Review article!

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COASTAL SEDIMENT PROCESSES

Meer nearshore *8167

macro

meso scale

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1 INTRODUCTION

The study of coastal sediment processes has been systematized over the past half century by coastal geomorphologists, while the dynamics of nearshore sediment movement has been treated only during the past twenty years by workers in the fields of coastal engineering and nearshore oceanography. For our purposes here, the term "coastal sediment processes" will cover the time history of the numerous phenomena related to sediment movement in the coastal area. From the perspective of engineering science, coastal sediment phenomena are closely related to various important practical problems such as siltation of harbor basins and beach erosion.

It was not so long ago that most coastal projects were carried out by trial and error because of a lack of knowledge of the underlying mechanisms of coastal sediment processes. During the last few decades, large quantities of data have been taken in studies of the transport of coastal sediment through field and laboratory investigations. Although these data are helpful, the phenomena are very complex and as yet most are understood only in a qualitative sense. Therefore, more basic research effort is required before we can confidently deal with practical problems. The extensive research activities carried out in recent years by groups all over the world hold promise of a better understanding of coastal sediment processes. This article gives a review of selected past studies and describes the present state of knowledge in this field.

2 VARIOUS APPROACHES AND RELATED ELEMENTS

Coastal sediment processes are extremely complex and include phenomena having quite different scales in space and in time. For purposes of

Table 1 Classification of coastal process scales

	Macroscale	Mesoscale	Microscale
Time scale	year	day/hour	second
Space scale	kilometer	meter	millimeter

clarification, Horikawa (1970) proposed classifying coastal phenomena into three categories as macroscale, mesoscale, and microscale. These scales can be roughly described as shown in Table 1. Specific examples for each scale will be given in the following sections.

Theoretically speaking, the complete superposition of microscale phenomena should compose the mesoscale phenomena and that of the mesoscale phenomena, the macroscale phenomena. At present, the above connections cannot be made. It is therefore suggested that coastal engineers devote their efforts to clarifying the mechanisms of these different scale phenomena in parallel, and to trying to fill in the great gaps still existing among them.

2.1 *Macroscale Approach*

To treat macroscale phenomena, the approach of the geologist and geomorphologist is quite helpful for understanding the general tendencies of coastal processes. But the time scale of their interest is usually too long for engineering purposes; hence coastal engineers have developed their own measures and devices for obtaining data on the relatively short-term variation of coastal processes. This "short" term, roughly the span of human life, is still quite long when considering normal engineering practice. Nevertheless, the historical sea-level change, ground-level variation due to crustal movement, as well as changes due to artificial interference, must all be taken into consideration at the locations where they occur.

In addition to natural processes, coastal engineers should be concerned with effects produced mainly by the construction of coastal structures as well as by various other similar sources. These man-made influences are occasionally the main causes of the beach erosion occurring on coasts everywhere.

Based on the above considerations, it is frequently realized that various natural and artificial causes combine at a given location. But in the future, man-made effects may steadily increase their contribution to the beach-erosion problem. Therefore, long-term coastal changes should be predictable with sufficient accuracy before starting any coastal construction project.

WAVE REFRACTION
DIFFRACTION
REFLECTION
BREAKING

NEARSHORE CURRENT

2.2 *Mesoscale Approach*

Changes in shoreline and sea-bottom topography, bar and cusp formation, and nearshore currents all fall in the category of mesoscale phenomena. Numerous investigations have been carried out on these subjects, and certain details are understood at least qualitatively. Because our understanding of nearshore dynamics has tremendously improved over the last decade, fluid motion may be predicted fairly precisely under appropriate assumptions. But actual coastal sediment processes have not been sufficiently investigated due to the complexity of coastal sediment movement and the formidable nature of the coastal environment. For example, no precise expression for the coastal-sediment transport rate is yet available. For this reason we cannot at present combine the fundamental equations of fluid motion and the conservation of sediment material to calculate the bottom change in the nearshore area.

In order to surmount this problem for practical applications, numerous longshore-transport formulas have been proposed based in large part on empirical results. Combining one of these formulas and the continuity equation derived under simplified conditions yields a prediction of shoreline change, some details of which will be given later. This kind of approach is quite useful for practical purposes, but the assumptions made in the above treatments are not always correct.

In order to simulate changes in the sea-bottom configuration, we start with the following equation:

$$(1-\lambda)(\partial h/\partial t)-\partial q_x/\partial x-\partial q_y/\partial y=0, \quad (2.1)$$

where x and y denote the horizontal axes, λ is the porosity of the sediment, h the water depth at a particular point (x, y) at a particular time t, and q_x, q_y the sediment transport rates through a unit length in either the y or x direction, respectively, per unit time. To close the above equation, q_x and q_y should be evaluated using the wave characteristics (wave height, wave period, and wave angle), water depth h, and grain characteristics (grain size d, density ρ_s, and porosity λ). As stated above, at present no reliable formulas for q_x and q_y are available.

Sediment particles will be transported by fluid motion. Fluid motion in the nearshore area is mainly caused by wave action. Therefore wave transformation in shallow water, that is, wave refraction, wave diffraction, wave reflection, and wave breaking should be determined beforehand. The computation of wave transformation can be successfully done, at least in a first approximation. The next step is the prediction of the nearshore current, which is presently possible with adequate accuracy.

2.3 *Microscale Approach*

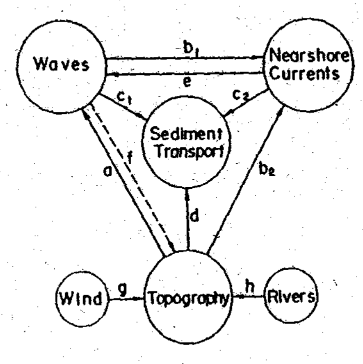
In order to correlate the sediment-transport rate with wave action, the detailed mechanisms of fluid and sediment movement must be known. However, our knowledge on these subjects is still inadequate, and further work is needed.

The important problems to be solved are listed below, and most are classified in the microscale regime. These problems are

1. Characteristics of oscillatory boundary-layer flow due to the coexistence of waves and currents, such as the velocity distribution and bottom shear stress, under hydrodynamically smooth or rough conditions,
2. Vertical distribution of the momentum-exchange coefficient (or the eddy viscosity) and of the diffusion coefficient in the wave field,
3. Vertical distribution of the mass-transport velocity,
4. Wave deformation and velocity field on a sloping bottom,
5. Interaction between waves and wave-induced currents,
6. Ripple formation and ripple size, disappearance of ripples and formation of sheet flow,
7. Vertical distribution of the suspended-sediment concentration inside and outside the breaker zone,
8. Detailed mechanisms of sediment transport due to waves and currents.

2.4 *Interactions among Waves, Currents, Topography, and Sediment Transport*

Figure 1 schematizes the relations among the various elements such as waves, nearshore currents, sediment transport, and topography, etc. The



a. Wave Deformation (Refraction, Diffraction, Reflection, etc.)
b. Nearshore Current
c. Sediment Transport Rate (Suspended and Bedload Transport)
d. Topographic Change due to Sediment Transport
e. Wave-Current Interaction
f. Mechanical Scouring (Breaking Point, Swash Zone)
g. Wind-blown Sand
h. Sediment Supplied from Rivers and Streams

Figure 1 Relationship among waves, nearshore currents, sediment transport, and topography (adapted from Horikawa, Harikai & Kraus 1979).

aim of our investigations is to clarify these complex relationships. In order to treat them, a suitable approach must be selected from among the three discussed above.

3 NEARSHORE DYNAMICS

Phenomena observed in the nearshore area are extremely complex, and seem to elude any kind of analytical treatment. Since the end of the 1940s, a number of workers have treated these problems through laboratory and field investigations, and numerous semi-empirical relations or correlations have been found involving nearshore phenomena. Our next interest is to understand their mechanisms, if only partially.

The radiation-stress concept proposed by Longuet-Higgins & Stewart (1960, 1962) opened a new era in the analysis of nearshore dynamics. Lundgren (1963) independently derived and named the same term as the wave thrust. In the present article, treatments relating directly to coastal sediment processes will be discussed briefly.

3.1 *Dynamical Equations and Definition of Radiation Stresses*

Following Phillips (1977), the dynamical equations for the coexistent field of waves and a current with vertically uniform velocity will be given. These are the conservation equations of mass, momentum, and energy derived by integrating the corresponding basic equations with respect to the vertical axis from the bottom ($z=-h$) to the water surface ($z=\zeta$) and taking a time average over many wave periods,

$$\frac{\partial}{\partial t}\left[\rho\left(h+\bar{\zeta}\right)\right]+\frac{\partial \tilde{M}_\alpha}{\partial x_\alpha}=0, \tag{3.1}$$

$$\frac{\partial \tilde{M}_\alpha}{\partial t}+\frac{\partial}{\partial x_\beta}\left(\tilde{U}_\alpha \tilde{M}_\beta+S_{\alpha\beta}\right)=T_\alpha+R_\alpha, \tag{3.2}$$

$$\begin{aligned}&\frac{\partial}{\partial t}\left[\frac{1}{2}\tilde{U}_\alpha \tilde{M}_\beta+\frac{1}{2}\rho g\left(\bar{\zeta}^2-h^2\right)-\frac{1}{2}M_\alpha^2/\rho\left(h+\bar{\zeta}\right)+\bar{E}\right]\\&\quad+\frac{\partial}{\partial x_\alpha}\left[\tilde{M}_\alpha\left(\frac{1}{2}\tilde{U}_\beta^2+g\bar{\zeta}\right)-\frac{1}{2}\tilde{U}_\alpha M_\beta^2/\rho\left(h+\bar{\zeta}\right)+U_\alpha\bar{E}\right.\\&\qquad\left.+F_\alpha+U_\beta S_{\alpha\beta}\right]+(\text{rate of energy loss})=0,\end{aligned} \tag{3.3}$$

where the horizontal axes x_1 and x_2 are taken on the still-water level and the vertical axis z in the upward direction, hence $\alpha, \beta=1$ and 2. The notations ρ, $\bar{\zeta}$, and $\bar{E}$ represent the fluid density, mean sea-level rise due to the interference between waves and current, and wave energy per unit surface area. In the present treatment, the horizontal velocity component u_α was split into the uniform velocity component U_α and the horizontal component u'_α of the wave-particle velocity. Also, the definitions

$$\tilde{M}_\alpha = \overline{\int_{-h}^{\zeta} \rho u_\alpha dz}, \quad \hat{M}_\alpha = \overline{\int_{-h}^{\zeta} \rho U_\alpha dz} = \rho(h+\bar{\zeta})U_\alpha,$$

and $M_\alpha = \overline{\int_{-h}^{\zeta} \rho u'_\alpha dz}$

were introduced. Therefore $u_\alpha = U_\alpha + u'_\alpha$, and $\tilde{M}_\alpha = \hat{M}_\alpha + M_\alpha$. The other terms included in the above equations are

$$S_{\alpha\beta} = \overline{\int_{-h}^{\zeta} (\rho u'_\alpha u'_\beta + p\delta_{\alpha\beta})\, dz} - \tfrac{1}{2}\rho g(h+\bar{\zeta})^2 \delta_{\alpha\beta} - M_\alpha M_\beta/\rho(h+\bar{\zeta}), \tag{3.4}$$

$$T_\alpha = -\rho g(h+\bar{\zeta})\partial\bar{\zeta}/\partial x_\alpha, \tag{3.5}$$

$$R_\alpha = \overline{\int_{-h}^{\zeta} \partial\tau_{\beta\alpha}/\partial x_\beta\, dz} + \bar{\tau}_{\zeta\alpha} - \bar{\tau}_{h\alpha}, \tag{3.6}$$

$$F_\alpha = \rho \overline{\int_{-h}^{\zeta} u'^2_\alpha \left[\tfrac{1}{2}u'^2_i + g(z-\bar{\zeta}) + p/\rho\right] dz}, \tag{3.7}$$

where p is the pressure, $\delta_{\alpha\beta}$ the Kronecker delta (i.e., $\delta_{11} = \delta_{22} = 1$, and $\delta_{12} = \delta_{21} = 0$), $\tau_{\beta\alpha}$ the stress component inducing the interference between waves and current, and $\bar{\tau}_{\zeta\alpha}, \bar{\tau}_{h\alpha}$ the mean shear stresses at the free surface and at the bottom respectively.

The term $S_{\alpha\beta}$ is the radiation-stress tensor, which corresponds to the excess momentum flux tensor, T_α the horizontal force per unit surface area induced by the free-surface gradient, R_α the frictional term consisting of the lateral and boundary frictional terms, and F_α the mean energy flux by the fluctuating motion alone.

The radiation-stress tensor S for the case of a train of waves $\zeta = \frac{1}{2} H \cos(x_1 k \cos\theta + x_2 k \sin\theta - \sigma t)$ is expressed by

$$S = \bar{E}\begin{bmatrix} (c_g/c)\cos^2\theta + \frac{1}{2}(2c_g/c-1) & \frac{1}{2}(c_g/c)\sin 2\theta \\ \frac{1}{2}(c_g/c)\sin 2\theta & (c_g/c)\sin^2\theta + \frac{1}{2}(2c_g/c-1) \end{bmatrix} \tag{3.8}$$

where H is the wave height, k the wave number, σ the angular frequency,

$\bar{E} = \rho g H^2/8$, ρ the fluid density, g the gravitational acceleration, θ the wave direction angle, c the wave celerity, and c_g the group velocity.

3.2 *Wave Set-Down and Wave Set-Up*

As a typical application of the radiation-stress concept to nearshore phenomena, we will consider the simple case where waves arrive normal to the shoreline (y axis) from offshore to a beach with a uniformly gentle slope. Taking the x axis from offshore toward shore, and considering the steady state, Equation (3.1) becomes $d\tilde{M}_x/dx = 0$, that is $\tilde{M}_x = \hat{M}_x + M_x = \rho(h+\bar{\zeta})U_x + M_x =$ constant. Due to the existence of the beach, the constant should be zero. Therefore Equation (3.2) can be written in the following form under the assumption that the frictional term R_x is negligible:

$$dS_{xx}/dx = -\rho g(h+\bar{\zeta})\, d\bar{\zeta}/dx. \tag{3.9}$$

Integration of the above equation outside and inside the surf zone separately under the appropriate assumptions yields the result that the mean sea level is reduced from the deep-water level to a certain value at the breaking point, then rises up toward the shore. The former and latter phenomena are called "wave set-down" and "wave set-up" respectively.

Laboratory investigations (Bowen, Inman & Simmons 1968) have confirmed that the above theoretical treatment predicts well the observed wave set-down and wave set-up, but that there are still some discrepancies remaining, especially in the vicinity of the breaking point.

3.3 *Longshore-Current Velocity Distribution*

It has been realized for many years that the longshore current is the most important agent for the longshore sediment transport. In order to establish a relationship between the longshore current and the longshore-sediment-transport rate, it is necessary to know the horizontal and vertical longshore-current velocity distributions in the nearshore area. Here we will take the simplest case, where the bottom slope is uniform, the depth contour is parallel to a straight shoreline, and the wave-induced current motion is steady. The x and y axes are taken perpendicular (towards offshore) and parallel to the shoreline, respectively. The basic equation in this case is simply written as follows:

$$dS_{xy}/dx = R_y. \tag{3.10}$$

We will consider the nearshore zone in separate regions, outside and inside the surf zone. Outside the surf zone, we can assume that the energy flux is conserved and that Snell's law is applicable. Applying this law, we can derive S_{xy} = constant. Therefore Equation (3.10) becomes

$$R_y = dS_{xy}/dx = 0. \tag{3.11}$$

Inside the surf zone, the wave height H is well expressed by $H=\gamma(h+\bar{\zeta})$, where γ is a constant. Bowen (1969a) and Longuet-Higgins (1970) evaluated dS_{xy}/dx under approximations leading to the equations,

$$R_y = \begin{cases} (1/4)\rho g\gamma^2(h+\bar{\zeta})\sin\alpha_b\cos\alpha_b d(h+\bar{\zeta})/dx \\ \quad \text{(Bowen)} & (3.12a) \\ (5/16)\rho g\gamma^2(h+\bar{\zeta})\sin\alpha d(h+\bar{\zeta})/dx \\ \quad \text{(Longuet-Higgins)} & (3.12b) \end{cases}$$

where α is the angle between the wave crest and bottom contour, and the subscript b denotes the value at the breaking point.

The next task is to determine the expression for the frictional term R_y. Considering Equation (3.6), and neglecting the frictional stress at the surface, $\bar{\tau}_{\zeta\alpha}$, we can construct the next equation as a general one,

$$R_y = \partial(\epsilon_\mu \partial v/\partial x)/\partial x - \bar{\tau}_{hy}, \tag{3.13}$$

where v is the longshore current velocity and ϵ_μ is the momentum-exchange coefficient. The first and second terms on the right-hand side are the lateral and bottom friction terms, respectively.

For the lateral friction term, Bowen and Longuet-Higgins used the expressions,

$$\partial(\epsilon_\mu \partial v/\partial x)/\partial x = \begin{cases} \rho(h+\bar{\zeta})\Lambda_h d^2v/dx^2 \\ \quad \text{(Bowen)} & (3.14a) \\ d\left[\mu_e(h+\bar{\zeta})\,dv/dx\right]\Big/dx \\ \quad \text{(Longuet-Higgins)} & (3.14b) \end{cases}$$

$$\mu_e = N\rho x\sqrt{g(h+\bar{\zeta})}, \qquad 0<N<0.016,$$

where Λ_h and N are constants, and x is measured from shoreline. According to these expressions, the momentum-exchange coefficient is assumed to increase monotonically from the shoreline to offshore beyond the breaker line. This behavior is not physically realistic. That is to say, the momentum-exchange coefficient should increase from the shoreline to the breaker line, but decrease in a certain manner beyond the breaker line.

For the bottom frictional term, Bowen and Longuet-Higgins took the expressions,

$$\bar{\tau}_{hy} = \begin{cases} \rho C v & \text{(Bowen)} & (3.15a) \\ \frac{1}{2}\rho f_w |\bar{u}_{wh}| v = \rho f_w u_{bm} v/\pi & \text{(Longuet-Higgins)} & (3.15b) \end{cases}$$

where C and f_w are both friction coefficients, $|\bar{u}_{wh}|$ the absolute-time average, and u_{bm} the amplitude of the orbital velocity at the bottom induced by wave action. The former expression is based on a linear friction law, while the latter is based on a quadratic law with the assumption that the longshore-current velocity v is small compared with the wave orbital velocity.

By using the above expressions for the lateral and bottom friction terms, Bowen and Longuet-Higgins obtained realistic velocity-distribution curves of the longshore current. Their contribution to the coastal engineering field was extremely important because they demonstrated that the wave-induced current can be calculated analytically. However, the assumptions listed below were made and, as a result, the applicability of the solutions is restricted to a certain range. These assumptions are: 1. The bottom beach slope is uniform; 2. The incident wave angle is small; 3. The lateral and bottom frictional terms are valid under certain limited conditions; and 4. The wave height inside the surf zone is simply expressed as being proportional to the water depth.

The last assumption seems to be applicable to only a uniformly sloping bottom. According to recent investigations (Mizuguchi, Tsujioka & Horikawa 1978, Hotta & Mizuguchi 1978) the proportionality constant γ is not constant in a real surf zone; it depends on the bottom configuration and also the breaker type.

Jonsson (1966, 1976), Kajiura (1964, 1968), and Kamphuis (1975) treated the frictional stress along a horizontal bottom due to waves. The diagram prepared by Jonsson (1976) is commonly used in longshore-current computations. Inside the surf zone, waves are superimposed obliquely on the longshore current. For a case where the wave orbital velocity is fairly large compared to the longshore-current velocity, the present treatment would be acceptable. However, in a case where the wave orbital velocity is comparable with the wave orbital velocity, it is expected that the bottom frictional term would be of a different form and that a new frictional coefficient would be required. Neither of these quantities has been given satisfactorily.

Concerning the effect of incident wave angle on the longshore-current velocity distribution, Liu & Dalrymple (1978), and Kraus & Sasaki (1979) independently treated the phenomenon and found a systematic influence of the angle of wave approach on the longshore-current velocity distribution.

3.4 *Nearshore Currents*

There are complex currents induced by numerous elements in the nearshore area. Among these currents, the wave-induced currents are most closely related to coastal sediment processes. Mass transport associated with waves, longshore currents, and rip currents are all wave-induced currents that form the nearshore current. When waves arrive almost perpendicular to the shoreline, the region bounded by two adjacent rip currents forms a unit cell. When the incident wave angle increases, the closed cell disappears and forms a meandering current or a longshore current.

If we use the horizontal velocity components u and v in place of $\tilde{U}_x$ and $\tilde{U}_y$, we can write $\tilde{M}_x=\rho(h+\bar{\zeta})u$ and $\tilde{M}_y=\rho(h+\bar{\zeta})v$. Introducing these expressions into Equations (3.1) and (3.2) and considering the steady state, we can derive the following equations:

$$\partial\left[(h+\bar{\zeta})u\right]/\partial x+\partial\left[(h+\bar{\zeta})v\right]/\partial y=0, \tag{3.16}$$

$$u\partial u/\partial x+v\partial u/\partial y=\left[T_x+R_x-(\partial S_{xx}/\partial x+\partial S_{xy}/\partial y)\right]/\rho(h+\bar{\zeta}), \tag{3.17a}$$

$$u\partial v/\partial x+v\partial v/\partial y=\left[T_y+R_y-(\partial S_{xy}/\partial x+\partial S_{yy}/\partial y)\right]/\rho(h+\bar{\zeta}), \tag{3.17b}$$

where ρ=constant. In order to satisfy Equation (3.16) a scalar function ψ is sometimes used, defined by

$$u(h+\bar{\zeta})=\partial\psi/\partial y,$$
$$v(h+\bar{\zeta})=-\partial\psi/\partial x. \tag{3.18}$$

The scalar function ψ was introduced by Arthur (1962) and called the transport stream function.

Remembering that $T_x/\rho(h+\bar{\zeta})=-g\partial\bar{\zeta}/\partial x$ and $T_y/\rho(h+\bar{\zeta})=-g\partial\bar{\zeta}/\partial y$, we cross-differentiate the first and second equations in Equations (3.17a, b) with respect to y and x, respectively, subtract the second equation from the first, and write the result using the vorticity component $\omega=\partial v/\partial x-\partial u/\partial y$,

$$\begin{aligned}-D[u\partial(\omega/D)/\partial x+v\partial(\omega/D)/\partial y]\\ =\partial(R_x/\rho D)/\partial y-\partial(R_y/\rho D)/\partial x\\ -\partial[\{\partial S_{xx}/\partial x+\partial S_{xy}/\partial y\}/\rho D]/\partial y\end{aligned} \tag{3.19}$$

where $D=h+\bar{\zeta}$. The term on the left-hand side, and the first and second terms on the right-hand side of Equation (3.19) are the nonlinear, frictional, and forcing terms respectively.

Bowen (1969b) treated the idealized nearshore-current pattern of a closed cell produced by waves arriving normal to the shoreline. He introduced linear expressions for the bottom friction terms and solved the resultant equation under the assumption that the wave height varies periodically along the shore. The important feature of Bowen's treatment was that it paved the way for numerical calculations of the nearshore-current system.

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Noda (1974) was the first to carry out numerical calculations for nearshore currents using realistic bottom contours and arbitrary incident-wave conditions. Bottom friction was included in this model, but nonlinear terms were neglected. In order to evaluate the frictional terms, he applied a quadratic resistance law with a constant-friction coefficient. Sasaki (1975) used essentially the same model with a variable-friction coefficient based on Jónsson's diagram for oscillatory flow, and calculated values of the friction coefficient at each mesh point. The calculated results gave rather good agreement with field-observation data in a qualitative sense.

4 MECHANISM OF COASTAL SEDIMENT MOVEMENT

In the treatment of coastal sediment transport, for simplicity it is quite common to consider separately sediment movement perpendicular and parallel to the shoreline. Needless to say, the two kinds of sediment movement are closely related to each other. However, onshore-offshore sediment movement is considered to be more significant for the short-term variation of coastal processes, while longshore sediment movement is more significant for the long-term variation of the coastal topography.

In the following section, we will discuss several topics related to the mechanisms of coastal sediment movement from the perspective of fluid mechanics. However, the essential point and goal of the present subject, that is, the evaluation of the sediment-transport rate, has not yet been fully clarified. The main reason for such an unfortunate state is the almost insuperable difficulty in measuring the rate and the direction of sediment transport in the coastal zone, which must be performed in conjunction with measuring the waves, currents, and bathymetry.

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4.1 *Inception of Sediment Movement*

The water depth at which sediment particles are first influenced by water motion is about 150 to 200 m, depending upon the environmental conditions, especially the wave characteristics. For engineering application, knowledge of the water depth where sediment particles move appreciable distances under wave action is important for determining the initial point of beach-profile change in the offshore region.

Considerable research has been conducted during the last three decades concerning the critical water depth for the inception of sediment movement. A detailed discussion of past treatments of this subject can be found elsewhere (Horikawa 1978b).

Horikawa & Watanabe (1967) treated the problem in the following simple manner. The equilibrium condition for a particle on the sea bottom can be expressed by

$$R_H = (W - R_V)\tan\phi, \tag{4.1}$$

where R_H and R_V are the horizontal and vertical components of the wave force acting on the particle, W is the immersed weight of the particle, and ϕ is the critical angle of static friction of a grain on the bottom in water. The values of W and R_H can be evaluated by the following equations:

$$W = (4\pi/3)(\rho_s - \rho)g(d/2)^3, \tag{4.2}$$

$$R_H = (1/4)K\pi d^2 \tau_{bm}, \tag{4.3}$$

where d and ρ_s are the grain diameter and density respectively, ρ is the fluid density, g the acceleration of gravity, τ_{bm} the amplitude of tangential stress acting on a grain due to the wave, and K a coefficient taking on different values depending upon the shape of the sediment particle and the type of movement. The magnitude of R_V was assumed negligibly small compared with W, and the following relationship was obtained:

$$\psi_m = \tau_{bm}/s\rho gd = 2/(3K) \tag{4.4}$$

where $s = (\rho_s - \rho)/\rho$ and ψ_m is a kind of Shields parameter. The theory of Kajiura (1968) for the bottom shear stress due to oscillatory flow was applied and comparisons made with experimental data on the inception of sediment-particle movement due to wave action. Based on these results, criteria were proposed for general movement in laminar or turbulent flow over a smooth surface, as well as for turbulent flow over a rough surface. Here, general movement is defined as the state where most of the sediment particles in the first layer are in motion. Based on the above, Horikawa & Sasaki (1970) compiled tables for the direct determination of the critical water depth for general sand motion under various wave conditions for sand grains having diameters between 100 μm and 1 mm. interment?

Shields parameter

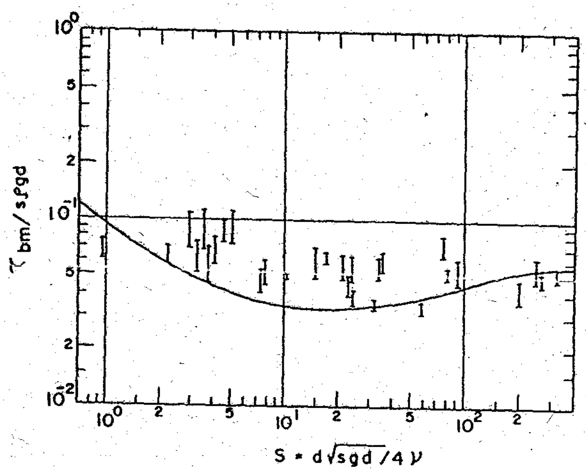

Figure 2 Experimental data on the initiation of sediment movement in oscillatory flow (adapted from Madsen & Grant 1976).

Madsen & Grant (1976) reanalyzed past results leading to the diagram shown in Figure 2, in which the ordinate is the Shields parameter $\psi_m = \tau_{bm}/s\rho gd$ and the abscissa is $S = d\sqrt{(sgd)}/4\nu$, where ν is the kinematic viscosity of the fluid. Considering the scatter in the data, we may well be able to assume ψ_m = constant. From this fact, Equation (4.4) still seems to be valid.

4.2 *Onshore-Offshore Sediment Movement*

In this section we will consider onshore-offshore sediment movement in a region at a depth shallower than the critical water depth. Dingler & Inman (1976) examined the bed form and type of onshore-offshore sediment movement in the nearshore area. In the offshore zone, sediment transport is induced by wave action as a combination of bed load and suspended load, and sand ripples on the bed play an important role in the mode of sediment transport. On the other hand, the sediment-transport mode inside the surf zone is considerably different from that outside. Agitation due to breaking waves is so strong that bed materials are suspended, and the concentration of suspended sediment in the vicinity of the breaking point is enormous. The amount of suspended sediment is strongly dependent on the breaker type (Kana 1978). Further shoreward, especially in the swash zone, sheet flow is predominant and sand ripples disappear.

In order to predict the time history of bottom-profile change, we must know the direction and the amount of net sediment transport due to wave action. However, at present, our knowledge on these subjects is still inadequate, and only a small number of results are available.

4.2.1 SAND RIPPLE FORMATION It has been known for many years that sand ripples are generated on the sea bed under the influence of waves. Bagnold (1946) was probably the first to systematically study the process of ripple formation. He carried out experiments using an oscillating board swung in an arc. Sediment particles of various sizes and specific gravities were spread over the board which was swung in a still body of water. The experimental procedure originated by Bagnold was continued by workers at the University of California, Berkeley. Using this kind of apparatus, Manohar (1955) obtained much data on sand ripples, and investigated fully the processes of generation, development, and disappearance of sand ripples. At about the same time, Inman (1957) measured the sizes of wave-generated ripples on the southern California coast.

The shapes of wave-generated ripples are rather symmetrical and are completely different from the shapes of ripples generated by unidirectional flow. Therefore the shape of a wave-generated ripple can be simply expressed by its height η and length λ. Hom-ma & Horikawa (1962) applied dimensional analysis to determine the functional relationships among ripple size, grain size, water depth, and local wave characteristics. These relationships are

$$F_1(\eta/\lambda, \lambda/d_0)=0, \tag{4.5a}$$

$$F_2(d_0/\lambda, u_{bm}d_0/\nu, w_0 d/\nu)=0, \tag{4.5b}$$

where $d_0 = H/\sinh(2\pi h/L)$, $u_{bm} = \pi H/[T\sinh(2\pi h/L)]$, h is water depth, H, T, and L are the wave height, wave period, and wave length, ν is the fluid kinematic viscosity, w_0 the fall velocity of a grain, and d the grain size. A subsequent study by Horikawa & Watanabe (1967) confirmed that the relationships expressed in Equations (4.5a, b) were also applicable to materials other than natural sand. Furthermore, Carstens, Neilson & Altinbilek (1969), Mogridge & Kamphuis (1973), and Dingler & Inman (1976) have continued investigations on the characteristics of wave-generated ripples in nearshore sands. It has been realized that sand ripples have an important role in sediment movement by wave action.

4.2.2 SEDIMENT TRANSPORT PATTERNS ON A BEACH Looking at the bottom configuration of a gently sloping beach, we can find a certain region where wave-generated sand ripples can be seen. In such a region the sediment-transport pattern is strongly influenced by the existence of the sand ripples in the following manner: Incident waves arriving to shore possess nonlinear characteristics, that is to say, the wave profile is peaked at a crest and flattened at a trough. Corresponding to these wave characteristics, the onshore velocity displays a large maximum over a shorter interval than that of the offshore velocity. Because of the asymmetrical characteristics of the velocity field, the onshore-offshore sediment transport due to wave action is not in balance over a wave period.

Sediment particles are, generally speaking, transported in the form of either bed load or suspended load. Owing to the wave nonlinearity discussed above, the net bed load in the present case would be in the onshore direction. On the other hand, suspended sediment particles are picked up by vortices generated behind ripples and transported onshore during the passage of a wave crest and offshore during the passage of a wave trough. Inman & Bowen (1963) demonstrated that offshore sediment transport occurs due to ripple asymmetry and resulting differences in intensity of wave-induced vortices. According to laboratory measurements of suspended sediment transport conducted by Sunamura, Bando & Horikawa (1978), the net suspended-sediment transport from solely these processes is in the offshore direction.

Based on observations of transport in a wave flume, Horikawa, Sunamura & Shibayama (1977) classified sediment-transport patterns due to wave action into the following four types:

Type 1: Bed-load transport is dominant, but no suspended sand cloud exists. Therefore the net sediment transport is in the onshore direction.

Type 2: Suspended sand clouds are formed, hence both bed load and suspended load should be considered. The net sediment-transport direction is either onshore or offshore depending on the dominance of the sediment-transport mode.

Type 3: Suspended-sediment transport is dominant. Suspended sand clouds are formed only on the onshore sides of ripples. Hence the net sediment transport is in the offshore direction.

Type 4: Suspended-sediment transport is dominant. Suspended sand clouds are formed on both sides of ripples, but the net sediment transport is in the offshore direction.

Shibayama & Horikawa (1980) presented limits for each transport type which are well described by the two parameters ψ_m and u_{bm}/w_0. Here u_{bm} is the amplitude of the near-bed fluid velocity and w_0 the fall velocity of a sand grain.

Tunstall & Inman (1975) treated fluid-energy dissipation due to vortices formed by oscillatory flow over a wavy boundary using a standing-vortex theory. Sunamura, Bando & Horikawa (1978) recorded sand-particle movement over a rippled bed using a 16-mm high-speed motion-analysis camera, and verified quantitatively that the movement of the suspended sand clouds is one of the main factors in producing net sediment transport. Sawamoto & Yamaguchi (1979) studied the motion of vortices above a rippled bed using potential-flow theory and estimated the suspended-sediment concentration above such a rippled bed. Shibayama & Horikawa (1980) introduced a dissipation model of time-dependent vortex circulation formed by wave action on a wavy boundary and calculated the water-particle path. Visual observation verified that the calculated water-particle path well represents the motion of the suspended sediment particles. Owing to the above efforts, the oscillatory velocity field in the vicinity of a wavy boundary can be analyzed, hence the motion of an individual suspended sediment particle seems to have been clarified in more detail.

4.2.3 SEDIMENT-TRANSPORT RATE At present, the available data for the bed-load-transport rate are only those obtained by Manohar (1955), Kalkanis (1964), and Abou-Seida (1965). Using these data and assuming that a Brown-type formula for the bed-load transport rate under a unidirectional-flow condition is applicable to the instantaneous bed-load transport rate by oscillatory flow, Madsen & Grant (1976) presented an empirical relationship for the average rate of sediment transport in an oscillatory flow, as shown in Figure 3. The ordinate, $\bar{\phi}=q_b/w_0 d$ is the

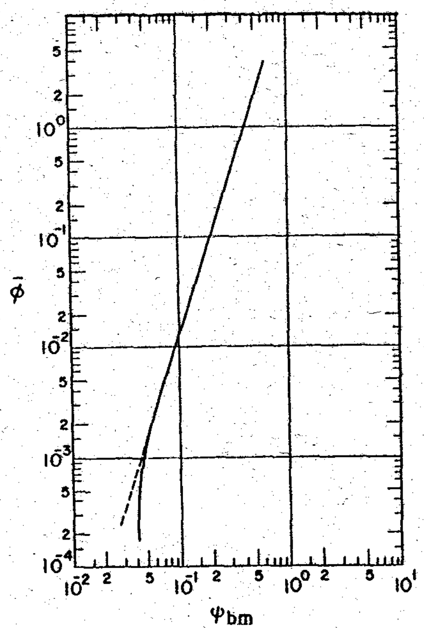

Figure 3 Empirical relationship for the average rate of sediment transport in oscillatory flow (adapted from Madsen & Grant 1976).

average bed-load transport rate, q_b, nondimensionalized by the fall velocity w_0 and the grain diameter d, while the abscissa, ψ_m, is the Shields parameter. Their proposed curve is expressed by

$$\bar{\phi}=12.5\psi_m^3. \tag{4.6}$$

Watanabe, Riho & Horikawa (1979) tested the above relationship between $\bar{\phi}$ and ψ_m outside the surf zone on a sloping sand bed in a wave flume by analyzing experimental data of wave characteristics and bottom-configuration change. The curves obtained are well fit by Equation (4.6).

Shibayama & Horikawa (1980) used the same procedure as Madsen & Grant to evaluate the suspended-sediment transport rate and obtained a similar relationship. Their treatment made use of the fact that bed-load particles are picked up, confined within vortices formed behind sand ripples, moved by the wave oscillatory flow, and finally deposited on the bed under gravitational force. In addition to these, Nielsen, Svendsen & Staub (1978) conducted intensive studies on the suspended-sediment transport mechanism. Sleath (1978) measured the quantity of bed materials in motion as bed load at each instant of the oscillatory-wave cycle.

At any rate, these relationships can be applied only to the sediment movement outside the surf zone. In order to look at the bottom change inside the surf zone, we must have some appropriate formula for the sediment transport rate in the sheet-flow region.

4.3 *Longshore Sediment Transport*

During the last two decades many approaches have been attempted to determine the longshore sediment-transport rate. Usually, one of the following general procedures is taken: 1 measuring the amount of deposition at the upstream side of a coastal structure such as a jetty or a breakwater, or of siltation inside a harbor basin; or 2 measuring the amount of the same kinds of sediment in a model basin. However, there are numerous problems to be solved, including the accuracy of the hydrographic survey, the limited extent of the surveyed area, the accuracy of the wave-energy evaluation, and the influence of grain size on the transport rate.

The longshore-transport rate should bear a close relationship with the longshore-current velocity, but the exact relation between them has not yet been established. On the other hand, many attempts have been made to correlate directly the wave-energy flux in the alongshore direction, E_y, with the rate of longshore transport, Q_y, according to the following form,

$$Q_y = \alpha E_y^m, \tag{4.7}$$

where α and m are constants to be determined empirically. Caldwell (1956) initiated the above treatment using field data. Subsequent to his work, numerous formulas have been presented during the last two decades. A comparison of these formulas was given by Galvin & Vitale (1976) and Horikawa (1978a).

One of the defects of Equation (4.7) is that it is not dimensionally correct. In order to eliminate the above defect, Inman & Bagnold (1963) suggested the use of the immersed-weight-transport rate I_t instead of the volume-transport rate. Komar & Inman (1970) established the following relationship between I_t and the longshore power at the breaking point, P_t:

$$I_t = KP_t. \tag{4.8}$$

The values of I_t and P_t are defined respectively as,

$$I_t = a'(\rho_s - \rho) g S_t, \tag{4.9}$$

$$P_t = \left(\bar{E} c_g\right)_b \sin\alpha_b \cos\alpha_b, \tag{4.10}$$

where S_t is the sand-volume-transport rate, ρ_s the sand density, ρ the fluid density, a' the correction factor for the pore space of the beach sand (approximately 0.6), g the acceleration of gravity, $\bar{E}$ the wave energy density, c_g the group velocity, α the angle between a wave crest and bottom contour, and the subscript "b" indicates the condition at the breaking point. The proportionality constant K is dimensionless and has been found equal to about 0.77 for a limited number of measurements. Komar & Inman (1970) correlated I_t with the longshore-current velocity based on the Bagnold (1963) model, and presented the following relationship,

$$I_t = 0.28\left(\bar{E} c_g\right)_b (\cos\alpha_b)\bar{v}/u_{bm}, \tag{4.11}$$

where $\bar{v}$ is the mean longshore-current velocity, and u_{bm} the maximum velocity under a breaking wave. The value of u_{bm} is calculated by

$$u_{bm} = \left(2\bar{E}_b/\rho h_b\right)^{1/2}. \tag{4.12}$$

Based on the calculation of Longuet-Higgins (1970), $\bar{v}$ is given by

$$\bar{v} = (5\pi/4)(\tan\beta/f_w)u_{bm}\sin\alpha_b \tag{4.13}$$

under the assumption that horizontal mixing is negligible, where $\tan\beta$ is ... slope. Substitution of Equation (4.13) into Equation (4.11)

and comparison with Equation (4.8) necessitates the conclusion,

$$\tan\beta/f_w = \text{constant}. \tag{4.14}$$

However, in the theory of Longuet-Higgins (1970), the mean longshore-current velocity depends on the parameter $P = 2\pi N \tan\beta/(\gamma f_w)$ which describes the importance of lateral mixing relative to bottom friction, N and γ being constants. In order to make clear the above results on the basis of a theoretical treatment, it is essential to clarify the dynamics in the nearshore area, especially inside the surf zone.

The above formulas, Equations (4.8) and (4.11), are applicable for the total alongshore sediment-transport rate inside the breaker line. In recent years, a great effort has been devoted to measuring the distribution of sediment-transport rate across the surf zone by, for example, using sand traps in a wave basin with a movable bed (Sawaragi & Deguchi 1978), or by taking a large number of core samples during dyed-sand tracer experiments in the field (N. C. Kraus, personal communication, 1979). This kind of approach, especially in the field, is truly laborious, but holds promise for advancing our knowledge of longshore sediment transport.

There is a standing debate on the subject of the dominant sediment-transport mode comprising longshore sediment transport. That is to say, which is more significant, suspended transport or bed-load transport? The impression that the suspended load is much more important within the surf zone has been supported traditionally (Dean 1973). On the other hand, Komar (1976) roughly estimated the suspended load and concluded that the suspended load comprises no more than approximately one fifth of the total transport. At present, it is fairly difficult to determine precisely the ratio between the suspended and bed transport rates. Therefore it may be a more suitable approach to focus on the total longshore transport rate. Dyed-sand experiments such as initiated by Komar & Inman (1970) are being actively performed by the Nearshore Sediment Transport Study Group (Seymour & Duane 1978) and in the writer's research group. Detailed analysis of core samples can yield important information on distribution of the moving-layer thickness and of the sand advection velocity of sediment particles across the surf zone. These data can be correlated with the wave and current characteristics at each sampling site.

5 SIMULATION OF SHORE PROCESSES

In order to treat coastal processes, as stated previously, we commonly separate the phenomena into two parts, namely onshore-offshore processes and alongshore processes. Beach-profile changes are considered to have a seasonal variation, that is to say, beach profiles seem to follow an approximately one-year cycle. In contrast, the alongshore change in beach topography is caused mainly by the local balance of alongshore sediment transport; variation in the longshore transport rate along the shoreline is the principal mechanism governing erosion and deposition of beach sediment. From this standpoint, Iwagaki (1966) formulated the following equation:

$$\partial \bar{h}/\partial t = (1 - \bar{h}/h_i)\partial h_i/\partial t + (\partial Q_y/\partial y)/(1-\lambda)B \qquad (5.1)$$

where B is the width of the littoral zone, Q_y the longshore transport rate, $\bar{h}$ the mean water depth in the littoral zone, h_i the water depth which determines the offshore limit of the littoral zone, λ the bottom sediment porosity, and the y axis is taken in the alongshore direction. Equation (5.1) indicates that coastal change has two contributions, the local variation of longshore transport, $\partial Q_y/\partial y$, and the time variation of h_i, which is determined by the time history of the incoming-wave characteristics. It is a consequence that

1. Even on a coast where $\partial Q_y/\partial y = 0$, beach erosion can occur with increasing wave height, because $\partial h_i/\partial t > 0$ in this situation and it follows that $\partial \bar{h}/\partial t > 0$.
2. When $\partial h_i/\partial t = 0$, erosion or deposition will completely depend on the sign of $\partial Q_y/\partial y$. That is to say, if $\partial Q_y/\partial y > 0$, erosion will occur because $\partial \bar{h}/\partial t > 0$, while if $\partial Q_y/\partial y < 0$, deposition will occur because $\partial \bar{h}/\partial t < 0$.

It can be said in general that the long-term variation in coastal topography is caused by local variation in the longshore transport rate. Through use of the above rules, if the rate of sediment transport along a beach can be evaluated, then equilibrium, erosional, or depositional regions along the coast can be determined as demonstrated in Figure 4. The prediction of shoreline changes occurring due to the presence of coastal structures can be predicted by this procedure as well. This approach has been developed and applied to practical problems.

The mathematical treatment of shoreline change was initiated by Pelnard-Considère (1956), who used a simple model as shown in Figure 5*a*. This concept was applied to the practical problems of predicting shoreline change due to the construction of groin. Price, Tomlinson & Willis (1972) developed a numerical model and checked the validity of its prediction by comparison with laboratory results. Hashimoto (1974) treated shoreline changes caused by the construction of a detached breakwater. Sasaki & Sakuramoto (1978) extended the one-line theory to a situation where the incident-wave direction varied with time, and

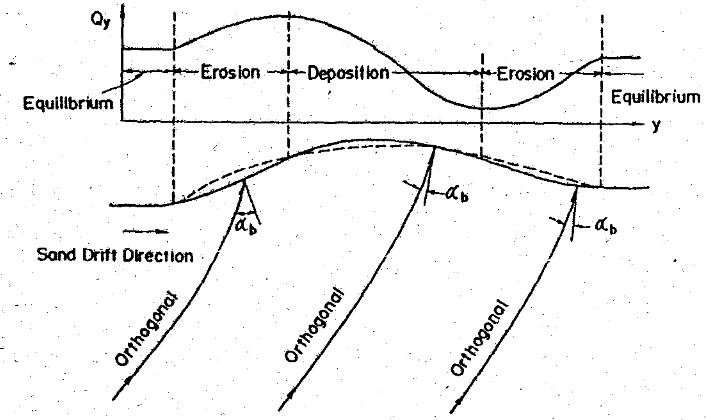

Figure 4 Mechanism of long-term beach change (adapted from Iwagaki 1966).

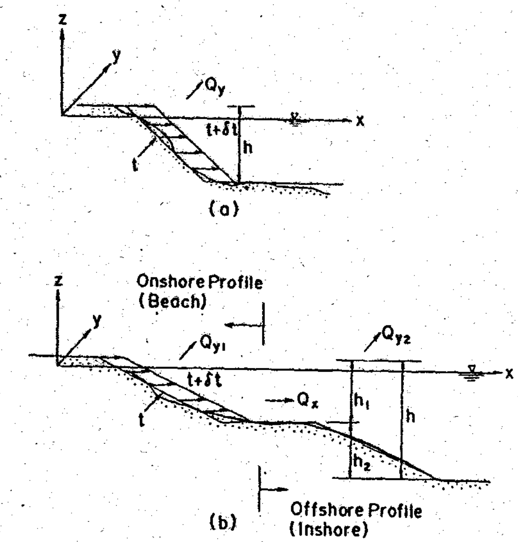

Figure 5 Schematic diagram for (*a*) the one-line theory and (*b*) the two-line theory [adapted from Pelnard-Considère (1956) for (*a*) and Bakker, Klein Breteler & Roos (1970) for (*b*)].

compared computed results obtained under certain reasonable assumptions with field-observation data. They confirmed that the agreement seemed to be fairly good for practical purposes.

In order to improve the modeling of beach-profile change, Bakker, Klein Breteler & Roos (1970) proposed the two-line theory as shown in Figure 5b, in which the onshore-offshore sediment transport is partially treated. More generally, the two-line theory could be extended to a multi-line theory. However, it should not be forgotten that further studies are needed to clearly define the physical meaning and limitations of various quantities that are used in such computations.

The two-dimensional approach seems to be rather powerful in evaluating shore processes for engineering purposes. However, the models used in the above treatments are too crude to predict the detailed structure of the sea-bottom configuration change. Therefore, as a goal we should aim to clarify three-dimensional coastal processes, even though the goal seems to be far away from our present position.

6 CONCLUDING REMARKS

The study of coastal sediment processes is one of the most difficult in coastal engineering. This field has been treated by scientists and engineers during the last few decades without any remarkable advancement. However, in recent years several large cooperative research groups have been organized in various countries such as Japan and the US with the strong intention of breaking through the difficult barriers obstructing the development of our understanding of coastal sediment processes. Therefore, the time when the state of the art can be completely reviewed will hopefully come in the not too distant future.

In preparing this review article, the writer collected and consulted a large amount of material. However, only a portion of the relevant literature could be cited owing to the lack of space. In addition to this, the subject matter had to be restricted, so that numerous related topics, such as the mass-transport velocity distribution, suspended-sediment concentration distribution, and wind-blown sand transport, were not covered. For these subjects, the reader should consult appropriate reference materials such as the *Proceedings of the International Conferences on Coastal Engineering*, *Coastal Engineering in Japan*, *Shore Protection Manual of the US Army Corps of Engineers*, or appropriate textbooks (Silvester 1974, Komar 1976, Horikawa 1978a).

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