

CHAPTER 84

FIELD AND MODEL STUDY ON THE PROTECTION OF RECREATIONAL BEACH AGAINST WAVE ACTION

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INTRODUCTION

The heightening of a sea wall is often proposed for the purpose of decreasing the amount of wave-overtopping. In a recreation beach, however, the heightening of sea wall is undesirable from the view-point of environmental preservation and beach utilization. In this paper, instead of it, a proposal is made of the sea wall with a wide frontal step as well as the widening of beach by artificial nourishment.

The frontal step is not only effective to decrease the amount of wave-overtopping, but also serves as a promenade for visitors. The widening of beach is known to serve to decrease wave-overtopping as well as to increase the utility for recreation. However, the protection of the nourished beach itself becomes sometimes a difficult problem. In this paper, the hydraulic characteristics of a wide frontal step and the effect of several protective measures for the nourished beach are described on the basis of model and field tests conducted for the improvement of Suma Beach.

Suma Beach is a recreational beach situated west of Kobe Port as shown in Fig. 1. The shore-line is about 2 km long and runs from east to west (Fig.11). The beach profile, as shown in Fig.2, has the narrow backshore at about 3 m above L.W.L. and a small step at about 1.5 m below L.W.L. The beach slope is 1/10 in the foreshore, 1/25-1/30 between 2 and 5 m below L.W.L and 1/60-1/80 in the offshore beyond about 6 m below L.W.L. The waves are predominant from the direction of SSW so that the beach materials tend to move eastward along the shore. The waves are in usual so small that most of them are less than 0.5 m in $H_{1/3}$ (significant wave height) and waves of more than 1.5 m in $H_{1/3}$ occur only a few times a year. But in the very rare cases, the beach have experienced the attack

of severe waves such as the later-described. The beach material is composed of coarse sand in the foreshore and fine sand in the offshore, except sand of a few millimeters in mean diameter at the bottom deeper than 8 m. The appearance of the coarse sand in the offshore seems to depend on the action of tidal currents of 1-1.5 knots in velocity. Also, the tidal range is 1.7 m in mean spring tide.

WAVE-OVERTOPPING CHARACTERISTICS OF A FRONTAL STEP

In 1965, a big typhoon hit Suma Beach and gave damage to houses behind of the beach. At that time, the significant wave was estimated to be 4.7 m high and 10 sec. in period and the tidal level was estimated to rise until 3.7 m above L.W.L. Therefore, model tests have been conducted in a channel and the profile shown in Fig.3 has been recommended as a future plan. The profile is obtained by adding a sand fill at the part above -1.5 m deep, heightening the sea wall by 0.5 m and attaching a wide frontal step of +3.5 m high to the original typical profile. Through this model test, wave-overtopping characteristics of a wide frontal step have been revealed as follows.

Measurement of wave-overtopping and calculation of expected rate of wave-overtopping

Test on wave-overtopping discharge has been conducted with a scale of 1/50 in a wave channel of 22 m long, 0.6 m wide and 55 cm high shown in Fig.4. In the model, the water topping over a sea wall due to wave action is guided into a water-trap. The water in the trap is pumped up by a vacuum pump in order to be measured in volume. For each test, the total amount of water topping the sea wall by only seven consecutive waves numbered from the 6th one of a regular wave train has been measured in order to exclude the influence of re-reflection of waves from the other end of the wave channel.

Fig.5 shows an example of the curve for overtopping discharge obtained from the test. The condition of test is 3.7 m above L.W.L. in water level and 10 sec. in wave period. From such curves, the expected rate of overtopping discharge for waves of 4.7 m in $H_{1/3}$ has been calculated as follows using Goda's method^{1),2)}:

$$q_{\text{exp}} = \sum_{j=1}^m q_j(H_j) \Delta p(H_j)$$

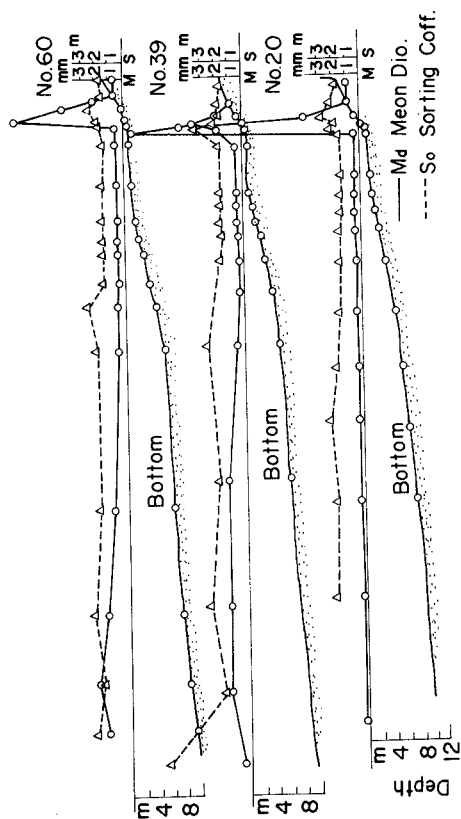


Fig.1 Situation of Suma Beach

Fig.2 Distribution of mean diameter of bed materials along the profile (Unit: m)

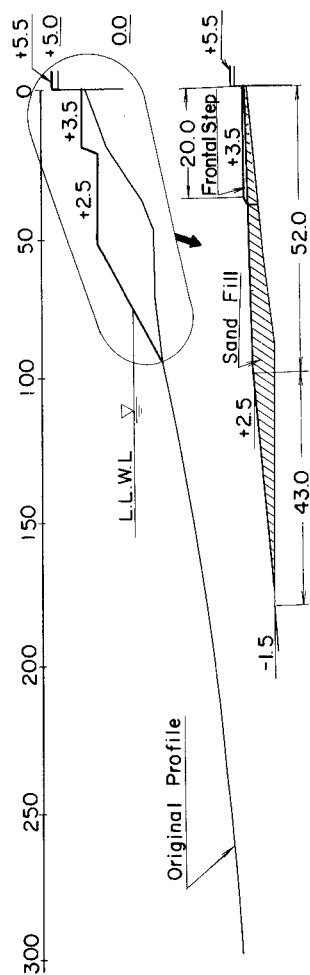
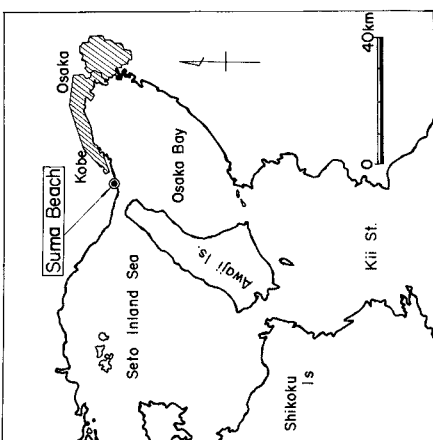


Fig.3 A recommended profile with a wide frontal step



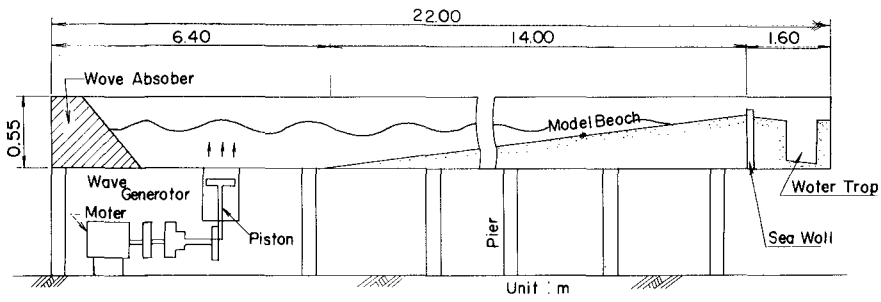


Fig. 4 A wave channel used for test on wave-overtopping

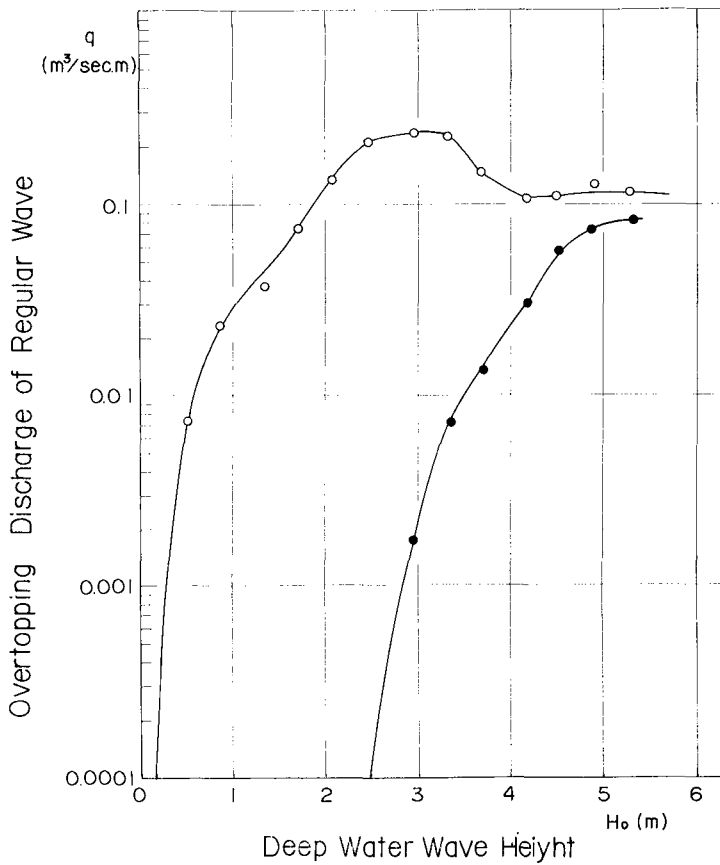


Fig. 5 Overtopping discharge of regular waves on the present profile and the profile with sand fill (Water Level: 3.7m above L.W.L.)

where q_{exp} is the expected rate of overtopping discharge, q_j is the rate of overtopping discharge of a regular wave of H_j in height, and $\Delta p(H_j)$ is the occurrence probability due to the Rayleigh distribution of waves between $(H_j - \Delta H/2)$ and $(H_j + \Delta H/2)$ in a irregular wave group of 4.7 m in $H_{1/3}$. q_j is obtained from experimental data such as Fig.5. In this study, ΔH is taken to be 58 cm.

Effect of wide frontal step against wave-overtopping

Fig.6 shows the expected rate of wave overtopping discharge q_{exp} for sea walls with different frontal steps under the same condition of waves and water level as Fig.5 mentioned above. In the figure, "Present Profile" corresponds to the typical profile of Suma Beach shown in (a) of Fig.7 and "Only Sand Fill" to the profile added with sand fill to the typical profile as shown in (b) of Fig.7. "6 m Sea Wall" is a profile with the sand fill of which the sea wall has been heightened until +6 m above L.W.L. The other each profile has the same sand fill as Fig.7 in addition to the corresponding each step shown in Fig.6. The profile F corresponds to the recommended profile shown in Fig.3.

In this figure, the expected rates q_{exp} of wave overtopping of the profiles A, B, C and D are less than the profile of "Only Sand Fill". The profile of "Only Sand Fill" has the same sea wall of 5 m above L.W.L. as the profiles A, B, C and D, but has not a frontal step as seen from Fig.7. Also, the profiles of E and F are nearly equal in q_{exp} to the profile of "+6 m Sea Wall" though they are +5.5 m in the height of sea wall. These facts should be attributed to the effect of frontal step. That is, it is clear that the height of sea wall can be decreased by adding a frontal step without increasing the amount of wave-overtopping.

Effect of a parapet

In Fig.6, each profile of A, B, E and G has a parapet of 50 cm high in the seaward end of the frontal step. It is interested that the profile B is larger in q_{exp} than the profile D without a parapet. In order to make more clear this matter, q_{exp} is compared in Fig.8 between the profiles with and without a parapet. In this figure, black dots show the result of test conducted under the same condition as white dots of 10 sec in wave period except that the wave period is 8 sec. Also, in the figure, the fact that dots are plotted in the right side of the actual line of 45° against the abscissa shown that q_{exp} increases due to attaching of a

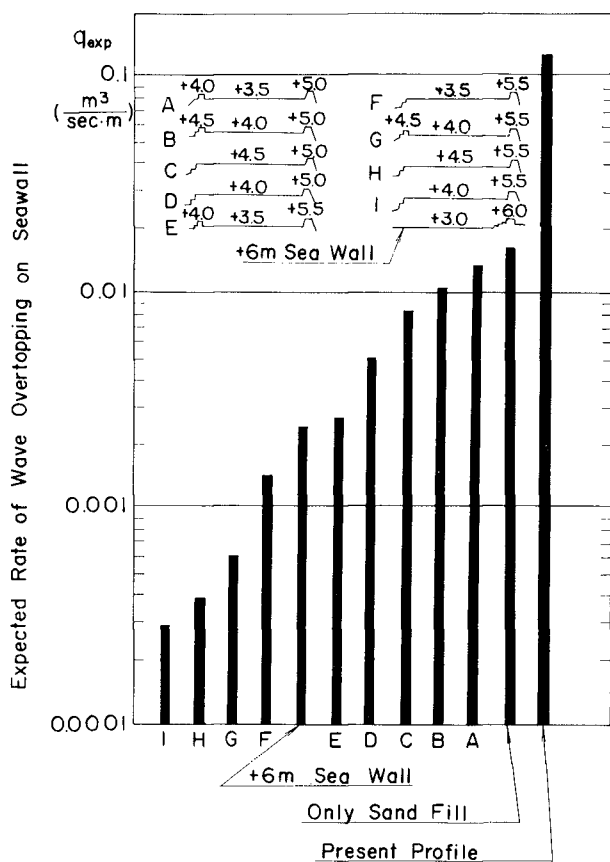


Fig.6 The comparison of the expected rates of wave-overtopping on different types of sea walls ($h=3.7m$, $T=10sec$, $H_{1/3}=4.7m$)

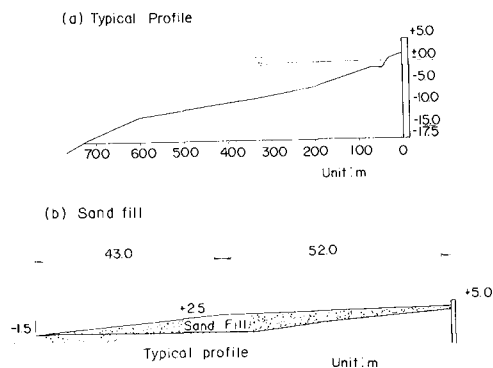


Fig.7 The typical profile and the sand fill

parapet of 50 cm high. All cases except the black dots of the profile D and B are plotted in the right side of the actual line. That is, in the most cases, the attaching of a parapet at the seaward end of a frontal step does not decrease the amount of wave overtopping, for the most cases it increases the amount. This depends on the fact that the wave running up the parapet disturbs the water filled between the parapet and the sea wall by the foregoing waves.

Effect of the height of a frontal step

In Fig.6, the comparison of q_{exp} between the profile F and H shows that the former with a lower frontal step is larger in q_{exp} than the latter with a higher one. But, the profile D is less in q_{exp} than the profile C, though the former is less in the height of a frontal step than latter. The difference of the height between the sea wall and the frontal step is one meter in the profile H and only 0.5 metres in the profile C. This seems to be the reason why the result of the comparison of q_{exp} between the profile F and H is different from between D and C. This shows that the waves running up the frontal step overtop easily the sea wall, unless the sea wall is so enough higher than the frontal step. Therefore, under the condition of a constant height of sea wall, the wave overtopping discharge does not always decrease with the increase of height of frontal step.

An additional test has been conducted to make more clear the above mentioned matter. In Fig.9, is shown the result of the test where the overtopping discharge of regular waves has been investigated for four profiles different in the height of frontal step under the condition of the sea water level of 3.4 m above L.W.L., the wave period of 10 sec and the crown elevation of sea wall of 5.5 m above L.W.L. The overtopping discharge q of regular waves of around 4.5 m high is the maximum for the profile R and the minimum for the profile P. Also, q of waves of around 5.5 m high is the maximum for the profile P and the minimum for the profile Q.

Though the effect of the height of frontal step on the wave overtopping discharge is not so simple as mentioned above, the expected rate q_{ecp} of wave overtopping for irregular waves of 4.7 m in $H_{1/3}$ is calculated on the basis of Fig.9 and shown in Fig.10. In the figure, F, I and H are for the cases that the waver level is 3.7 m above L.W.L. and also plotted on the basis of Fig.6. From the figure, in general, the wave

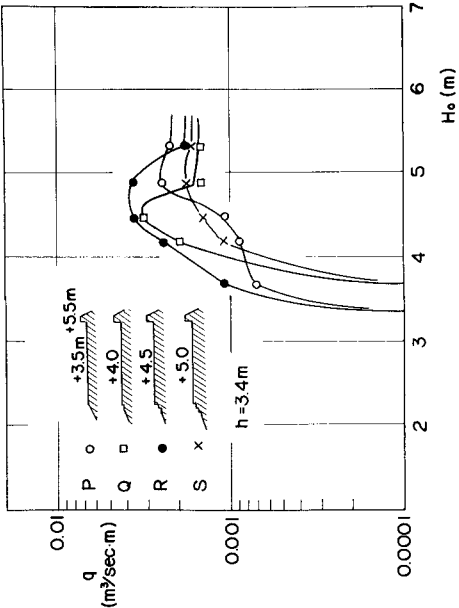


Fig.9 Overtopping discharge of regular waves against the profiles different in the height of frontal step ($h=3.4m$, $T=10sec$)

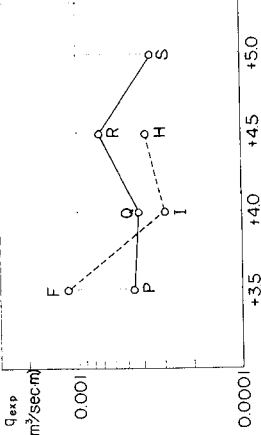


Fig.10 The change of q_{exp} due to the height of frontal step.

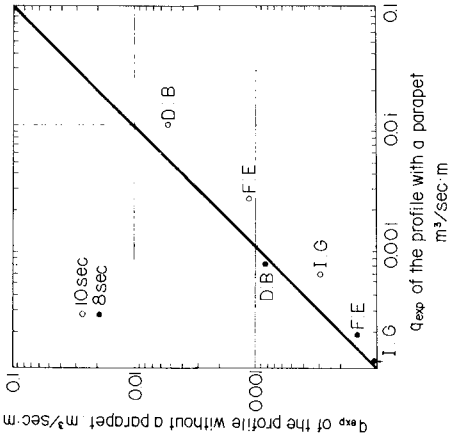


Fig.8 Comparison of expected rate of wave overtopping between the profiles with and without a parapet

overtopping discharge seems first to decrease, then to increase and again to decrease as the height of the frontal step is heightened gradually from the beach level. In the second step showing the increase of discharge such as from Q to R, the decrease of the ability to prevent the wave running up on the frontal step from moreover overtopping the sea wall, which is caused by the decrease of the relative height of sea wall against the frontal step, seems to exceed the increase of the ability to prevent the wave from running up on the frontal step by the heightening the frontal step. In the last step showing the decrease of the discharge, the frontal step is seemed to become so high as to decrease very much the volume of waves running up on it, despite the decrease of the relative height of the sea wall against the frontal step.

EFFECT OF PROTECTIVE MEASURES OF NOURISHED BEACH

In order to examine the stability of sand fill recommended above, a foreshore part between two groins shown in Fig.11 was nourished on trial with sand of 10,000 cubic meters in the summer of 1970, of which grain size is shown in Fig.12. But in one year after the nourishment, the foreshore was eroded by 4,000 cubic meters, despite there was no attack of typhoon. Therefore, field and model tests have been conducted to find out the measures suitable to protect the nourished beach against waves.

Model test

The model test has carried out in a wave basin with a scale of 1/50 in vertical and horizontal. Sand of 0.2 mm in mean diameter was used as the material of the model beach. The area reproduced in the wave basin is shown in Fig.11.

The experimental condition such as waves and water level in the model should be determined so that the profile change in the prototype is reproduced in the model as exactly as possible. Fig.13 shows the comparison of equi-depth lines between just after the sand-filling in 1970 and about one year after it. The part above the equi-depth lines of 1.5 m below L.W.L. is eroded except the portion near to the east-side groin. The sand eroded during this one year amounts to about 4,000 cubic meters, as mentioned above. But, the seaward-part of about 20 m long of the both groins consists of steel piles built so as to coincide with the slope of the nourished beach in the level of their crown. This may be

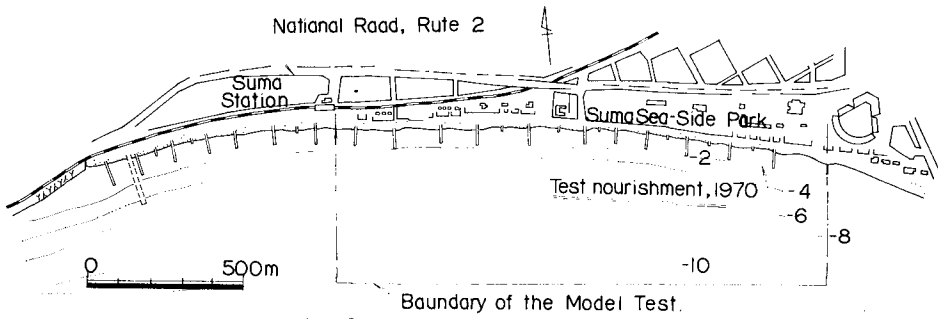


Fig. 11 Suma Beach

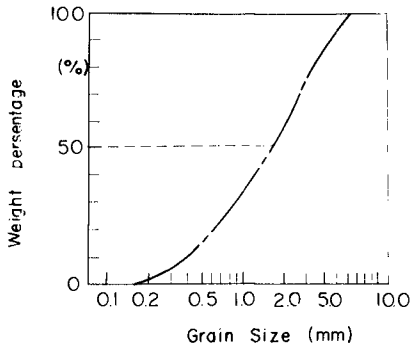


Fig.12 Grain size distribution of the nourished sand in the prototype

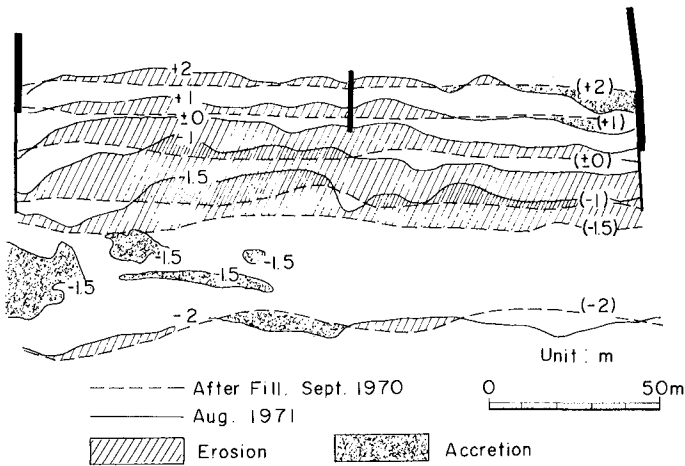


Fig. 13 The change of equi-depth lines in the section nourished artificially in 1970

the main reason why the relatively large amount such as 4,000 cubic meters has been eroded from the nourished beach in 1970. Anyhow, by the preliminary test, the wave of 3 cm high, 0.86 sec period and SSW direction with the water level of 3 cm above L.W.L. has been selected as a model condition suitable to reproduce the change of the equi-depth line in the prototype shown in Fig.13. The model wave corresponds to the wave of 1.5 m high and 6 sec in the prototype and the model water level to 1.5 m above L.W.L. in the prototype. The erosion corresponding to 4,000 cubic meters in the field has occurred in 4 hours of wave action in the model, when the equi-depth lines changed as shown in Fig.14. The change is approximately similar to Fig.13, though the contrast between erosion and accretion is stronger than the prototype.

Then, at first, the extension of groins and the offshore breakwater have been tested under the above selected wave and water level. In this case, the extended part of groins corresponds to 60 m in length and 2 m above L.W.L. for the seaward part of 20 m long and 3 m for the other part in the crown elevation. The offshore breakwater corresponds to 3 m above L.W.L. in the crown elevation. Fig.15 shows the change of bottom of four sections in the central part of the model beach, where equi-depth lines are shown by dotted lines for the initial profile and by actual lines for one after the wave action of 4 hours. The material volume above L.W.L. has been compared between before and after of the wave action, of which the results is shown in the figure with the notion of "Erosion" or "Accretion". From the figure, it is seen that the extension of groins is not enough to protect the nourished beach only by itself from the waves even the year of the normal weather condition without severe typhoons, but that the addition of the offshore breakwater is enough to protect the beach.

However, the nourished beach should be retained even against the large waves in the typhoons. Therefore, the model test for the waves of 3 m high and 8 sec has been carried out under the conditions of the same wave direction and water level as the fore-going test. The results are shown in Fig. 16, where the change of the material volume above L.W.L. in the same as the fore-going test is also shown for each section. In the test, the elevation of the submerged breakwater is 1.5 m below L.W.L. at the crown. The followings are seen from the figure;

- (1) The offshore breakwater of the case 4 is the least in the erosion of beach, and changes the shore line to be convex.

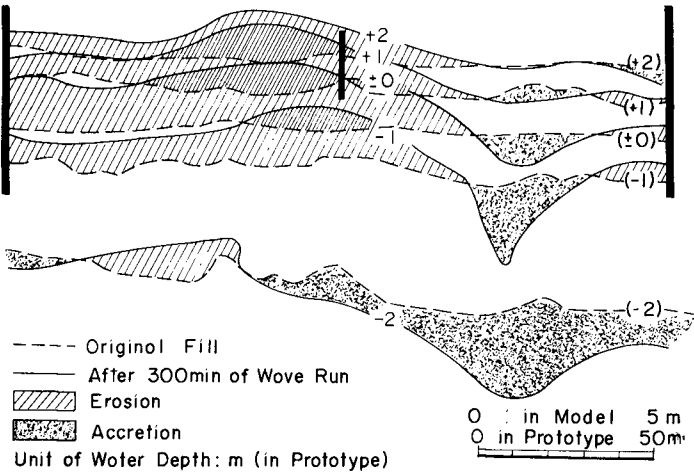
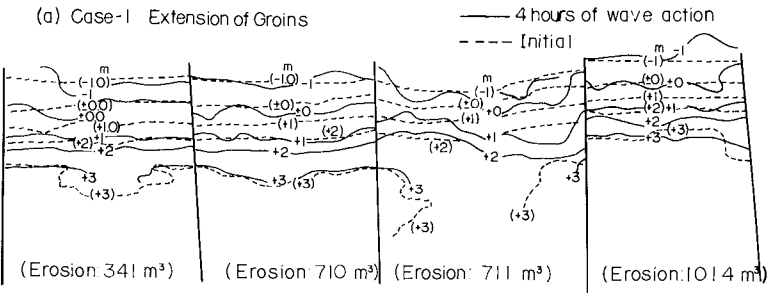


Fig. 14 The change of equi-depth line in 4 hours of wave action in the model ($H=1.5\text{m}$, $T=6\text{sec}$, $h=1.5\text{m}$)

(a) Case-1 Extension of Groins



(b) Case-2 Offshore-Breakwater in Addition to Extension of Groins

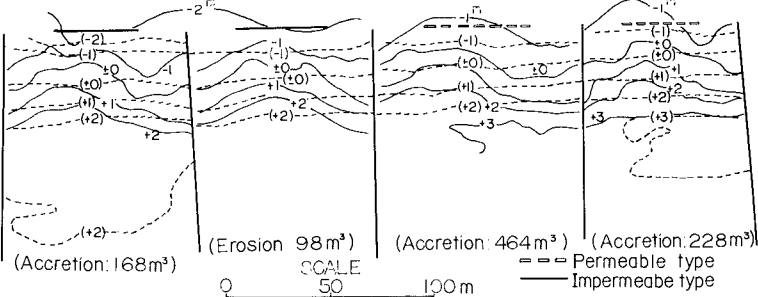
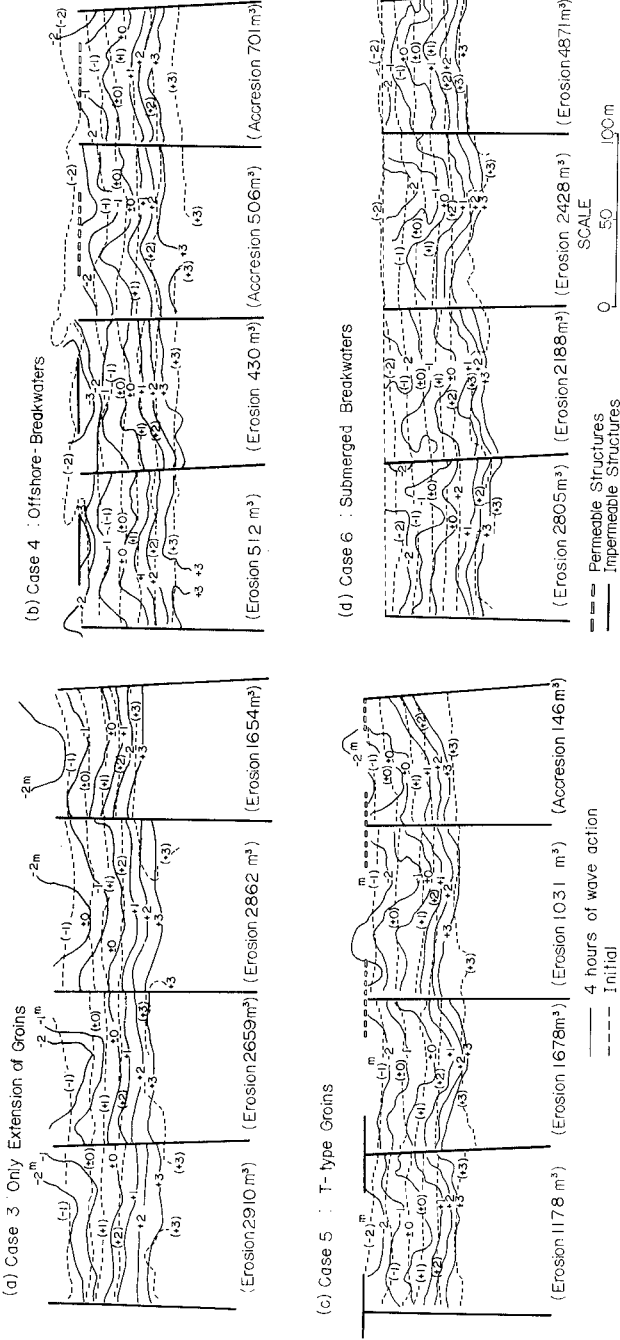


Fig. 15 The change of equi-depth lines due to the wave of $H=1.5\text{m}$ and $T=6\text{sec}$ in the model.



(2) The T-type groin of the case 5 is inferior to the offshore breakwater and superior to the submerged offshore breakwater in the ability retaining the beach material, and changes the shore line to be concave.

(3) The shore line seems to be more straight at the offshore breakwater than at the T-type groin.

Field test

On the other hand, in the summer of 1972, a T-type groins, a permeable offshore breakwater and a submerged offshore breakwater have been constructed in the field as shown in Fig.17, and each section has been nourished with sand of 7,300, 6,600 and 8,000 cubic meters respectively. The structures of T-type groins, offshore breakwater and submerged breakwater are impermeable, permeable and impermeable, respectively as shown in Fig. 17.

Fig. 18 shows the comparison of equi-depth lines between September 1972 and January 1974 for each section. Fig. 19 shows the change of profiles along each broken lines drawn in Fig. 18. Moreover, Fig. 20 shows the volume change of each material above L.W.L. for each section, where the change is expressed by the volume difference of each surveying data against September 1972 which is just after the beach nourishment. From these figures 18 to 20, are seen the follows:

(1) The section of the offshore breakwater is less in the erosion of beach than the T-type groins, as same as in the model tests (see Fig.20).

(2) The section of the submerged breakwater is the most changeable in the volume of beach material (see Fig. 20). This means the submerged breakwater is inferior to the other measures in the ability of protecting beach.

(3) The shore line is more straight in the section of the offshore breakwater than the T-type groins, as same as in the model test.

(4) The erosions has occurred mainly in the foreshore which extends from about - 1~-1.5 to +2.0~+2.5 m L.W.L. (see Fig.19). This might be relative to the fact that the beach nourishment have been conducted so as to push out offshoreward the foreshore of -1.5 m L.W.L. to +3.0 m L.W.L.

(5) In the section of T-type groins, the slope of foreshore tends to be gentle in the center and steep in the both side (see Fig.19). In the section of offshore breakwater, it should be steep in the center and gentle in the both side, but such a tendency have not appeared clearly.

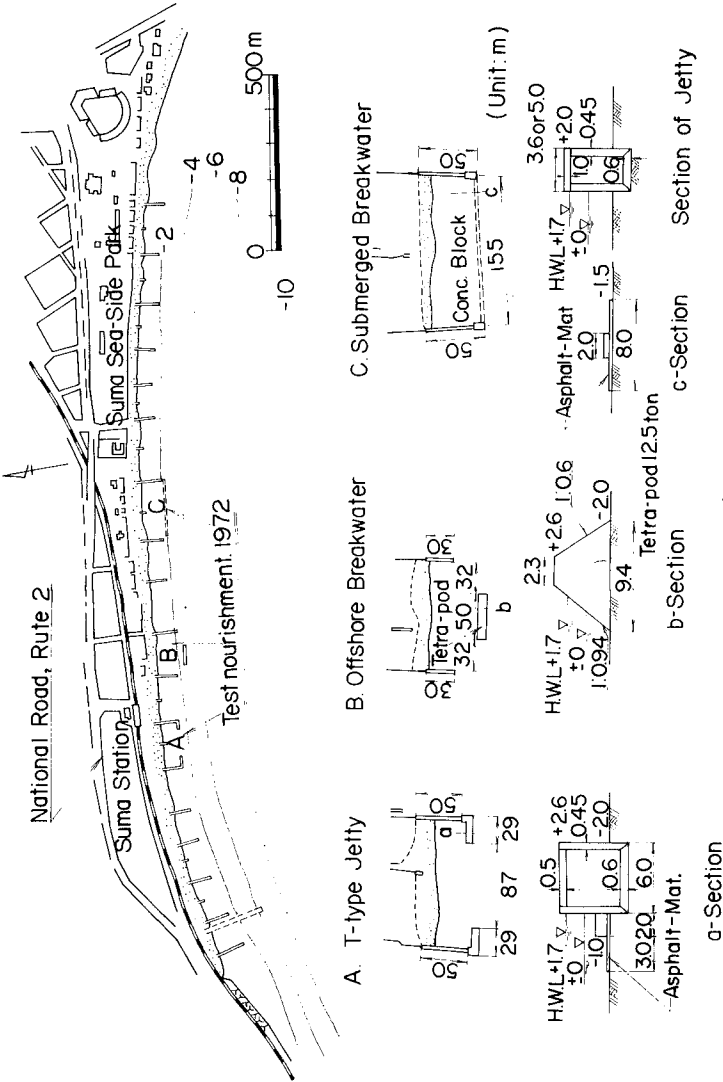


Fig. 17 Protective measures constructed in the field in 1972 for the comparison of their ability on beach protection

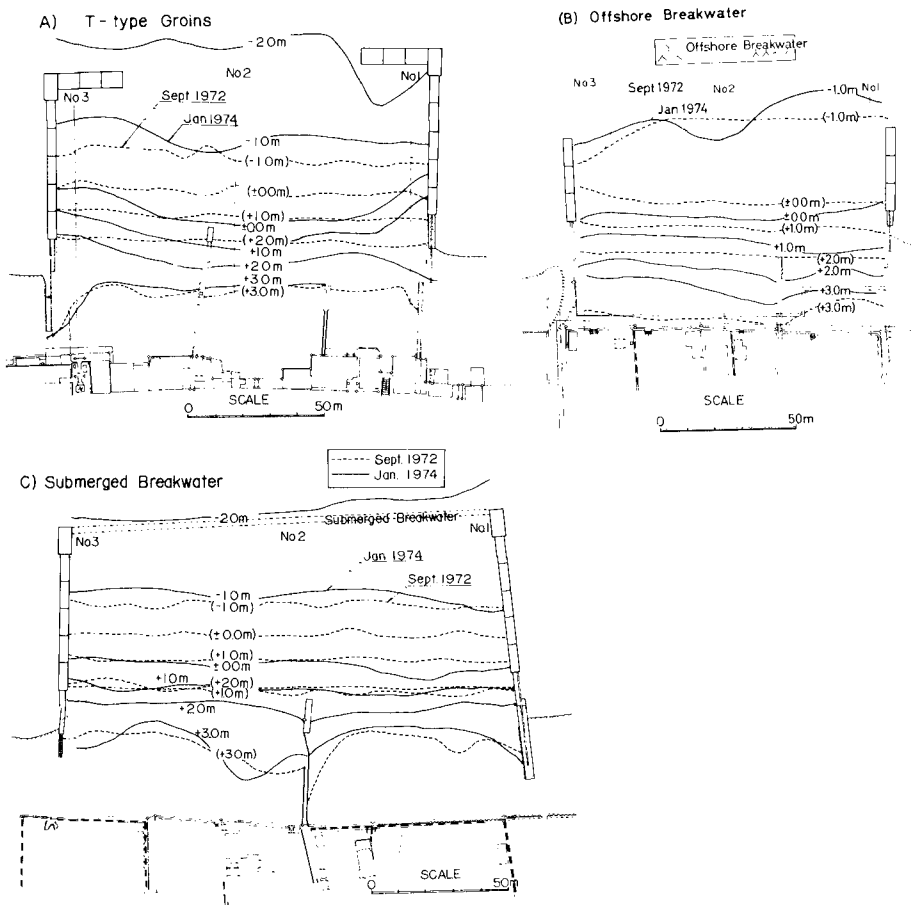


Fig. 18 The comparison of equi-depth lines between September of 1972 and January of 1974

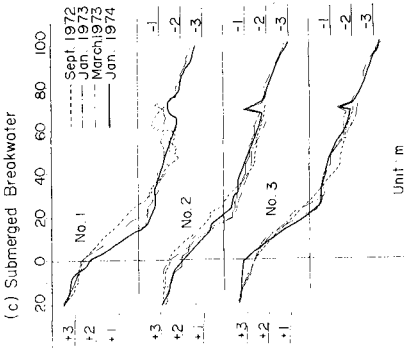
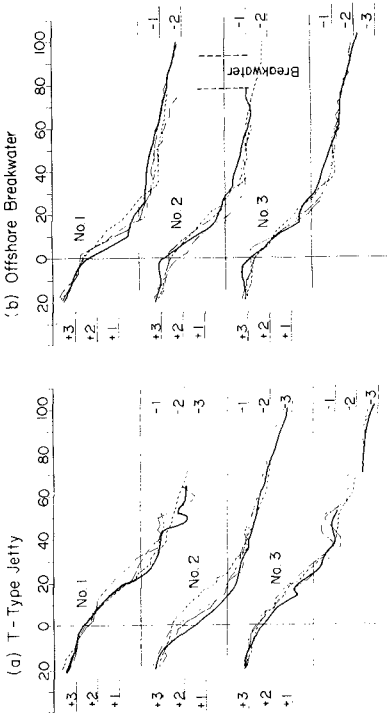


Fig. 19
The change of beach
profile after sand-
fill in the prototype

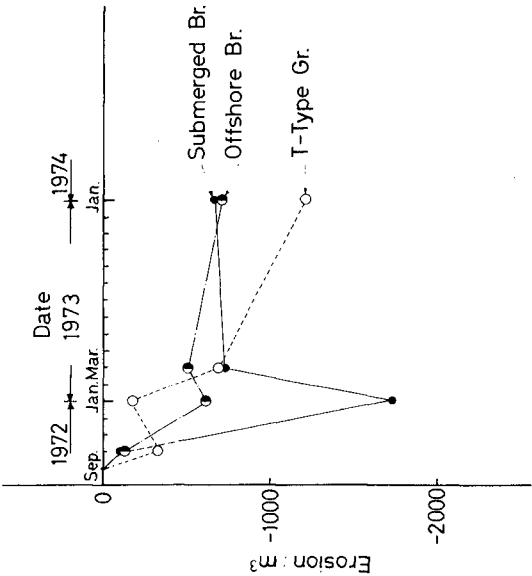


Fig. 20 Volume change of the material
on the beach above L.W.L.

As seen from the above mentioned, the results of model test considers fairly well with the field test in the qualitative consideration. But it is difficult to compare the model test with the field test in the quantitative consideration.

CONCLUSION

In a recreational beach, the heightening of sea wall is undesirable from the view-point of environmental preservation and beach utilization. Therefore, a proposal has been made of the sea wall with a wide frontal step as well as the widening of beach by artificial nourishment. And the follows have been made clear by the model and field tests.

On the wide frontal step;

- (1) The height of sea wall can be decreased by adding a wide frontal step without increasing the amount of wave overtopping discharge.
- (2) The attaching of a parapet of 50 cm high at the seaward end of a frontal step does not serve to decrease the amount of the wave overtopping; for the most cases it increases the amount.
- (3) The amount of wave overtopping first decreases, then increases and again decreases as the height of frontal step is raised from the beach level.

On the protective measures of the nourished beach;

- (1) The offshore breakwater is superior to the T-type groins in the ability retaining the beach material. And T-type groins are superior to the submerged breakwater.
- (2) The shore line changes into convex shape in the section of the offshore breakwater and into concave shape in the section of the T-type groins.
- (3) The shore line is kept more straight in the section of the offshore breakwater than the T-type groins.
- (4) The results of model test coincide fairly with the field test in the qualitative consideration.

At last, the authors express their thanks to Mr. Tensetsu Hattori and Mr. Katsunori Sasaki in the same Institute as the authors for their cooperation in this work.

REFERENCES

- 1) Senri Tsuruta and Yoshimi Goda: Expected discharge of irregular wave overtopping, Proc. of 11th Conference on Coastal Engineering, Vol. II, Sept. 1968.
- 2) Yoshimi Goda: Expected rate of irregular wave overtopping of seawalls, Coastal Engineering in Japan, Vol. 14, J.S.C.E., Dec. 1971, pp.43 - 51.