Review

Dynamic behavior of coastal sediment

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(Communicated by Takao INUI, M. J. A.)

Abstract: The study of coastal sediment processes was carried out initially by coastal geomorphologists in the past century. However, the dynamics of nearshore sediment movement has been treated by researchers in the fields of coastal engineering and nearshore oceanography over the past half century. The aim of this paper is to review the achievement of the related researches up to the present and to suggest finally the subjects which should be studied more deeply in the near future. This review article covers 1) the significance of coastal sediment study as an introduction, 2) an outline of coastal sediment behavior, 3) the critical water depth for the inception of sediment movement, 4) various modes of coastal sediment movement, 5) numerical simulation models of beach transformation, and 6) conclusions and recommendations for future studies.

Key words: Bed load; critical water depth for the inception of sediment movement; oscillatory boundary layer flow; sand ripple; sheet flow; suspended load.

Introduction. Tanaka *et al.*[1] investigated the long term variation of the Japanese coastal area by using topographic maps issued by the Geographical Survey Institute, and reported that the yearly rate of the vanishing area of sand and gravel beaches during the recent 15 years was 160 ha/yr. Dividing this value by 9,499 km, the total length of the sand and gravel beaches, induces 1/6 m/yr, which is the yearly rate of beach width recession. Assuming the average beach width in Japan to be 30 m, all sand and gravel beaches will disappear within 180 years. This prediction indicates that the beach erosion in Japan is so severe that the coastline should be protected in some way. At present 43% of the total length of the Japanese coastline, 34,837 km, has been designated to be preserved.

In the USA,[2] in contrast, the total length of coastline is 135,570 km, among which 32,990 km (24.3%) and 4,350 km (3.2%) have suffered from beach erosion significantly and seriously, respectively.

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On the other hand, the coastal zone plays an important role in sustaining the ecosystem in the coastal region, protecting the coast against disastrous forces, and giving us the opportunities to develop and utilize the coastal area. Therefore, it is an essential matter in the developed countries in particular to harmonize the development of land and its preservation. From that view point, understanding coastal sediment behavior should be clarified as one of the important tasks.

Coastal sediment behaviors. The study of coastal sediment processes has been systematized over the past century by coastal geomorphologist. A typical contribution is “Shore Processes and Shoreline Development” written by D. W. Johnson in 1919.[3] From this book we can learn a tremendous amount of features of shoreline development through photographs as well as illustrated figures. Hence, this book is quite instructive to not only coastal geomorphologists but also coastal engineers about how to grasp qualitatively the general idea of shoreline development. The author also tried to explain how these characteristic shorelines were formed in connection with the natural forces such as waves, currents, tide, and wind. However, the dynamic behavior of shoreline change was not clear enough to be understood. Since then the aspect of nearshore sediment movement has been treated during the past half century by the scientists in the fields of coastal engineering and nearshore oceanography.

Extensive studies on coastal sediment processes have been carried out in numerous countries for the purposes of coastal protection as well as coastal development. The following three contributions are representative outcomes of the above activities. The first one is "Beach Processes and Sedimentation" written by P. D. Komar in 1976[4] from the view point of coastal geomorphologist. The second is "Sea Bed Mechanics" written by J. F. A. Sleath in 1984,[5] and the third is "Nearshore Dynamics and Coastal Processes" edited by K. Horikawa in 1988[6] both of which are from the stand point of coastal engineering.

For our purposes here, the term "*coastal sediment processes*" will cover the chronological history of the numerous phenomena related to important practical problems such as siltation of harbor basins and beach erosion.

It was not so long time ago that most coastal projects were carried out by trial and error because of a lack of knowledge of the underlying mechanism of coastal sediment processes. Therefore, the coastal engineers have realized that the coastal sediment study is one of the most difficult subjects among the numerous coastal engineering matters. However, during the last half century, large quantities of data have been gathered in studies of the transport of coastal sediment through field as well as laboratory investigations. As our understanding of coastal sediment behavior was expanded so rapidly that it is now practically possible to predict the coastal features by using numerical models developed appropriately over this period.

As the coastal sediment processes are extremely complex and include phenomena having quite different scales in space and in time. For purposes of clarification, Horikawa[7] proposed classifying coastal phenomena into three categories such as macroscale, mesoscale, and microscale. These scales can be roughly described as shown in Table.[8] The target of our study is to cover the whole regions and to connect the microscale phenomena with the mesoscale ones, and then the mesoscale phenomena with the macroscale ones. At the present stage the above task is not completed but is nearing completion. The above concept should be kept in mind for studying the coastal sediment behavior.

A large number of coastal engineers around the world have engaged in the coastal sediment studies up to the present. However, the writers of this review article would like to concentrate their attention to the achievement made mainly at the Coastal Engineering Laboratory, the University of Tokyo during the last fifty years.

Outline of coastal sediment processes. Figure 1[9] schematizes the relations among the various elements such as waves, nearshore currents, sediment transport, bottom topography, *etc*. The aim of our investigations is to clarify these complex relationships. In order to treat them, a suitable approach must be selected from the three scales discussed above.

Figure 2 illustrates the on-offshore beach profile along which various phenomena of waves and sediment movement appear. The wave coming from offshore changes its height and wavelength in response to the water depth at each location. The waves then break at a certain water depth, and progress as a broken wave. Another characteristic feature is the wave-induced current, which is called as nearshore current. The nearshore current consists of the longshore current and the rip current as shown in Fig. 3.[10] The bed material will be moved by waves and nearshore currents. Our task is to clarify the forms as well as the rate of sediment transport with its direction.

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

Critical water depth for sediment movement. The water depth at which sediment particles on the sea

Table. Classification of coastal process scales (Horikawa, 1981).

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

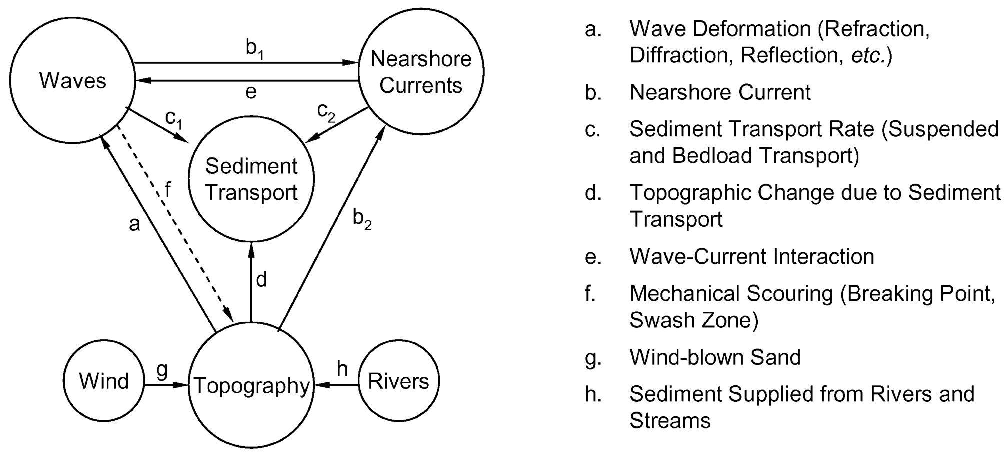

Fig. 1. Relationship among waves, nearshore currents, sediment transport, and topography (Horikawa, Harikai and Kraus, 1979).

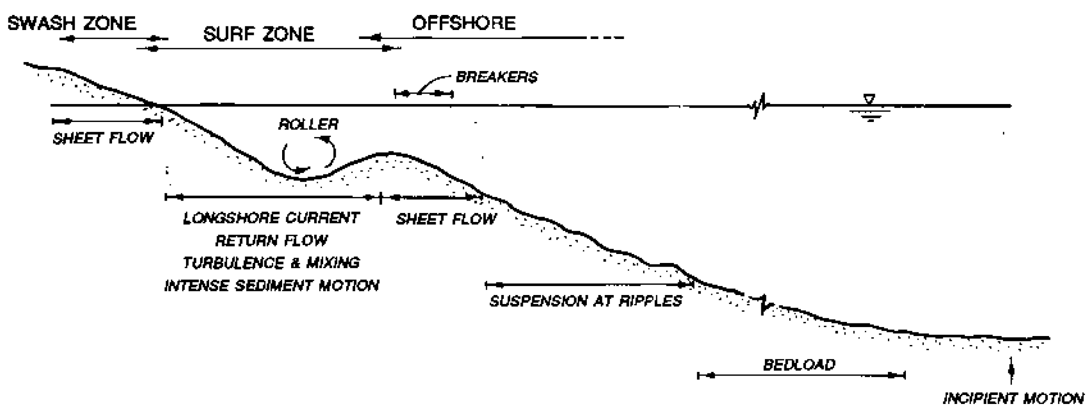

Fig. 2. Major region of the nearshore.

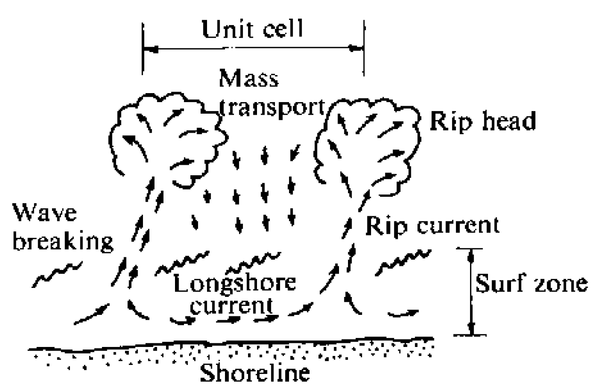

Fig. 3. Nearshore current system (Horikawa, 1978).

bed are first affected by wave motion is said to be 150 to 200 m, depending upon the environmental conditions, particularly the wave characteristics. For engineering application, knowledge on water depth where sediment particles move appreciable distance under wave action is important for determining the initial point of beach profile change in the offshore region. The stated critical water depth defines the outer boundary of the interested region for our investigation.

Considerable investigation on the critical water depth for the sediment movement has been conducted in the period of the 1950s by Japanese researchers.[10) They carried out their studies mainly in laboratory. While Sato *et al.*[11) conducted their extensive field study by installing radioactive glass sand on the sea bed and then tracing the stated sand particles by a scintillation counter. The latter data are comparatively extremely valuable comparing with others.

These empirical formulas are summarized in the following common expression

$$\frac{H_o}{L_o} = \alpha\left(\frac{d}{L_o}\right)^n\left(\sinh\frac{2\pi h_i}{L}\right)\left(\frac{H_o}{H}\right) \qquad [1]$$

where H_o and L_o are the wave height and wavelength, respectively, in deep water, h_i the critical water depth for the inception of sediment movement, H and L the wave height and wavelength at h_i, respectively, and d the grain size of bed material. The coefficient α and exponent n were determined empirically by each of the authors.

Horikawa and Watanebe[12) treated the same subject rationally as described as follows: It is well known that the velocity distribution of oscillatory boundary layer was given by Longuet-Higgins[13) in case of laminar flow. On the other hand Kajiura[14) analyzed the oscillatory boundary layer in case of turbulent flow by introducing a model to the momentum exchange coefficient, so-called the eddy viscosity K_z. The basic equation inside the boundary layer is written in the following: Referring the expression of K_z for steady flow, Kajiura proposed the expression of K_z for oscillatory flow from the bed to the outer edge of the boundary layer. The boundary layer is divided into three layers, namely the inner layer, overlap layer, and outer layer. Then the fundamental equation was solved to give the velocity distribution inside the boundary layer.

In order to determine the critical condition for the onset of sediment movement under wave action, the following equation was introduced to express the equilibrium condition for a particle on the sea bed

$$R_H = (W - R_V)\tan\phi \qquad [2]$$

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 following expressions:

$$W = \left(\frac{4\pi}{3}\right)(\rho_s - \rho)g\left(\frac{d}{2}\right)^3 \qquad [3]$$

$$R_H = \left(\frac{1}{4}\right)K\pi d^2\tau_{bm} \qquad [4]$$

where d and ρ_s are the grain diameter and density, respectively, ρ 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 different values depending upon the shape of the sediment particle and the type of movement. The magnitude of R_V was assumed to be negligibly small compared with W, and the following relationship was obtained

$$\psi_m = \frac{\tau_{bm}}{\rho sgd} = \frac{2}{(3K)} \qquad [5]$$

where $s = (\rho_s - \rho)/\rho$ and ψ_m is a kind of Shields parameter. The theory of Kajiura for the bottom shear stress due to oscillatory flow was applied and comparisons were made with experimental as well as field data on the inception of sediment-particle movement due to wave action. The data were classified into the following categories, such as laminar boundary layer case, turbulent boundary layer case with hydraulically smooth bottom, and turbulent boundary layer case with hydraulically rough bottom. Figures 4 and 5 are the criteria for laminar flow and turbulent flow with a smooth surface and for turbulent flow with a rough surface, respectively.

Madsen and Grant (1976)[15) reanalyzed past results leading to the diagram shown in Fig. 6, in which the ordinate is the Shields parameter $\psi_m = \tau_{bm}/\rho sgd$ and the abscissa is $S = d\sqrt{(sgd)}/4\nu$, where ν is the kinematic viscosity of fluid. The term S is a Reynolds number defined on the basis of a grain size of sediment. Considering the scatter in data, we may well be able to assume ψ_m = const. or ψ_m is variable depending upon the grain size.

Various modes of coastal sediment movement. When offshore waves come into the water which is shallower than the critical water depth for the inception of sediment movement, the sediment particles of bed material are forced to move back and forth by the wave-induced bed shear stress. This kind of bed materi-

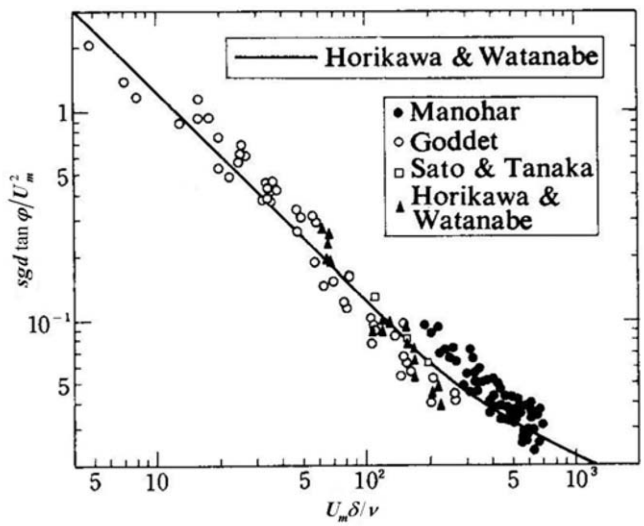

Fig. 4. Criteria for general movement for laminar flow and turbulent flow with a smooth surface. In the figure δ denotes the thickness of viscous sublayer (Horikawa and Watanabe, 1967).

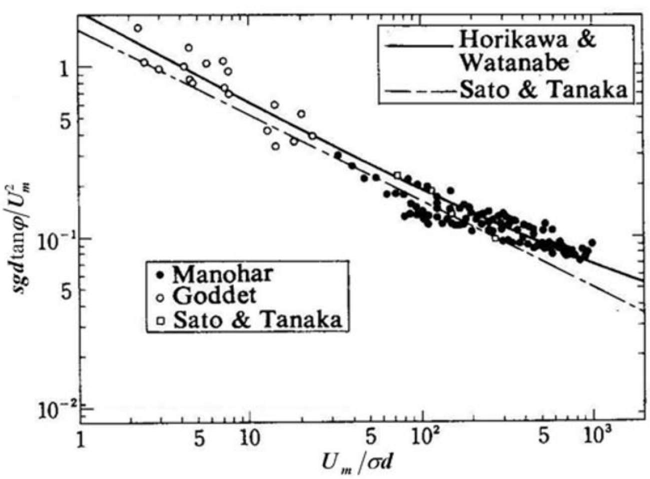

Fig. 5. Criteria for general movement for turbulent flow with a rough surface (Horikawa and Watanabe, 1967).

al movement is called **bed load transport**. The bed load movement is in general asymmetric in the sense of magnitude as well as the direction of movement owing to the asymmetric profile of waves on a sloping bed. Therefore, the onshore transport of bed load is different in amount from the offshore transport. Figures 7 (a) and (b) show the cross-shore transport rate in the direction of onshore and offshore, respectively.[16] The plotted data were obtained in both small wave flumes and a prototype wave flume. The abscissa of these figures is the non-dimensional sediment transport rate $\Phi = q/w_0 d$ and the ordinate is the Shields parameter $\psi_m = \tau_{bm}/\rho sgd$, where q is the bed load transport rate, w_0 the fall velocity of a grain, d the grain size, τ_{bm} the amplitude of bed shear stress acting on a grain. As for determining the direction of net sediment movement Fig. 8[17] can be utilized, where the abscissa is the flow intensity defined by $\psi' = (d_0 \sigma)^2/sgd$, and the ordinate is the Ursell para-

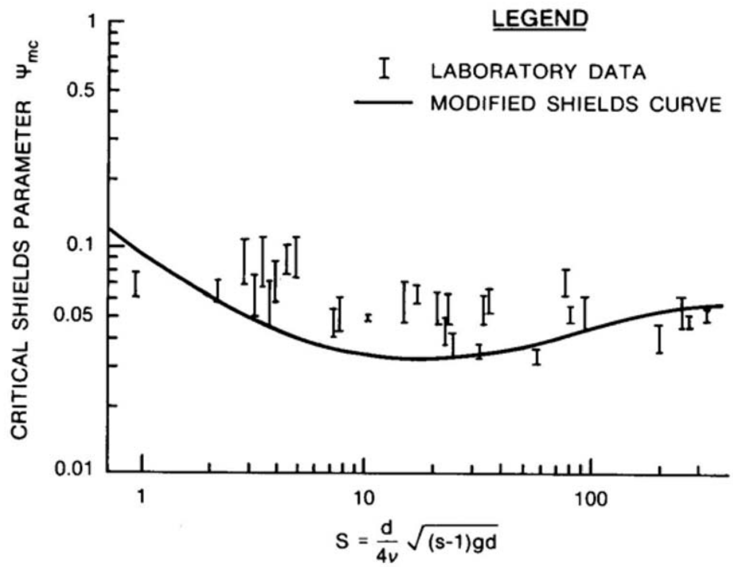

Fig. 6. Critical conditions for the initiation of sediment movement under oscillatory flow (Madsen and Grant, 1976).

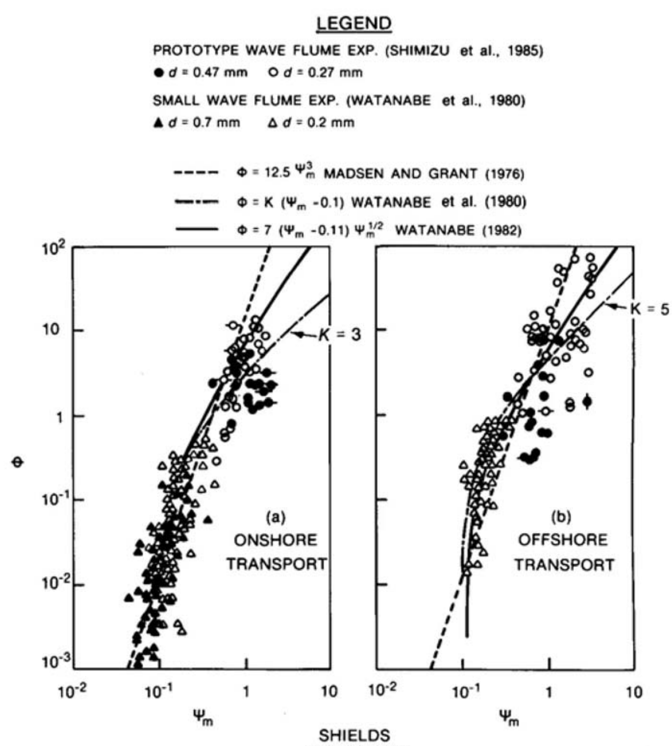

Fig. 7. Comparison of measurements of cross-shore transport in small and large flumes and semi-empirical predictive relations (Shimizu *et al.*, 1985).

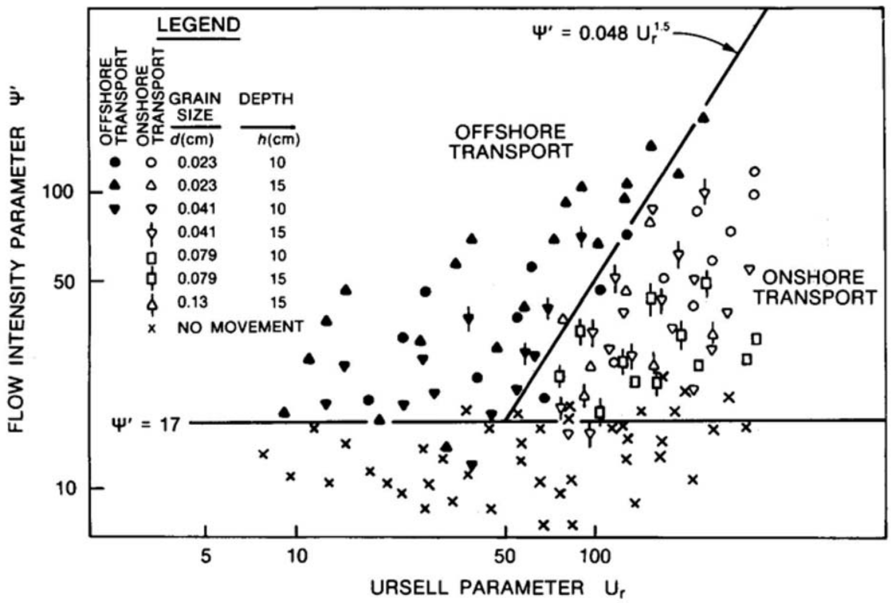

Fig. 8. Classification of net cross-shore transport direction (Sunamura, 1982).

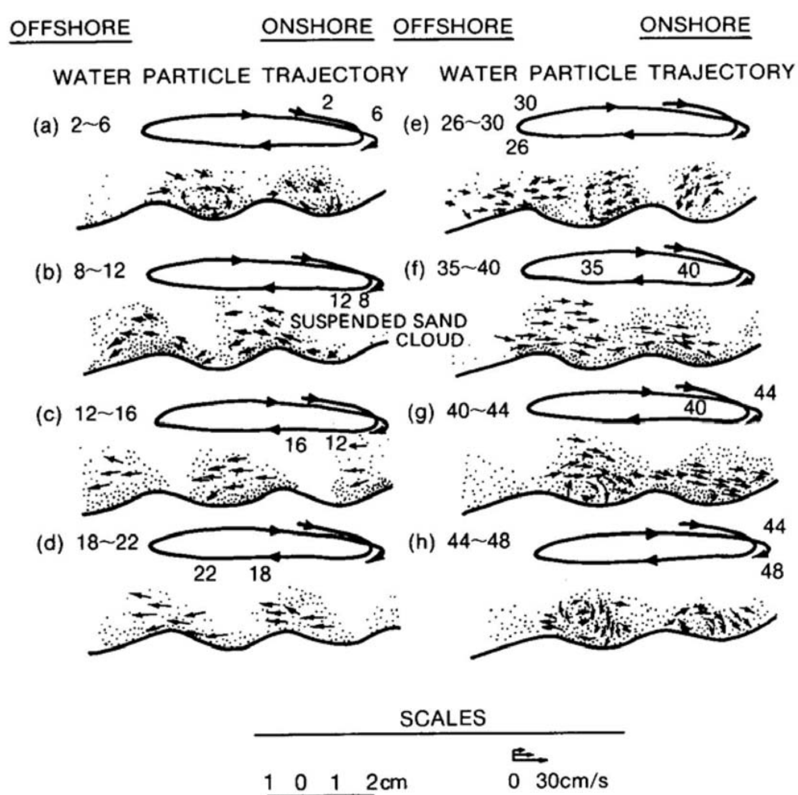

Fig. 9. Movement of sediment cloud over a ripple (Sunamura, Bando, and Horikawa, 1978).

meter $Ur = HL^2/h^3$. In these terms, $d_o = H/\sinh(2\pi h/L)$ is the excursion length of the near bottom orbital motion, H and L are the wave height and wavelength, respectively, at the water depth h, $\sigma = 2\pi/T$, and T the wave period.

As the wave advances further into the shallower region, the movement of bed material is more active to form sand ripples on the bed. Looking at the fluid motion induced by waves in the vicinity of sand ripples in more detail, it is clearly seen that a vortex is formed behind the ripple crest, and sand particles are confined into the vortex. The vortex with sand particles is then transported by the reversal flow, and the sand particles finally settle down to the bed. This type of sediment movement is called **suspended load**. Figure 9[18] illustrates the process of sediment cloud observed in a wave flume. It is clearly observed that the suspended sediment cloud is originated in the vortex which develops during the shoreward flow. The sediment cloud thus formed is transported backward during the stage of offshoreward flow, and a part of the sediment particle settles down to the bed. Detailed observation of Fig. 9 points to the following conclusion: that is to say, the suspended sediment is finally transported toward the offshore direction.

Bagnold (1946)[19] was probably the first to systematically study the process of ripple formation. Since then a number of studies were carried out to collect laboratory as well as field observation data. Hom-ma and Horikawa (1962)[20] applied the dimensional analysis to determine the functional relationships among the ripple rise and pitch, grain size, water depth, and local wave characteristics. Sato (1987)[21] continued his effort to introduce more general functional forms between λ/d_o and $d_o/d \cdot (\psi_{rms})^{1/2}$ in Fig. 10 (a), and η/λ and ψ_{rms} in Fig. 10 (b). Where d_o is the excursion length of the near bottom orbital motion, d grain diameter, and ψ_{rms} the root mean square value of the Shields parameter, and η and λ are ripple rise and ripple pitch, respectively. Figures 10 (a) and (b) show the characteristic relations among the ripple sizes, grain size, and oscillatory flow characteristics. There is not much difference in characteristic curves for symmetrical as well as asymmetrical flow cases. In some cases the ripple form is 3D instead of 2D. However, 3D ripples also follow the same characteristic curves as 2D ripples.

Figure 11[22] shows the comparison of flow velocity distributions in the vicinity of ripples. The upper figure is the measured one at the phase of $\sigma t = 3\pi/10$ by using a Doppler anemometer and the lower figure is the calculated one at the same phase. The flow patterns illustrated in (a) and (b) are in reasonably close agreement each other. Figure 12[22] shows the vertical distribution curve of mean concentration of suspended sediment calculated and the data obtained in a wave flume. The agreement is fairly good. Then the rate of suspended sediment transport in either offshore or onshore direction can be evaluated.

Ripple formation has interested many researchers. Figure 13 indicates the region of ripple formation.[23] The ordinate and abscissa in this figure are the Shields parameter and the Reynolds number based on the grain diameter, respectively. The lower limit is defined by the modified Shields curve given by Madsen and Grant (1976)[15] and the upper limit is the inception of sheet flow given by Horikawa *et al.* (1982),[24] respectively. The former corresponds to the inception of sediment movement and shows the initiation of ripple formation, while the latter indicates the condition for disappearance of sand ripples owing to high speed of oscillatory flow.

The sand transport above the rippled bed is not normally observed in the field, but flow condition usually meets with another mode of sediment transport which is called **sheet flow**. In order to generate oscillatory flow with velocity strong enough to wash away sand ripples, the oscillatory flow tank and a system of measuring apparatus as shown in Fig. 14 were provided at the University of Tokyo. Figure 15 gives a typical example of observed data[24]: the first column gives the vertical distribution of sediment concentration at various phases, the second column does the vertical distribution of sand particle velocity, and the third column does the vertical distribution of the sand flux calculated by the product of the first and the second columns. Based on the experimental data stated above and other available ones, the non-dimensional transport rate and the Shields parameter are correlated as shown in Fig. 16.[24] From this figure it is possible to evaluate the sediment transport rate under the sheet flow condition. Since Horikawa *et al.* presented their paper in 1982, a number of researchers have treated the same subject from various view points. Dibajnia and Wananabe[25] proposed sand transport formula under sheet flow, while Asano (2000)[26] summarized this in his review article.

Numerical simulation model of beach transformation. It was seriously desired in the 1970s to develop appropriate numerical simulation models of beach transformation in order to predict long term variation of beach change induced by the construction of coastal structures. Then Horikawa thus organized a

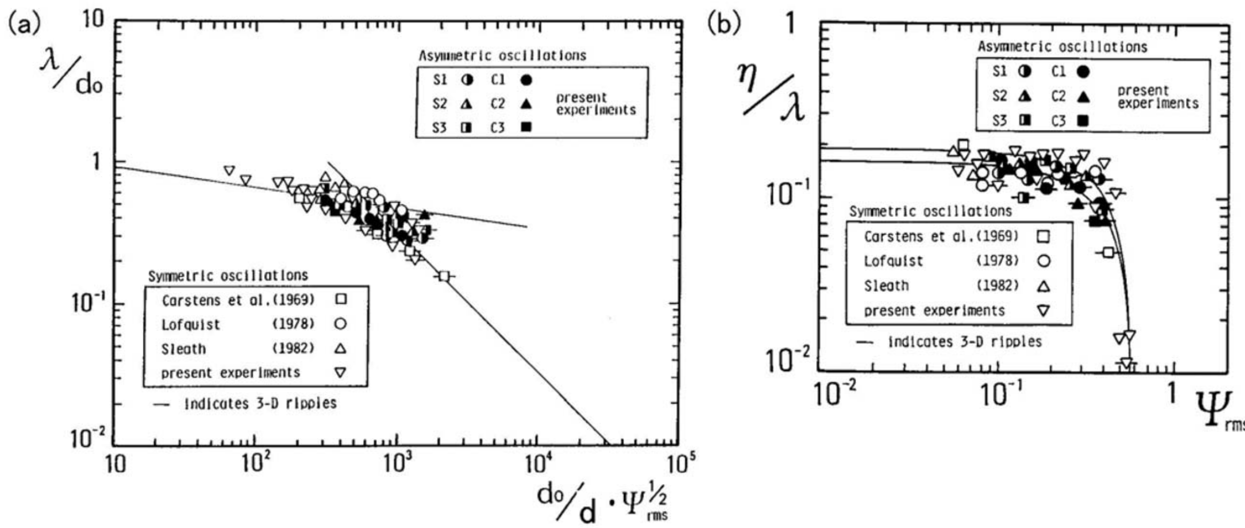

Fig. 10. Relationships between λ/d_o and $d_o/d \cdot (\psi_{rms})^{1/2}$ (a), and between η/λ and ψ_{rms} (b) (Sato, 1987).

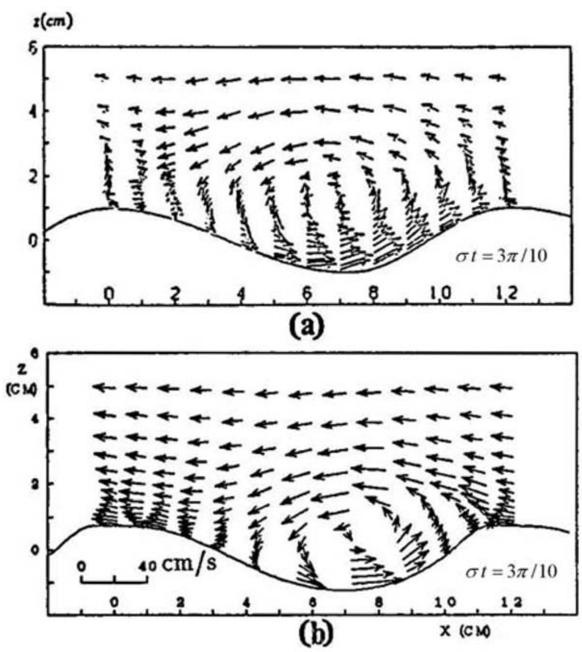

Fig. 11. Comparison between measured and calculated flow field above ripples (Pena-Santana *et al.*, 1990).

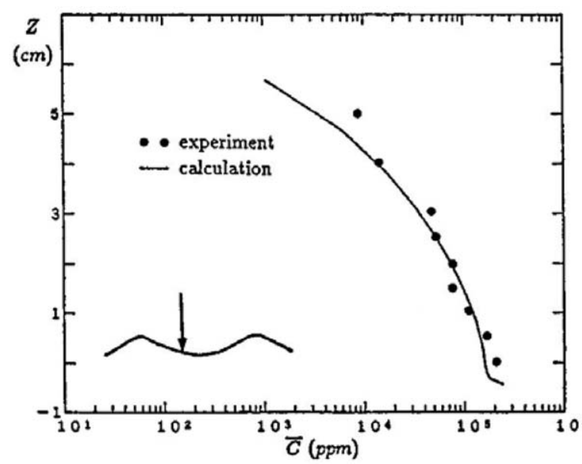

Fig. 12. Comparison of measured and calculated vertical distribution of suspended sediment concentration (Pena-Santana *et al.*, 1990).

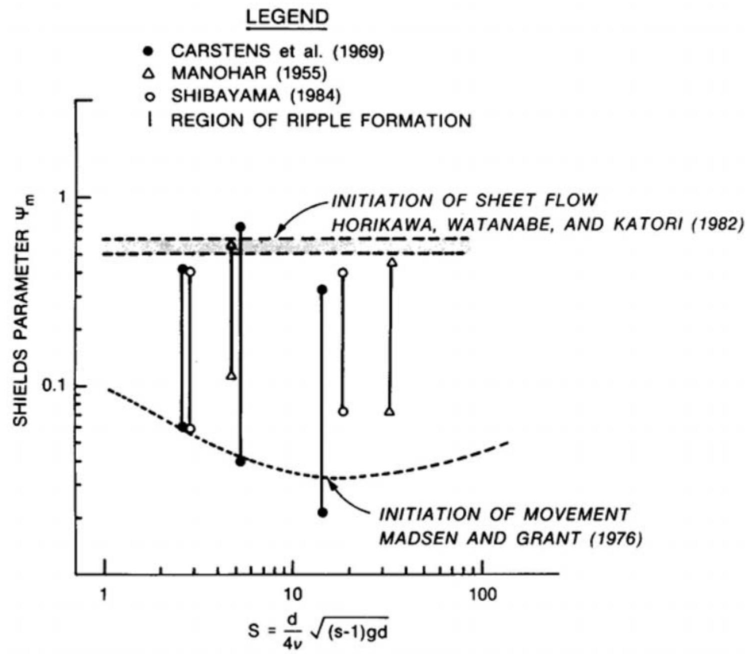

Fig. 13. Region of ripple formation (modified from Komar and Miller, 1975).

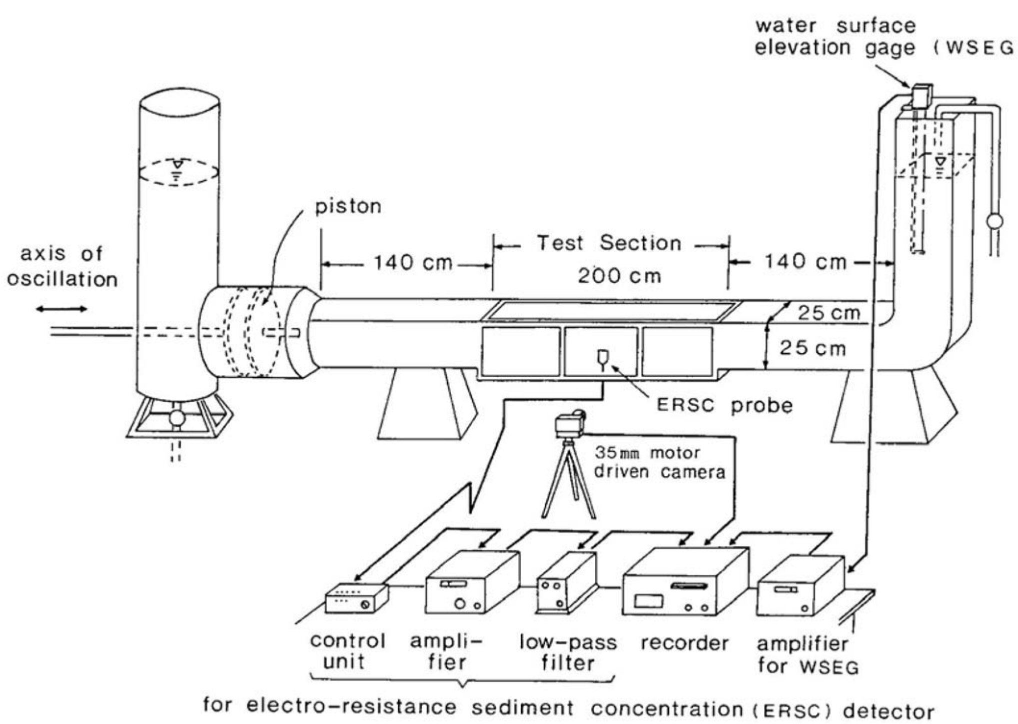

Fig. 14. Oscillatory flow tank and measuring apparatus.

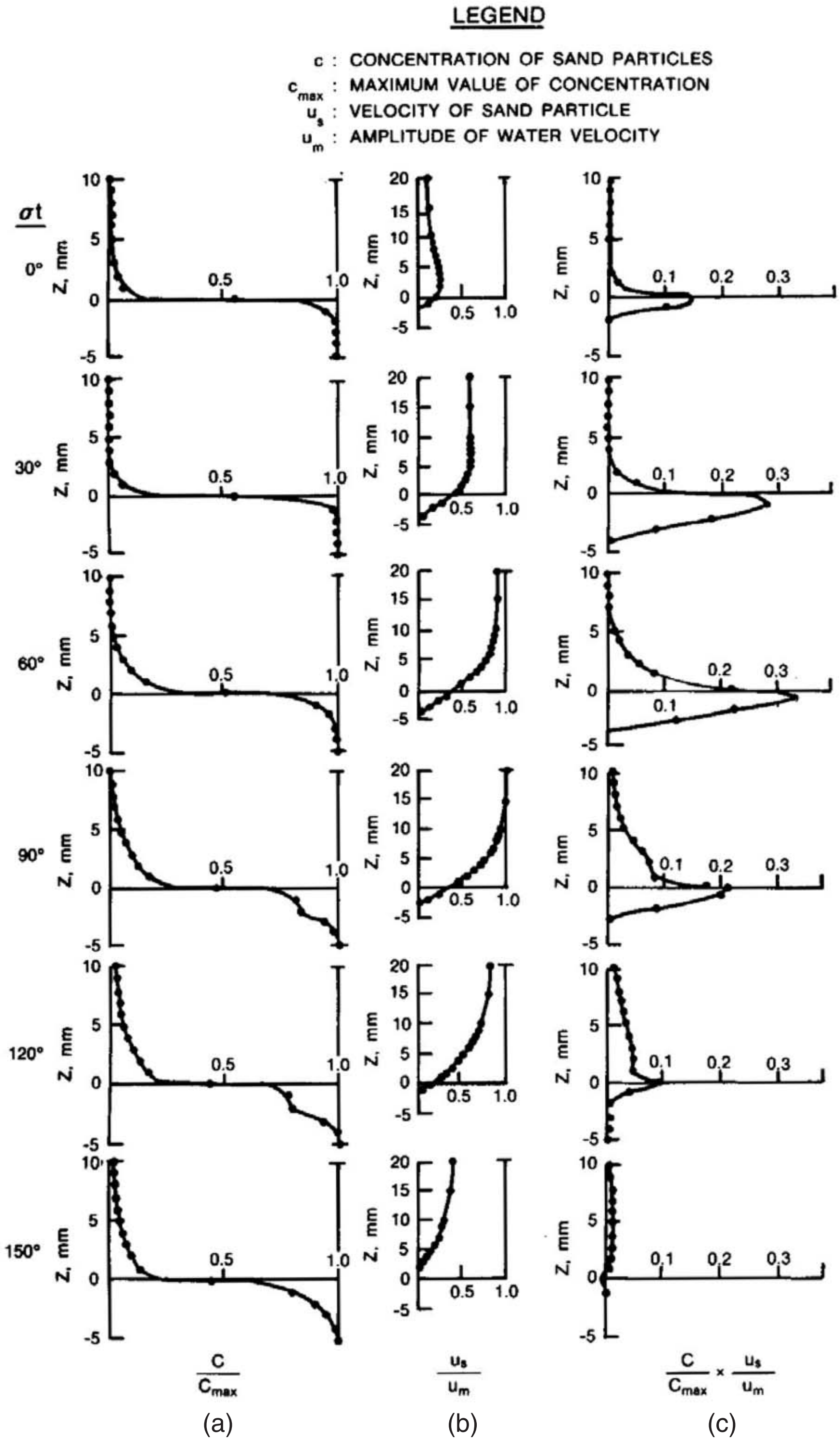

Fig. 15. Measurements of the time history of vertical distribution of sheet flow, showing (a) sand concentration, (b) sand velocity, and (c) calculated sand flux (Horikawa *et al*., 1982).

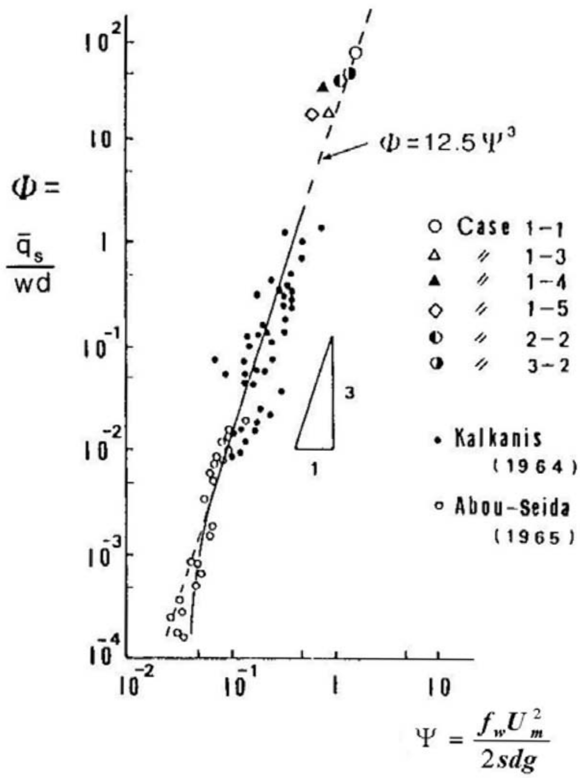

Fig. 16. Relation between non-dimensional transport rate and Shields parameter (Horikawa *et al.*, 1982).

joint research group in 1977 under the financial support of the Toyota Foundation and conducted preparatory research work including field observations. This project was continued for another six years (1978 to 1984) under the financial support provided by Electric Power Industry of Japan. The outcome was compiled finally as a book entitled "Nearshore Dynamics and Coastal Processes".[6] It is interesting that the Nearshore Sediment Transport Study Program in the United States took place almost in parallel to our program. The common interest was to carry out intensive field observations to understand the real phenomena in field. In our case, voluminous data of nearshore waves, nearshore currents and sediment transport were collected simultaneously in order to correlate each other.

Figure 17 is the observed velocity vectors in a rip current by way of the balloon camera system developed by our research group.[27] Figure 18 shows wave profiles and current velocity measured simultaneously in the surf zone.[6] The stated information is valuable to simulate the waves and current in a model.

Beach profile change model. The waves coming from the offshore induce the movement of beach material to change the original beach profile to the new one in a relatively short period. In order to predict the new beach profile, the wave height at each location should be predicted first, and then the on-offshore transport rate and its direction must be evaluated. Thus the new beach profile can be predicted on the basis of conservation of beach material. Figure 19 is a typical example showing the comparison of the calculated beach profiles and the laboratory ones. The agreement is satisfactory for our practical purposes. The above procedures are applied frequently to reach the acceptable results. In recent years the above beach profile change model was improved by using a transport model based on two-phase flow by Mina and Sato (2004).[28]

In order to predict the beach-deformation such as beach erosion as well as beach accretion, it is quite necessary to predict the future transformation of beach area. As stated above, a comprehensive investigation program was organized by Horikawa to develop effective numerical simulation models. Then a guideline to develop various numerical models was presented based on the time scale as well as spatial scale as shown in Fig. 20.[6]

Shoreline model. This model is applicable to predict the shoreline change in a moderate period of several years for a certain range of shorelines. The basic concept of this model is illustrated in Fig. 21, in which the beach profile is assumed to move offshoreward or onshoreward in parallel to the original one.[6] Let us take two sections apart Δy in the alongshore direction, and define Q as the volume rate of longshore sediment transport, and D_s as the depth at the outer edge of littoral zone. The quantity q denotes the rate of sediment entering and leaving the profile from the landward and seaward boundaries, i.e.,

$$q = q_s + q_o \qquad [6]$$

in which q_s is the rate of sediment volume entering or leaving per unit width of shoreline and q_o is the corresponding rate for the seaward boundary. Therefore, the quantity q can represent, for example, a sediment discharge from a river or a loss due to sand mining. Then the continuity equation governing the shoreline position x_s becomes:

$$\frac{\partial x_s}{\partial t} = \frac{1}{D_s}\left(\frac{\partial Q}{\partial y} - q\right) = 0 \qquad [7]$$

In order to solve the above equation, two terms D_s and Q must be evaluated. As a result of discussion the

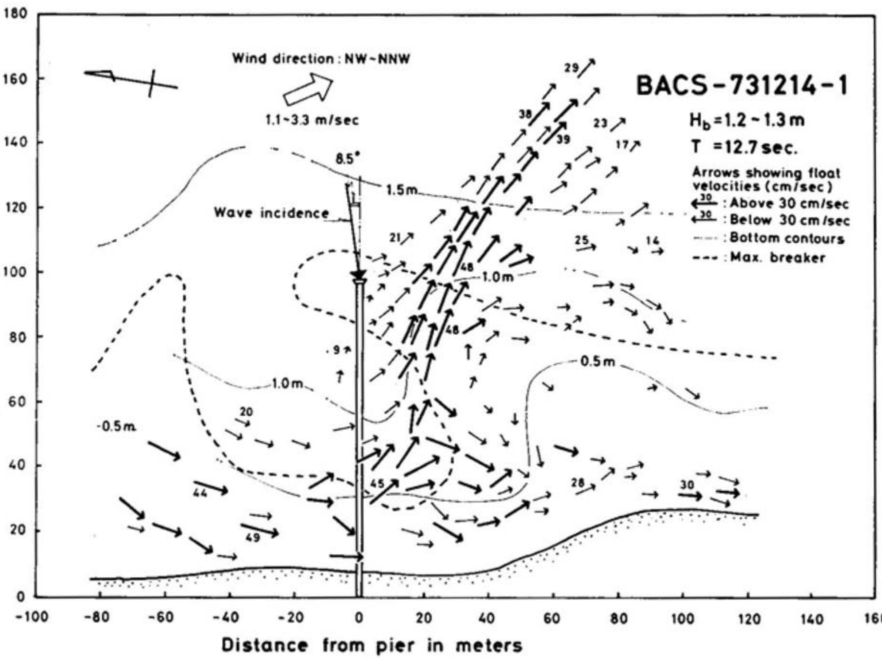

Fig. 17. Velocity vectors in a rip current (Sasaki and Horikawa, 1975).

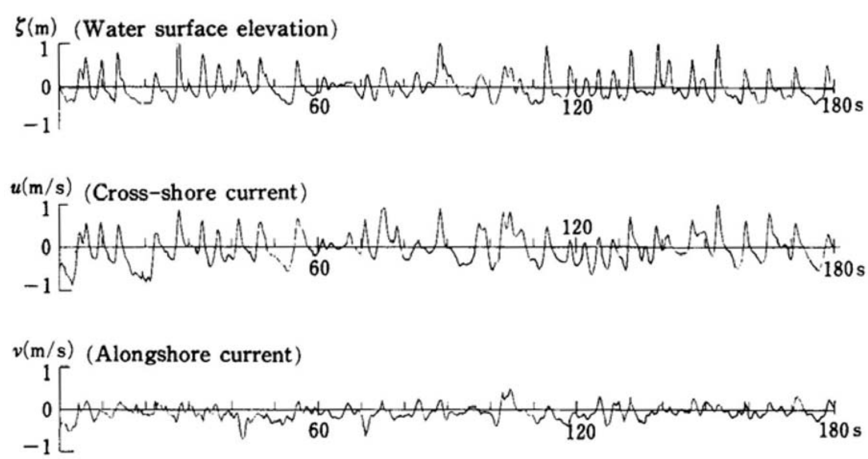

Fig. 18. Wave profile and current velocity measured in the surf zone (Horikawa, 1988).

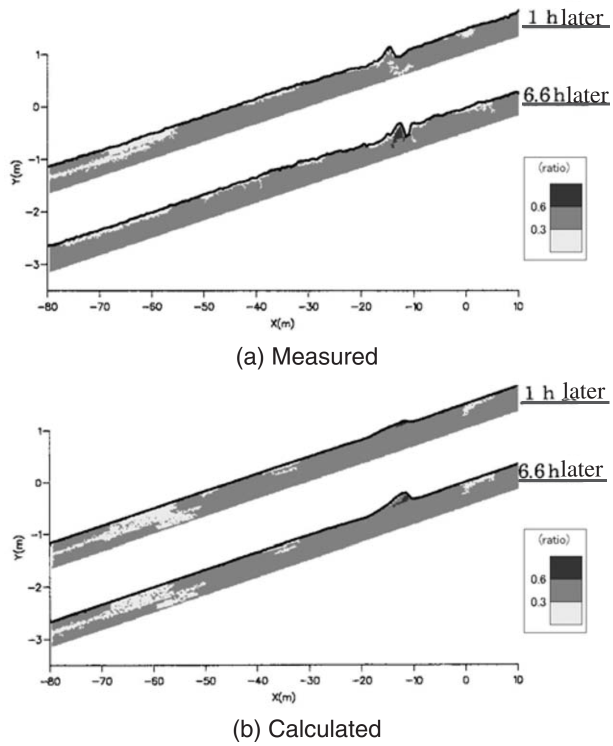

Fig. 19. Comparison between measured and calculated cross-shore bottom profile and ratio of coarse and fine sands.

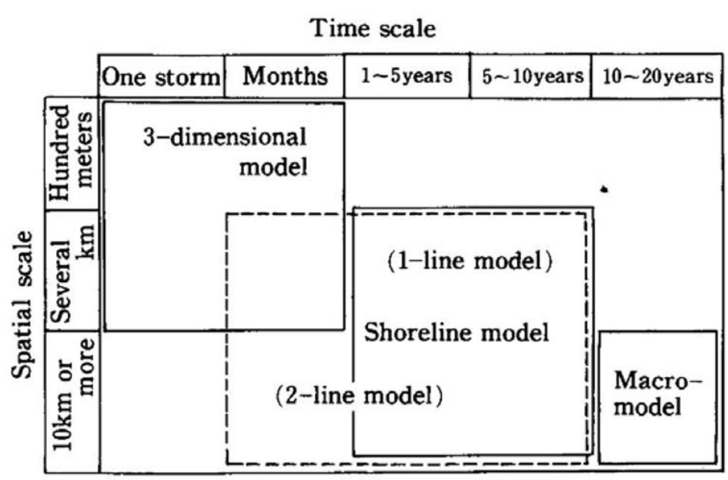

Fig. 20. Application ranges of beach evolution predictive models (Horikawa, 1988).

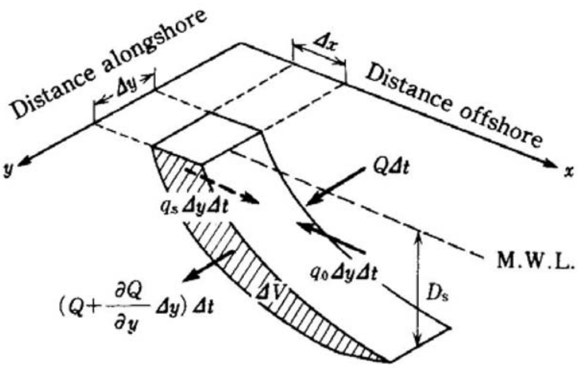

Fig. 21. Basic concept of the shoreline predictive model (Horikawa, 1988).

following expression proposed by Hallermeier (1983)[29] was recommended to use:

$$D_s = (2.28 - 10.9\frac{H_o}{L_o})H_o \quad [8]$$

As for the longshore sediment transport rate Q, since Bagnold (1963)[30] proposed a kind of power model as shown in the following

$$I = K'(EC_g)_B\frac{V}{u_m} \quad [9]$$

where I is the immersed weight longshore sediment transport rate and defined by the next equation

$$I = (\rho_s - \rho)\boldsymbol{g}(1 - \lambda_V)Q \quad [10]$$

where ρ_s and ρ are densities of sediment and water, respectively, $\boldsymbol{g}$ acceleration of gravity, λ_V the ratio of sediment voids to the bulk volume of sediment. While in Eq. [9], $(EC\boldsymbol{g})_B$ is the incident wave energy flux evaluated at the breaker line, V the representative value of the longshore current velocity, such as the mean current velocity, u_m the maximum value of the horizontal wave orbital velocity at the breaker line, and K' is a dimensionless empirical coefficient. Figure 22 shows the correlation between measured longshore sediment transport and energy-based predictive formula. The data plotted in this figure were obtained in USA and Japan. That is to say, the data at El Moreno Beach and Silver Stand Beach were observed in USA, while the other data were obtained by our research group in Japan.[31),32)]

However, Ozasa and Brampton (1980)[33] presented the following expression for the longshore sediment transport rate affected by a coastal structure.

$$I = \left(EC_g\right)_B\left(K_1 \sin\alpha_B \cos\alpha_B - \frac{K_2}{\tan\beta}\cos\alpha_B\frac{\partial H_B}{\partial y}\right) \quad [11]$$

where K_1 and K_2 are empirical coefficients, α_B the incident wave angle with respect to the shoreline at the breaking point, and y the alongshore axis. The first term is the contribution from oblique wave incidence and the second term is the contribution from alongshore variation in breaking wave height induced particularly by the existence of a coastal structure such as a breakwater.

The present model was applied to the shoreline change induced by the construction of a jetty. The shoreline change was surveyed for about two years. Figure 23 shows the comparison between the measured final shoreline and the calculated one staring from the measured initial shoreline.[34] This model has been utilized at numerous location for the practical purposes.

Three dimensional model. This model is to predict the bottom configuration in the nearshore area by the way of a rational method. The process of computation is as follows: First of all, nearshore wave field in the interested area is calculated on the basis of an appropriate wave model such as the mild slope equation presented by Berkhoff (1972),[35] Watanabe and Maruyama (1984)[36] and Isobe (1986, 1994).[37),38)] Then the calculation of nearshore current velocity will be carried out on the basis of the governing equations given by Phillips (1977).[39] In order to proceed this procedure appropriately, the evaluation of friction and lateral mixing terms were of extremely importance. Hence a great effort was devoted to get valuable information on these terms by doing laboratory experiments as well as field observations. Particularly the observed nearshore current velocity fields as in Fig. 17 were compared with the numerical simulation results. This kind of laborious work was essential to select the stated terms appropriately. The next step is how to estimate the local sediment transport rate induced by waves and nearshore current. Watanabe *et al.* (1984)[40] proposed the expression for evaluating the local flux of sediment transport induced by the coexistence of waves and current in referring to the results presented by foregoing researchers. In these expressions several unknown coefficients were introduced. Therefore, these coefficients should be selected appropriately to match the calculated results with the experimental or, if possible, field observation results.

The change in local bottom elevation, z_b, or water depth, h, can readily be computed, once the spatial distribution of sediment transport rates, q_x and q_y, are given,

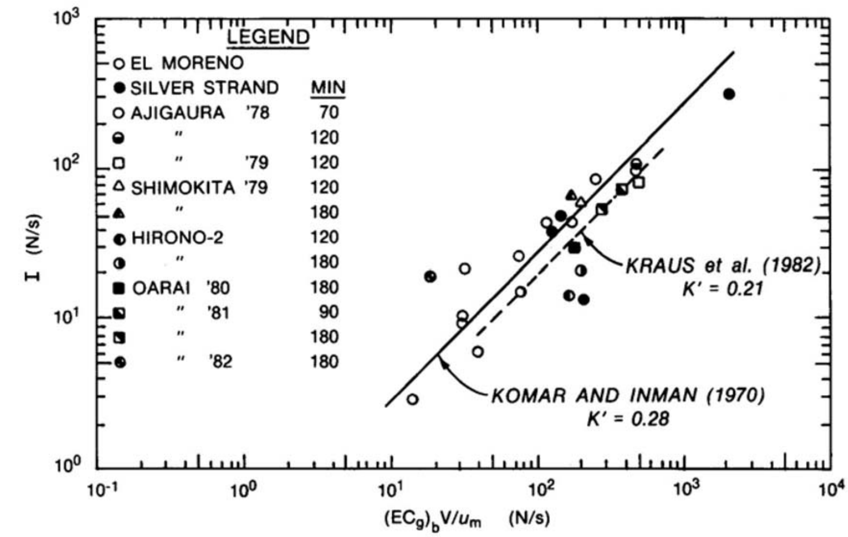

Fig. 22. Measured total immersed-weight sand transport rate versus the theoretical predictive equation of Inman and Bagnold (1963) (Kraus *et al.*, 1982).

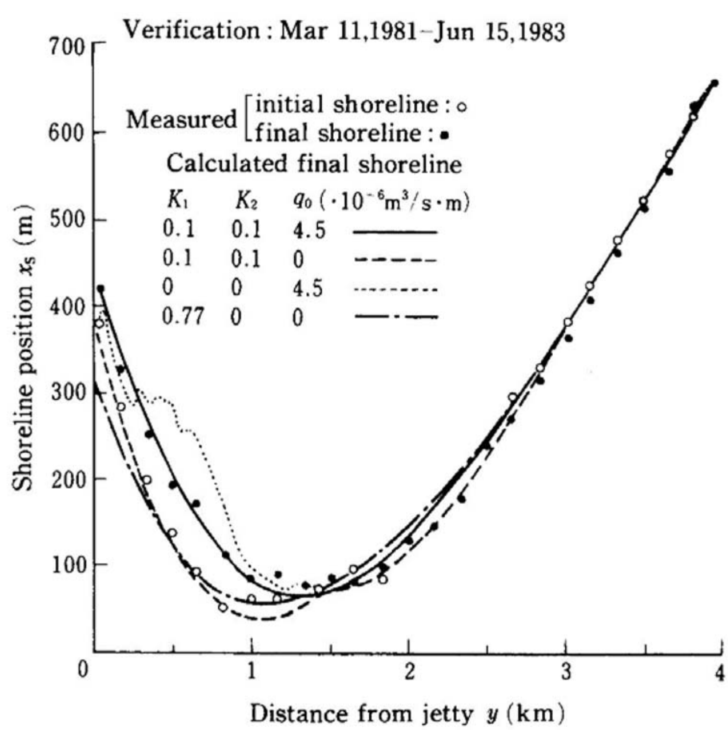

Fig. 23. Measured and calculated shoreline change for the verification interval (Kraus *et al.*, 1984).

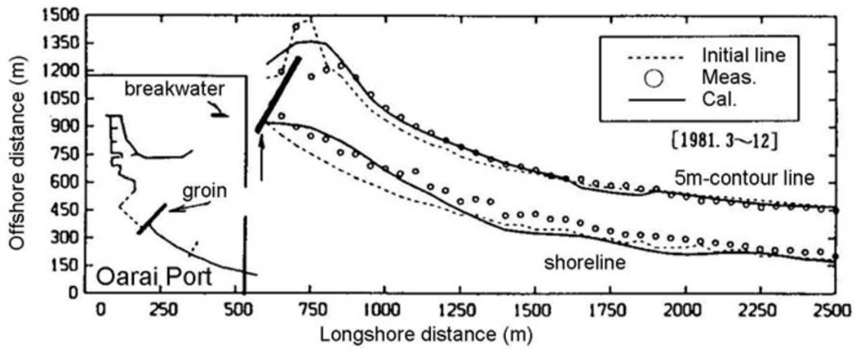

Fig. 24. Comparison of measured and calculated shoreline and 5 m-contour line.

by solving the conservation equation for sediment mass:

$$\frac{\partial z_b}{\partial t} = -\frac{\partial h}{\partial t} = -\frac{\partial q_x}{\partial t} - \frac{\partial q_y}{\partial_y} \qquad [12]$$

The present model was applied to the laboratory investigation on the bottom topography change induced by an offshore detached breakwater set in parallel to the original shoreline in a wave basin. Selecting appropriate values of unknown coefficients, the predicted bottom topography was quite similar to that in the wave basin. Since then at several real coastal sites the 3-D model was applied to verify the applicability to the real beaches. As an example Fig. 24 will be presented here. This field survey was conducted at Oarai Harbor, where the shoreline and the 5 m depth contourline were picked up from the hydrographic survey map. The dotted lines are the initial condition in March 1981. The solid lines are computed by using the model stated above and the circles indicate the measured locations of the shoreline as well as the 5 m water depth contour. There are some irregularities particularly at the shoreline, which might be caused by the tidal fluctuation. However, the calculated result is satisfactory for our purpose.

Conclusions and recommendation for future study. As stated in Introduction, beach erosion has been recognized as one of the most serious problems since the beginning of the coastal engineering activities.

In 1954 when Horikawa initiated his coastal engineering research works, our knowledge on coastal sediment was extremely limited due to the difficulty in measuring the physical quantities such as wave characteristics, nearshore currents, and sediment transport rate particularly in field. However, the achievement of our study was so remarkable as stated in this review article, that the beach evolution can be predicted appropriately by either the shoreline model or the three dimensional model. Even though these models are now effective enough to be applied to the practical problems, it is desirable to improve more of these models by accumulating practical experience. In particular, effort to fully understand the sediment movement mechanism is indispensable. The fluid-sediment two-phase flow model will be a promising tool for this purpose. More accurate prediction of wave and current field taking into account the nonlinearity and randomness of phenomena will improve the model. Finally, behavior of suspended sediment near the wave breaking point and sediment in the swash zone will be the keys to deepen our understanding of the beach process and to improve the modeling.

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(Received Aug. 30, 2005; accepted Nov. 14, 2005)

Profile

Kiyoshi Horikawa was born in 1927 in Tokyo. He graduated from the Faculty of Engineering at the University of Tokyo in 1952 and continued his study at the Graduate School until July 1954, when he was appointed Assistant Professor at the University of Tokyo. Since then to the present he has engaged intensively in the research activities related to the coastal engineering. This specified discipline was newly established as a branch of civil engineering in 1950 by the leadership of Dean M. P. O'Brien at the University of California, Berkeley. In 1953, the Typhoon No. 13 attacked the Japanese Island producing severe storm surge damage along the coast of Ise Bay. This unexpected accident became a trigger to introduce the coastal engineering research activity to Japan in order to reduce the natural hazard caused by waves, typhoons and tsunamis. During the last fifty years, he devoted himself to systematize this new engineering field in Japan. His research interest has a variety of subjects, such as nearshore waves, nearshore currents, storm surges, tsunamis, coastal sediment movement, wave action on coastal structures, and coastal environment. Among these subjects, his major interest has been in the coastal sediment movement. He was promoted to Professor at the University of Tokyo in 1967. As a result of his distinguished contribution to the advancement of coastal sediment mechanism, he received numerous awards, such as the JSCE* Award in 1969, the International Coastal Engineering Award of ASCE** in 1981, the Purple Ribbon Medal in 1993, and the Japan Academy Prize in 1997. He was also praised to the Person of Cultural Merits in 1999. He is Honorary Member of JSCE since 1993 and Honorary Member of ASCE since 1997. He was Dean, Faculty of Engineering at the University of Tokyo (1984 to 1986), President of JSCE (1989 to 1990), President of Saitama University (1992 to 1998), and President of Musashi Institute of Technology (1998 to 2004). Now he is Professor Emeritus at the University of Tokyo since 1988, at Saitama University since 1998, and at Musashi Institute of Technology since 2004.

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* JSCE: Japan Society of Civil Engineers
** ASCE: American Society of Civil Engineers

Profile

Masahiko Isobe was born in 1952 and graduated from the Faculty of Engineering at the University of Tokyo, specializing in the civil engineering. He continued his study at the Graduate School of Engineering and obtained master's and doctoral degrees in the field of coastal engineering under the supervision of Professor Kiyoshi Horikawa. He began his professional career as Research Associate in 1978 at the University of Tokyo. Since then he has experienced Assistant Professor and Associate Professor at the Yokohama National University, and Associate Professor and Professor at the University of Tokyo. At present, he is serving as the Dean of the Graduate School of Frontier Sciences. His contribution in research is mainly on the wave hydrodynamics and coastal environment. He has been working on the nonlinear wave dynamics in the nearshore area and water quality problems in the enclosed sea through physical and biological processes. As a result of his achievements, he was awarded annual incentive prize and annual research award of the Japan Society of Civil Engineers in 1986 and 1997, respectively.