Journal of Coastal Research	4	4	531-541	Charlottesville, Virginia	Fall 1988

Current Shore Protection Works in Japan

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ABSTRACT

SAWARAGI, T., 1988. Current shore protections works in Japan. Journal of Coastal Research, 4(4), 531-541. Charlottesville (Virginia). ISSN 0749-0208.

The function to prevent disaster has been given the most priority in the design of shore protection works in Japan. However, in recent years, the importance of easy access to coasts and waterfronts is increasingly recognized. Therefore, the design of shore protection works requires consideration of coastal scenery and utilization of the beach. To cope with these requirements, new types of shore protection works have been designed and investigated in the field and in laboratories. This paper considers one example of the procedure for the determination of the shore protection works that control beach erosion. Aspects of sea dikes of gentle slope and artificial reefs are discussed as two examples of new shore protection works in Japan.

ADDITIONAL INDEX WORDS: *Beach erosion, gentle slope sea dike, wave overtopping, zonal protection works, artificial reef.*

INTRODUCTION

In Japanese Coastal Law, enacted in 1953, two kinds of engineering works that mitigate against coastal disasters are regulated. One group mitigates against storm surges and the other against beach erosion. Needless to say, there are also engineering works for coastal disasters caused by wave overtopping, tsunamis, blockage of river mouths, and so on. Various kinds of coastal structures such as sea dikes, seawalls, lockgates, offshore detached breakwaters, groins, artificial reefs, wave absorbing block mounds, and so on, have been used to construct protective works against these disasters.

Figure 1 illustrates the ratio of increase of the region where these kinds of coastal structures were constructed, using 1962 as a base year (TOYOSHIMA, 1986). The increase in the number of offshore detached breakwaters is remarkable. This is because the offshore detached breakwater effectively reduces and absorbes incident wave energy. However, the offshore detached breakwater, as well as the wave absorbing block mound in front of seawalls, detract from natural coastal views and prevent effective utilization of many coastal regions.

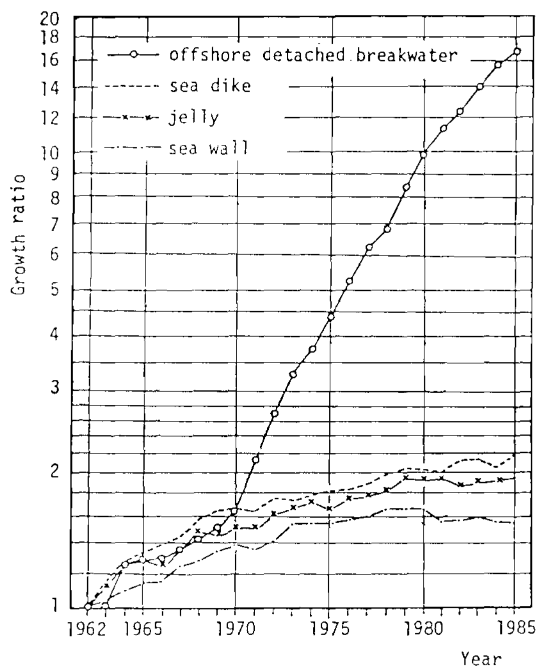

Figure 1. Growth ratio of construction sites for coastal structures.

88002 received 6 January 1988; accepted in revision 13 April 1988.

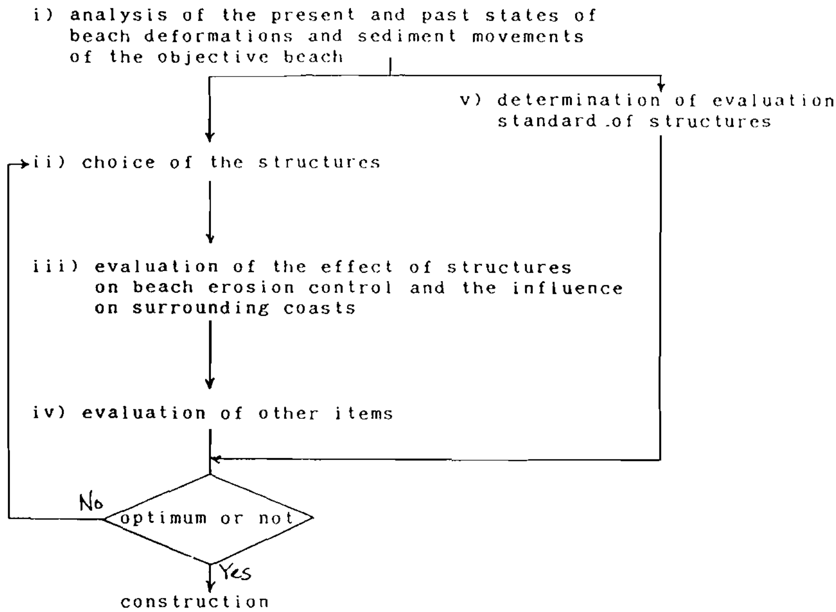

Figure 2. Flow chart for the determination of protection works.

Recently, there is increased concern with preservation of coastal environments and easy access to the shoreline. In order to cope with these requirements and to create pro-waterfront, the following coastal protection works have been newly planned and investigated in Japan: (1) sea dikes of gentle slopes, (2) zonal protection systems (a typical example is artificial reefs consist of offshore submerged breakwaters and artificially nourished beaches behind them), (3) beach nourishment by sand by-passing, and (4) head-land defense works.

The applicability of these engineering works depends on the causes and local situations of potential disasters. In the following sections, the procedure for the determination of the shore protection works intended for beach erosion control is firstly described and then some aspects of sea dikes of gentle slope and artificial reefs, as an example of zonal protection systems, are discussed in terms of experimental results.

PROCEDURE FOR DETERMINING SHORE PROTECTION WORKS WITH RESPECT TO EROSION CONTROL

Figure 2 is a flow chart which illustrates the procedure for determining proper shore protection works. Items in the flow chart are explained as follows: (1) Investigation of present and past states of beach deformation and sediment movement at the coast. In this investigation, bottom topography and characteristics of the beach are analyzed according to the following criteria: (a) *Bathymetric data.* Relation between changes of beach profiles, ΔA and shifts of the location of the shoreline $\Delta 1$ (so-called $\Delta A - \Delta 1$ correlation) is analyzed. If there is a strong relation between ΔA and $\Delta 1$, a one-line-theory is applied. Prediction of shoreline location, and characteristic height of beach change ($\bar{h}$ required in the calculation of shoreline location) can be determined (Uda *et al.*, 1982). Empirical eigen function methods are also used to determine characteristics of beach deformation. (b) *Meteorological data of the past.* Characteristics of incident waves are estimated if there is no measured wave data. (c) *Oceanographic data.* Representative waves, tidal currents and tidal ranges are determined. (d) *Bed material data.* Longshore and cross-shore distribution of sand-sized grains and mineral composition help determine the dominant direction of sediment transport.

(2) Choice of structures. Structure of protection works must be determined by considera-

tion of beach deformation and dominant mode of sediment movement. In Table 1, various kinds of structures which have been used as protection works in Japan are classified by function and ability to control sediment transport.

(3) Evaluation of the effect of structures on beach erosion control and the influence on the surrounding coast. This phase is based on the prediction of topographic change of the coast, with and without structures. At present, the following procedures are used to predict topographic changes: (a) Numerical model. (a1) Numerical prediction of the shoreline location (or contour lines) based on so-called Single-(or Multi-)line theory in which the total longshore sediment transport plays the most important role. (a2) Numerical prediction of the change of water depth using local sediment transport rates. Figure 3 illustrates a "decision tree" concerning the choice of a1 and a2. (b) Physical model. Predictions are based on a two- or three-dimensional scale model experiment on a movable bed. (c) Hybrid model. In this model, changes in water depth are numerically simulated using information from wave and current fields (measured in the scale model experiments on a fixed bed).

In the numerical model, it is difficult to simulate wave deformation, including wave breaking in the vicinity of structures. Also, in the law of similitude, the bed material is not fully established. In the hybrid model, these problems, *i.e.* in the numerical and physical models, are compensated. Figure 4 provides a flow chart for the prediction of beach deformation by these three models.

SEA DIKES OF GENTLE SLOPE

In Japan, sea dikes are a popular form of shore protection works. Because front slopes used to be relatively steeper than 1:1, the foreshore, in front of the sea dikes, often disappeared after storms due to erosion caused by reflected waves. Now, however, new sea dikes with gentle slopes are covered with permeable revetments, as shown in Photograph 1, are increasingly common in Japan. Table 2 compares the number of newly constructed sea dikes of steep and gentle front slopes for 1983 and 1986.

TOYOSHIMA (1986) pointed out, based on experimental results, that wave run-up height on permeable gentle-slope sea dikes is reduced to 0.45 to 0.65 times that on impermeable gentle slope sea dikes. The effects of gentle-slope sea dikes on beach deformation is not well

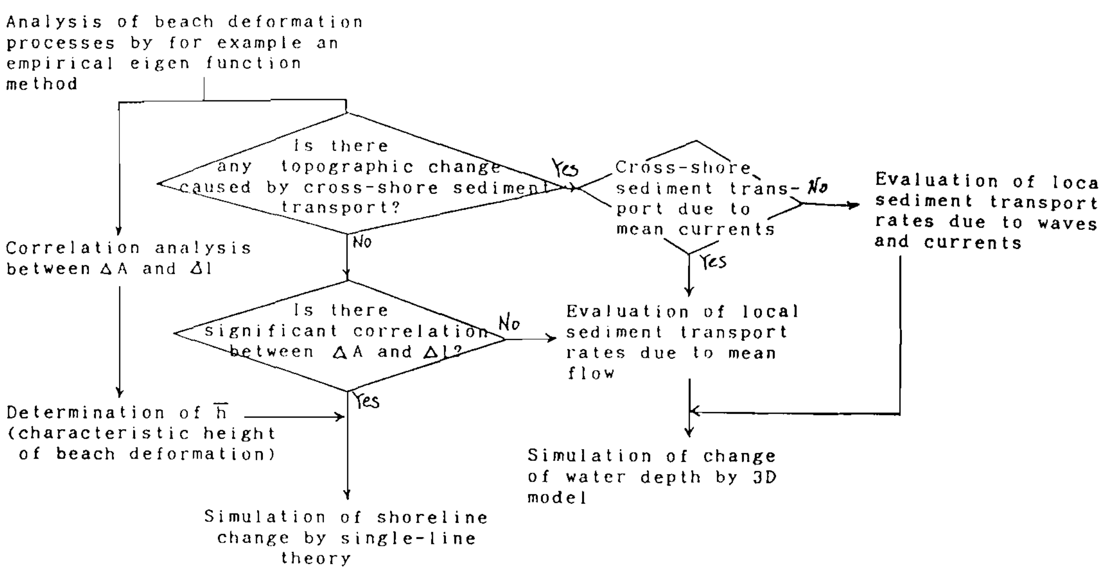

Figure 3. Flow chart for the determination of a beach deformation model for numerical prediction.

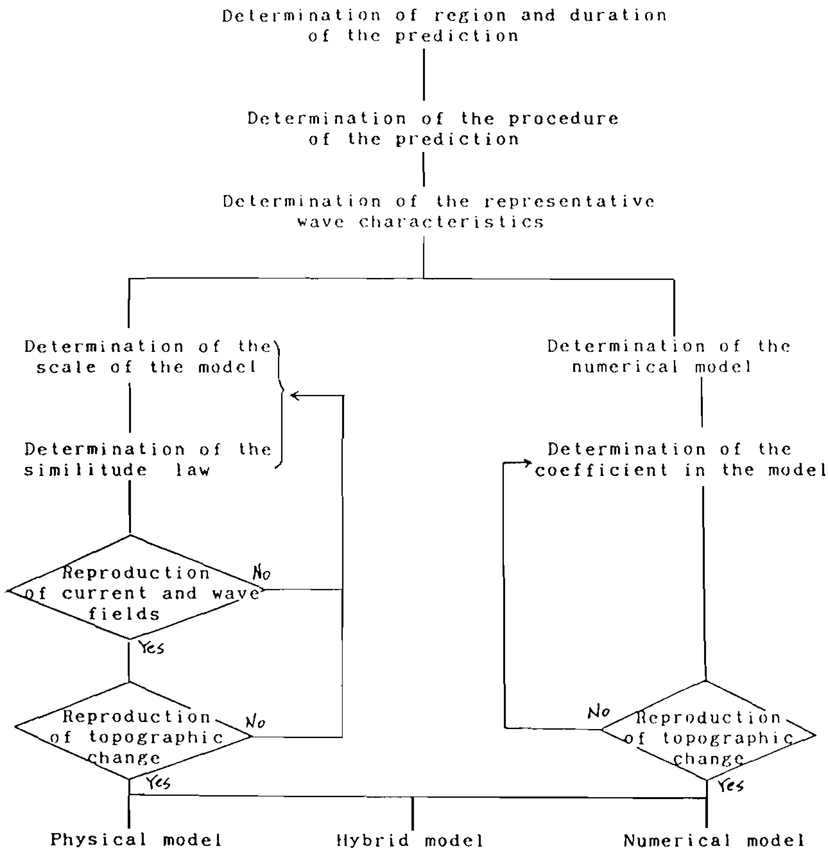

Figure 4. Flow chart for the prediction of beach deformation.

known. DEGUCHI (1984) has, however, discussed the fact that gentle-slope sea dikes tend to favor sand accumulation in front of the dikes under attack by accretive (flat) waves. This observation is based on two-dimensional movable bed experiments. Figure 5 shows some experimental results (DEGUCHI, 1983). The vertical axis is the volume of sand deposited in front of the sea dikes. V_{dep} is calculated from the changes of water depth. X_i is the distance between the initial shoreline and the foot of sea dikes measured offshore and X_b is the distance between the shoreline and the wave break point. V_{dep} for natural beach and vertical sea dikes are also shown in Figure 5. When $X_i/X_b<0.5$, V_{dep} in front of gentle-slope sea dikes are almost the same as V_{dep} of a natural beach. With the increase in X_i/X_b from 0.5, V_{dep} decreases and when $X_i/X_b = 1.0$, V_{dep} becomes about half that of a natural beach. However, it seems evident that gentle-slope sea dikes encourage deposition of sand in front of sea dikes, that is compared to the effects of vertical sea dikes. TOYOSHIMA (1987) reports that sand accretion in front of gentle-slope sea dikes is presently taking place at several locations in Japan.

ZONAL PROTECTION WORKS

The usual individual protection works such as offshore detached breakwaters, sea dikes, offshore submerged breakwaters and so on protect the shore as a line. On the other hand, shore protection systems which consist of offshore submerged or offshore detached breakwaters and artificially nourished beaches behind them are called *zonal protection works.*

Photo 1. An example of a sea dike of gentle slope constructed in Japan.

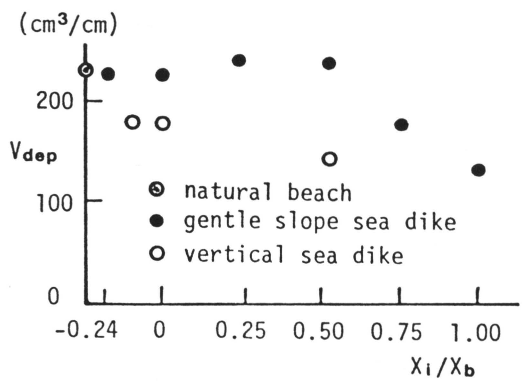

Figure 5. Sand volume deposited in front of sea dikes with different slopes at various locations.

Table 1. *Functions of structures as shore protection works.*

sediment transport to be controlled	structures	conceivable hydraulic functions
longshore sediment transport	groins	trapping of longshore sediment, reduction of longshore current
	head-lands	reduction of longshore energy flux and current
longshore and cross-shore sediment transport	offshore detached breakwaters and submerged breakwaters	diffraction of incident waves, reduction of wave energy and longshore sediment transport
cross-shore and longshore sediment transport	sea dikes sea walls and artificial reefs	reduction of return flow and longshore sediment transport
replenish or supply of sand	sand by-pass and beach nourishment	

The layout of a typical zonal protection work, identified as an artificial reef, is shown in Figure 6. The slope of an artificially nourished beach is usually 1:10 to 1:20. The offshore submerged breakwater in the artificial reef must

Table 2. *Numbers of sea dikes constructed in Japan, 1983 and 1986.*

slope	1:2	1:2.5	1:3	1:4	1:5	1:8	1:10	Total
1983	26	6	26	4	0	1	0	60
1986	30	1	80	1	1	1	2	123

provide enough width and height so that large incident waves break forcibly, hence reducing their energy and preventing replenished sand behind them from flowing offshore.

At present, test sites with artificial reefs are being studied in Japan. Various data concerning wave attenuation and beach deformation are being collected for analysis. The results will be reported within a few years. In the following section, effects of artificial reefs on the reduction of wave overtopping and effects of offshore submerged breakwaters for controlling cross-shore sediment transport on artificially nourished beaches are discussed. These observations are based on experimental results.

EFFECT OF ARTIFICIAL REEFS ON WAVE OVERTOPPING RATES

The effects of artificial reefs constructed in front of vertical sea-walls on wave overtopping rates were investigated in our laboratory. The symbols used in the investigation are shown in Figure 7. In an effort to evaluate effects of artificial reefs on wave overtopping, wave transformation and forced wave breaking criteria were investigated (SAWARAGI *et al.*, 1988a).

The wave breaking criteria on a uniformly sloping beach, as proposed by GODA (1985), is given by Eq. (1).

$$\frac{H_b}{Lo} = A\,[1-\exp(-1.5\,\frac{\pi h_b}{Lo}\,(1+(\tan\theta)^{4/3}))] \quad (1)$$

where H_b and h_b are the wave height and water depth at wave breaking point, tan Θ the bottom slope, L_o the wave length at the deep water and A the experimental coefficient which will take the value between 0.12 to 0.18. It is found that Eq. (1) can be applied to the forced wave breaking on the artificial reef and A is a following function of the relative height of the reef h_s/h_o and h_o/L_o where h_s is the height of the reef and h_o the depth at the foot of the reef:

$$A = a\,(hs/ho) + b \quad (2)$$

$$a = 2.18\,(ho/L_o) + 0.451,$$

$$b = 0.201\,(ho/Lo) + 0.107$$

It is also found that the wave transformation and change of mean water level after forced wave breaking takes place on the reef are predicted by solving equations of conservation of the energy and momentum fluxes. In the calculation of wave height after breaking, the energy dissipation rate derived from the bore model is used (MASE *et al.*, 1982). Figure 8 shows some examples of the comparison

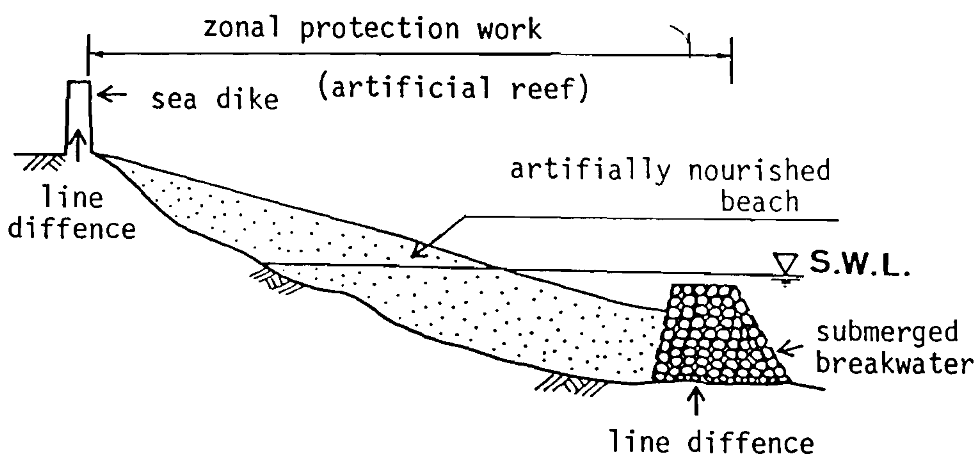

Figure 6. Sketch of a zonal protection system, artificial reef.

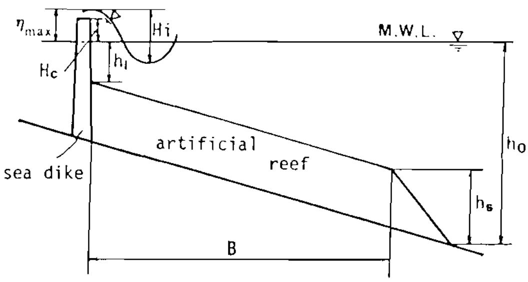

Figure 7. Sketch showing wave overtopping.

between calculated and measured wave heights and mean water levels.

Secondly, to estimate wave overtopping rates from the vertical sea dike in front of the artificial reef, a weir model was applied (SAWARAGI *et al.*, 1988b). The weir model was originally proposed by KIKKAWA *et al.* (1972) for predicting wave overtopping rates from sea dikes on a uniformly sloping beach when clapotis is formed without breaking. The weir model was modified to explicitly take the effect of incident wave steepness into account as follows:

$$q = \frac{Q}{\sqrt{gL_iH_i^2}} = \frac{4\sqrt{2}}{3} mk^{3/2} \sqrt{\frac{Hi}{Li}} \int_{t_1/T}^{t_2/T} [F(t/T) - \frac{Hc}{kH_i}]^{3/2} d(t/T) \quad (3)$$

where q is a nondimensional overtopping rate, Q the overtopping rate per unit time and unit width of the sea dike, H_i and L_i the wave height and length in front of the sea dike, H_c the crest height of the sea dike from the mean water level, F(t/T) the nondimensional surface elevation in front of the seawall, t_2 and t_1 are the time when the surface elevation Z becomes the same as H_c and the maximum Z_{max}, $k = Z_{max}/H_i$ and m the experimental constant which will take the value of 0.5.

Using the experimental data conducted by many investigators and assuming that F(t/T) is a sinusoidal function, k is determined as follows:

When clapotis is formed in front of the sea dike without breaking

$$k = k_c = 0.594(Hc/Hi) + 0.590 \quad (4)$$

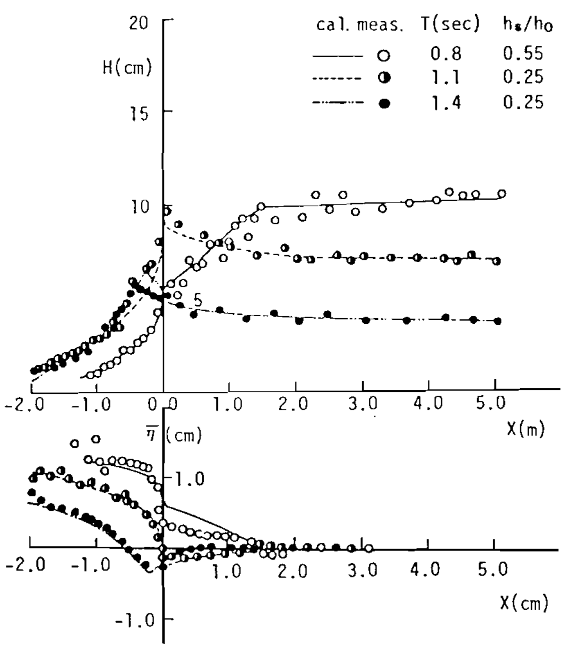

Figure 8. Wave heights and mean water levels on artificial reefs.

When incident waves break in front of the sea dike on the uniformly sloping beach

$$k = k_b = 0.620(Hc/Hi) + 0.877 \quad (5)$$

When forcible wave breaking takes place on the artificial reef constructed in front of the sea dike

$$k = k_r = 0.640(Hc/Hi) + 0.730 \quad (6)$$

Figure 9 shows the relations between non-dimensional overtopping rate q from the vertical wall and non-dimensional crest height H_c/H_i in the cases of different steepnesses of incident waves. Closed symbols in the figure indicate q measured in the case where clapotis is formed in front of the sea dike and open symbols show the results obtained in the case where the sea dike is located in the breaker zone. In the Figure 9, q from the sea dike in front of which the artificial reef was constructed are also distinguished.

It can be seen from Figure 9 that when H_c/H_i is the same, q measured in the case where clapotis without breaking formed in front of the sea dike is the smallest. However, when the forced breaking of incident waves took place on the artificial reef, q becomes smaller than that of the case when the sea dike is in the breaker zone of the uniformly sloping beach. Therefore, it can be judged that the artificial reef is effective in reducing wave overtopping rates from the sea dike by forcing the breaking of incident waves. Three lines in Figure 9 show the predicted overtopping rate from Eqs.(3)-(6).

EFFECT OF SUBMERGED BREAKWATER ON CONTROLLING CROSS-SHORE SEDIMENT MOVEMENT OF THE REPLENISHED SAND BEHIND IT

Maintaining the function of an artificial reef, as mentioned above, requires that the artificial reef be prevented from significant deformation, especially an outflow of replenished sand in the offshore direction beyond the submerged breakwater. Therefore, determination of location, the height hs and the width B of the submerged breakwater, grain size of the replenished sand D and plane spacing of the reefs becomes an important problem in the design of an artificial reef.

A few studies have investigated the effects of grain size of replenished sand and the submerged breakwater on beach deformation. However, it is not always adequate in practice to determine the dimensions of an artificial reef based on these results.

In our laboratory, a series of investigations have been conducted in an effort to determine the dimensions of a submerged breakwater (perched, artificial type reef) for the prediction of wave deformation (SAWARAGI *et al.*, 1988a). It was thus found that the region where significant erosion takes place on the artificial reef corresponds to the region where

$$u_*/w_f = 0.5 \sim 0.6 \quad (7)$$

in which w_f is the settling velocity of the replenished sand and u_* is a shear velocity, which is given by using Jonsson's friction factor f_w (JONSSON, 1967) and the amplitude of the water particle velocity at the bottom u_{bm} as follows:

$$u_*^2 = \rho f w u_{bm} 2/2 \quad (8)$$

u_{bm} is calculated based on the linear long wave theory,

$$u_{bm} = H\sqrt{g/(h+Z)}/2$$

For f_w, the following approximate expression given by SWART (1974) is used

$$f_w = \exp[-5.977 + 5.213(a_m/k_s)^{0.194}] \quad (9)$$

where $a_m = u_{bm}T/2/\pi$, $k_s = 2D_{90}$ where D_{90} is the grain size that 90% of the replenished sand grains are finer.

It was also discovered that the eroded region, the length of which is defined as X_e, is not necessarily replaced by the submerged breakwater in full length as shown in Figure 10. The thick line in Figure 10 indicates the profile of the artificial reef measured after 2 hrs wave generation, in a case where the submerged breakwater was replaced by a thin wall. Other lines in this figure show the temporal change of profiles when 80% of X_e was replaced by the submerged breakwater.

The author and others investigated the relation between the length of the submerged breakwater B, which is wide enough to prevent the artificial reef from significant erosion, and the length of breaker travel (Xs), as defined by GALVIN (1969), by considering that almost all breaker types of forced breaking were plungers. GALVIN (1969) defined 'breaker travel' as the region between the wave breaking point and splash touch-down point and gave the following

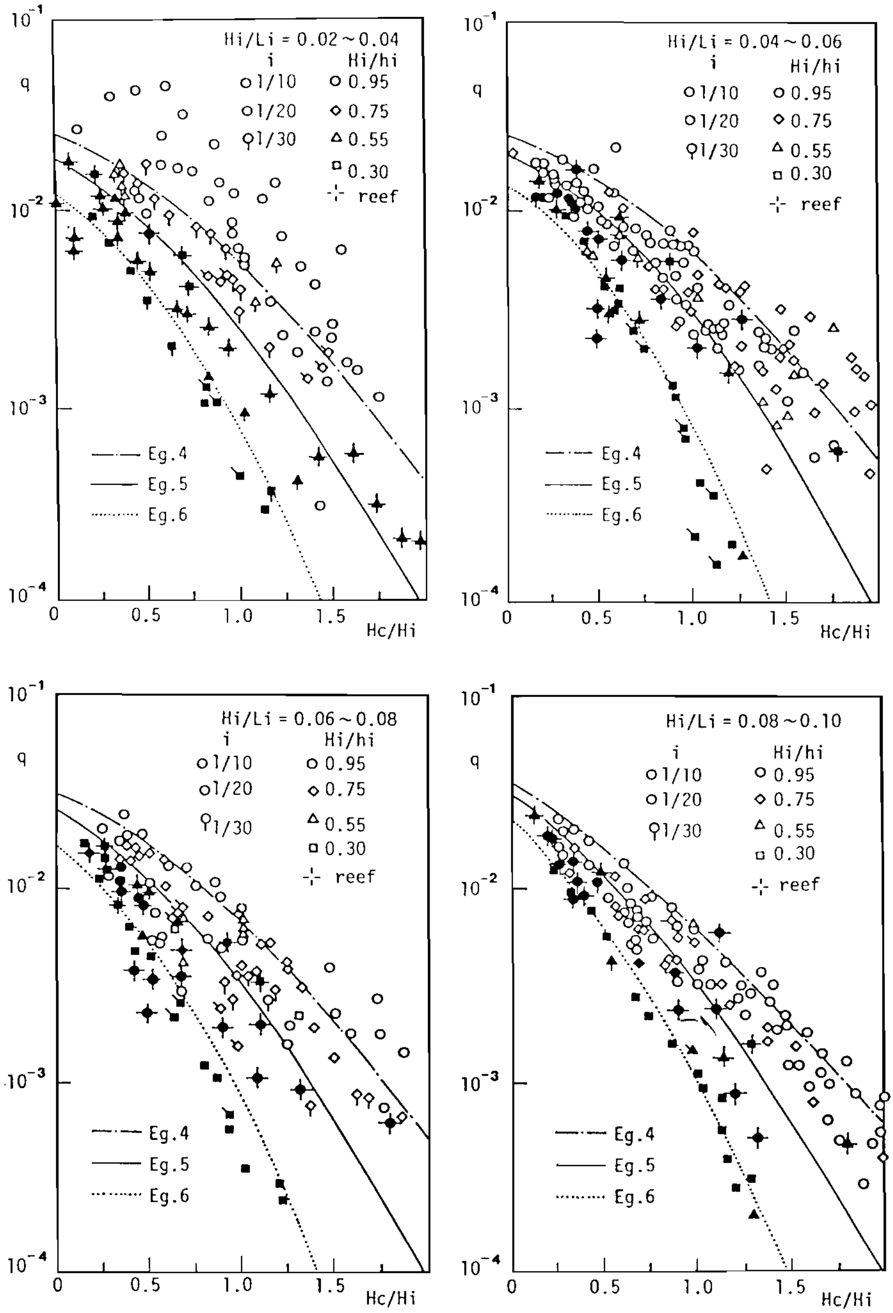

Figure 9. Relation between nondimensional wave overtopping rates and nondimensional crest height of sea dikes.

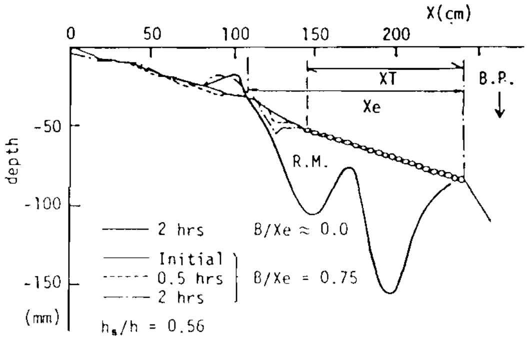

Figure 10. Effect of the width of submerged breakwaters on the deformation of artificial reefs.

expression based on experiments (see Figure 11):

$$X_s = X_{pg} + X_s'$$

$$X_{pg} = (4.0 - 9.25\tan\theta)H_b,\ X_s = X_{pg}$$

where X_{pg} is the distance between wave breaking point and plunging point, X'_s is the distance between plunging point and splush touch-down point. The following modified expression of X_s is used to evaluate breaker travel on the artificial reef X_t as shown in Figure 11

$$X_t = 2[(4.0 - 9.25\tan\theta)H_b - (h_b - h_o - h_s)] \quad (10)$$

The results indicate that B roughly corresponds to X_t. Therefore, the width of the submerged breakwater must be wider than the length of the breaker travel on the artificial reef.

Finally, as for plane spacing of artificial reefs, only a few experimental studies have been conducted (*e.g.* UDA *et al.*, 1984; DEGUCHI, 1986). Close investigations on three-dimensional behavior of the artificial reef, especially the dissipation rate of replenished sediment in the longshore direction, are required for practical design.

CONCLUDING REMARKS

In this paper, some aspects of new shore protection works, *i.e.* gentle-slope sea dikes and artificial reefs which have been increasingly constructed in Japan, are discussed briefly. It is shown experimentally that gentle-slope sea dikes encourage sand deposition in front of them. Artificial reefs tend to reduce wave overtopping from vertical sea walls behind them. A procedure for determining an effective width of the submerged breakwater, *i.e.* of the artificial reef, that will prevent replenished sand from flowing offshore is also proposed. In addition to these coastal protection works, headland defenses and sand by-passing are deployed to advantage in Japan. The effectiveness of these structures, however, requires more research.

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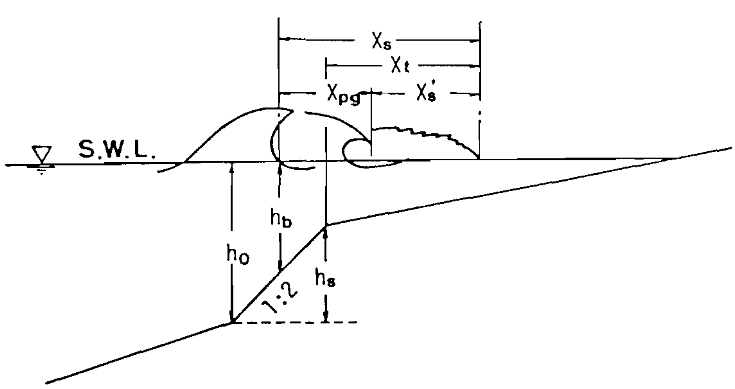

Figure 11. Sketch of breaker travel.

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□ RESUMEN □

La prevención de desastres ha recibido en el Japón la máxima prioridad en el diseño de obras de protección de costas. No obstante, en estos años se ha venido reconociendo la importancia de accesos simples a la costa. Es, por tanto, necesario incorporar al diseño de las obras de protección estos accesos. Con este objetivo se han estudiado nuevos tipos de obras de protección de costas tanto en el laboratorio como en la naturaleza.

En este articulo se introduce, primeramente, el procedimiento para la determinación de las obras de protección. A continuación se discute algunos aspectos relacionados con diques en talud de pendiente suave y con arrecifes artifiales utilizados en el Japón.—*Department of Water Sciences, University of Cantabria, Santander, Spain.*

□ ZUSAMMENFASSUNG □

In Japan wurde bei der Formgestaltung von Küstenschutzverbauungen der Verhinderung von Flutkatastrophen Priorität eingeräumt. Jedoch ist in den letzten Jahren ein wachsendes Bedürfnis festgestellt worden, welches auf einen direkten und einfachen Zugang zur Küste ausgerichtet ist. Aus diesem Grund sind bei der Formgebung von Küstenschutzmaßnahmen Überlegungen notwendig, die sich mit der optischen Gestaltung der Küstenlinie und der Strandnutzung auseinandersetzen. Um diese Aufgabe zu bewältigen wurden neue Formen von Küstenschutzmaßnahmen erarbeitet und im Freiland sowie im Labor untersucht. In dem Artikel wird beispielhaft ein Verfahren zur Festlegung der erforderlichen Küstenschutzarbeiten zur Begrenzung der Stranderosion erläutert. Weiterhin werden anhand experimenteller Arbeiten Aspekte diskutiert, die sich im Zusammenhang mit der Einführung nur leicht geneigter Deiche bzw. Dämme und "künstlicher" Riffe ergeben.—*Ulrich Radtke, Geographisches Institut, Universität Düsseldorf, F.R.G.*