京 都 大 学 防 災 研 究 所 年 報　第48号 B　平成17年4月
Annuals of Disas. Prev. Res. Inst., Kyoto Univ., No. 48 B, 2005

A simple formulation of the non-cohesive sediment-transport

Benoit CAMENEN*, Magnus LARSON** and Takao YAMASHITA*

* Research Center for Disaster Environment, DPRI, Kyoto University, Japan
** Lund University, Sweden

Synopsis

This manuscript presents a simple and robust total load formula (bed load and suspended.load) valid for the nearshore region. It takes into account the effects of a wave and current interaction, as well as the effects of the breaking waves and of the possible phase lag between the instantaneous velocity and the sediment concentration (in case of bed load for the sheet flow regime). The bed load formula is based on the bottom shear stress concept. For the suspended load transport, a simple formula is proposed assuming an exponential profile for the sediment concentration, and a mean sediment diffusivity and current velocity over the depth. A large set of laboratory and field experimental data was used to validate the formulas.

Keywords: sediment transport, bed load, suspended load, waves, current, nearshore

1. Introduction

The prediction of non-cohesive sediments transport is vital importance for the maintenance and management of coastal environments. The complexity of the phenomena entails the use of semi-empirical formulas which can be used in depth averaged models. This paper presents simple and robust formulas for the bed load and the suspended load for a wave and current interaction, including breaking wave effects.

2. Bed load

2.1 Steady current

Sand bed load transport was first studied in case of steady flow. The earliest formulas still widely used were based on the concept that bed load is a function of the bottom shear stress (Meyer-Peter & Müller, 1948; Einstein, 1950). The bed load appeared to be proportional to the dimensionless shear stress (or Shields parameter θ) to the power $n \approx 1.5$. The Shields parameter is defined as follows:

$$\theta_c = \frac{\tau_c}{(\rho_s - \rho) g d_{50}} \tag{1}$$

where τ_c is the current related shear stress, ρ_s and ρ the sediment and water density, g the acceleration of the gravity, and d_{50} the median grain size. A critical value of the Shields parameter θ_{cr} was used as a limit beyond to it no transport occurs, such as the dimensionless sediment flux,

$$\Phi_b = \frac{q_{sb}}{\sqrt{(s-1) g d_{50}^{\ 3}}} = A\left(\theta - \theta_{cr}\right)^n \tag{2}$$

where $s = \rho_s / \rho$ is the relative sediment density.

This kind of expression however appears to overestimate the sediment flux when the Shields parameter is a few times larger than its critical value.

Moreover, the prediction of the critical Shields stress is subjected to some uncertainties. Thus, as observed in Fig. 1, some sediment transport may occur even when $\theta < \theta_{cr}$ because of the uncertainties on its prediction.

The data used were selected assuming bed load was prevailing: *i.e.* laboratory data with flat beds (from Brownlie, 1981), river data with gravels (Smart, 1984, 1999; Nikora & Smart, 1997), and sheet flow data in pressured close conduits (Wilson, 1966, Nnadi & Wilson, 1992).

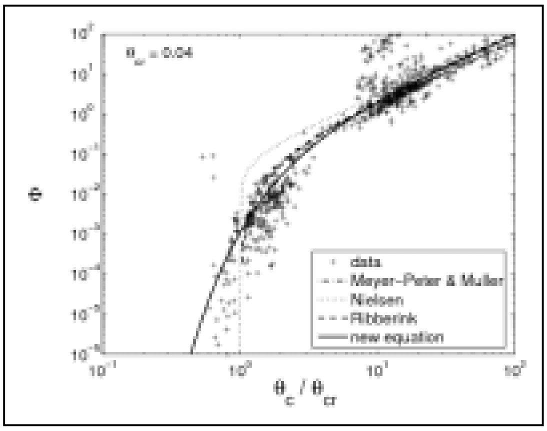

Fig. 1: Effect of the critical Shields parameter on bed load transport rate: comparison between data and the studied formulas.

A new expression for the bed load transport was thus introduced to improve the prediction of the sediment transport for low values of the Shields parameter by introducing an exponential expression of the critical Shields parameter effects:

$$\Phi_b = a\theta_c^{3/2} \exp\left(-b\frac{\theta_{cr}}{\theta_c}\right) \quad (3)$$

where a calibration with the data yields $a = 12$ and $b = 4.5$.

Tab. 1 presents the overall results for the studied formulas (Meyer-Peter & Müller, 1949, Nielsen, 1992; and Ribberink, 1998) and the new equation. It appears clearly that the new relationship improves the prediction of the bed load fluxes. In Fig. 2 is presented the results obtained with Eq. 3 compared to the experimental data. It appears that the results do not depend on the data sets except for the Willis et al. data where a large underestimation is observed. However, as fine sediments were used for this experiment ($d_{50} = 0.1$ mm), suspended load is suspected to have occurred during the experiment.

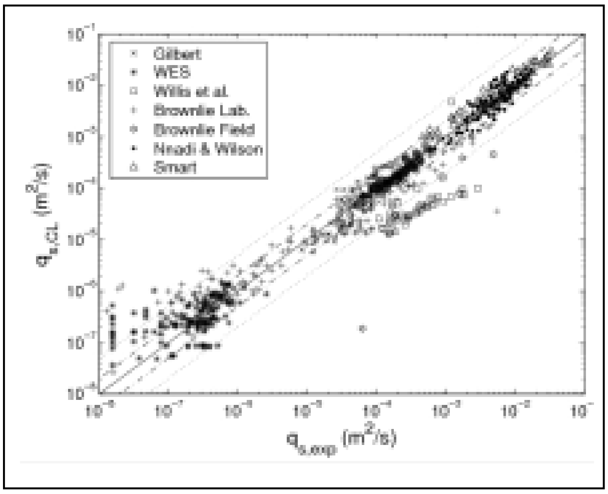

Fig. 2: Comparison between bed load transport predicted by the new formula (Eq. 3) and experimental data.

Table 1: Prediction of the bed load transport rate within a factor 2 and 5 of the measured values as well as the root-mean-square (rms) errors.

Formula	Pred x2	Pred x5	E_{rms}
Meyer-Peter	66%	87%	0.30
Nielsen	57%	75%	0.46
Ribberink	69%	89%	0.25
Eq. 3	78%	93%	0.15

2.2 Wave and current interaction

(1) Development of the formula for a wave and current interaction

In order to generalize the proposed formula to include the effect of waves, Eq. 3 was written in the wave direction and its normal direction:,

$$\Phi_{bw} = a\sqrt{\theta_{cw,net}}\,\theta_{cw,m} \exp\left(-b\frac{\theta_{cr}}{\theta_{cw}}\right) \quad (4a)$$

$$\Phi_{bn} = a\sqrt{\theta_{cw,mc}}\,\theta_{cw,m} \exp\left(-b\frac{\theta_{cr}}{\theta_{cw}}\right) \quad (4b)$$

where *a* and *b* are coefficients (with the tentative values of 12 and 4.5, respectively, as given by comparison with steady current data) and the subscript *cw* refers to waves and current in combination. Eq. 4 was proposed a bit *ad hoc* and would describe sediment transport as a product between a transporting term ($\theta_{cw,net}$) and a stirring term ($\theta_{cw,m}$). The stirring term may be estimated based on the mean combined shear stress from waves and current, which for an arbitrary angle φ (*cf.* Fig. 3) between the waves and the current is written,

$$\theta_{cw,m} = \sqrt{\theta_{cw,mw}^{\;2} + \theta_{cw,mc}^{\;2}} \tag{5}$$

where the wave related and current related mean Shields parameter are defined as follows:

$$\theta_{cw,mw} = \frac{1}{T_w}\int_0^{T_w} \frac{f_{cw}(U_c \cos\varphi + u_w(t))^2}{2(s-1)gd_{50}}dt \tag{6a}$$

$$\theta_{cw,mc} = \frac{f_{cw}(U_c \sin\varphi)^2}{2(s-1)gd_{50}} \tag{6b}$$

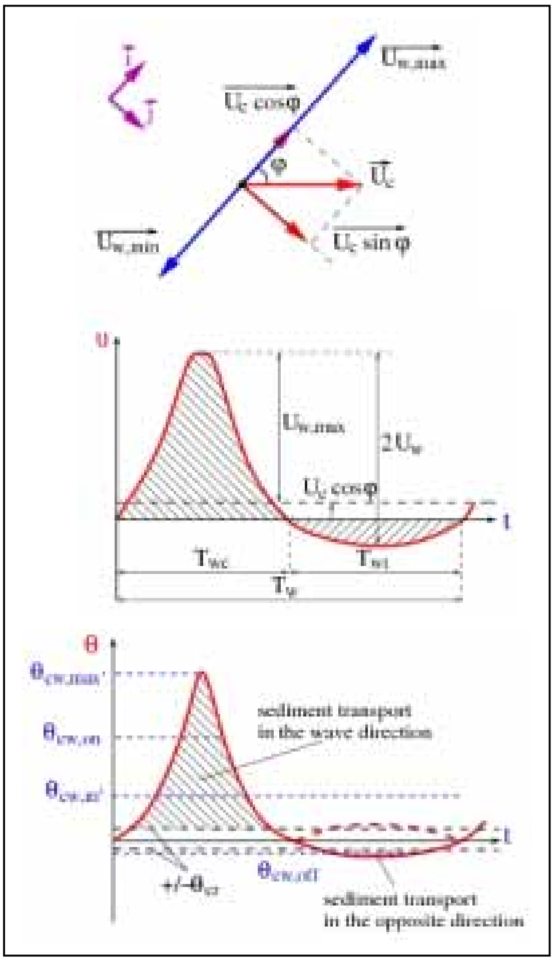

Fig. 3: (a) Definition sketch for wave and current interaction and (b) a typical velocity profile over a wave period in the direction of the waves including the effect of a steady current, and (c) induced instantaneous Shields parameter profile.

The transporting term is defined as the difference between the mean value of the onshore instantaneous Shields parameter and the mean value of the offshore instantaneous Shields parameter

$$\theta_{cw,net} = \theta_{cw,on} - \theta_{cw,offt} \tag{7}$$

with,

$$\theta_{cw,on} = \frac{1}{T_{wc}}\int_0^{T_{wc}} \frac{f_{cw}(U_c \cos\varphi + u_w(t))^2}{2(s-1)gd_{50}}dt \tag{8a}$$

$$\theta_{cw,off} = \frac{1}{T_{wt}}\int_{T_{wc}}^{T_w} \frac{f_{cw}(U_c \cos\varphi + u_w(t))^2}{2(s-1)gd_{50}}dt \tag{8b}$$

where T_{wc} and $T_{wt} = T_w - T_{wc}$ are the portions of the wave cycle for which the combined velocity of the waves and the current is positive and negative, respectively, T_w the wave period, f_{cw} the friction factor for waves and currents combined, u_w the instantaneous wave velocity, and t time. As a first approximation, f_{cw} is taken to be constant, although it should vary with time. Madsen and Grant (1976) suggested a linear combination between the friction coefficients for current (f_c) and waves (f_w) according to,

$$f_{cw} = Xf_c + (1-X)f_w \tag{9}$$

where $X = U_c/(U_c + U_w)$.

(2) Comparison with data

Data on bed-load transport under waves and current are more limited than corresponding data for steady currents. In spite of this, several data sets were compiled from the literature and analyzed for the purpose of comparison with predictions by Eq. 4. Most of the data are from oscillatory wave tunnels (OWT, *cf.* Horikawa *et al.*, 1982, Sawamoto & Yamashita, 1986, Ahilan & Sleath, 1987, Watanabe & Isobe, 1990, King, 1991, Dibajnia & Watanabe, 1992, Ribberink & Chen, 1993, Ribberink & Al Salem, 1994, Dojmen-Janssen, 1999, and Ahmed & Sato, 2003). Previously, experimental studies were often carried out using an oscillating tray (OT; oscillating bed in a tank of still water, *cf.* Kalkanis, 1964, Abou-Seida, 1965 and Sleath, 1978). The OWT data have the advantage of producing large orbital velocities for mainly bed-load conditions. The experimental cases involved both symmetric and asymmetric waves with and without a steady current. In case of symmetric waves without a current the half-cycle transport was evaluated. A recent data set using large wave flume (LWF) was obtained recently by Dohmen-Janssen & Hanes (2002). For all these laboratory experiments, the bed load was prevailing.

In case of experimental data with waves only, the calibration of Eq. yields to the coefficient $a = 6$, which is surprisingly lower than for the current case. However, Soulsby (1997) found similar results using the Meyer-Peter & Müller equation with the maximum

Shields parameter due to the waves. For this reason, in case of an interaction between waves and current, the following empirical equation for a is provided:

$$a = 6 + 6Y \tag{10}$$

where $Y = \theta_c / (\theta_c + \theta_w)$.

Tab. 2 presents the overall results for the studied formulas and the new equation using the experimental data on a half cycle. It appears that Eq. 4 yields the best results among the studied formulas, especially the root-mean-square error.

Table 2: Prediction of the bed load transport rate within a factor 2 and 5 of the measured values as well as the root-mean-square errors for wave and current interaction (half cycle).

Formula	Pred x2	Pred x5	E_{rms}
Bailard & Inman	48%	83%	0.34
Dibajnia & Watanabe	34%	81%	0.43
Ribberink (k_s=$2d_{50}$)	37%	84%	0.43
Eq. 4	64%	96%	0.15

Tab. 3 presents the overall results for the studied formulas and the new equation using the experimental data on a full cycle. The dispersion of the results is much larger. But as Dibajnia & Watanabe (1992) observed, some phase lag may occur between the sediment concentration and the instantaneous shear stress. This may decrease the net sediment transport, and even induce a net sediment transport in the opposite direction.

Table 3: Prediction of the bed load transport rate within a factor 2 and 5 of the measured values as well as the root-mean-square errors for wave and current interaction (full cycle).

Formula	Pred x2	Pred x5	E_{rms}
Bailard & Inman	45%	67%	4.4
Dibajnia & Watanabe	42%	75%	7.1
Ribberink (k_s=$2d_{50}$)	32%	55%	12.6
Eq. 4	48%	73%	9.8
Eqs. 4 and 12	54%	82%	4.5

(3) Phase lag effects

Dohmen-Janssen (1999) made an extensive study on sediment phase lag in case of the sheet flow regime. She found that phase lag effects start to occur when the following criterion is reached:

$$p_{pl} = \frac{2\pi\delta_s}{W_s T_w} > 0.35 \tag{11}$$

where $\delta_s = 10 d_{50} \theta_w$ is the sheet flow layer and W_s is the settling velocity of the sediment. It appears on Fig. 4 (where predicted sediment bed load using Eq. 4 were plotted versus experimental data for cases without current) that all the data that are badly predicted correspond to cases where phase lag effects are non negligible following Eq. 11.

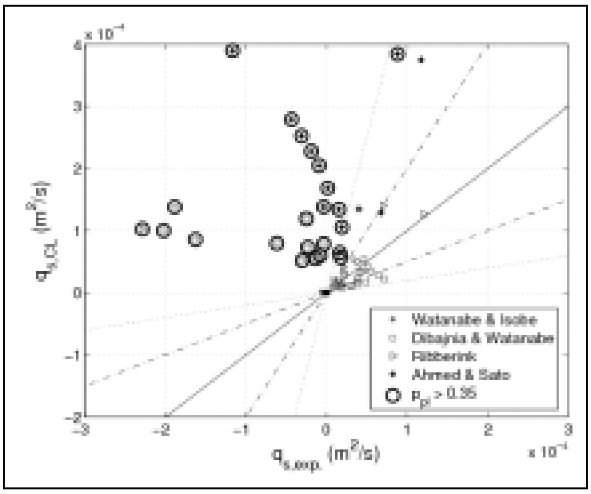

Fig. 4: Comparison between bed load transport predicted by the new formula (Eq. 4) and experimental data where the phase lag effects may have occurred.

To improve Eq. 4, a modification of the term $\theta_{cw,net}$ was introduced in order to take account the wave lag effects:

$$\theta_{cw,net} = (1 - \alpha_{pl})\theta_{cw,on} + (1 + \alpha_{pl})\theta_{cw,off} \tag{12}$$

where the coefficient α_{pl} has been calibrated using experimental data where phase lag obviously occurred.

$$\alpha_{pl} = \alpha_c - \alpha_t \tag{13a}$$

$$\alpha_j = \frac{\nu^{0.25} U_{wj}^{\ 0.5}}{W_s T_{wj}^{\ 0.75}} \exp\left[-\left(\frac{U_{w,cr}}{U_{wj}} \right)^2 \right] \tag{13b}$$

The Fig. 5 shows two examples where an decrease of the wave period (increase of the wave orbital velocity) induce a decrease of the net sediment transport. The Dibajnia & Watanabe formula is the first quasi-steady formula which is able to take into account the effects of the sediment phase lag. In Fig. 5, it appears clearly how significant the improvement of Eq. 4 is by introducing Eq. 12. The general improvement of the results is also presented in Tab. 3. Eqs. 4 and 12 presents the best overall results among the studied formulas.

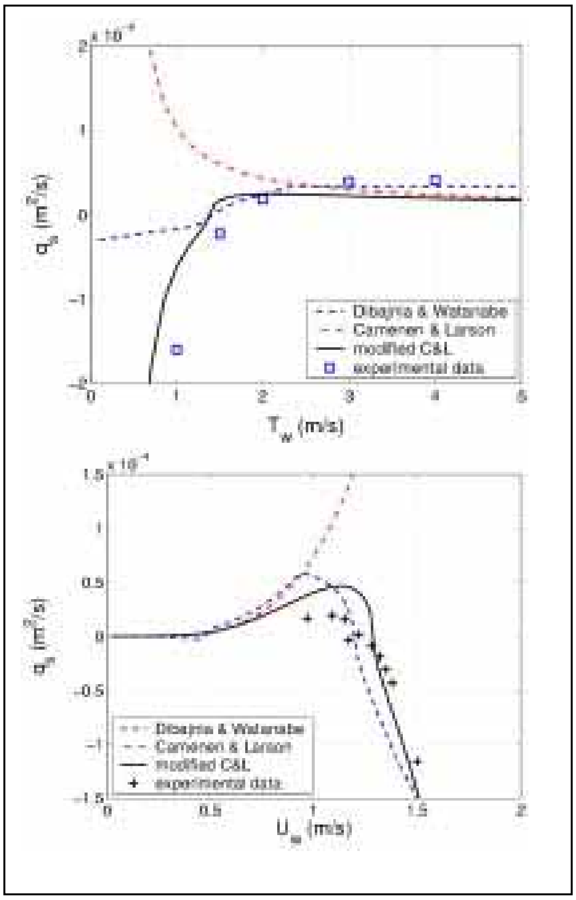

Fig. 5: Comparison between bed load transport predicted by the Dibajnia & Watanabe formula, Eq. 4, and Eq. 4 and 12 and experimental data for a varying wave period (a) and a varying wave orbital velocity (b).

3. Suspended load

The traditional approach for calculating suspended load is to determine the vertical distribution of suspended sediment concentration and velocity, after which the product between these two quantities is integrated through the vertical. Several different expressions have been derived for the concentration profile, but most of them rely on the steady state vertical diffusion equation expressed as,

$$\varepsilon \frac{\partial c}{\partial z} + W_s c = 0 \qquad (14)$$

where ε is a constant for vertical diffusion of sediment, c sediment concentration, z a vertical coordinate, and W_s the sediment fall speed. Depending on the expression selected for ε analytic solutions to Eq. 14 of different type may be found. A constant ε was assumed. It yields an exponential decay with distance from the bottom according to,

$$c(z) = c_R \exp(-\frac{W_s}{\varepsilon} z) \qquad (15)$$

where c_R is the reference concentration at the bottom located at z=0. The sediment concentration profile may thus be described by two parameters which are the bottom sediment concentration and the sediment diffusivity.

The suspended load is equal to the integration over the depth of the product between the concentration and the velocity:

$$q_{ss} = \int_0^h u(z)c(z)\,dz = U_c \int_0^h c(z)\,dz \qquad (16)$$

where h is water depth, u the horizontal velocity (varying through the vertical in the general case), z a vertical coordinate, and U_c the mean horizontal velocity. As a first approximation, when determining q_{ss} the vertical variation in u is neglected. Thus, from Eq. 15, a simple formula for the suspended sediment load is obtained:

$$q_{ss} = U_c\, c_R \frac{W_s}{\varepsilon}\left[1 - \exp\left(-\frac{W_s h}{\varepsilon}\right)\right] \qquad (17)$$

3.1 Validity of the hypothesis

To validate the two main hypothesis of this formula, *i.e.* an exponential profile of the sediment concentration and a constant velocity over the depth, a comparison is proposed between the experimental estimation of the sediment suspended load and Eq. 17 using the fitted value to the observed data for c_R and ε. A large data set on suspended sediment transport was compiled including cases with current only, and cases with a wave and current interaction. Most of these data come from the compilation provided by the SEDMOC program (2001). It appears on Tab. 4 that very accurate results are obtained for the data with a current alone. The two hypotheses, *i.e.*, an exponential concentration profile and a constant velocity over the water depth are thus verified for this case. For the cases with a wave and current interaction, the results are more scattered. This is partly due to the lack of measurement close to the bed for some of the experiments but also because of the complex velocity profiles that often occur for cross-shore measurements (undertow). The mean velocity over the depth under the through of the waves is more appropriate but still induces some uncertainties in the results. The two hypotheses could nevertheless be considered as verified for a wave and current interaction as the overall results are still quite good.

Table 4: Prediction of the suspended load transport rate within a factor 2 and 5 of the measured values as well as the root-mean-square errors using Eq. 17 and the observed data for c_R and ε.

Eq. 17	Pred x2	Pred x5	E_{rms}
Current only	99%	100%	0.09
Waves and current	44%	83%	0.43

3.2 Sediment diffusivity

Following the CHETN by Kraus & Larson (2001) on infilling of navigation channels, the sediment diffusivity may be related to the energy dissipation:

$$\varepsilon = \left[k_c \left(\frac{D_c}{\rho} \right)^{1/3} + k_w \left(\frac{D_w}{\rho} \right)^{1/3} + k_b \left(\frac{D_b}{\rho} \right)^{1/3} \right] h \tag{18}$$

where k_c, k_w, and k_b are constants related to the current, wave, and breaking wave dissipation, D_c, D_w, and D_b respectively.

(1) Current alone

The energy dissipation in the bottom boundary layer due to a current may be written:

$$D_c = \tau_c u_{*c} \tag{19}$$

where u_{*c} is the shear velocity due to the current only. Eq. 19 allows to find the same results as the classical mixing length approach, *i.e.*,

$$\varepsilon_c = k_c u_{*c} h = \frac{\sigma_c}{6} \kappa u_{*c} h \tag{20}$$

where σ_c is the Schmidt number.

Using the selected data with current only, the Schmidt number was estimated and compared with previous studies by Van Rijn (1984b) and Rose & Thorne (2001). A different expression is proposed that should be valid for larger scale of the suspension parameter W_s / u_{*c}, especially when $W_s / u_{*c} >> 1$ (u_{*c} very small) where the Schmidt number should tend toward 1.

$$\sigma_c = \begin{cases} A_{c1} + A_{c2} \sin^{2.5}\left(\dfrac{\pi}{2} \dfrac{W_s}{u_{*c}} \right) & if \dfrac{W_s}{u_{*c}} \le 1 \\ 1 + \left(A_{c1} + A_{c2} - 1 \right) \sin^{2.5}\left(\dfrac{\pi}{2} \dfrac{W_s}{u_{*c}} \right) & if \dfrac{W_s}{u_{*c}} > 1 \end{cases} \tag{21}$$

with $A_{c1} = 0.7$ and $A_{c2} = 3.6$.

In Fig. 6 is Eq. 21 plotted versus the suspension parameter for all the data. Even if some dispersion exists, the predictive results using Eq. 21 are better as those given by the existing formulas, with 90% of the data predicted within a factor 2 and a standard deviation lower than 0.2.

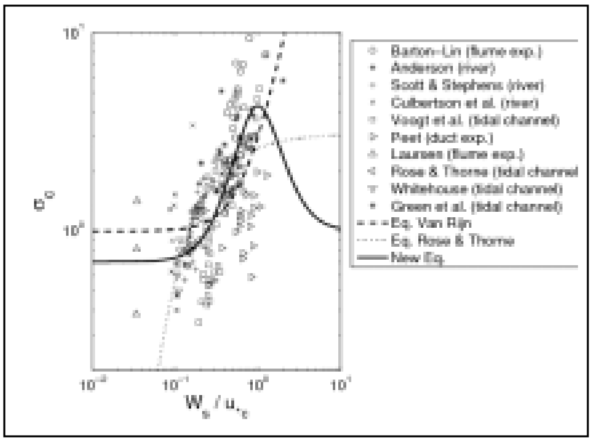

Fig. 6: Sediment diffusivity versus the suspension parameter as well as the Van Rijn (dashed line), Rose & Thorne (dotted line) relationships and Eq. 21 (full line).

(2) Waves alone

Following the study for current only, the energy dissipation in the bottom layer due to the waves, as well as the sediment diffusivity may be written as follows:

$$D_w = \tau_w u_{*w} \tag{22}$$

$$\varepsilon_w = k_w u_{*w} h = \frac{\sigma_w}{6} \kappa u_{*w} h \tag{23}$$

The Schmidt number was studied in the same manner. However, in case of the waves data, as the shear velocity cannot be obtained directly from the data, some adding dispersion is added because of the calculation of the shear velocity using predictive formulas for the bed forms and the roughness height. A similar expression to Eq. 21 was obtained with much lower coefficients: $A_{w1} = 0.09$ and $A_{w2} = 1.4$. This may be explained simply because the friction velocity due to waves is generally much larger than the one due to current and then the mixing due to the waves much larger. The results are not as good as for the current data with 65% of the data predicted within a factor 2 and a standard deviation lower than 0.4. However, it may be observed in Fig. 5 that some dispersion of the results may be due to the shear velocity estimation.

In case of a wave and current interaction, a unique Schmidt number should be used for both the current

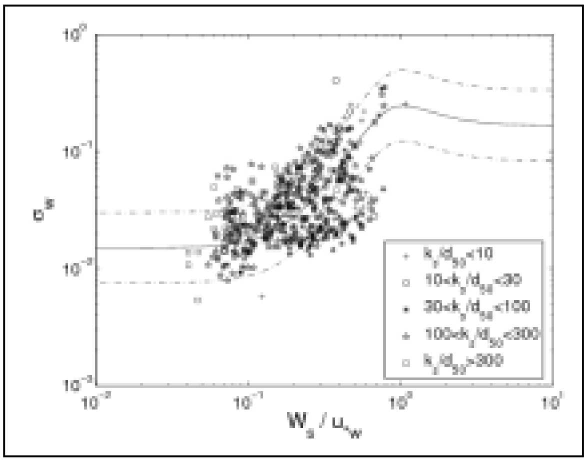

Fig. 7: Sediment diffusivity versus the suspension parameter with the roughness height emphasised as well as Eq. 21 (full line) using the coefficients

related and wave related sediment diffusivity. A relationship is proposed as a function of the ratio X (see Eq. 9):

$$\sigma_{cw} = X^5 \sigma_c + (1 - X^5)\sigma_w \tag{24}$$

(3) Breaking waves

Using a wave model, the estimation of the wave energy dissipation is found from the onshore decrease of the wave energy flux:

$$D_b = \frac{1}{h}\frac{dF_w}{dx} \tag{25}$$

where $F_w = E_w C_g$ with E_w the wave energy and C_g the group celerity. Using the collected data, it appears that Eq. 18 with a constant value for $k_b = 0.015$ yields correct results: more than 70% of the data predicted within a factor 2 and a standard deviation close to 0.3.

3.3 Reference concentration

The reference concentration strongly depends on the hypothesis on the concentration profile and is subject to large uncertainties. Following Madsen (1993) method, the reference volumic bed concentration may be estimated from the volumic bed load, assuming $q_{sb} = c_R U_s$ where U_s is the speed of the bed load layer. The bed load may be written following the results by Camenen & Larson (2005). As Madsen (1993) proposed, as a first approximation, the speed of the bed load layer may be proportional to the shear velocity, namely $U_s \propto \theta^{1/2}$. The bed reference concentration may thus be written as follows,

$$c_R = A_{cR}\theta_t \exp\left(-4.5\frac{\theta_m}{\theta_{cr}}\right) \tag{26}$$

where θ_t is the transport-dependence Shields parameter and θ_m is the maximum Shields. In case of the current alone, $\theta_m = \theta_t = \theta_c$.

(1) Current alone

Using the data with a steady current, an improvement of the results has been obtained by calibrating A_{cR} as a function of the dimensionless grain size:

$$A_{cR} = 3.5\ 10^{-3}\exp(-0.3d_*) \tag{27}$$

It appears clearly in Tab. 5 and Fig. 6 that Eqs. 26 and 27 improve the predictive results compared to the existing formulas. A non negligible dispersion however still remains.

Table 5: Prediction of the bottom sediment concentration within a factor 2 and 5 of the measured values as well as the root-mean-square errors in case of a current alone.

Formula	Pred x2	Pred x5	E_{rms}
Madsen	27%	50%	0.83
Nielsen	13%	50%	0.43
Eqs. 26 and 27	49%	84%	0.51

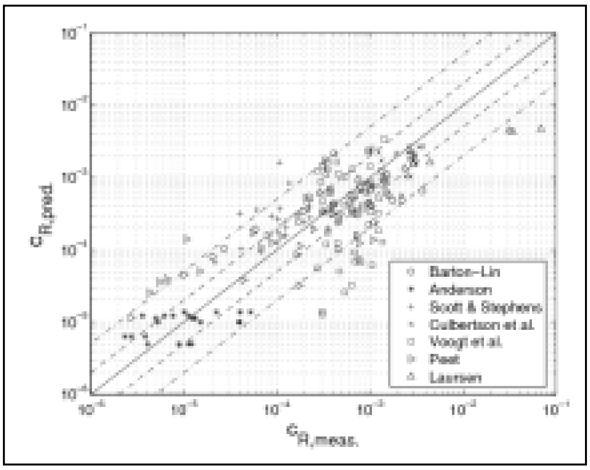

Fig. 8: Comparison between bottom reference concentrations predicted by the Eqs. 26 and 27 and experimental data using data with current.

(2) Waves alone

In case of a waves only, following the results by Camenen & Larson (2004) on bed load transport, the mean shear stress $\theta_{cw,m}$ is used for the transport-dependence term. Eq. 26 with Eq. 27 found for the current alone still presents the best results compared to the Madsen (1993) and Nielsen (1986, 1992) formulas (see Tab. 6), although the effect of the grain size seems not to be as significant as for the results with a current alone. However, compared to the data set for current, the range of value of d* for the is not as wide, and the grain size distribution of the data quite different.

Table 6: Prediction of the bottom sediment concentration within a factor 2 and 5 of the measured values as well as the root-mean-square errors in case of non-breaking waves.

Formula	Pred x2	Pred x5	E_{rms}
Madsen	31%	62%	0.58
Nielsen	24%	48%	1.26
Eqs. 26 and 27	47%	81%	0.58

As shown in Fig. 7, some of the uncertainties come from the errors in the prediction of the shear velocity or roughness height in case of waves.

(2) Waves and current interaction

In case of a wave and current interaction, Eq. 26 and 27 can still be used. However, a large dispersion of the results is observed. For this data set, it appears that using a constant value for A_{cR} induces better results (*cf.* Tab. 7). But as it was observed for the waves alone, the effect on the quality of the results of the calculation of the shear velocity is significant.

Table 7: Prediction of the bottom sediment concentration within a factor 2 and 5 of the measured values as well as the root-mean-square errors in case of non-breaking waves.

Formula	Pred x2	Pred x5	E_{rms}
Madsen	03%	23%	0.47
Nielsen	28%	47%	1.08
Eqs. 26 and 27	37%	74%	0.50
Eq. 26 with A_{cR}=5 10^{-4}	50%	87%	0.46

3.4 Suspended load rate

The classical formulas for suspended load in case of wave-current interaction are often either integrating the suspended load over the depth (Einstein, 1950; Van Rijn, 1993) or estimating the total load sediment transport using empirical formula. (Bailard, 1981; Watanabe 1982; Van Rijn, 1984a and b; Dibajnia & Watanabe, 1992). The first method allows a better inclusion of the physical processes but is generally time consuming and too sensitive to some parameters. The second one yields more robust predictions but not

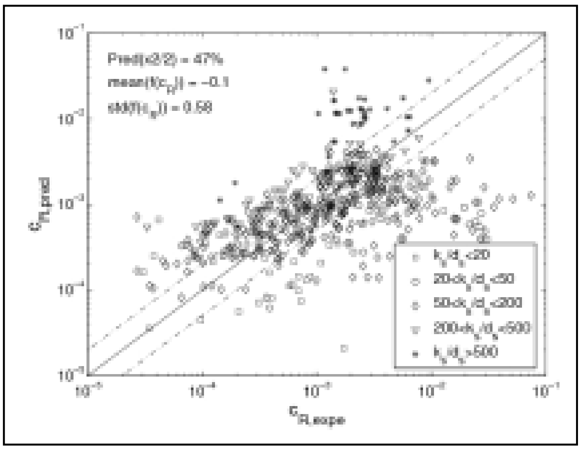

Fig. 9: Comparison between bottom reference concentrations predicted by the Eqs. 26 and 27 and experimental data using data with waves.

always accurate. The aim of this study is to propose a robust and as accurate as possible relationship (Eq. 17) whatever the hydrodynamic conditions are, which does not need an integration over the depth, but yet takes into account physical parameters like c_R or ε .

(1) Steady current

In case of a steady current, Eq. 17 with the sediment diffusivity from Eqs. 20 and 21, and the bottom reference concentration from Eqs. 26 and 27, yields the best results among the studied formula (*cf.* Tab. 8; A comparison with the total load formula for current only by Engelund & Hansen, 1972, is added as a comparison). It also appears that most of the dispersion comes from the dispersion in the prediction of the bottom reference concentration. Around more than 40% (80%) of the data are predicted within a factor 2 (5) and a standard deviation lower than 0.6.

Table 8: Prediction of the suspended load within a factor 2 and 5 of the measured values as well as the rms errors in case of data with steady current.

Formula	Pred x2	Pred x5	E_{rms}
Bijker	24%	45%	1.04
Engelund-Hansen	31%	55%	0.85
Bailard	33%	72%	0.69
Van Rijn	30%	69%	0.98
Present work	41%	79%	0.58

(2) Wave and current interaction

A comparison between the predicted and observed suspended sediment load in case of wave and current interaction is presented in Tab. 8 and Fig. 8. It appears that the proposed formula (Eq. 17) presents reasonably good results but much more dispersed compared to the current data. The obtained results are again highly dependent on the estimation of the reference concentration, and thus as shown in Sec. 3.3, on the estimation of the roughness height and total shear stress.

Table 9: Prediction of the suspended load within a factor 2 and 5 of the measured values as well as the rms errors in case of data with a wave and current interaction (in brackets are the results for breaking waves only).

Formula	Pred x2	Pred x5	E_{rms}
Bijker	21(40)%	47(80)%	0.64(0.67)
Bailard	27(58)%	66(86)%	0.55(0.50)
Van Rijn	36(14)%	65(38)%	0.86(1.18)
Present work	38(31)%	69(74)%	0.74(0.63)

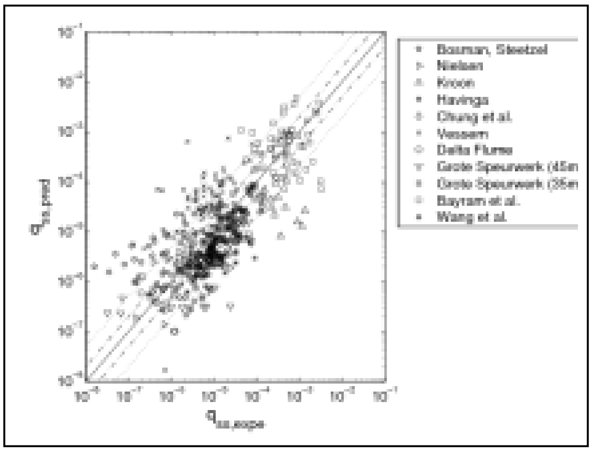

Fig. 10: Comparison between suspended sediment load predicted by the Eqs. 17 and experimental data using data with a wave and current interaction.

In case of a wave and current interaction without breaking waves, the present work as well as the Van Rijn formula yields the best results even if some scattering still exists especially for the Van Rijn formula. For this latest formula, the large number of parameters and its relative complexity may explain the observed scattering as it is much more sensitive to any parameters. The results obtained using the Van Rijn formulas are much poorer when waves are breaking. On the other hand, it appears that the Bijker and especially Bailard formula present the best results among the studied formulas. This may be explained as these formulas were calibrated for the estimation of the suspended load in the surf zone. The Bailard formula also yields the less dispersed results. Indeed, this formula is not as sensitive to the shear stress (only an average friction coefficient is introduced) and simple enough to avoid a dispersion of the results. In case of
Eq. 17, a large improvement of the results may be obtained having a better prediction of the total shear stress and including the effect of breaking waves on the bottom reference concentration.

4. Conclusions

A bed load formula was presented in this paper including wave and current interaction, as well as possible phase-lag effects. The best overall results were obtained compared to the studied formulas.

To complete the proposed total load formula, a suspended load formula was also presented assuming an exponential concentration profile and a constant velocity over the depth. The diffusion parameter was calculated as a function of the total energy dissipation (current, waves, and breaking waves). And the reference concentration was estimated as a function of the Shields parameter (waves + current) and the grain size. The best overall results were obtained compared to the studied formulas; but some improvement in case of wave and current interaction is needed especially for the breaking wave effects.

Acknowledgements

This work was conducted under the Inlet Modeling System Work Unit of the Coastal Inlets Research Program, U.S. Army Corps of Engineers, and the Japanese Society for the Promotion of Science.

References

Abou-Seida, M. (1965), Bed load function due to wave action, Technical Report HEL-2-11, Hydraulic Engineering Laboratory, University of California, Berkeley, California.

Ahilan, R. & Sleath, J. (1987), `Sediment transport in oscillatory flow over flat beds', J. Hydraulic Eng. 113(3), 308-322.

Ahmed, A. & Sato, S. (2003), `A sheet-flow transport model for asymmetric oscillatory flow, part I: uniform grain size sediments', Coastal Eng. J. (in Japan) 45(3), 321-337.

Bailard, J. (1981), 'An energetics total load sediment transport model for a plane sloping beach', J. Geophysical Res. 86(C11), 10938–10954.

Brownlie, W. (1981), Compilation of alluvial channel data: laboratory and _eld, Technical Report KH-R-43B, California Institute of Technology, Passadena, California, U.S.A.

Camenen, B. & Larson, M. (2005), 'A bedload sediment transport formula for the nearshore', Estuarine, Coastal and Shelf Science 63, 249–260.

Dibajnia, M. & Watanabe, A. (1992), Sheet flow under nonlinear waves and currents, in `Proc. 23rd Int. Conf. Coastal Eng.', ASCE, Venice, Italy, pp. 2015-2029.

Dohmen-Janssen, M. (1999), Grain size influence on sediment transport in oscillatory sheet-flow, phase-lags and mobile-bed effects, PhD thesis, Delft University of Technology, ISBN 90-9012929-4, The Netherlands.

Dohmen-Janssen, C. & Hanes, D. (2002), `Sheet flow dynamics under monochromatic nonbreaking waves', J. Geophysical Res. 107(C10), 13:1-13:21.

Einstein, H. (1950), The bed-load function for sediment transportation in open channel flows, Technical Report 1026, U.S. Dept. of Agriculture, Techn. Bulletin, Washington D.C., USA.

Engelund, F. & Hansen, E. (1972), A Monograph on Sediment Transport in Alluvial Streams, technical press edn, Copenhagen.

Horikawa, K., Watanabe, A. & Katori, S. (1982), Sediment transport under sheet flow conditions, in `Proc. 18th Int. Conf. Coastal Eng.', ASCE, Cape Town, South Africa, pp. 1335-1352.

Kalkanis, G. (1964), Transportation of bed material due to wave action, Technical Memo 2, Coastal Engineering Research Center, U.S. Army Corps of Engineers, Washington, D.C.

King, D. (1991), Studies in oscillatory flow bed load sediment transport, PhD thesis, University of California, San Diego.

Kraus, N. & Larson, M. (2001), Mathematical model for rapid estimation of infilling and sand bypassing at inlet entrance channels, Technical Note CHETN-IV-35, Coastal and Hydraulics Laboratory, U.S. Army Corps of Engineers, Vicksburg, Mississippi, USA.

Madsen, O. (1993), Sediment transport on the shelf, in ‘Sediment Transport Workshop Proc. (DRP TA1)’, CERC, Vicksburg, Mississippi, USA.

Meyer-Peter, E. & Müller, R. (1948), Formulas for bed-load transport, in `Rep. 2nd Meet. Int. Assoc. Hydraul. Struc. Res.', Stockholm, Sweden, pp. 39-64

Nielsen, P. (1986), ‘Suspended sediment concentrations under waves’, Coastal Eng. 10, 23–31.

Nielsen, P. (1992), Coastal bottom boundary layers and sediment transport, Vol. 4 of Advanced Series on Ocean Engineering, World Scientific Publication.

Nikora, V. & Smart, G. (1997), `Turbulence characteristics of New Zealand gravel-bed rivers', J. Hydraulic Eng. 123, 764-773.

Nnadi, F. & Wilson, K. (1992), `Motion of contact-load particules at high shear stress', J. Hydraulic Eng. 118(12), 1670-1684.

Ribberink, J. (1998), `Bed-load transport for steady ows and unsteady oscillatory flows', Coastal Eng. 34, 52-82.

Ribberink, J. & Al Salem, A. (1994), `Sediment transport in oscillatory boundary layers in cases of rippled beds and sheet flow', J. Geophysical Res. 99(C6), 707-727.

Ribberink, J. & Chen, Z. (1993), Sediment transport of fine sand under asymmetric oscillatory flow, Delft Hydraulics, Report H840, part VII, The Netherlands.

Rose, C. & Thorne, P. (2001), ‘Measurements of suspended sediment transport parameters in a tidal estuary’, Continental Shelf Res. 21, 1551–1575.

Sawamoto, M. & Yamashita, T. (1986), `Sediment transport rate due to wave action', J. of Hydroscience and Hydraulic Eng. 4(1), 1-15.

SEDMOC : Sediment Transport Modelling in Marine Coastal Environments (2001) Edts: L.C. Van Rijn, A.G. Davies, J. Van der Graff, and J.S. Ribberink, Aqua Publications, ISBN 90-800346-4-5", Amsterdam, The Netherlands

Sleath, J. (1978), `Measurements of bed load in oscillatory flow', J. Waterways, Port, Coastal and Ocean Eng. (- Division) 10(4), 291-307.

Smart, G. (1984), `Sediment transport formula for steep channels', J. Hydraulic Eng. 111(3), 267-276.

Smart, G. (1999), `Turbulent velocity pro_les and boundary shear in gravel bed rivers', J. Hydraulic Eng. 125(2), 106-116.

Soulsby, R. (1997), Dynamics of marine sands, a anual for practical applications, Thomas Telford, ISBN 0-7277-2584X, H.R. Wallingford, England.

Van Rijn, L. (1984a), ‘Sediment transport, part I : bed load transport’, J. Hydraulic Eng. (Hydr. Division) 110(10), 1431–1456.

Van Rijn, L. (1984b), ‘Sediment transport, part II : suspended load transport’, J. Hydraulic Eng. (Hydr. Division) 110(11), 1613–1641.

Van Rijn, L. (1993), Principles of sediment transport in rivers, estuaries and coastal seas, Aqua Publications, The Netherlands.

Watanabe, A. (1982), ‘Numerical models of nearshore currents and beach deformation’, Coastal Eng. J. (in Japan) 25, 147–161.

Watanabe, A. & Isobe, M. (1990), Sand transport rate under wave-current action, in `Proc. 22nd Int. Conf. Coastal Eng.', ASCE, Delft, the Netherlands, pp. 2495-2506.

Wilson, K. (1966), `Bed-load transport at high shear stress', J. Hydraulic Eng. (Hydr. Division) 92(11), 49-59.

沿岸域における全漂砂量の簡便式の提案

Benoit Camenen*・Magnus Larson**・山下隆男

*日本学術振興会外国人特別研究員
**ルンド大学，スウェーデン

要旨

本研究は，沿岸域における簡易で広範囲の条件下で信頼できる全漂砂量（掃流，浮遊漂砂）の算定式の提案とその適用性を示したものである。算定式は，砕波乱流やシートフローでの流れと底質移動の位相遅れの効果に加えて波と流れの相互作用効果も考慮している。

掃流漂砂量の定式化は波・流れ共存場での海底せん断応力の表示に依存している。一方，浮遊漂砂量には，浮遊砂濃度の鉛直分布が指数関数で表示できると仮定して，平均漂砂拡散係数と鉛直方向に積分平均化した流速を用いた簡便な定式化を提案した。多くの漂砂量に関する実験，現地観測データを用いて，提案した漂砂量則の適用性を検証した。

キーワード: 漂砂量則，浮遊漂砂，掃流漂砂，波浪，海浜流，沿岸域