CHAPTER 68

BEACH PROFILES AND ON-OFFSHORE SEDIMENT TRANSPORT

by

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ABSTRACT

The on-offshore sediment transport due to waves on a sloping beach is studied by analyzing the laboratory test data on two-dimensional beach deformation. The net rates of sediment transport both inside and outside the breaker zone are evaluated from beach profile changes and are related to the nondimensional bottom shear stress or the Shields parameter. The importance of the critical shear stress and of asymmetrical to-and-fro water partical motion near the bottom is pointed out.

INTRODUCTION

Much effort has been devoted during the last few decades to elucidate sediment movement phenomenon in coastal zone, but it was not so long since intensive attempts were started to set up numerical models simulating beach processes. Prediction of beach transformation requires knowledge about the functional relationship of sediment transport rate to conditions of waves, currents, water depth, bottom configuration, sediment property and so on. There have been many, almost innumerable, studies on sediment transport rate, and considerable amount of knowledge on this problem has been accumulated so far. Yet it seems that we are still far from a reliable formula estimating the rate of sediment transport in particular perpendicular to coastline. This directional mode of sediment movements, or on-offshore sediment transport plays a vital role in the short-term beach deformation such as severe beach erosion due to storm surge. This may be also quite essential for more detailed understanding of longshore sediment transport, because the latter is expected to correlate with the former through the sediment sorting and entraining mechanics.

In the present study the on-offshore sediment transport was dealt with by analyzing the data obtained in laboratory experiments. In order to attain our purpose of getting the on-offshore transport rate formula of practical use in predicting beach deformation inside and outside the breaker zone, experiments were conducted under sloping beach condition. Two-dimensional beach deformations under various wave conditions were measured. Any equipment, such as a sand trap, to measure directly the sediment transport rate was not used in this test, because such apparatus would cause severe disturbance to the fluid motion and the sediment movement. Instead the net rates of sediment transport were estimated from the beach deformation data. Attempts to find the relationship between the net transport rates and wave conditions were then made.

EXPERIMENTAL PROCEDURES

Experiments were conducted in a two-dimensional wave flume, 25 m long, 1.5 m deep and 0.8 m wide. As mentioned above, the net rate of on-offshore sediment transport was evaluated from the beach profile changes. In order to make simple the experiments, data handling and interpretation of results, tests were limited to a uniform initial slope and to the wave action of one-hour for each wave condition, while wide range of wave conditions was adopted.

Table 1 is a list of experimental cases. Two kinds of quartz sand with mean diameters of 0.2 mm and 0.7 mm were used as bed materials, both being well sorted with the Trask sorting coefficient of 1.1. Combination of these two grain sizes, two initial slopes of 1/10 and 1/20, three wave periods of 1.0, 1.5 and 2.0 seconds, and four or five wave heights made fifty-eight experimental cases altogether.

In each experimental case, a beach with prescribed uniform slope, 1/10 or 1/20, was initially set up, and its deformed profile after one-hour wave action was measured. Measurement of beach profiles in this experiment was done by tracing them on transparent sheets attatched on a side glass wall of the flume, although a bottom profiler of electrical-resistance type, by which continuous profile data can be recorded on a magnetic tape, has become available in the latest experiment. In the middle of wave action, wave profiles were measured by capacitance type wave gages at locations of 10 to 50 cm intervals above the beach as well as at the offshore end of the flume.

EXPERIMENTAL RESULTS

Beach Profile Change

Figures 1 (a) to (d) show the beach profiles after one-hour wave action for all the cases listed in Table 1. It should be noted that profile changes in some cases with the initial slope of 1/20 and large wave heights stretch beyond the extent of these figures. The locations of breaking points are indicated by triangle symbols; open triangles for the initial stage and solid triangles for the final stage.

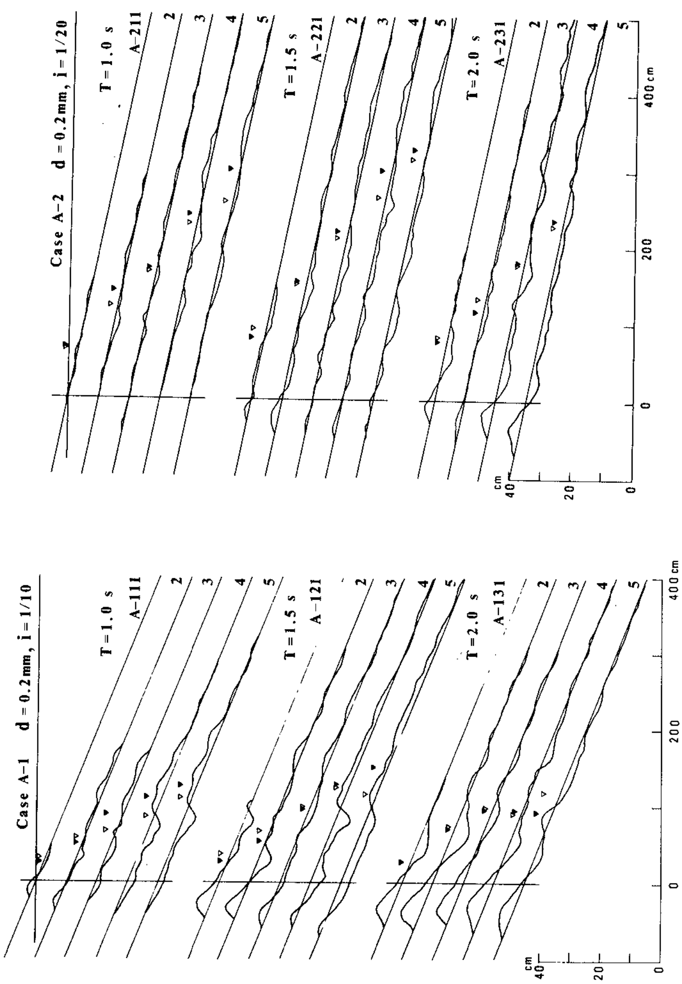
Table 1 Test cases and condition.

Case A-1	$d = 0.2$ mm		$i = 1/10$	Case B-1	$d = 0.2$ mm		$i = 1/10$
Case No.	T (sec)	H_0 (cm)	H_0/L_0 ($\times 10^{-2}$)	Case No.	T (sec)	H_0 (cm)	H_0/L_0 ($\times 10^{-2}$)
A-111		2.6	1.7	B-111		2.6	1.7
2		5.1	3.3	2		5.0	3.2
3	1.0	6.9	4.4	3	1.0	6.4	4.1
4		8.9	5.7	4		8.5	5.4
5		10.5	6.7	5		10.0	6.4
A-121		3.2	0.9	B-121		3.0	0.8
2		5.8	1.7	2		5.3	1.5
3	1.5	8.3	2.4	3	1.5	7.7	2.2
4		10.5	3.0	4		10.5	3.0
5		12.1	3.5	5		11.3	3.2
A-131		3.1	0.5	B-131		3.5	0.6
2		5.4	0.9	2		5.3	0.9
3	2.0	6.1	1.0	3	2.0	6.7	1.1
4		7.5	1.2	4		8.2	1.3
5		9.2	1.5	5		9.6	1.5

Case A-2	$d = 0.2$ mm		$i = 1/20$	Case B-2	$d = 0.2$ mm		$i = 1/20$
Case No.	T (sec)	H_0 (cm)	H_0/L_0 ($\times 10^{-2}$)	Case No.	T (sec)	H_0 (cm)	H_0/L_0 ($\times 10^{-2}$)
A-211		2.9	1.9	B-211		2.9	1.9
2		5.2	3.4	2		5.1	3.3
3	1.0	7.3	4.7	3	1.0	7.0	4.5
4		9.6	6.2	4		8.9	5.7
5		11.5	7.3	5		10.6	6.8
A-221		3.1	0.9	B-221		3.2	0.9
2		5.7	1.6	2		5.4	1.5
3	1.5	7.9	2.3	3	1.5	7.7	2.2
4		10.2	2.9	4		9.7	2.8
5		11.7	3.3	5		12.1	3.5
A-231		2.7	0.4	B-231		2.7	0.4
2		4.0	0.6	2		4.3	0.7
3	2.0	6.2	1.0	3	2.0	6.1	1.0
4		8.2	1.3	4		7.8	1.3

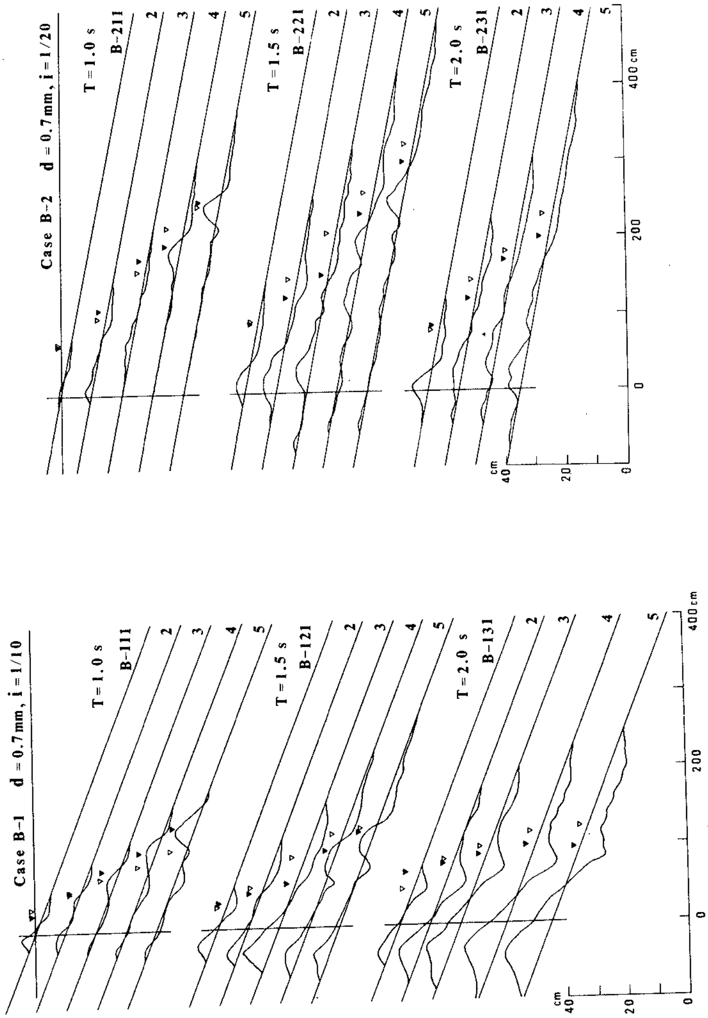
d : grain diameter, i : initial beach slope, T : wave period

H_0 : deep water wave height, H_0/L_0 : deep water wave steepness



(a) (b)

Fig. 1 Beach profiles after one-hour wave action.



(c)

(d)

Fig. 1 Beach profiles after one-hour wave action (continued).

It is recognized by refering to Table 1 that the bar-type profiles are formed for relatively large wave steepness conditions, showing systematic increase in bar size with the increase of wave steepness. Under the condition of smaller wave steepness, a wide berm becomes noticeable in particular for the cases with initial slope of 1/10, implying the shoreward transport of nearshore bed material. It was observed during the experiment that the shoreward limits of significant beach change in most cases were determined by the locations of maximum runup. However no remarkable nearshore beach change took place for the cases with initial slope of 1/20 and wave period of 1.0 sec, where flow intensity of uprush was quite weak.

In case of 0.2 mm-sand, sand ripples always appeared outside the breaker zone and the sediment transport seemed predominantly in the form of suspension. When the initial slope was 1/20, sand ripples were observed in the surf zone as well, but vortex formation around those ripples were not so remarkable as it was in the offshore region.

By contrast, in case of 0.7 mm-sand, sand ripples were scarcely observed under those experimental conditions. Only in a few cases such as B-133 to 135, sand ripples appeared and vortex formation was recognized, but amount of suspended sands was pretty small. Predominant mode of sediment transport of 0.7 mm-sand was in general in the form of bed load.

Evaluation of Net Sediment Transport Rate

The net rate of on-offshore sediment transport is evaluated from two-dimensional beach deformation data as follows. Suppose a beach profile change as given in Fig. 2. In virtue of the conservation of the bed material amount, the local net transport rate $q(x,t)$ per unit width is related to the bed elevation $z(x,t)$ measured from a certain datum as:

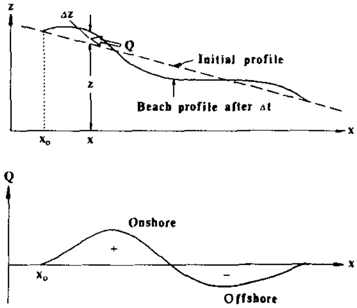


Fig. 2 Evaluation of net sediment transport rate.

$$\frac{\partial z}{\partial t} = \frac{1}{1 - \varepsilon}\frac{\partial q}{\partial x} \tag{1}$$

where ε is the sediment porosity. Therefore the effective net transport rate $Q(x)$ averaged over a time duration Δt of wave action is evaluated from the bed elevation change $\Delta z(x)$ during the same period by:

$$Q = \frac{\bar{q}}{1 - \varepsilon} = \int_{x_o}^{x} \frac{\Delta z}{\Delta t}\, dx \tag{2}$$

in which x is positively seaward and x_o means the shoreward limit of beach deformation. After such calculation, we shall obtain the spatial distribution of transport rate $Q(x)$ as shown in the bottom of the same figure.

Figures 3 (a) to (d) show spatial distributions of the effective net transport rate $Q(x)$ averaged over one-hour of wave action thus evaluated from beach profiles given in Fig. 1. Positive values of Q indicates the net transport towards the onshore direction, while negative Q means the offshore transport. The abscissa x is the horizontal distance from the initial shoreline. The locations of breaking point are again indicated by triangle symbols.

It is seen from Figs. 3 (a) and (b) that in case of 0.2 mm-sand, sediment transport towards the offshore direction is predominant in particular outside the surf zone. In few cases such as A-131 to 135 with the initial slope of 1/10 and relatively small wave steepness, the onshore net transport dominates. It is interesting to note that this onshore transport was mostly replaced by the seaward transport for the initial slope of 1/20 even under the similar deep water wave conditions. (Compare Cases A-133 and 134 with A-233 and 234, respectively.)

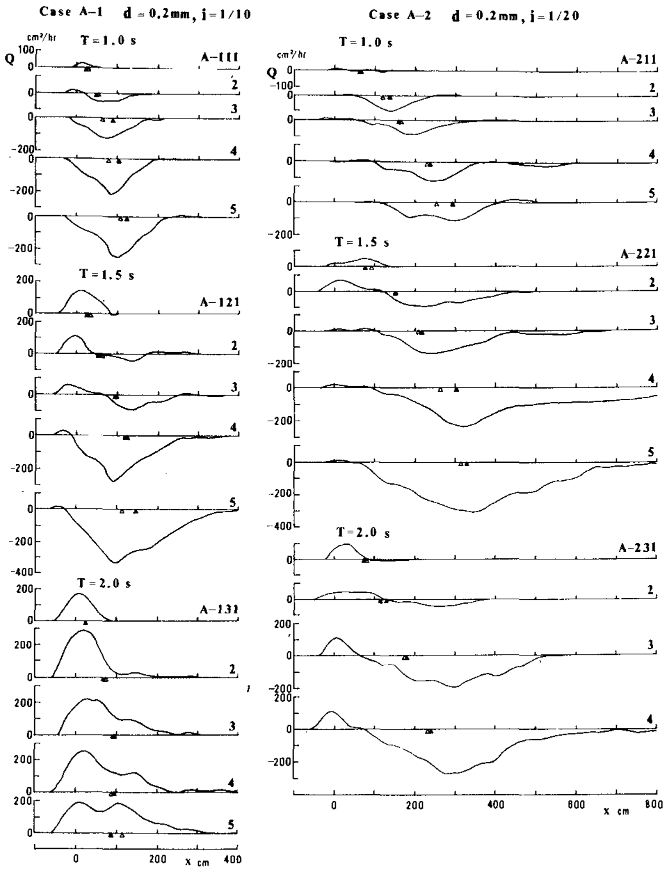
Figures 3 (c) and (d) for 0.7 mm-sand indicate that, in contrast to the former cases of 0.2 mm-sand, the net transport is mostly shoreward, with the exception of some cases such as B-115 where the wave steepness as well as beach slope are large. Predominant shoreward transport might be simply explained by the asymmetric to-and-fro water particle motion under large amplitude waves, since the coarse sands are transported essentially as bed load.

DISCUSSIONS

Beach Profile Changes

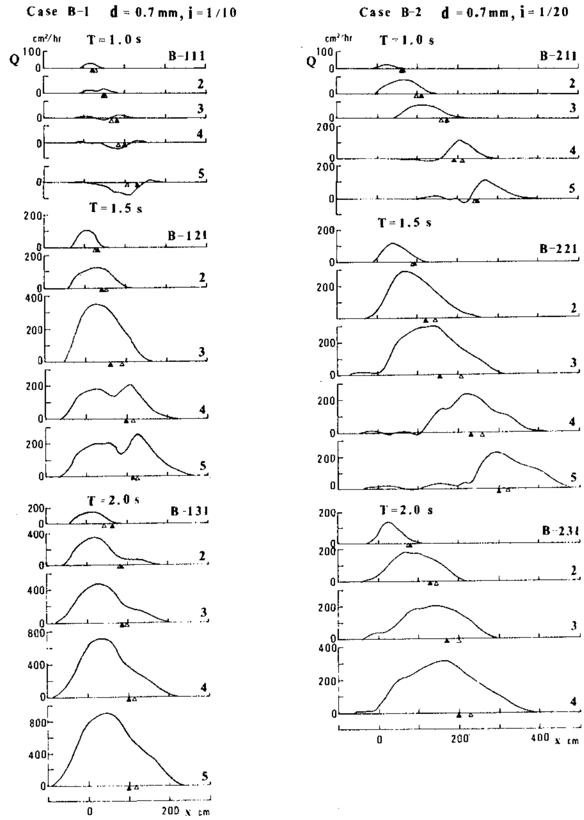
Two-dimensional beach transformation has been intensively studied by numerous investigators. There are many attempts to determine parameters classifying beach profiles, such as by Johnson (1949), Iwagaki and Noda (1962), Nayak (1970), Sunamura and Horikawa (1974), and Hattori and Kawamata (1980).

Sunamura and Horikawa (1974) have proposed a beach profile classification based on the displacement of topography from the initial beach slope: Type I = a shoreline retrogresses and sand accumulates in offshore



(a) (b)

Fig. 3 Distributions of effective net transport rates.



(c) (d)

Fig. 3 Distributions of effective net transport rates (continued).

zone, Type II = a shoreline advances and sand piles up offshore, and Type III = a shoreline progresses and no sand deposition takes place offshore. According to them, these three types are distinguished by a nondimensional parameter,

$$C = (H_o / L_o) (\tan \beta)^{0.27} (d / L_o)^{-0.67} \qquad (3)$$

as:

$$\text{Type I: } 8 < C, \quad \text{Type II: } 4 < C < 8, \quad \text{Type III: } C < 4 \qquad (4)$$

where H_o and L_o : deep water wave height and length, $i = \tan \beta$: initial beach slope, d : grain diameter. Critical values of C have been determined based on the experimental results on the equilibrium beach profiles after more than 160-hour wave action.

Application of this classification to the present data given in Fig. 1 yields the result shown in Fig. 4. In spite of that the present data are for quite short wave duration, they are distinguishable by the proposed values of the parameter C.

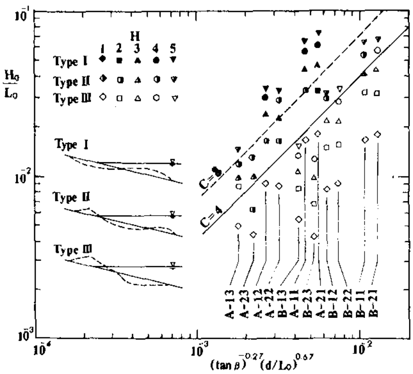


Fig. 4 Classification of beach profile changes.

Net Transport Direction of Suspended Sediment

As mentioned above, the predominant direction of net bed load transport is onshore, but that of the net transport of suspended load depends on conditions. For example, the predominant onshore transport in Cases A-133 and 134 are replaced by seaward transport in Cases A-233 and 234. One interpretation of the predominant directions of net suspended load in the present experiments will be given by Fig. 5.

Since the significant height of sediment suspension would be subject to the pitch of sand ripples, the abscissa $\lambda / w_o T$ was adopted as a param-

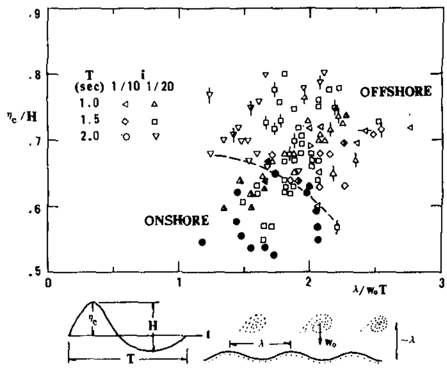


Fig. 5 Net transport direction of suspended sediment.

eter representing the excursion length of suspended sands, where λ is the ripple pitch, w_o the settling velocity of sands, and T the wave period. The ratio of the wave crest height η_c above the mean water level to local wave height H was chosen as another parameter in order to account for the effect of asymmetry of the to-and-fro water particle motion. Solid symbols mean the onshore transport, while open symbols the offshore transport. Figure 2 indicates that these two parameters are not always effective in classifying the sediment transport direction.

Inman and Tunstall (1972) have pointed out that the asymmetry of the sand ripple forms is important in determining the direction of the net sand transport. Although the present authors agree to their view, it should be mentioned that the asymmetry of ripple forms was not significant in the present experiment. The direction of sediment transport is indeed a very crucial problem and further intensive study on this point is definitely required.

Net Sediment Transport Rate and the Shields Parameter

Madsen and Grant (1976) have found that the so-called Shields parameter is effective in quantifying the fluid-sediment interaction in oscillatory flow as well as steady flow. They have proposed an empirical relationship of the sediment transport rate as:

$$\Phi' = 12.5\ \Psi^3 \qquad (5)$$

where $\Phi' = \overline{q'}/w_o d$: dimensionless transport rate, $\overline{q'}$: transport rate averaged over half a period of oscillatory flow, w_o: the settling velocity, d : the grain diameter, $\Psi = \tau_{o_m}/(\rho - \rho_s)gd$: the Shields parameter, ρ and ρ_s: the densities of fluid and sediment, respectively, and τ_{o_m} is the bottom shear stress amplitude defined by $\tau_{o_m} = f\rho u_{o_m}^2/2$ in which u_{o_m} is the maximum velocity at the bed and f the wave friction factor given by Jonsson

(1966). This formula is found to agree well with the experimental data for the bed load rates obtained by oscillating a plate containing sediment in still water. It should be mentioned that the quantity Φ' corresponds to the transport rate averaged over half a period, and that the net transport rate, which is directly related to beach deformation, must be evaluated as the difference between transport rates of onshore and offshore directions.

The dimensionless net transport rates $\Phi = \overline{q}/w_o d$ averaged over one-hour wave action were calculated from the present experimental data. The relations between Φ and Ψ are shown in Figs. 6 (a) to (d), where symbols are distinguished depending on the location; triangles are data obtained inside the surf zone, squares are at breaking point and circles are outside the breaker zone. Clear correlations between two parameters are recognized, though the overall scatter is not small enough.

It is interesting in particular that the relations of Φ and Ψ outside the breaker zone are almost identical, independently of bed slope, wave period and even of the transport direction. The values of Φ for a certain Ψ value inside the surf zone are mostly larger than outside; this should be investigated further by improving the estimation of bottom shear stress in the surf zone.

Now we shall consider only the data obtained outside the breaker zone. All of them are plotted together in Fig. 7 for 0.2 mm-sand and 0.7 mm-sand, respectively, distinguished by different symbols dependent on the transport direction and wave period. It is seen that the magnitude of Φ is somewhat larger for the offshore transport than for the onshore transport. Besides the larger wave period gives larger transport rate Φ.

The Madsen-Grant formula (Eq. 5) mentioned before is indicated by a broken straight line in each figure, and it seems by no means to fit well the experimental data. Since Eq. (5) corresponds to the transport rate averaged over half a period, it ought to overestimate the net rate which must be essentially the difference of the onshore and offshore transport rates. The tendency appearing in Fig.7 is contrary to this expectation except for small Ψ values.

Experimental data shows the tendency that the Φ value increases rapidly with the Ψ value while the latter is small, but its increase rate gradually diminishes as Ψ increases; namely, the relation of Φ and Ψ is convexly nonlinear in logarithmic scales. This trend implies the importance of critical shear stress for the onset of sediment movement. According to the study of Madsen and Grant (1976), the critical value of nondimensional shear stress or the Shields parameter Ψ_{cr} is about 0.07 for fine sands and 0.05 for coarse material. Those values are found to fit well with the critical conditions of the so-called initial movement reported by Horikawa and Watanabe (1967). We shall find from the result of Horikawa and Watanabe (1967) that the critical Shields parameter for the onset of the general movement is to be around 0.11 for fine sands and 0.08 for coarse sands. (The terms, *coarse* and *fine*, are in fact not so appropriate because of their vague meaning. The terminology,

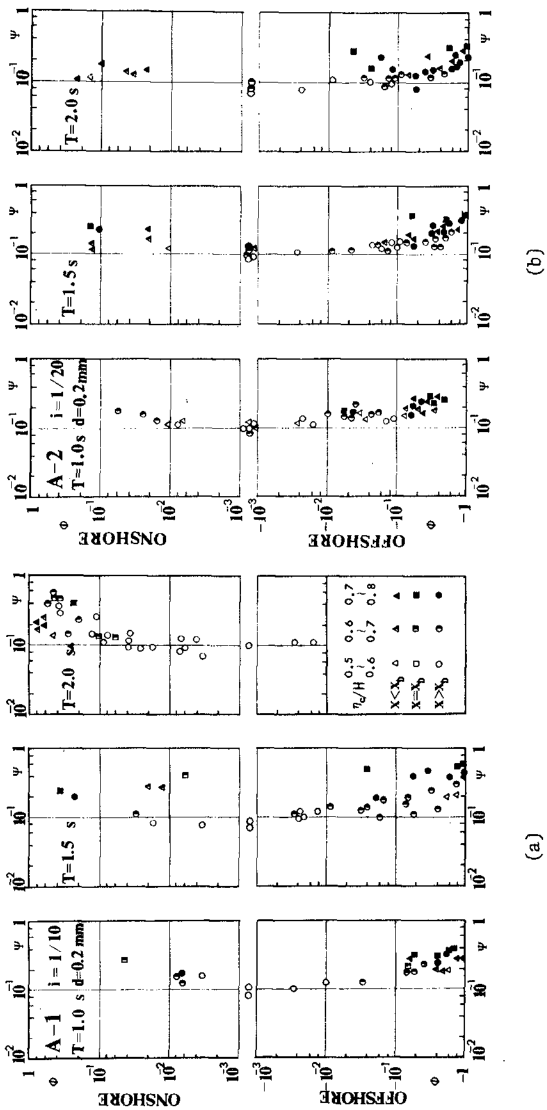


Fig. 6 Relationship between net transport rate and Shields parameter.

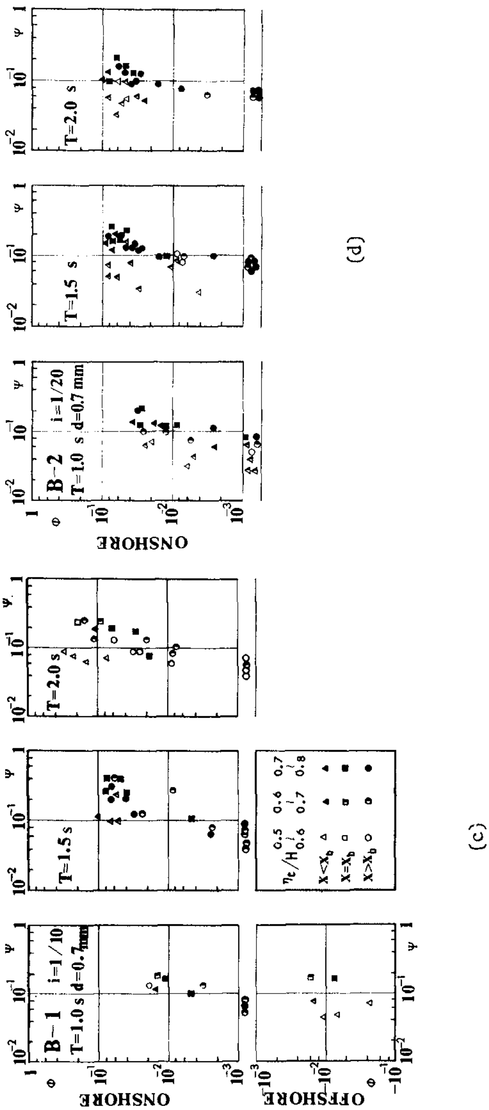


Fig. 6 Relationship between net transport rate and Shields parameter (continued).

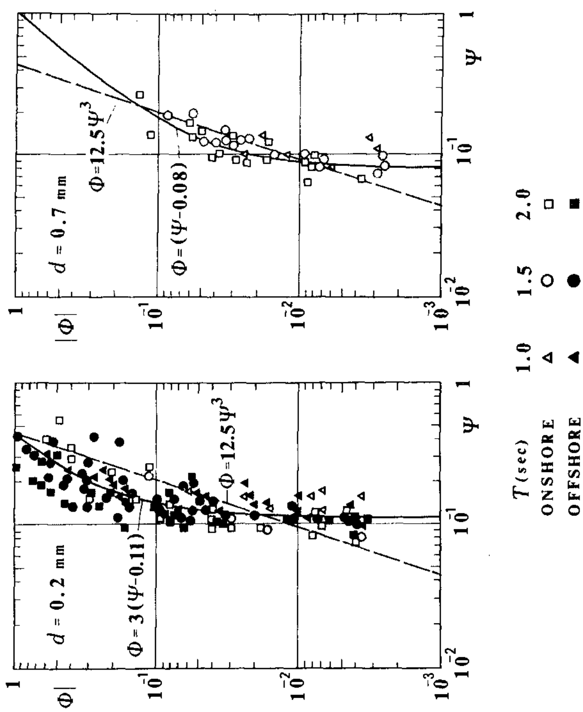


Fig. 7 Summarized relationship between net transport rate and Shields parameter.

hydrodynamically *smooth* and *rough*, will be better. See Horikawa and Watanabe (1967).)

We shall assume the net transport rate is proportional to the excess of Ψ value over its critical value for the general movement just mentioned as:

$$\Phi = A\,(\Psi - \Psi_{cr}) \tag{6}$$

The values of proportionality constants A to give good fit with the present data are 3.0 and 1.0, respectively, as shown in Fig. 7. It should be mentioned that Eq. (6) is just an empirical relation and is not physically well-founded yet. Probably we should not expect so simple relations between Φ and Ψ, because the net transport rate must be subject not only to the shear stress amplitude rather simply evaluated but also to the local bed slope, asymmetry of near-bed velocity, mass transport velocity, ripple form, and so on. Thorough investigation of the data and more physical consideration will be needed.

CONCLUDING REMARKS

Some interesting, though unsatisfactory yet, results on the on-offshore sediment transport on a sloping beach have been gained by analyzing the experimental data of two-dimensional beach deformation. The authors are proceeding the study on wave deformation and near-bottom velocity field on a sloping beach in parallel with the laboratory tests similar to experimentation herein reported. Such investigation will be quite useful to improve the interpretation of experimental data.

REFERENCES

Hattori, M. and R. Kawamata (1980): Onshore-offshore transport and beach profile change. Summaries of 17th Conf. Coastal Eng., pp. 254-255.
Horikawa, K. and A. Watanabe (1967): A study on sand movement due to wave action. Coastal Eng. in Japan, Vol. 10, pp. 39-57.
Inman, D. L. and E. B. Tunstall (1972): Phase dependent roughness control of sand movement. Proc. 13th Conf. Coastal Eng., pp. 1155-1171.
Iwagaki, Y. and H. Noda (1962): Laboratory study of scale effects in two-dimensional beach processes. Proc. 12th Conf. Coastal Eng., pp. 194-210.
Johnson, J, W. (1949): Scale effect in hydraulic model involving wave motion. Trans. A. G. U., Vol. 30, pp. 517-525.
Jonsson, I. G. (1966): Wave boundary layers and friction factors. Proc. 10th Conf. Coastal Eng., Vol. 1, pp. 127-148.
Madsen, O. S. and W. D. Grant (1976): Qualitative description of sediment transport by waves. Proc. 15th Conf. Coastal Eng., pp. 1093-1112.
Nayak, I. V. (1970): Equilibrium profiles of model beaches. Proc. 12th Conf. Coastal Eng., pp. 1321-1340.
Sunamura, T. and K. Horikawa (1974): Two-dimensional beach transformation due to waves. Proc. 14th Conf. Coastal Eng., pp. 920-938.