

CHAPTER 204

EXPERIMENTAL STUDY ON THE EFFECT OF GRAVITY DRAINAGE SYSTEM ON BEACH STABILIZATION

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Abstract

In order to develop a new beach preservation technique which has two functions of protecting the beach in a storm and enabling utilization of beach during calm sea, we consider the system of controlling a groundwater level by burying a permeable layer under the foreshore. Since most of the components are buried underground, people on the beach do not notice the presence of the system, enabling preservation of the beach in its most natural state. The greatest challenge of the system is, however, whether installation of the permeable layer alone will fully prevent erosion. Authors therefore conducted an experiment to evaluate beach stabilization effect of the present system. The result indicates that the permeable layer buried in the sand can prevent rise in the groundwater level and the wave set-up. The critical wave height for the foreshore erosion is 1.66 times higher than that on the beach without the permeable layer.

1. Introduction

Against severe natural conditions in Japan, we have constructed seawalls with wave breaking blocks, jetties, detached breakwaters, and so on. These protection measures have stopped further erosion, and also protected the hinterland and people from large waves in a storm. However, in a calm sea condition, the structures keep the people away from the waterfront. Therefore, we are developing a new beach stabilization technique, which has functions of disaster prevention in a storm and beach utilization in a calm. In order to develop such a new beach stabilization technique, first of all a beach erosion mechanism

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during stormy weather was investigated in the field (Kato and Yanagishima: 1992, 1993). These investigations elucidated the relation between rise in the watertable due to wave run-up and the beach erosion. That is, when waves run-up beyond the berm crest in a storm, the sea water stays for a good while on the horizontal area of berm, which accelerates the infiltration of water into the beach. As a result, the watertable becomes higher, and the water exfiltrates through the surface of foreshore. The seepage level of water corresponds to the critical level of berm erosion. Taking this result into consideration, there are two countermeasures for reducing the beach erosion in a storm. One is a construction of a structure offshore to decrease the wave energy and to suppress the wave run-up on the beach. Another is to lower the watertable by some method.

As a latter measure, we consider a permeable layer setting up under the beach. In a storm, waves run-up to the high level and the water infiltrate into the beach. However, the groundwater may be gravitationally drained to the offshore through the layer. Concerning to the effect of permeable layers, two- and three-dimensional experiments were conducted with regular waves to compare the groundwater levels and the profiles on the beach with and without the permeable layer.

2. Details of Experiments

2.1 Aims of Experiments

Aims of two- and three-dimensional experiments are as follows:

- (1) Two-dimensional experiments : A purpose is to know the possibility of controlling watertable and reducing the beach erosion for three different kinds of permeable layers.
- (2) Three-dimensional experiments : There are two purposes in the three-dimensional experiments. One is to confirm the effect of the convergence type drainage system, in which the water gathered by the permeable layer is drained offshore through one pipe. Another is to evaluate the effect of permeable layer on beach stabilization quantitatively.

2.2 Conditions and Methods in Two-dimensional Experiments

The experiments were carried out in a experimental flume of 38.0 m long, 0.5 m wide and 1.5 m high. The movable profiles were made with sand of 0.135 mm in a median grain size. The permeability coefficient of sand is 6.28×10^{-3} cm/s, which is the result of permeability test.

As the permeable layer, three types shown in Figure 1 were tested. Features of these types are as follows, respectively.

- (1) Gravel Permeable Layer (GPL) : Gravels of 13 - 20 mm in size were laid with the thickness of 10 cm from the backshore to the breaker zone (Figure 1, top). Plankton nets were interposed between the sand layers and the gravel layer to prevent sand from intruding into the gravel layer. The sand layer above the permeable layer was 10 cm thick. A still water level was situated inside the permeable layer of horizontal part.

(2) Gravel Permeable Layer with Drainage Pipe (GPL-DP) : The thickness of the permeable layer and the sand layer were decreased to half that of GPL, respectively (Figure 1, middle). The permeable layer in the backshore was therefore situated above the still water level. A drainage pipe (44 mm in inner diameter) extended from the permeable layer near the shoreline toward offshore. The pipe was buried along the wall of experimental flume, with its outlet vent located at the wave breaking point.

(3) Permeable Pipe with Drainage Pipe (PP-DP) : Three permeable pipes were used in place of the permeable layer (Figure 1, bottom, Figure 2). The permeable pipe was a spring coil of 15 mm diameter wrapped up with a plankton net as shown in Figure 3. Groundwater percolates into the pipe through the plankton net. Three permeable pipes were buried under the beach with a 15 cm interval in the traverse direction. To protect the permeable pipes, a gravel layer was placed above the pipes. This gravel layer did not act as a permeable layer because sand was filled in the voids of gravel layer. The drainage pipe extending toward the offshore direction from near the shoreline was the same as that used in the GPL-DP.

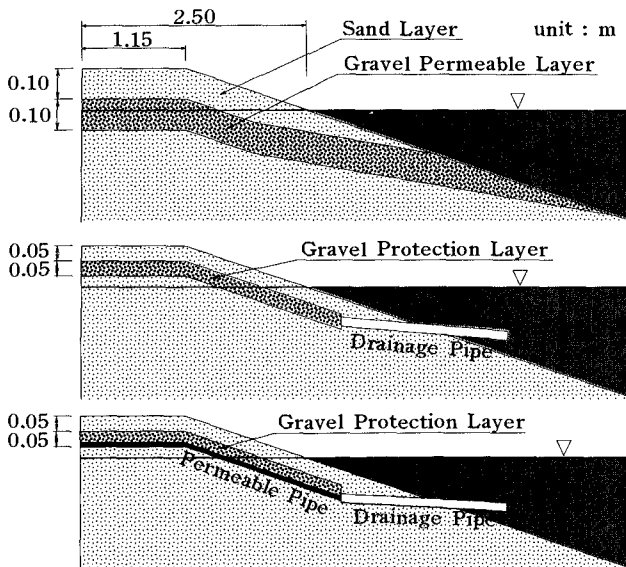


Figure 1: Types of Permeable Layer in Two Dimensional Experiment.

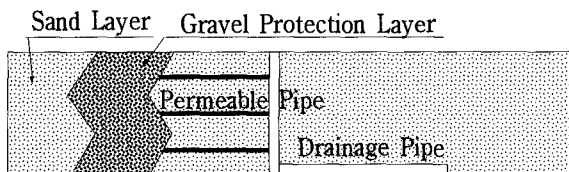


Figure 2 : Plane view of PP-DP.

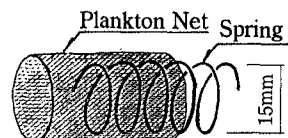


Figure 3 : Structure of Permeable Pipe.

Table 1 : Waves in Two Dimensional Experiments.

H_o (cm)	T (s)	H_o / L_o	GPL	GPL-DP	PP-DP	Without a Permeable Layer
8.4	1.34	0.03	○	○	○	○
10.0	1.79	0.03	○	○	○	○
15.0	1.79	0.03	○			○

The wave conditions in the two dimensional experiments are listed in Table 1, in which the cases with a circle were conducted. In the experiment, the level of groundwater in the section from the backshore to the shoreline, wave set-up in the surf zone, and the profile changes were measured. To measure the groundwater level and the wave set-up, a manometer using a vinyl tube 6 mm in inner diameter was used. Measurements were carried out with an interval of 25 cm along the wall of the flume in the cross-shore direction. Using a sand level meter, the profile changes were measured along three measurement lines in the traverse direction with an interval of 10 cm. The mean value of the profiles along the three measurement lines is calculated and used for further analysis of the profile changes. These result are compared with those obtained in the profile without permeable layer. In the profile without a permeable layer and the profile with GPL, a dye (methylene blue) was injected close to the glass wall of the flume. The seepage velocity inside the permeable layer and the sand layer above the permeable layer respectively were estimated from the travelling distance of the gravitational center of the dye patch and its elapsed time.

2.3 Conditions and Method in Three-Dimensional Experiments

Test conditions were primarily the same as those in the two dimensional experiments except the following.

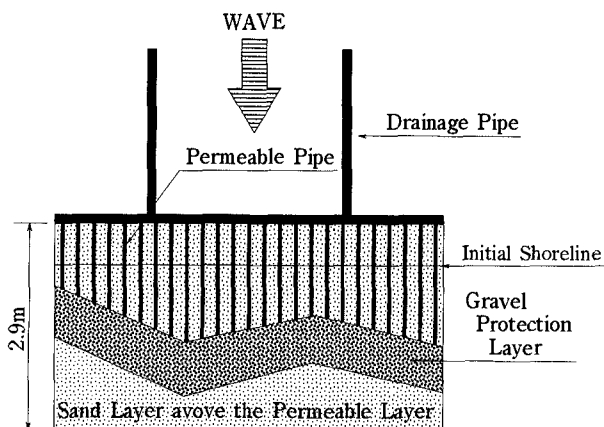


Figure 4 : Layout of Permeable Layer in Three-Dimensional Experiments.

A wave basin (35.0 m L $\times$ 1.1 m H $\times$ 15.0 m W) was divided into two sections (7.5 m W) to enable simultaneous experiments on the two beaches with PP-DP and without the permeable layer. Figure 4 shows the layout of PP-DP: 100 permeable pipes measuring 2.9 m in length and 15 mm in inner diameter were buried with a 7.5 cm longshore interval and eight drainage pipes measuring 44 mm in inner diameter with a 94 cm longshore interval. The outlet vent of the drainage pipe was positioned at the wave breaking line. The median grain size of the sand used in the experiments was $d_{50} = 0.16$ mm and the permeability coefficient was $k = 1.14 \times 10^{-2}$ cm/s.

Table 2 : Waves and Profile Changes in Three-dimensional Experiments.

T (s)	H _o (cm)	Value of C	Classification of Profile Changes	
			Without a Permeable Layer	PP-DP
1.34	1.5	2.0	II, III	III
	3.0	4.0	I, II	II, III
	4.5	6.0	I	II
	6.0	8.0	I	I
	7.5	10.0	I	I
	☆ 8.4	11.2	I	I
2.01	1.8	1.8	III	III
	3.7	3.7	I, II	II, III
	5.4	5.5	I	II
	7.2	7.4	I	I
	9.0	9.2	I	I
	☆ 12.5	12.8	I	I

The wave conditions used in the experiments are listed in the first two columns of Table 2. The waves were acted for one hour in each case. The three columns on the right in the table give information on the shoreline changes; the details are given later.

The groundwater level was measured with a 1.25 m interval in the longshore direction along the two measurement lines, the initial shoreline (shoreline at the time of still water) and the line near the tip of the wave run-up. Profiles were measured with a 0.125 m interval in the cross-shore direction along the three measurement lines set with a 2.25 m interval in the longshore direction. For the cases with the wave height values marked with asterisks in Table 1, additional measurements of profile were done along the measurement lines set with 0.25 m interval in the longshore direction.

3. Results of Experiments

3.1 Comparison of Groundwater Levels

Figure 5 shows the groundwater level in the two-dimensional experiment in which waves of 8.4 cm in height and 1.34 s in period were acted on the beach without the permeable layer. The reference level for elevation on the ordinate is the initial still water

level. The groundwater level begins to rise immediately after waves are acted, and substantially levels off after 11 minutes. The peak of the watertable near the location of 2.0 m substantially coincides with the tip end of the run-up, and the water level exceeds the still water level by 2 cm. The wave set-up at the shoreline located at 2.5 m is 1 cm above the still water level.

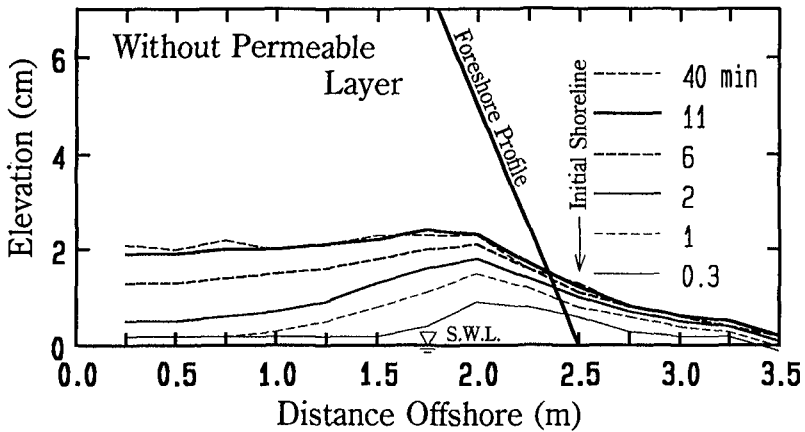


Figure 5 : Changes of Groundwater Level with Time in the Beach without the Permeable Layer (two-dimensional experiment).

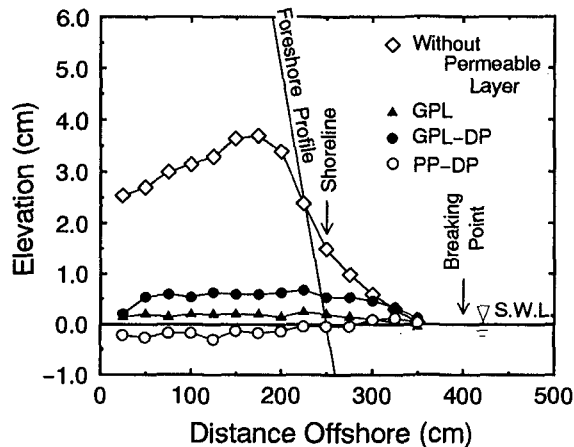


Figure 6 : Comparison of Groundwater Levels between Beaches with and without the Permeable Layer (two-dimensional experiment).

Figure 6 shows the distributions of groundwater level and the wave set-up measured ten minutes after the waves of 10.0 cm in height and 1.79 s in period were acted on the beaches without the permeable layer and with three types of permeable layer. Because of

the severe wave conditions, rise in the groundwater level and the wave set-up on the beach without a permeable layer are greater than those shown in Figure 5. In the beaches with the permeable layers, both the rise in the groundwater level and the wave set-up are quite insignificant. In particular, the groundwater level in the beach with PP-DP is lower than the initial still water level. This is considered to be the effect of wave set-down because the terminal end of the drainage pipe on the offshore side is located at the wave breaking point.

3.2 Wave set-up

It is confirmed that the permeable layer has a function to suppress rise in the groundwater level. The experiment results, however, suggest that it can also prevent the wave set-up. Figure 7 shows the relation between the wave set-up at the initial shoreline and the wave steepness in the two-dimensional experiment. For all cases, the values of wave set-up measured 10 minutes after the start of wave action are plotted. The broken line in the figure is a theoretical value of wave set-up according to Goda's theory (1975) when the bottom gradient is 1/10. The wave set-up on the beach without a permeable layer substantially agrees with Goda's theoretical value. On the beach with the permeable layer, particularly with PP-DP, no wave set-up is observed.

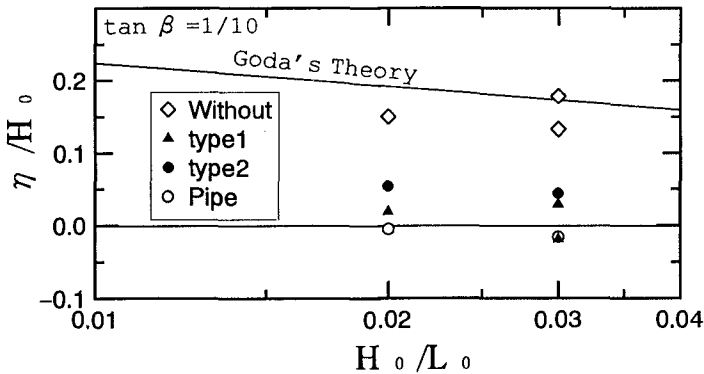


Figure 7 : Relation between Wave set-up and Wave Steepness
(two-dimensional experiment).

Figure 8 shows the longshore distributions of wave set-up at the shoreline on the beaches with and without the permeable layer in the three-dimensional experiments. The wave height was 12.5 cm and the period was 2.01 s, and the measurements were taken 30 minutes after the waves were acted. On the beach without the permeable layer, the wave set-up agrees with the theoretical value of Goda (1975) which is shown by a horizontal line in the figure. On the beach with the permeable layer, on the other hand, the average wave set-up is about 0.5 cm. The relative position between the measurement point of groundwater level and the position of the drainage pipe, which is shown by $\uparrow$ in the Figure 8, changes from a location to a location. The wave set, however, is considered to

be constant in the longshore direction, although a slight fluctuation is observed. This result indicates that groundwater within the range of 0.94 m along the longshore direction could be discharged by one drainage pipe, suggesting that convergence type drainage system is feasible.

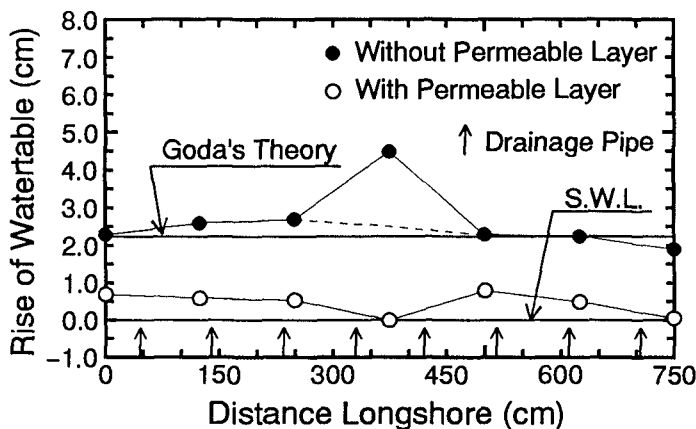


Figure 8 : Comparison of Groundwater Levels between on the Beaches with and without the Permeable Layer (three-dimensional experiment).

3.3 Seepage velocity in the Sand Layer and the Permeable Layer

Figure 9 shows the seepage velocity in the sand layer and the permeable layer. The top portion shows the values for the beach without the permeable layer, the bottom portion those for the GPL. The velocity in the profile without the permeable layer is smaller than 0.15 mm/min. The velocity in the sand layer located above the still water level is directed onshoreward, whereas the flow in the sand layer below the still water level is directed toward the offshore along the sea bottom. In the beach with the permeable layer, the seepage velocity is two to three times that in the beach without the permeable layer, with a offshoreward velocity of about 30 cm/min (about 1000 times the velocity in the sand layer; note that velocity vectors are in logarithmic expression in the figure). It must be noted that the flow in the permeable layer fluctuated with the same period as wave motions, but the time average velocity is shown in Figure 9.

A reason of the fast velocity in the permeable layer is studied. Firstly, a numerical simulation of two dimensional unsteady flow was conducted to evaluate the amount of water infiltration into the beach due to the wave run-up. The simulation was carried out for the entire model beach. The water infiltration into the beach in the portion from the shoreline to the run-up level was given, so that the calculated groundwater level agreed with the measured value. The rate of infiltration into the beach without the permeable layer was estimated to be $1.1 \text{ cm}^3/\text{min}/\text{cm}$. As for the beach with the permeable layer, if the same amount of water is assumed to infiltrate into the beach due to the wave swashing,

the seepage velocity in the permeable layer (10 cm thick) would be 0.37 cm/min, given the void ratio of 0.3. Thus, the fast velocity in the permeable layer cannot be explained solely by this infiltration. In the following consideration, therefore, the experimental fact that no wave set-up is observed on the beach with the permeable layer is taken into account.

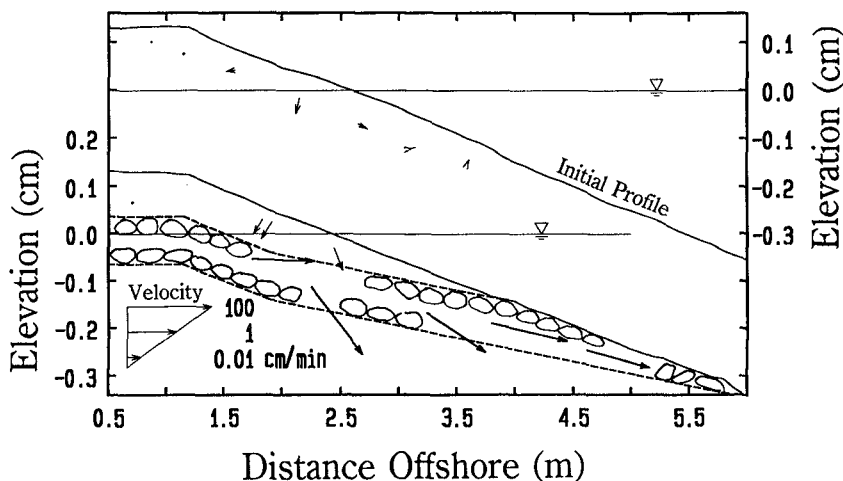


Figure 9 : Seepage Velocity in the Sand Layer and the Permeable Layer
(two-dimensional experiment).

Assuming two-dimensional steady-state condition and ignoring the term of horizontal diffusion, the equation of motion in the breaker zone is

$$\frac{d}{dx} (\rho U^2 (h + \eta) + S_{xx}) = -\rho g (h + \eta) \frac{d\eta}{dx} - \tau_b \quad \dots\dots\dots (1)$$

with the offshore direction taken along the positive x-axis, where U is the mean velocity, h water depth, η wave set-up, S_{xx} radiation stress and τ_b bottom shear stress. Since there is no wave set-up, $\eta = 0$. Then, by assuming $\tau_b = 0$, the equation (2) can be easily integrated.

From the radiation stress given by the equation below,

$$S_{xx} = -\frac{3}{16} \rho g \gamma^2 \tan^2 \beta \quad \dots\dots\dots (2)$$

which is approximated by the linear shallow-water theory by assuming a uniform slope ($h = x \tan \beta$) and a constant proportionality between the wave height and depth ($\gamma = H/h$), then it holds:

$$U = -\frac{\gamma}{4} \sqrt{3 g x \tan^3 \beta} \quad \dots\dots\dots (3)$$

If it is assumed that a flow which compensates the onshoreward discharge ($U \times h$) is generated in the permeable layer, then the velocity V_i in the permeable layer is

$$V_i = \frac{\gamma}{4 \lambda D} \sqrt{3 g x^3 \tan^3 \beta} \quad \dots\dots\dots (4)$$

where λ is the porosity ($=0.3$) and D is the thickness of the permeable layer ($=10$ cm). By assuming $\gamma = 0.78$, V_i at a distance of 5 cm offshore ($h=0.5$ cm) from the shoreline is calculated to be 75 cm/min, which is almost 2.5 times larger than the value actually measured in the permeable layer. This is probably because the velocity is overestimated in equation (3). The calculated value would be much closer to the actual measurement if considerations are given to the factors that the onshore velocity in the surf zone is subject to the bottom shear stress, that any small rate of wave set-up would act to decelerate the velocity, and that there is the effect of the permeability coefficient in the sand layer above the permeable layer. It can be said that the fast velocity in the permeable layer is subject to the strong influence of the gradient of the radiation stress.

3.4 Profile changes

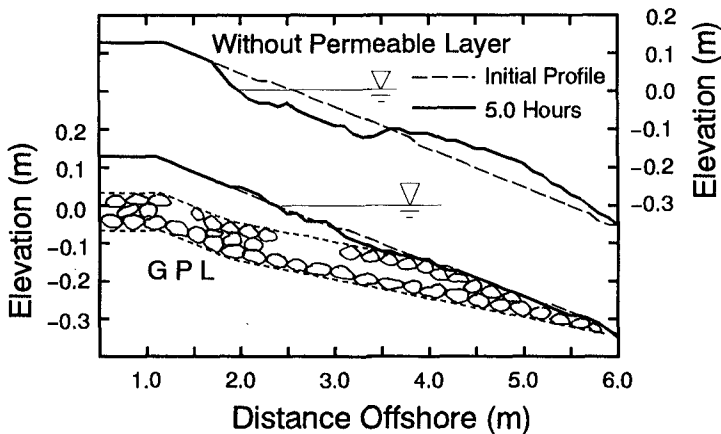


Figure 10 : Comparison of Profile Changes
(two-dimensional experiment : $H=8.4$ cm, $T=1.34$ s).

Figure 10 shows the profile changes on the beach without the permeable layer and with GPL, respectively acted with waves of 1.34 s in period and 8.4 cm in height for five hours. The profile without the permeable layer eroded in the range including the initial shoreline, and the sand deposited on the offshore forming a bar. The shoreline receded for about 0.5 m. On the beach with GPL, on the other hand, hardly any profile change is observed except for a small berm slightly on the onshore side of the shoreline. No change

in the shoreline position is observed. In this case, the permeable layer is effective in preventing erosion near the shoreline.

Figure 11 shows the profile changes acted with waves of 1.79 s in period and 15.0 cm in height for five hours. Because of severe wave conditions, the beach without the permeable layer eroded in the range from 1.3 m to 4.3 m. At a point about 4.1 m, there is formed a trough, of which on the offshore side a bar is formed. Although similar erosion is observed on the beach with GPL, no trough is formed since the erosion do not extend deeper than the permeable layer. Sand from the layer above the permeable layer is transported toward offshore direction and deposits on the outlet vent of the permeable layer. This phenomenon was observed from an early stage of wave action. The drainage capacity of the permeable layer was reduced by the clogging of the outlet vent. The erosion occurring at the initial stage of wave action was attributable to the insufficient drainage capacity of the permeable layer, but the eventual erosion similar to that occurring on the beach without the permeable layer was probably because the outlet vent was clogged.

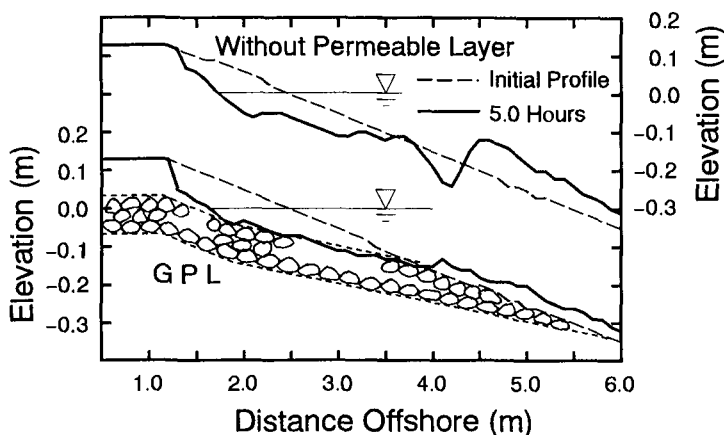


Figure 11 : Comparison of Profile Changes

(two-dimensional experiment : $H=15.0$ cm, $T=1.79$ s).

The top part of Figure 12 shows the topographical change on the beach with the permeable layer, PP-DP. Waves of 8.4 cm in height and 1.34 s in period were acted for one hour. The locations of drainage pipes are clearly shown in the figure, but as they were actually buried underground, only the outlet vents were visible. Because of the wave incident angle of 90° to the shoreline, the profile change is two dimensional and there is no three dimensional effect of draining the water offshore through the eight drainage pipes.

As for the changes in the cross-shore direction, the foreshore is eroded and a bar is formed offshore at about 4 m. The outlet vents of the drainage pipes was located on the immediately onshore side of the bar. However, the vents were not clogged by the sand because the terminal end of the drainage pipe was set slightly above the sea bottom (see Figure 1). Then, the groundwater controlling effect of the permeable pipes is included

even in this foreshore erosion. To demonstrate this point, we conducted a similar experiment using a beach without the permeable layer. The result is shown in the bottom part of Figure 12. On the beach without the permeable layer, erosion on the foreshore is extensive, with a substantial amount of deposition on the offshore bar.

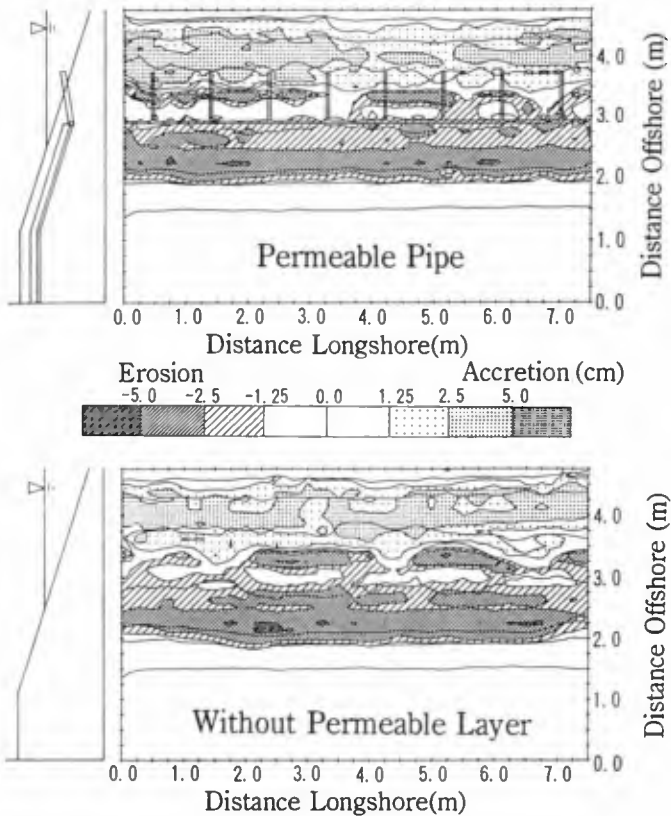


Figure 12 : Comparison of Topographical Changes
(three-dimensional experiments : $H=8.4$ cm, $T=1.34$ s).

As discussed above, in some cases erosion do occur on the beach with the permeable layer, although its extent is slightly less. To have quantitative understanding on the effect of permeable layer, we conduct an analysis using the constant C of the beach profile type classification (Sunamura and Horikawa, 1974) which is evaluated by the equation (5), in which an averaged profile is considered in each case.

$$C = (\tan \beta)^{0.27} \left(\frac{d}{L_0} \right)^{-0.67} \left(\frac{H_0}{L_0} \right) \dots\dots\dots (5)$$

According to Sunamura et al, profile changes can be classified into three types depending on the value C . Characteristic features of each type with regard to the change of the shoreline position can be described as follows:

- $C > 8$ Type I : Recession
 $8 > C > 4$ Type II : Stable
 $4 > C$ Type III : Advancement

The value C calculated by the equation (5) for each case is listed in the third column of Table 2. The value C is the same so long as the wave conditions are identical, without regarding to whether or not the permeable layer is employed. However, different types of shoreline changes are observed on the beaches with and without the permeable layer even if the value C is identical. The types of shoreline changes were investigated for all cases in the three-dimensional experiment. The results are listed in Table 2 and shown in Figure 13. Referring to Figure 13, it is noted that the value C in the case of experiment without the permeable layer is smaller than that of Sunamura et al. This is probably because of the following differences in the experimental conditions: the present experiment is the three-dimensional one (two-dimensional in Sunamura et al); the foreshore slope was 1/10 (many cases gentler than 1/10 are contained in Sunamura et al); and, waves were acted for one hour (more than 40 hours in Sunamura et al). Because of these differences, we evaluate the effect of permeable layer by comparing the results obtained for the beaches with and without the permeable layer instead of comparing with the results obtained by Sunamura et al. Referring to Figure 13, it is seen that the value C is larger on the beach with the permeable layer. In the case without a permeable layer, the mean value of C in the category of shoreline stable is 3.2, while in the case with the permeable layer it is 5.3. In short, the value of C is larger in the case with the permeable layer by a factor of 1.66.

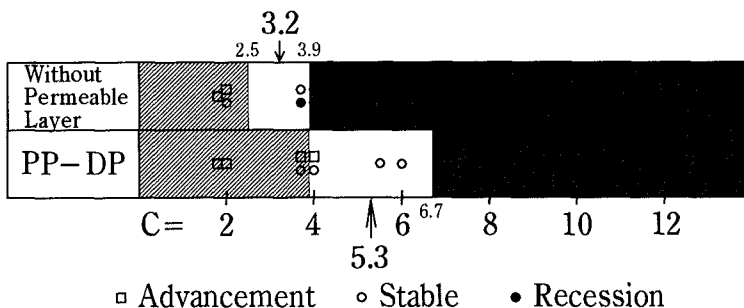


Figure 13 : Classification of Types of Shoreline Changes
in Three-Dimensional Experiment.

When the changes in the value C are viewed from a different angle, the following can be said. When a beach with the permeable layer is subjected to waves having a value C_s , shoreline changes of the beach will be the same as those observed in the natural

beach (without the permeable layer) subjected to waves of $C_a / 1.66$. In other words, the permeable layer is effective in reducing the value C_a to about 60 %. This effect can be achieved also by a wave controlling structure. Generally, a wave controlling structure can control the wave height and not the wave period. Thus, given a constant bottom gradient, grain size and wave length (period) in equation (5), the value C will be in direct proportion only to the wave height. This in turn means that reduction of the value C to about 60 % attained by the permeable layer is comparable to the effect of a structure which is capable of decreasing the wave height to about 60 %. This wave decreasing effect is achieved by a submerged breakwater with a slightly larger crown depth with slightly narrower crown width. It should be noted, however, that a permeable layer has additional effect in that hardly any wave set-up near the shoreline occurs, which is a function not obtained by a submerged breakwater.

4. Conclusion

Effects of the permeable layer are as follows:

- ① Substantially no wave set-up occurs at the shoreline.
- ② Rise in the groundwater level can be suppressed to a greater extent.
- ③ Erosion near the shoreline can be mitigated.
- ④ According to our experiment, combination of permeable pipe and drainage pipe is most effective.
- ⑤ Effect of permeable layer in respect of shoreline changes is comparable to that obtained by reducing the wave height by about 40 %.

It can be concluded that the effect of permeable layer is demonstrated by our laboratory model experiments. Presently, we are conducting filed experiments at a beach in order to complete the present technique. (Katoh and Yanagishima, 1996).

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