(ii) Linear fit of $\varepsilon(z)$-values result in ε_o- en μ-values;
(iii) Fit of $C(z)$-values in combination with ε_o and μ result in $C(z)$-equation and C_o-value.

As a result of each test both concentration and diffusion distributions are available. The latter will be used as an input for the description of the secundary current (undertow).

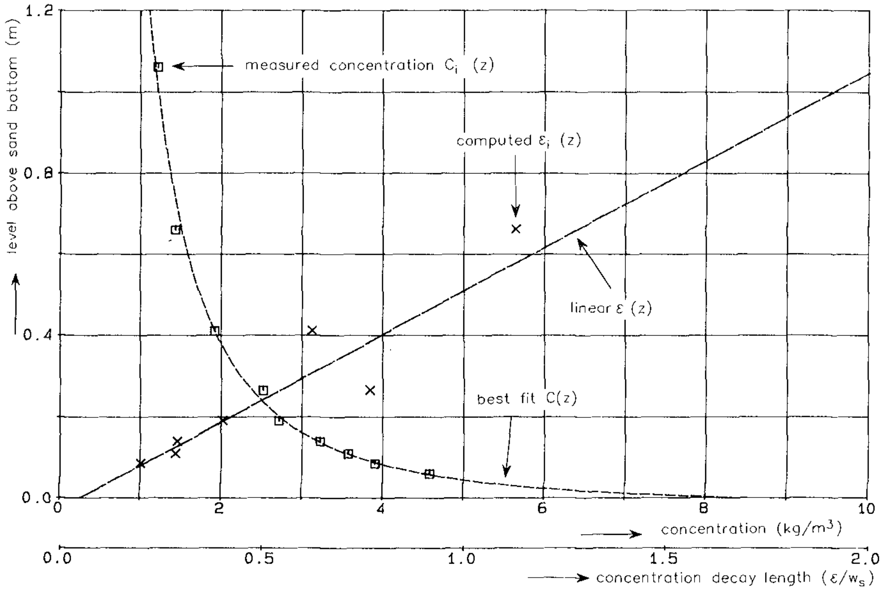


Figure 1 Example of C(z)-fit-procedure (test T1F1)

3.4 Mean velocity distribution

The suspended sediment load is transported by the mean velocity profile, which is present due to the mass transport of the breaking waves above the level of the wave troughs and results in the (compensating) so-called undertow below this level.

The description of this mean velocity profile below the trough level is started with the assumption of a constant vertical gradient in the (time mean) shear stress (Stive and Wind, 1986):

$$\frac{1}{\rho} \cdot \frac{\partial \bar{\tau}}{\partial z} = \frac{\partial}{\partial z} \left(\varepsilon(z) \cdot \frac{\partial u}{\partial z}\right) = \alpha$$

After integration follows:

$$\alpha = \frac{\bar{\tau}_t - \bar{\tau}_o}{\rho \cdot d_t} \qquad [m^2/s]$$

in which:

- $\bar{\tau}_t$ the mean shear stress at the wave trough level [N/m^2]
- $\bar{\tau}_o$ the mean shear stress at the reference level [N/m^2]
- ρ the mass density of the fluid [kg/m^3]
- d_t the vertical distance between reference and wave trough level [m]

The velocity gradient can be derived from:

$$\frac{\partial u}{\partial z} = \alpha \cdot \frac{z}{\varepsilon(z)} + \beta \cdot \frac{1}{\varepsilon(z)}$$

in which the integration constant β equals:

$$\beta = \varepsilon_o \cdot \frac{\partial u}{\partial z}(z=0) = \frac{1}{\rho} \cdot \bar{\tau}_o \qquad [m^2/s^2]$$

The final velocity profile can therefor be derived from:

$$u(z) = u_o + \alpha \cdot \int_o^z \frac{z}{\varepsilon(z)}\, dz + \beta \cdot \int_o^z \frac{1}{\varepsilon(z)}\, dz \qquad [m/s]$$

in which u_o stands for the mean velocity at the reference level. This type of equation is also presented by Okayasa et al (1988).

The vertical diffusion (mixing) coefficient is now taken equal to the sediment diffusion coefficient $\varepsilon(z)$.
The use of a non-constant but continuous diffusion coefficient distribution in undertow description differs from the approach of several other authors who use two layers of constant diffusion (e.g. de Vriend and Stive 1987).
After some integrations the mean velocity profile can be shown to be described by:

$$u(z) = u_o + K_{lin} \cdot z + K_{log} \cdot \ln\left[1 + \frac{\mu . z}{\varepsilon_o}\right] \qquad [m/s]$$

where: $K_{lin} = \frac{\alpha}{\mu}$ [1/s]

$$K_{log} = \left(\beta - \frac{\alpha}{\mu} \cdot \varepsilon_o\right)/\mu \qquad [m/s]$$

The form and magnitude of the velocity distribution below the wave trough depends on the mass flux m and the shear stress at the trough level $\bar{\tau}_t$.

For continuity reasons the mass flux m equals

$$m = \rho \int_{z=0}^{d_t} u(z)\, dz \qquad [kg/m/s]$$

Both mass flux and shear stress at trough level are based on relations by de Vriend and Stive (1987).
The mean shear stress at the reference level $\bar{\tau}_o$ is related to the mean velocity u_o at this level.

3.5 Nett local transport computation

The nett transport has to be split up into two parts: the transport below the (mean) trough level and the transport above this level.
In the lower part the nett seaward transport S_l follows from the integral of the product of mean velocity u(z) and mean concentration C(z), according to:

$$S_l = \int_{z=0}^{d_t} u(z) \cdot C(z) \cdot dz \qquad [kg/m/s]$$

From the equations for u(z) and C(z) the result of this integral can be shown to be:

$$S_l = C_o\, [u_o \cdot I_1 + K_{lin} \cdot I_2 + K_{log} \cdot I_3] \qquad [kg/m/s]$$

with:

$$I_1 = \frac{\varepsilon_o}{\mu} \cdot \frac{1}{K_1} \cdot [K_2^{K_1} - 1] \qquad [m]$$

$$I_2 = \frac{\varepsilon_o}{\mu} \cdot \frac{1}{K_1} \cdot [K_2^{K_1} (d_t - \frac{\varepsilon_o}{\mu} \cdot \frac{K_2}{K_1+1}) + \frac{\varepsilon_o}{\mu} \cdot \frac{1}{K_1+1}] \qquad [m^2]$$

$$I_3 = \frac{\varepsilon_o}{\mu} \cdot \frac{1}{(K_1)^2} \cdot [K_2^{K_1} (K_1 \ln K_2 - 1) + 1] \qquad [m]$$

and:

$$K_1 = 1 - w_s/\mu \qquad [-]$$

$$K_2 = 1 + (\mu/\varepsilon_o) \cdot d_t \qquad [-]$$

In the upper zone the nett landward transport S_u follows from:

$$S_u = \int_{z=d_t}^{\infty} u(z) \cdot C(z)\, dz \qquad [kg/m/s]$$

Since vertical gradients in C(z) near the mean water level are rather small this integral can be simplified to:

$$S_u = C_d \cdot \int_{z=d_t}^{\infty} u(z)\, dz = C_d \cdot m/\rho \qquad [kg/m/s]$$

in which:

C_d the concentration at the mean waterlevel [kg/m³]

m the mass flux above the wave trough level [kg/m/s]

Finally the total nett transport S_x can be computed from:

$$S_x = S_l + S_u$$

An example of this nett transport computation is shown in Figure 2. The right side of this figure shows the measured and best-fit concentration distribution, whereas the left part shows the measured and best fit nett velocity profile below the wave trough level. Computing nett transport results in a seaward transport of 0.77 m³/m/hr below the trough level, while the landward transport above the trough level amounts 0.33 m³/m/hr. As a consequence the total nett transport is about 0.44 m³/m/hr seaward.

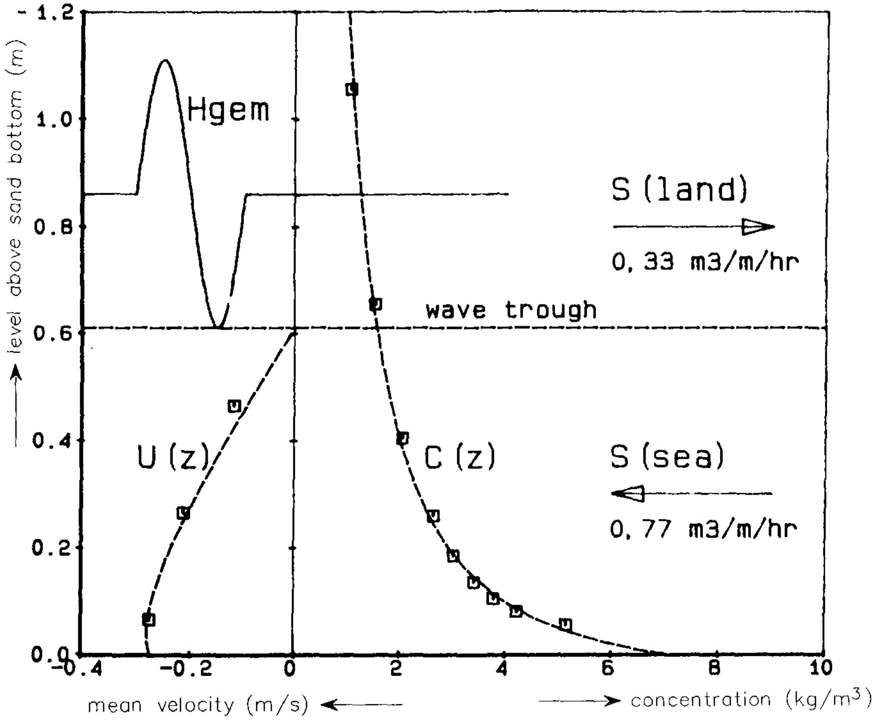


Figure 2 Transport at x = 181 m based on velocity profile and concentration profile (test T3I4); Computed transport: S(net) = 0.44 m³/m/hr

3.6 Justification

In order to give some justification on this method instantaneous transports, computed by integration of the product of C(z) and U(z) over the water depth, are compared with transports computed from frequently measured bottom profiles.

Figure 3 shows the profile development for a partly protected dune due to both 9.5 and 12.5 hrs of wave attack. In this time interval the amount of erosion above and in front of the revetment slowly increases. From this the transport at every position and time can be computed using mass balance equations.

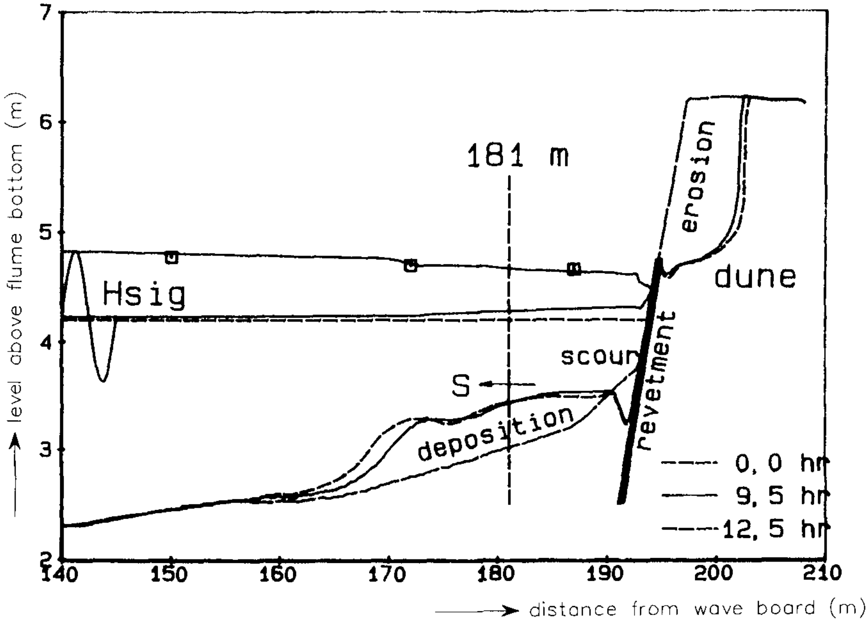


Figure 3 Example of profile development and transport in a large scale model with revetment (test T3I4); Measured transport at t = 11.25 hr: S(net) = 0.40 $m^3/m/hr$

In this case position and time correspond with the situation from Figure 2 and result in a nett transport of 0.40 $m^3/m/hr$.
This "measured" transport deviates only 10% from the former computed transport.

Figure 4 shows a comparison between 22 measured and computed transports. With the dotted line indicating a deviation of 20%, the agreement was concluded to be rather good.
It was therefor assumed that the presented method of nett transport computation in case of breaking waves gives a reliable estimate of the actual nett transport.

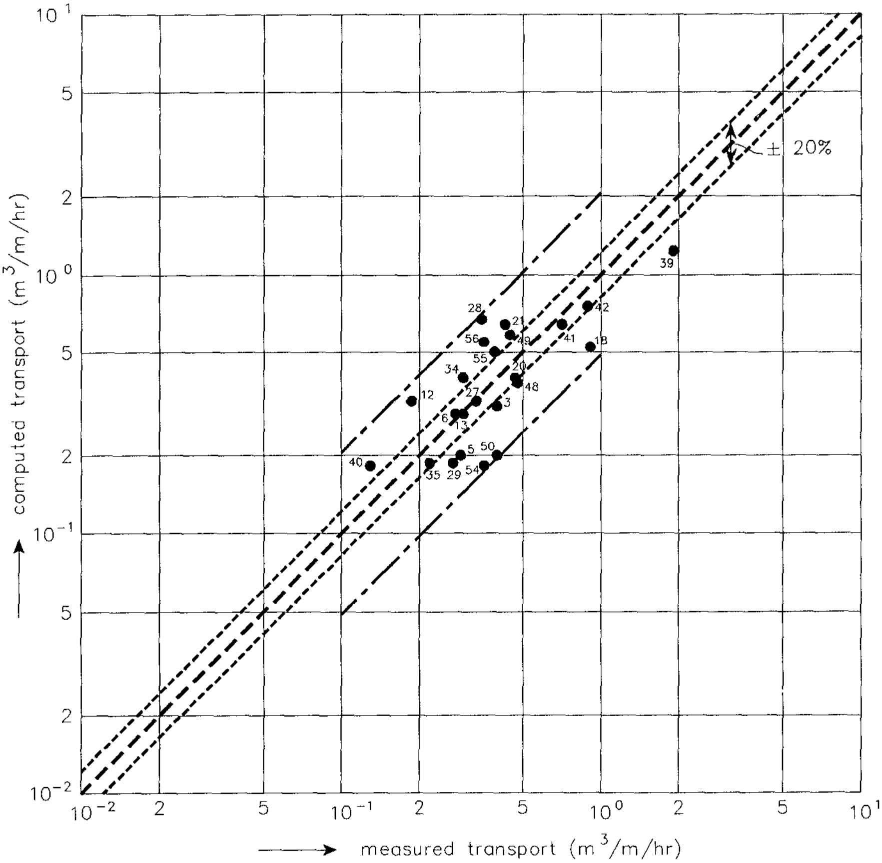


Figure 4 Comparison between measured transports (from profile development) and computed transports (using C(z) and U(z))

4. Model calibration

The calibration of the dynamical model is carried out for the "internal" process parameters, whereas the verification is based on "external" process results, such as profile development and the amount of dune erosion above the maximum surge level.

In this case the transport model is calibrated by the use of approximately 150 simultaneously measured concentration and velocity profiles during storm surge conditions in both small and large scale model tests. For calibration the relevant parameters (e.g. mass flux m and reference concentration C_o) are related to the hydraulic conditions which are computed by the use of results of a calibrated wave decay model.

The reference concentration C_o is related to both the intensity of breaking and the way of breaking (spilling or plunging) according to:

$$C_o = \rho_s \cdot K_c \cdot F_D \cdot [\rho/\bar{\tau}_{cr}]^{3/2} \cdot [F_k(y)]^{3/2} \cdot [Diss/\rho] \qquad [kg/m^3]$$

in which:

K_c a constant [-]
K_D a constant related to the sediment diameter [-]
F_k a function which describes the effect of the kind of breaking [-]
Diss the dissipation of turbulent kinetic energy [W/m²]

In case of the dissipation term there has been made a distinction between the dissipation source term in the wave energy balance equation and the dissipation of the turbulent kinetic energy Diss. The former is actually a production term of turbulent kinetic energy [Roelvink and Stive, 1989].

In order to take in account the effects of the way the waves are breaking, a relation between the near bottom magnitude of the depth and time mean turbulent energy k_o and the total turbulent energy K is suggested:

$$k_o = K \cdot F_k(y) \qquad [m^2/s^2]$$

in which y is the ratio H_{rms}/d.

Considering a exponential downward decrease in turbulence level and a characteristic penetration depth below the mean water level equal to $\alpha_k \cdot H_{rms}$, this function is:

$$F_k(y) = [\alpha_k \cdot y \, (\exp\,(1/\alpha_k \cdot y) - 1]^{-1} \qquad [-]$$

In case of $\alpha_k = ½$ this equation can be simplified to:

$$F_k = \begin{cases} 0 & y < 0.33 \\ 0.47(y-0.33) & y \gtrsim 0.33 \end{cases} \qquad [-]$$

In case of spilling breakers, wave breaking does not contribute to the reference concentration.

Figure 5 shows a comparison between measured and computed reference concentrations. With respect to the closed symbols the agreement was concluded to be satisfactory.

5. Model verification

The final verification of the model is done by comparing measured dune erosion quantities with computed profile development for 43 model tests and a small number of prototype data. Figure 6 shows a comparison of measured and computed erosion quantities in a small scale model due to different wave heights, wave periods, water levels and dune heights for constant and varying hydraulic conditions.

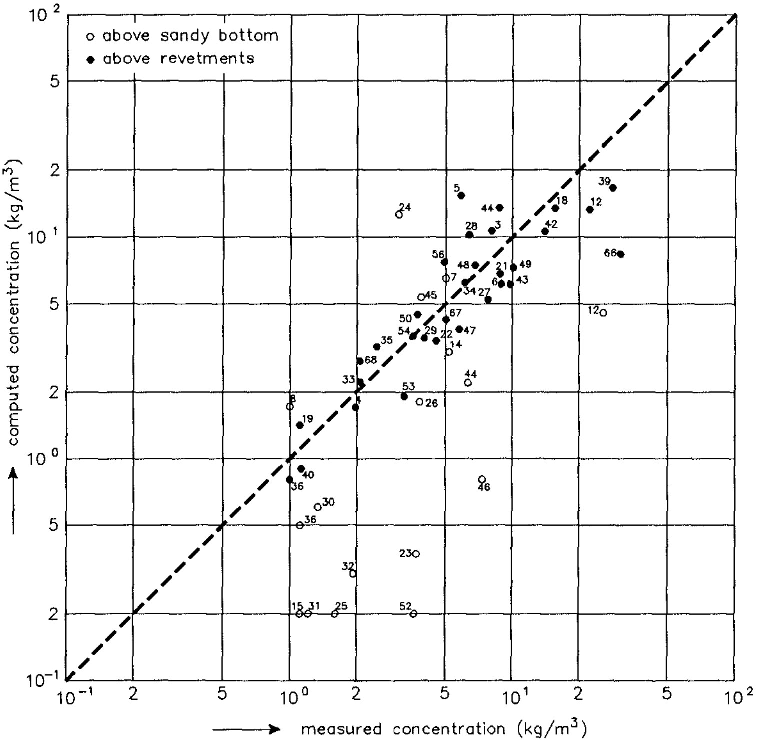
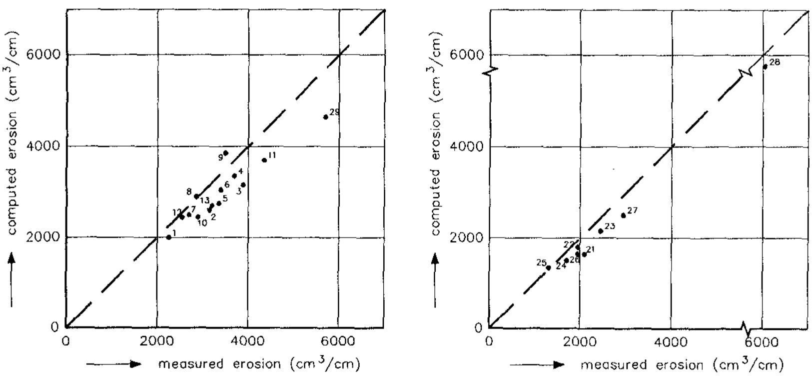


Figure 5 Comparison between measured and computed reference sediment concentrations (open symbols indicate positions above revetment)



constant hydraulic conditions varying hydraulic conditions

Figure 6 Comparison between measured and computed erosion quantities for constant and varying hydraulic conditions (small scale)

6. Conclusions

During extreme wave attack the suspended transport based on time mean concentrations and velocities is dominant. A predictive model for sediment load and transport due to extreme wave attack is presented which gives a reliable description of the transports and profile development during a storm surge. By the use of this model the amount of erosion for arbitrary coastal profiles, e.g. with bars and tidal gully or with protected dunes (dune revetments) due to arbitrary storm surge conditions can be determined.
Finally it should be revailed that in order to increase reliability of the model some further verification and (perhaps) re-calibration for unused data sets will be carried out.

7. Acknowledgement

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References

Battjes, J.A. and J.P.F.M. Janssen (1978),
Energy loss and set-up due to breaking of random waves.
Proc. 16th Conf. on Coastal Engineering, Vol. I, pp. 569-589.

Bosman, J.J. and Steetzel (1986),
Time and bed averaged concentrations under waves.
Proc. 20th Conf. on Coastal Engineering, Vol. II, pp. 986-1000.

DELFT HYDRAULICS (1987),
Systematic research on the effectiveness of dune toe revetments; large scale model investigations.
Research report H298 part I (in Dutch)

DELFT HYDRAULICS (1989),
Cross-shore transport due to extreme hydraulic conditions.
Research Report H298 part III (in Dutch).

Graaff, J. van de (1988),
Sediment concentration due to wave action.
Ph.D. Thesis, Delft Univ. of Technology, Delft, The Netherlands.

Nielsen, P. (1984),
Field measurements of time averaged suspended sediment concentration
Coastal Engineering, Vol. 8, pp. 51-72.

Okayasu, A., T. Shibayama and K. Horikawa (1988),
Vertical variation of undertow in the surf zone.
Proc. 21th Conf. on Coastal Engineering, Vol. I, pp. 478-491.

Ras, S.L. and J.A. Amesz (1989),
Concentration and diffusion coefficient distribution due to irregular and breaking waves.
M.Sc. Thesis, Delft Univ. of Technology, Delft, The Netherlands (in Dutch).

Roelvink, J.A. and M.J.F. Stive (1989),
Bar-generating cross-shore flow mechanism on a beach.
Journal of geophysical research, Vol. 94, No. C4, pp. 4785-4800, April 1989.

Songvisessomja, S. and N. Samarasinghe (1988),
Profile of suspended sediment due to prototype wave.
Proc. 6th Congress Asian and Pacific Regional Division,
Intern. Ass. for Hydr. Res., pp. 97-104, Japan.

Steetzel, J.H. (1987),
A model for beach and dune profile changes near dune revetments.
Proc. Coastal Sediments ´87, ASCE, pp. 87-97.

Stive, M.J.F. and H.G. Wind (1986)
Cross-shore mean flow in the surf zone.
Coastal Engineering, Vol. 10, pp. 325-340.

Vellinga, A.P. (1986)
Beach and dune erosion during storm surges.
Ph.D. Thesis, Delft, Univ. of Technology, Delft, The Netherlands.

Vriend, H.J. de, and M.J.F. Stive (1987),
Quasi-3D modelling of nearshore currents.
Coastal Engineering, Vol. 11, pp. 565-601, Amsterdam.