LITTORAL DRIFT AS A SOURCE OF HARBOUR SILTATION

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INTRODUCTION

The sediment transport in coastal waters, estuaries and tidal rivers are of great importance for the planning of harbours and navigational channels. Any obstacle or coastal construction changes a natural state of balance, thus the equilibrium of sediment transport of the area may be distorted. This might cause a very strong change of the sediment budget and perhaps create a need of heavy shore protection works and dredging operations.

When constructing wave breakers and berthing areas, the lay-out of the former coast line will be modified causing thus a change in sediment circulations of the area. Waves and tidal currents are the most important phenomena controlling the behaviour of sediment in the coastal zone, notably shorewards of the breaker line. The tidal currents prevail well far offshore while the wave-induced currents are dominant inside the surf zone.

The topic of this paper is mainly to demonstrate a method of calculating the sediment transport due to the oblique breaking waves at the shore line. The method followed herein is originally presented by Lonquet-Higgins, later modified by Komar.

It is well known that storms attacking the shallow coastal waters will influence heavily on the littoral drift thus increasing the total load remarkably. In our country the littoral drift causes in general no problems in common engineering works with harbour and sea way design. This is mainly due to the lack of severe tidal variations and due to the sheltered seas and specific sea bottom conditions with rather sparse movable type granular soils. However, when trying to keep pace with the state of art in coastal engineering the different solutions of special problems must be known. Knowing the different aspects of coastal phenomena is a vital prerequisite for the possibility to co-operate with foreign countries and companies, say for example in the arctic coastal regions.

In connection with the littoral drift calculation the simple mathematical modelling of more global sediment transport in marginal seas and coastal waters are shown.

CALCULATION OF LITTORAL DRIFT

A total amount of suspended sediment and the character of its distribution across the surf zone depends much on the grain size distribution of local sediments and on the bed topography, and changes thus irregularly with distance from the shore line. A mass transport and the wave-induced oscillatory motion of sediments are equally important seaward and shorewards of the breaker line. Reflection of waves modify considerably the resulting fields of water velocities.

The oblique waves breaking at the shoreline generate a longshore current which primarily is confined to the surf zone. This phenomenon causes the sand transportation along the shoreline /2/.

Here the total load consisting of the sum of suspended and bed load due to the oblique breaking waves at the shoreline is calculated. The littoral drift parallel to the hypothetical coast line was evaluated using the method of Refs. /3/ and /4/. The selected environmental conditions have their equivalence in full scale. Thus, the verification later with full scale measurements is possible. In this very moment the full scale measurement process is still going on, and only a qualitative data is available.

Selecting the input data

The environmental data used in the calculations are shown in table 1.

Table 1. Basic data used in calculations

Wave height	H_0	2.2 m
Period	T	6.35 s
Wind direction	α_w	~ 45^0 N
Wave length	L_0	63 m
Water depth	H	varies

Subscript $_0$ refers to the deep water conditions.

The profile of the beach was assumed to be regular with a constant slope angle. The slope angle was selected according the full scale observations and is equal to 2.1^0.

From deep water the waves are refracted and shoaled until the breaking point by applying the usual refraction and shoaling theory for linear waves. However, due to the strong nonlinear character of shallow water waves and the breaking phenomenon the limits of the linearity assumptions should be taken into account.

The wave breaking conditions were estimated by using the following formulae /1/ and /5/:

(Miche approx.) $$\left(\frac{H_w}{L}\right)_{max} = 0.142 \tanh \frac{2\pi h}{L} \quad (1)$$

(Komar & Byrne approx.) $$H_b = 0.39 g^{1/5} (H_0{}^2 T_b)^{2/5} \quad (2)$$

(Goda approx.) $$\frac{H_b}{h_b} = 0.17 \, (L_0/h_b) \, \{1-\exp[1.5\pi (h_b/L_0) (1-1.55)^{4/3}]\} \quad (3)$$

where H_w is wave height
H_b is breaking wave height
g is acceleration due to gravity
h_b is water depth at breaking point

Thus the breaking wave height was estimated to be 2.5 m at the depth of 3.3 m.

After breaking, the wave height decreases over a certain distance approaching a kind of equilibrium state, where the local wave height is very close to half the local water depth /2/. In the case of irregular waves the significant wave height should be used instead of a wave height.

The breaking conditions are governed by the surf similarity parameter.

$$\xi_b = \frac{S}{\sqrt{H_b/L_0}} \tag{4}$$

where S is slope inclination ~ tanß

ß is slope angle

According to Ref. /5/ the following limits can be obtaied:

$\xi_b \leq 0.2$	for spilling breaker
$\xi_b \leq 0.4$	for plunging breaker
$0.2 < \xi_b < 0.4$	for intermediate breaker

Different breaker types are shown in Fig. 1. The breaking condition relevant for the littoral drift is the spilling breaker.

Using the general solutions of wave refraction and assuming that the bottom contours are parallel with the shoreline gives for the breaking angle between the waves and the shoreline a value 32.7^0. Taking morover into account the slope of inclination of 2.1^0 the surf zone width is 90 m.

Calculating principles

The longshore velocity distribution over the surf zone was calculated using the following equations /3/:

$$V = AX + B_1 X^{P_1} \qquad 0 < X < 1 \tag{5}$$

$$V = B_2 X^{P_2} \qquad 1 < X \tag{6}$$

where $V = v/v_0$, $X = x/x_b$.

v is the longshore current velocity at distance x from the shoreline. x_b is the width of the surf zone /3/. v_0 is given by

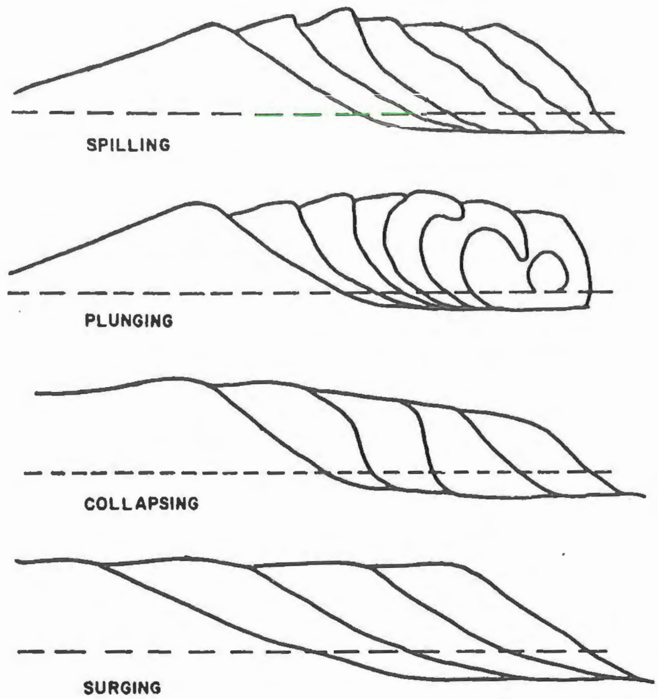

Figure 1. Wave breaking types at shallow water /1/.

$$v_0 = \frac{5\pi}{16} \quad \frac{\gamma\xi^2}{(1+\gamma)^{1/2}} \sqrt{gh_b} \, \frac{\tan\beta}{c} \, \sin\alpha_b \cos\alpha_b \tag{7}$$

where γ is the ratio of wave height to water depth
c is the frictional drag coefficient
β is the angle of beach slope

The coefficients of the formula (7) are

$$P_1 = -\frac{3}{4} + \sqrt{\left(\frac{9}{16} + \frac{1}{\xi P}\right)} \tag{8}$$

$$P_2 = -\frac{3}{4} - \sqrt{\left(\frac{9}{16} + \frac{1}{\xi P}\right)} \tag{9}$$

$$A = \frac{1}{(1-2.5\ P\xi)} \tag{10}$$

$$B_1 = \frac{P_1 - 1}{P_1 - P_2} A \qquad (11)$$

$$B_2 = \frac{P_1 - 1}{P_1 - P_2} A \qquad (12)$$

$$P = \frac{\pi N \tan\beta}{\gamma^c} \qquad (13)$$

$$\xi = \frac{1}{(1+0.375\gamma^2)} \qquad (14)$$

The velocity distribution is shown in Fig. 2 where the straight line represents the velocity at mid surf position calculated by the equation /4/:

$$\upsilon_1 = 1.17\ (gH_b^{1/2})\ \sin\alpha_b \cos\alpha_b. \qquad (15)$$

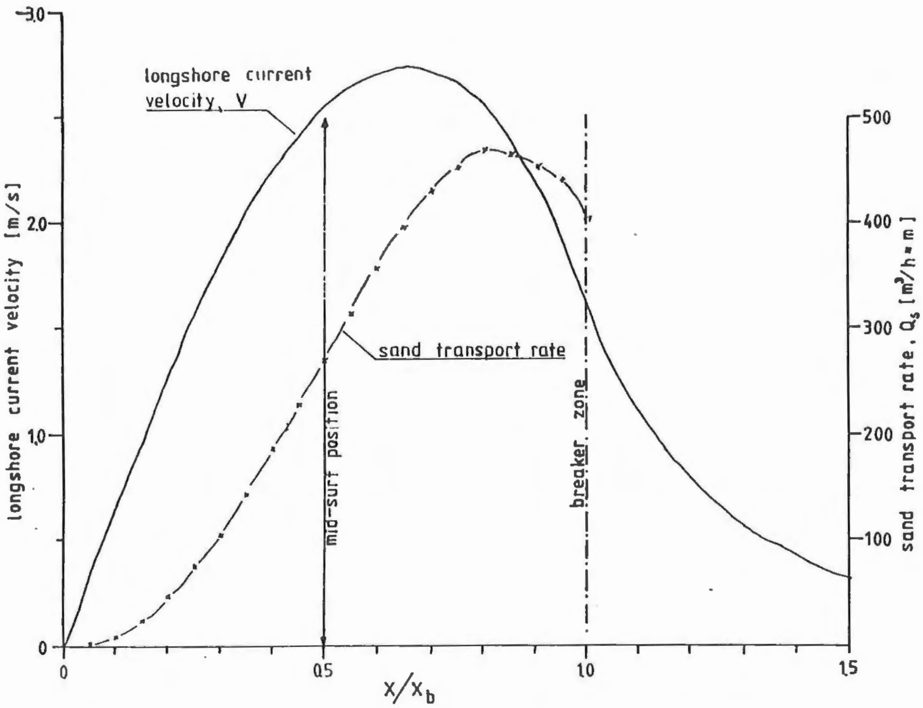

Figure 2. The velocity and sediment transport distribution curves.

The total sediment transport rate was evaluated by the formula:

$$I_l = K\ (ECn)_b\ \sin\alpha_b \cos\alpha_b = KP_l \qquad (16)$$

where the coefficient $K \sim 0.77$
and $(ECn)_b$ is the wave energy flux per unit wave crest length. The term $(ECn)_b$ multiplied by $\cos\alpha_b$ places it on the basis of a unit shoreline length and multiplication by $\sin\alpha_b$ yields the longshore component. The term P_l is commonly referred as the longshore component of the wave energy flux /4/.

The dimension of I_l is the immersed weight of sand transport rate and it is possible to relate it to the volume transport rate Q_s by:

$$I_l = (\rho_s - \rho)\ g\ a'\ Q_l \qquad (17)$$

where a' corresponds to the pore space of the sediment.

Because the maximum load will be evaluated the porosity of the bottom soil is assumed to be 0.4 which yields 1-0.4 = 0.6 for the term a'.

The local longshore sediment transport rate can be given as /3/:

$$i_l = \frac{\pi K_l}{4}\ c'\rho g \gamma^2 hv \qquad (18)$$

where v is given by equation (5) or (6) and c' is the drag coefficient for the wave motion given by:

$$c' = \frac{1}{2} f_w \quad \text{where} \qquad (19)$$

$$f_w = e^{[-5.977 + 5.213(a_0/r)^{-0.194}]} \qquad (20)$$

and a_0 is the amplitude of orbital motion.

Here r is the roughness of sea bottom which equals the diameter of soil particle by neglecting the effects of ripples and dunes.

The coefficient K_l is according to Ref. /2/:

$$K_l = \frac{0.77\ (1+\gamma)}{\frac{5\pi^2}{8}\gamma\,\xi^2\left(\frac{A}{3} + \frac{B_1}{P_1+2}\right)} \qquad (21)$$

Integrating the equation 18 over the surf zone and identifying it with equation 16, yields the total immersed weight of sand transport

$$I_l = \int_0^{x_b} i_l \, dx = KP_l \tag{22}$$

Results

The solution of equation 16 gives 1.47 m^3/s for the regular wave height of 2.2 m which overestimates the longshore total transporting rate. The use of the significant wave height as the regular wave height means higher value of the energy flux in the terms of $(ECn)_b$: The energy density of irregular waves can be given by

$$E = \rho g m_0 \tag{23}$$

Here m_0 is the zeroth moment:

$$m_0 = \int_0^{\infty} \omega^0 S(\omega) d\omega = \int_0^{\infty} S(\omega) d\omega \tag{24}$$

where $S(\omega)$ is the spectral density

For the regular waves it yields:

$$E = \frac{1}{2} \rho g a_0^2 = \frac{1}{8} \rho g H^2 \tag{25}$$

Taking into account the relation between the significant wave height and m_0, $H_s = 4 \sqrt{m_0}$, we get the relation of the significant wave height versus the mean wave height:

$$\frac{H}{H_s} = \frac{1}{\sqrt{2}} \tag{26}$$

Thus the solution of equation 16 can be reduced to:

$$I_l \left(\frac{1}{\sqrt{2}}\right)^2 = \frac{I_l}{2} \tag{27}$$

which yields the volume of sediment transport $Q_s = 0.75$ m^3/s.

The sediment rate distribution along the surf zone is shown in Fig. 2 with dashed line.

When observing the sediment distribution curve, it can be noted that the highest sediment concentrations, thus the highest sediment transport intensity, are encountered at places of intensive wave breaking. Generally the vertical profiles of sand concentration can be categorized in the following three groups /7/:

Type 1 represents the shoaling zone distribution, far seawards of the breaker line.

Type 2 with less district decrease in the upper layer is encountered in the zone of wave dissipation (surf zone beyond the places of wave breaking).

Type 3 occurs at the breaker lines with inherent intensive turbulent mixing.

The distribution curves are shown in Fig. 3. It should be noted that Fig. 3 represents merely the suspended distribution as the total load.

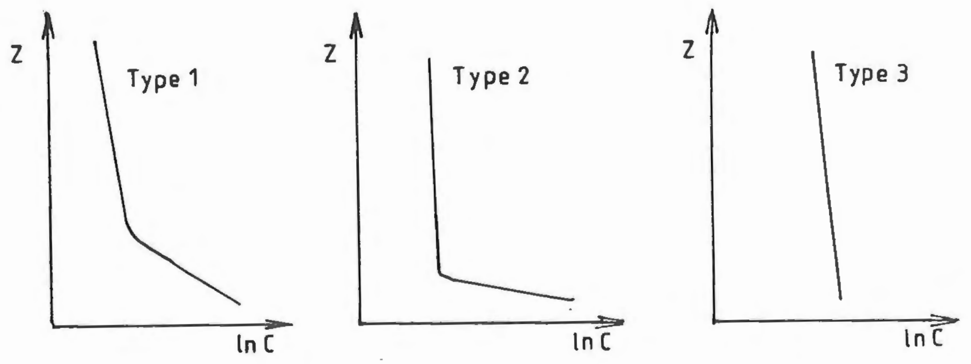

Figure 3. Different sediment distribution-types near the shore line.

The approximate formula of Ref. /1/ was used as a verification.

$$Q_s = 1.4\ (10^{-2})\ (a_0^2)\ (c_0^2)\ K_R^2\ \sin\alpha_b \cos\alpha_b \qquad (28)$$

yields the value $Q_s = 0.95\ m^3/s$ for the same conditions.

Thus in spite of above mentioned assumptions the results seem to be of same order than the solution of general CERC-formula for "the average conditions".

GLOBAL SEDIMENT TRANSPORT IN COASTAL AREA

Because the littoral drift phenomenon explains only one part of the whole sediment circulation process more global evaluation is needed to solve the total sediment transport of the coastal area.

When attempting to evaluate sediment transport in coastal waters in total, the interaction between flow, sediment transport and the bottom topography must be known. One reasonable way trying to solve sediment transport character is to model the dynamics of pure water and use computed velocity and pressure fields to the sediment transport calculations.

The equation of motion, the so called Reynolds equation is possible to show in the form:

$$\frac{\partial v_i}{\partial t} + v_j \frac{\partial v_i}{\partial x_j} + f\ v_j = \frac{1}{\rho} \left(k_i - \frac{\partial P}{\partial x_i}\right) + \frac{\partial}{\partial x_j} \left(A_j \frac{\partial v_i}{\partial x_j}\right) \qquad (29)$$

where v_i is flow velocity in x_i-direction
P is pressure
k_i is external force in x_i-direction
f is coriolis tensor
A_j is eddy viscosity coefficient in x_j-direction

The equation of continuity is:

$$\frac{\partial v_j}{\partial x_j} = 0 \qquad (30)$$

The analytical integration of the equations above is impossible mainly due to the varying topography of coastal waters and the nonlinear shallow water effects. The equations can be solved by using finite difference or finite element techniques. For the horizontal flow field the equations above can be modified to the form /6/:

$$\frac{\partial\bar{u}}{\partial t} + \bar{u}\,\frac{\partial\bar{u}}{\partial x} + \bar{v}\,\frac{\partial\bar{u}}{\partial y} - f\bar{v} + \frac{r}{h+\eta}\left|\bar{u}^2 + \bar{v}^2\right|^{1/2}\bar{u} - A_x\left(\frac{\partial^2\bar{u}}{\partial_x{}^2} + \frac{\partial^2\bar{u}}{\partial_y{}^2}\right) + g\frac{\partial\eta}{\partial x} = 0$$

$$\frac{\partial\bar{v}}{\partial t} + \bar{u}\,\frac{\partial\bar{v}}{\partial x} + \bar{v}\,\frac{\partial\bar{v}}{\partial y} - f\bar{u} + \frac{r}{h+\eta}\left|\bar{u}^2 + \bar{v}^2\right|^{1/2}\bar{v} - A_x\left(\frac{\partial^2\bar{v}}{\partial x^2} + \frac{\partial^2\bar{v}}{\partial y^2}\right) + g\frac{\partial\eta}{y} = 0$$

$$\frac{\partial\eta}{\partial t} + \frac{\partial}{\partial x}\left((h+\eta)\,\bar{u}\right) + \frac{\partial}{\partial y}\left((h+\eta)\,\bar{v}\right) = 0 \qquad (31)$$

The term A, the eddy viscosity coefficient, describes the turbulence effect on the kinematic viscosity of the flow and can be related as:

$$A = N - (1/\kappa)\ln N \qquad (32)$$

where κ is von Karman constant.

According to the Prandtl-mixing layer theory the term N can be stated as:

$$N = u \;/\; u_* = u_*\;\delta/\nu \qquad (33)$$

where ν is the viscous sublayer.

Assuming a logarithmic vertical velocity profile to the averaged velocity the shear stress velocities u_* and v_* (horizontal and vertical directions) are possible to write in the forms:

$$u_* = 0.007\bar{u} \qquad (34)$$

$$v_* = 0.007\bar{v} \qquad (35)$$

The sediment transport rate is now possible to calculate using for example the general total load formulas of Ref. /8/.

CONCLUSIONS

The total sediment transport rate due to the oblique waves was calculated. The main topic was to demonstrate the sediment transport concepts in the purpose of harbour design: the loose sea bottom and shore sediments can build a source of heavy harbour siltation and decrease the navigability of the ship channels, too. In connection with the calculation procedure some simplified assumptions were made. The assumptions used and the regular wave simplification did not decrease the accuracy of results essentially. This was checked by using the formula developed for "average conditions".

The method can easily be widened to evaluate the annual total rate of sediment transport. Then both the percentage distribution of wind directions and wave height and period distribution must be known.

Because the littoral drift phenomenon explains only one part of the whole sediment transport budget of the coastal area a simple mathematical modelling was presented for the more global sediment transport evaluation.

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