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NUMERICAL STUDY ON BEACH PROFILE EVOLUTION DUE TO RANDOM WAVES

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ABSTRACT

A numerical model for beach profile evolution due to random waves has been developed. To incident the actual random wave condition, the Goda model (1975,1983) is employed as the wave sub-model, since the model can predict any representative wave height distribution across the shore and includes the effect of surf beat. Then, a cross-shore sediment transport model which is related to the wave energy flux is coupled with a mass conservation equation to obtain a new beach profile. The present model is more capable of predicting a milder beach profile than regular wave condition. Finally, the model is applied to simulate dune-beach erosion under severe storm condition.

INTRODUCTION

The waves generated by winds propagate for a distance from the ocean and approach a beach. This random sea is a dominant force to the beach profile evolution and many other coastal engineering subjects. As the water depth becomes shallow, the orbital motion of the water particle commences to have an influence on the sea bottom material. Then, waves tend to dissipate energy by friction, release most of the energy by breaking and cause a change in a beach profile. Through these physical processes, a wave height distribution across the shore may change to a non-Rayleigh distribution.

In the last few decades, numerous studies have been done on wave transformation and breaking waves. Therefore, wave height distributions for regular waves and random waves can be numerically predicted to some degree, to date. In contrast to a number of wave models, practical engineering methods for beach profile evolution due to random waves are still a few, for instance Stive and Battjes (1984), Briand and Kamphuis (1990), and Sato and Mitsunobu (1991). In fact, for convenience and engineering usefulness, regular wave conditions are widely accepted for physical modeling as well as numerical experiment for cross-shore sediment transport. However, the mild transformation of random waves may result in the mild bar profile compared with the distinct bar-trough feature due to regular waves. On the other hand due to the technology innovation, new powerful and faster desk-top PC machines are now available to most of the coastal engineers and scientists. In addition, some large scale wave tank data on beach profile evolution due to random waves (e.g. Kajima et al 1982, Dette and Uliczka 1986, and Kraus et al 1992) is available, too. Therefore, it is considered that the numerical simulation of beach profile evolution due to random waves is not such a time-consuming job and not an unrealistic approach, but will become a practical method even if a little bit complicated.

To simulate actual conditions in terms of incident waves, the Goda model is employed as the wave sub-model in the model, since his model predicts information on several kinds of representative wave height distributions, for instance: maximum wave, significant wave, and mean wave height distributions. This is an advantage when compared to the Battjes and Janssen (1978), Thornton and Guza (1983) models. It is emphasized that the purpose of this study is the development of numerical model to predict mainly the storm beach profile, especially macro-scale features due to random seas. The transport model is based on a kind of Dean's sediment transport model in which the transport rate is a function of the wave energy dissipation.

NUMERICAL MODEL

The present model consists of three sub-models as well as other 2-D models i.e.: (1) the wave model, (2) the sediment transport model, and (3) the mass conservation equation for bed materials. Regarding the wave model, there are some numerical ways to take account the randomness of incident waves into the simulation. The first method may be the decomposed method, in which random waves are decomposed into individual regular waves by a zero cross-up method or FFT, and the individual regular wave condition is applied to a sediment transport formula. The second method is to use a probablistic wave height deformation, such as proposed by Goda (1975, 1983), Battjes and Janssen (1978), Thornton and Guza (1983), and Larson and Kraus (1992). In fact, the Larson and Kraus method (1992) can be applied to the first method, as well.

In this study, Goda's method is chosen as the wave model, since the wave height distributions of any kind of representative waves across the shore, e.g., maximum wave, significant wave, and mean wave heights are available, as well as the probability density function. In addition, the model includes the wave nonlinearlity and surf beat effect.

Following the wave field, the cross-shore sediment transport rate is computed based on the energy flux dissipation per unit water volume originally proposed by Dean (1977) and Moore (1982), since the sediment transport in the surf zone shows a good correlation with the energy flux dissipation (Larson and Kraus, 1989). As can be expected, for the engineering applicability, the model structure is quite simple, so that the user of the program can easily improve the model. It is noted that the above mentioned wave model does not directly consider the contribution of randomness of the incident random seas into the micro-mechanics of the sediment movement and resulting transport rate, yet.

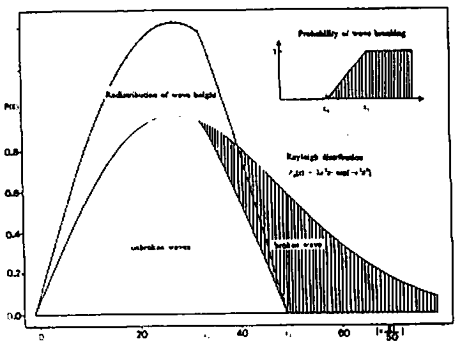

Fig. 1 Probability of wave height distribution after and before breaking

RANDOM WAVE MODEL

Before the development of a complete three dimensional model, the plane sloping beach with straight parallel contours and the normal incident wave angle are assumed. Therefore, wave diffraction and refraction as well as reflection are not taken into account in the current model. To apply the model to a non-uniform beach profile, the modified Goda method by Sato and Kobe (1983) is employed here in the wave transformation model.

(a) Probability distribution of wave height proposed by Goda (1975, 1983)

Goda (1975,1983) proposed a transformation and deformation model for the random seas, in which the wave height distribution of random waves in deep water is of a Rayleigh distribution as shown in Fig. 1. Once breaking commences in the surf zone, it is assumed that a wave which is greater than breaking criteria, $H_b = x\ H.$ breaks as the wave propagates to the shallow water in the same way as regular wave deformation. It is also assumed that the probability of breaking waves changes linearly in the range (x_2, x_1) as shown in Fig.1. As a result, the probability of unbroken waves is shown by a solid line. In this process, the energy of a breaking wave in the range from x_2 to x_∞ will not disappear, and is redistributed to small wave reformation. In the reformation process, it is assumed that the probability of breaking waves is distributed proportionally to the probability of unbroken waves. So, the distribution of wave heights in deep water and unbroken waves, and redistribution of wave heights are formulated as follows:

(i) In deep water:

$$P_0(x) = 2a^2x \cdot \exp\left[-a^2x^2\right] \tag{1}$$

where, $x=H/H_*$, $a=H_*/H_{rms}$, $P_0(x)$ = Rayleigh distribution, x= normalized wave height, and the reference wave height H_* is set to be the significant wave height in this study so that the value of a is 1.416.

(ii) The unbroken wave probability is given as follows:

$$\begin{aligned} p_r(x) &= P_0(x) \quad ; \quad x \le x_2 \\ &= P_0(x) - \frac{x-x_2}{x_1-x_2}P_0(x) \quad ; \quad x_2 < x \le x_1 \\ &= 0 \quad ; \quad x_1 < x \end{aligned} \tag{2}$$

(iii) The reformed wave probability is written as follows:

$$P(x) = \alpha \cdot P_r(x) \quad ; \quad 1/\alpha = \int_0^{x_1} P_r(x)dx \tag{3}$$

(b) Wave breaking criterion

The wave breaking criterion is necessary to calculate the random wave breaking, so Goda (1975) proposed an empirical criterion which includes the effect of local bottom slope and wave conditions. Because of his slope term in the formula, Goda's breaking criteria is restricted to calculations of the random wave transformation on positive sloping beaches. Sato and Kobe (1983) modified the Goda model to allow us to compute the wave transformation on an arbitrary beach profile. They set the slope term in the breaking limit to the absolute value of the local beach slope as follows:

$$x_b = \frac{H_b}{H_0} = \frac{A}{H_0/L_0}\left(1 - \exp[-1.5\frac{\pi h}{H_0}\frac{H_0}{L_0}(1+\kappa|\tan\theta|^{4/3})]\right) \tag{4}$$

where, $\kappa=15$, $L_0=gT^2/2\pi$, and the coefficient A takes the value 0.17 for regular waves. In the random wave breaking, A is set to 0.18 for the upper breaking limit of random waves at $x=x_1$ and 0.12 for the lower breaking limit $x=x_2$.

(c) Shoaling coefficient

The shoaling coefficient is also necessary to calculate the wave transformation. Since the shoaling coefficient becomes larger than the value derived from the above small amplitude assumption in the natural condition, the shoaling coefficient for finite amplitude waves proposed by Shuto (1974) is employed in the model. The shoaling coefficient is calculated as follows

$$k_s = \frac{H}{H_0} = [\ (1+\frac{2kh}{sinh2kh}) \cdot tanhkh]^{-1/2} \quad ; \quad \frac{gHT^2}{h^2} \leq 30$$

$$Hh^{2/7} = Constant \quad ; \quad 30 \leq \frac{gHT^2}{h^2} \leq 50 \tag{5}$$

$$Hh^{5/2}[\sqrt{\frac{gHT^2}{h^2}} - 2\sqrt{3}] = Constant \quad ; \quad 50 \leq \frac{gHT^2}{h^2}$$

where, ks is the shoaling coefficient, $k=2\pi/L$ is a wave number, H_0 is the equivalent deep water wave height, g= gravitational acceleration, and gHT/h is the Ursell parameter in a shallow water.

CROSS-SHORE SEDIMENT TRANSPORT AND CONTINUITY EQUATION

Based on modified Goda model, it is possible to obtain any representative wave height distribution, namely, root-mean-square wave height, significant wave height, 1/10 maximum wave height, and maximum wave height. While the p.d.f is restricted to the Rayleigh distribution in deep water, each probablistic wave height is related as follows:

$$\begin{aligned} H_{1/10} &= 1.80H_{rms} \\ H_{1/3} &= 1.416H_{rms} \\ H_1 &= 0.886H_{rms} \end{aligned} \tag{6}$$

However, once the breaking occurs, the wave height is redistributed as shown by $p_r(x)$, then the above relationship is not assured, because the pdf is not certified to be a type of Rayleigh distribution inside the surf zone. So, it is now necessary to select one representative wave to input the sediment transport model. Regarding the representative wave height for beach profile evolution, Otsuka et al (1984) suggests based on their small scale experiment that the mean beach profile has a correlation with mean wave height (H_m) which presents the energy of incident random waves, while the critical water depth for the sediment movement and dimensions of sand ripples have correlation to the significant wave height. Vellinga (1986) used the significant wave height in his dune erosion profile model. Nishi et al. (1990) also conducted a series of random wave experiments to study a equivalent wave height between regular wave and random wave conditions. The results showed that a mean beach profile due to regular waves and due to random waves were quite different, and the critical water depth for depth change can be correlated by a significant wave parameter. Regarding field data, Kraus et al. (1991) used significant wave height and spectral peak wave period into their beach profile predictor. So, it seems that we need more knowledge to define a representative wave height which can be used in a sediment transport model.

In the current model, the significant wave height is used for the models, since the results of the significant wave gives us more erosion than that of the mean wave. It is more safer for engineering application. As described by Larson and Kraus (1989), wave energy flux dissipation per unit volume of water ($1/h \partial F/\partial h$) shows a good correlation with a net sediment transport rate in the surf zone. So, we have used Dean's type of sediment transport (1977) in which the sediment transport is a function of energy flux. Later, this sediment transport model was modified by Larson and Kraus to extend to the bar-trough type beach profile. The sediment transport rate in the surf zone is computed as follows:

$$q = K[D - D_{eq} + \frac{\epsilon}{K}\frac{\partial h}{\partial x}] \quad for\ D > [D_{eq} - \frac{\epsilon}{K}\frac{\partial h}{\partial x}]$$
$$q = 0 \quad for\ D \leq [D_{eq} - \frac{\epsilon}{K}\frac{\partial h}{\partial x}] \tag{7}$$

where, K is the transport coefficient set to be $1.1 * 10^{-6}$ m^4/s, which is approximately half the value originally obtained by Moore (1982). The ϵ is the transport coefficient for slope term.

CONTINUITY EQUATION FOR BED MATERIAL

The wave field and the corresponding net sediment transport rate are obtained, so the change in beach profile can be calculated at each time step based on the continuity equation for bed material. The continuity equation is written as follows:

$$\frac{\partial q}{\partial x} = \frac{\partial h}{\partial t} \tag{8}$$

NUMERICAL RESULTS

Simple method to simulate the beach profile evolution due to random waves has been described in previous sections. Based on these mathematical formula, the numerical computations have been carried out for random wave and regular wave incidence. The wave height distribution for regular wave are shown in Fig. 2 (a) and (b). The point of maximum wave height in cross-shore wave height distribution is called peak point of wave height distribution in contrast to the breaking point for regular wave in this study, because the breaking occurs through the surf zone for random wave incidence on the plane beach, and only the probability of breaking waves can be define able for random waves in contrast to the single or multiple breaking points for regular wave incidence. In fact, the regular wave has a clear breaking point, whereas the random wave does not have a certain breaking point. It is shown that the random wave height changes gradually compared to the regular wave, especially around the peak point of wave height distribution.

The corresponding sediment transport rates are shown in Fig. 3 (a) and (b) respectively. As seen in Fig. 3, the sediment transport rate due to random wave show a wide distribution, whereas the sediment transport rate due to regular wave shows a narrow range distribution.

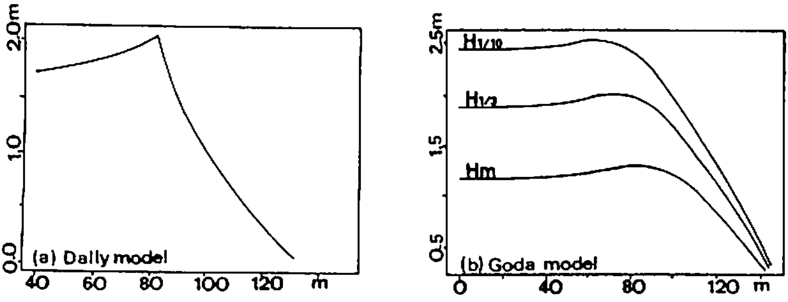

Fig. 2 Wave height distributions

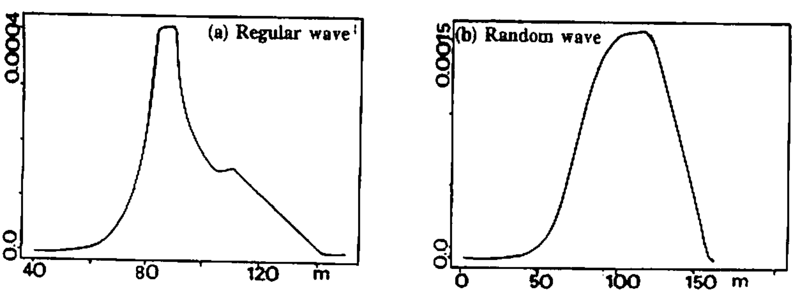

Fig. 3 Sediment transport rates

A beach profile evolution due to random waves is shown in Fig. 4. The incident wave height is 2.0 m (significant wave height) and wave period is 6.0 sec, and the duration of wave action is 10.0 hours for this case. This wave condition is expected to cause an offshore sediment transport based on the beach profile predictor by Kraus et al. (1991). As can be seen in the figure, the storm waves erode the beach and cause a shoreline recession. The eroded sand is transported offshore and deposited across the shore, so the shape of sand deposition is similar to a terrace bar. The beach profile from the shoreline to offshore is flat as in the data obtained from a large wave tank by Dette (1991). The beach slope from the peak point of the wave height distribution to offshore is steeper than the beach slope to onshore. It is also noted that the rate of beach profile change is decreasing as the duration of wave action is increasing. However, the trough feature is not reproduced in this case. As an application to the dune erosion shown in Photo. 1, the current model was run to the wave condition of H_{sig}=2.5m, T=6.0sec. The numerical result is shown in Fig. 5.

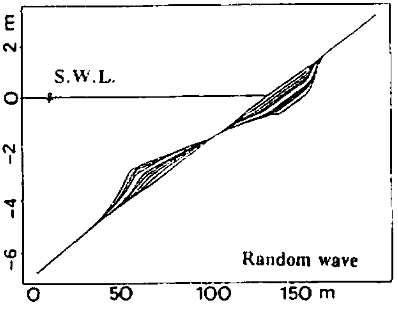

Fig. 4 Beach profile evolution due to random wave

Photo. 1 Dune beach erosion at North Sea

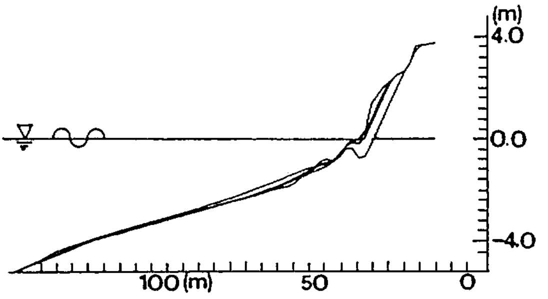

Fig. 5 Dune-beach erosion due to random waves

CONCLUDING REMARKS AND FURTHER STUDY

The proposed model which is simple and easy to apply to engineering projects generates a milder beach profile compared to that due to regular wave conditions and qualitatively agrees with the laboratory data by Dette, however, distinct bar trough features can not be reproduced. The numerical results show that the model is also applicable to dune erosion, but we need more prototype scale data to verify and calibrate the model.

It is noted that the sediment transport does not directly relate to the randomness of each incident wave in this report, since the method is based on the average and mean sediment transport rate longer than the incident wave periods. In other words, the sediment transport rate is a function of averaged energy dissipation in this study. So, the randomness of precedent and antecedent waves can be taken into account to improve the prediction. For instance, the randomness of incident waves can be explained by a time Series of Integrated Wave Energy History (SIWEH) in a macro-scopic sense and can be used in the model.

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