CHAPTER 102

STABILITY OF BREAKWATERS CONSTRUCTED ON DREDGED SAND MOUND

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ABSTRACT

The possibility for the construction of "sand mound breakwater", which is a caisson type breakwater on a artificial sand mound, has been studied. First the wave force characteristics acting on the composite sand mound breakwater model are described, to identify the conditions where impulsive breaking wave pressures are generated. Second, the scouring of the toes of rubble mounds is discussed. Finally there is an outline of field experiments, scouring of sand mounds, and the status of the deposits. The results of experiments provide useful information for the design of breakwater constructed on the artificial sand mound or on a similar double-sectioned sea bottom.

1. INTRODUCTION

The construction of composite breakwaters in relatively deep sea areas requires a large rubble mound foundation that takes up a considerable proportion of the total construction cost. This is due to limitations imposed on the capacity of the caisson plant, which force an increase in the height of the rubble mound. In harbors where dredging of waterways and anchorages yields large quantities of good sand, the construction costs of breakwaters and the cost incurred in disposing the dredged sand may be reduced if the sand is used in the breakwater mound.

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The construction of "Sand Mound Composite Breakwaters" requires a solution to the technical problems with wave force characteristics, scouring, bearing capacity, liquefaction characteristics, all of which are important in breakwaters on the artificial sand mound. Since 1986, the Hokkaido Development Bureau has been conducting experiments with hydraulic models and field demonstration tests at Tomakomai East Port to help solve these problems.

This report presents an evaluation of tests with the hydraulic model, with partial results of the field tests. First the wave force characteristics acting on the composite sand mound breakwater model are described, to identify the conditions where impulsive breaking wave pressures are generated. A number of studies on impulsive breaking pressures (e.g. Tanimoto, 1976) acting on normal composite breakwaters have been carried out, but studies relative to the breakwater on a double-sectioned sea bottom are virtually non existent.

Second, the scouring of the toes of rubble mounds is discussed. Much effort has been made to avoid the scouring of toes of rubble mound, however, no generally applicable countermeasures have been established to meet particular local requirements. This presents a design method for effective countermeasure work based on tests with hydraulic models: (i) determining the scouring positions of breakwaters by structural types, and (ii) determining the length of effective counter-scouring work.

Finally there is an outline of field experiments, scouring of sand mounds, and the status of the deposits.

2. WAVE FORCE CHARACTERISTICS ACTING ON BREAKWATER

2-1 Testing Equipment and Method

The tests were conducted in a two-dimensional wave tank equipped with an irregular wave generator (24m long, 80cm wide, 1m high). Dimensions were 1/75, with the channel bed slope 1/100. The wave spectrum employed was the modified Bretschneider-Mitsuyasu frequency spectrum. The measurement time was set at the significant wave period x200 sec., to ensure measurements of more than 200 waves. The data was transformed to digital at the sampling frequency of 100Hz. Figure 1 shows a typical section of the test model.

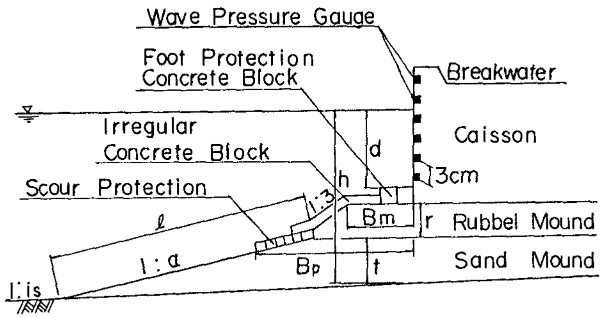


Fig.1 Typical section of test model

The wave pressures were defined as the maximum average pressures of individual waves, and the representative wave pressures were defined with a same way for the wave heights such as the maximum wave pressure, the 1/10 maximum one-tenth wave pressure, and the maximum one-third wave pressure or significant wave pressure, expressed as p_{max}, $p_{1/10}$, and $p_{1/3}$ respectively.

There are two major types of tests. Test 1 on a fixed bed with the sand mound slope, and thickness and the depth varied. The rubble mound had a standard width of front berm Fm of a minimum 14cm (in-situ size 10.5m) assuming the foot protection concrete block and the armor concrete block as paired. For reasons of economy the thickness was kept as low as possible, 4cm (3m in-situ) including the thickness of the armor concrete block. The breakwaters were then evaluated with significant waves of 6 wave heights x 3 periods.

Test 2 was performed on a movable bed and the variation in wave pressures after scouring was investigated. A movable bed was prepared in the same section as the fixed bed by replacing the sand mound and seabed with fine sand (median particle size d_{50}=0.16mm). Considering characteristics of littoral drift, this was equivalent to a median particle size of d_{50}=1mm in-situ. The slope of the sand mound and the width of the counter-scouring works were varied. After measuring the sand surface after scouring by the precision bottom gauge with wave motion for 4 hours, the wave pressures were measured at similar conditions as in Test 1.

2-2 Results and Discussion

(a) The Effect of Slope of Sand Mound

Figure 2 shows variations in the coefficient of wave pressure intensity with the slope of the sand mound at 1/15, 1/30, and 1/50 and with t=6cm, and h=24cm. The figure indicates that the coefficients often become large when the steepest slope is a=1/15, and it also indicates that there are cases where coefficients become greater with the longer periods and flatter slopes. This shows that impulsive breaking wave pressures are possibly generated more commonly with flat sloped sand mounds of moderate slope length.

(b) Shock Wave Pressure Generation

The above confirmed that very strong shockwave pressures could act on the sand mound composite breakwater under various conditions. Figure 3 graphically shows the possibility of shock pressures leading to breaking waves. It uses two sets of nondimensional quantities, and the ratio of significant wave height ($H_{1/3}$) to own depth of mound (d) on the ordinate and mound slope length (l) to wavelength (L) on the abscissa and the coefficients of the maximum cotidal average intensity of wave pressure ($\overline{p}_{max}/w_0H_{max}$). The broken and chained lines indicate boundaries where the coefficients exceed 1.5 and 2.0 suggesting the possibility of shock pressure from breaking waves to be significant above these boundaries.

The design of conventional composite breakwaters for sea areas with bedrock and cross sections similar to sand mounds, require special attention to the possibility of producing similar wave pressure characteristics.

(c) The Effect of Scouring of Sand Mound

The wave pressure tests above were all on fixed beds, and relate to the problems of the effect of various conditions of wave on the stabilization of the breakwaters immediately after completion. Another important problem in the stability of sand mound breakwaters is scouring and the possible effects of wave pressures when the shape of the sand mound changes due to scouring in front of breakwaters.

Figure 4 shows results with h=24cm and t=6cm as in Table 2, when $H_{1/3}$=8.5cm, and $T_{1/3}$=1.3s was active for 4 hours. The scouring is greater with a=1/15 than with 1/50. It is evident that the face of the narrower anti-scouring installations suffers more from scouring. This indicates problems with actual breakwaters as the maximum scouring depth with the model can be assumed to be 1 to 1.5m.

In $\overline{p}_{1/3}$ there are no appreciable changes among the values before and after scouring for different wave pressure intensities. For $\overline{p}_{max}$, the intensities before

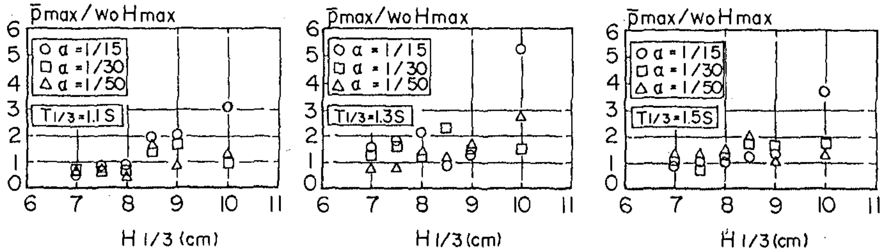


Fig. 2 Differences of wave pressures by sand mound slopes

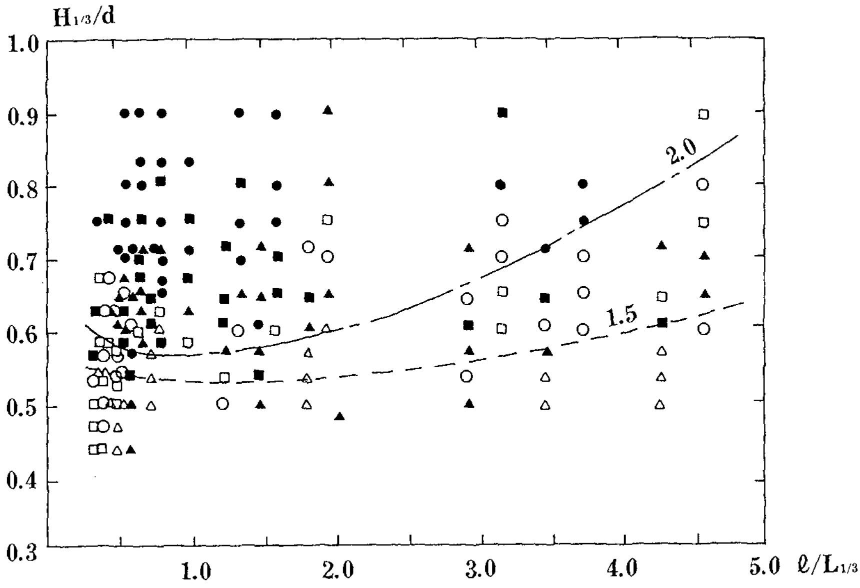
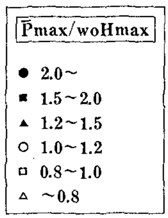


Fig. 3 Possibility of shock pressure



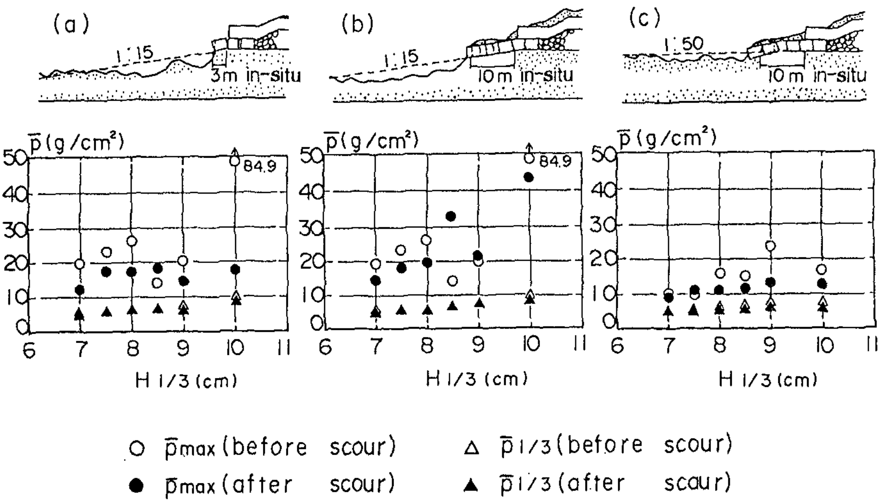


Fig. 4 Differences of wave pressures due to before and after scours

scouring was greater. Other periods show similar results, and it is assumed that there are no significant differences in the characteristics of wave pressures with different scouring configurations.

3. SCOURING IN FRONT OF BREAKWATERS AND ITS PREVENTION

3-1 Test Method

The tests were carried out in the two-dimensional wave tank used for the wave pressure tests. The scale, except for the bottom materials, were based on the Froude Model Law of 1/40 without distortion. The bottom materials were the same as used in the previous wave pressure tests. The particle size of the sand equal a d_{50}=0.5mm size of bottom materials. The rubble-filled wire cross baskets for the scour prevention were 1/40 of the regular baskets (2.0m x 3.0m x 1.0m) with the weight adjusted to equal 10 tons in-situ. With regular waves as test waves scouring was studied on 3 kinds of wave periods T=7, 10, and 13 sec.

3-2 Test Results and Discussion

(a) Structural Type of Breakwater and Place of Scouring

Irie (1984) pointed out that because of standing waves bottom is scoured at nodes and accumulated at loops in front of the actual breakwaters. A remarkable damage of rubble mound would take place when the first node of standing waves L/4 comes near the toe of rubble mound where L is the length of a progressing wave.

Introducing this into a design poses problems: (i) establishing a basis for depth determination when calculating L of a composite breakwater, and (ii) determination of the location of the reflection face (the origin for L/4) for breakwater armored with wave dissipating blocks or slit caisson breakwater. To evaluate (i) and (ii), these 3 types of breakwater were examined. Figure 5 shows the models of wave dissipating type breakwater. The height of the crown was set so the breakwater would be non-overtopping.

Figure 6 shows an example of the scouring location where: h=37.5cm (15m in-situ), and T=2.06sec (13 sec). The ○ mark in the figure indicates the use of (h) as the depth when determining the wave length and △ indicates when (d) was used. Kr is the reflection coefficient. The locations of L/4, L/2, and 3L/4 are shown on the structures. Hence, black indicates the scouring locations and white the deposition. Based on this, the conversion into L/4 shows only differences of a few meters with long T=13 sec periods, which indicates that it is relevant to the pickup (h) as the depth to determine the wavelength.

Figure 7 shows the scouring locations of the wave dissipating armor block structure and the 2-stage slit breakwater. The ○ and □ in the figure show the locations of L/4, L/2, and 3L/4 seen from structures using (h) to calculate the wavelength as with the composite breakwater. The ○ mark indicates upright wall reflection of the wave dissipating armour block structure and the □ mark indicates the intersection between the slope face of the wave dissipating blocks and the still water level.

For the slit caisson breakwater, ○ indicates the offshore side slit wall face and □ indicates the rear upright wall. In wave dissipating armor block breakwaters, visual inspection was considered necessary to establish the upright wall face as a reflector, based on the reflected waves around the structure and the scouring location shown in the figure.

With the slit caisson breakwater, the reflecting face was set on the offshore side of the upright face up to T=10sec. However, for periods of T=13sec. the reflection face of the scouring location is thought to be located somewhere between the offshore side slit wall face and the rear upright wall. In practice it is

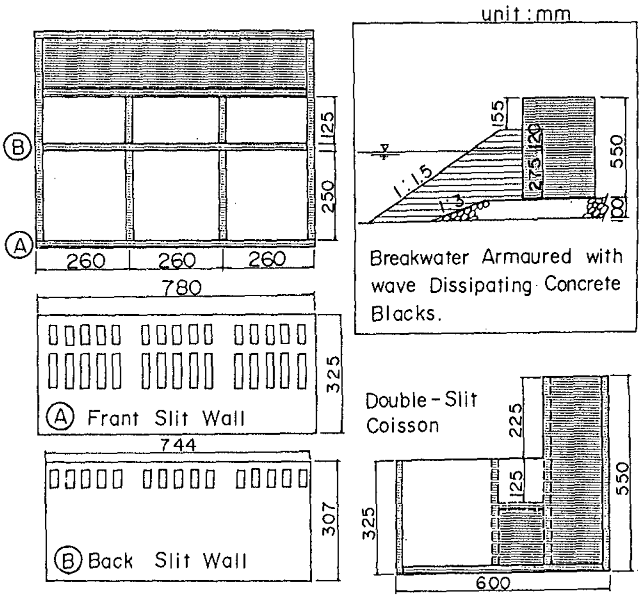


Fig. 5 Models of wave dissipating type breakwater

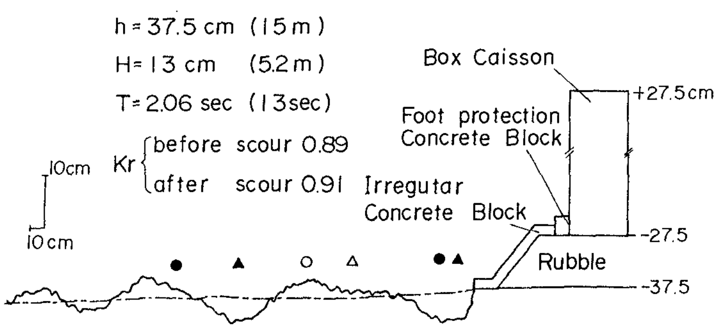


Fig. 6 Scouring location in front of composite breakwater

strongly recommended to set all on the offshore side of the slit wall face.

Comparing the scouring of common composite breakwaters with low-reflection type breakwaters shows that the absolute quantities of scouring and deposition are smaller with the low-reflection type, suggesting that the reduction in reflected waves is more effective to control the scouring.

(b) Length of Scour Prevention Works

The experiments with the composite breakwater used the wavelength as the parameter to attempt to determine the length of rubble-filled wire cross baskets for the scour prevention. Wavelength L was determined with the installation depth, and the scouring conditions were compared by successively increasing the length of the wire crosses from the offshore side (i) L/4 (3L/12), (ii) L/3 (4L/12), and (iii) 5L/12, with the upright wall face as the origin (face of reflection). Two sea bottom slopes were studied 1/100 and 1/15.

Figure 8 shows one set of results. The bottom slope did not affect moving the wire crosses nor cause scouring when the basket length was 5L/12, and these dimensions was considered effective for scour control. Scour control with L/3 were also generally effective although there was a slight dipping of one basket at the end. With L/4 baskets, the movement of baskets in the front row was significant, and particularly with steep bottom slopes, and the scour quantity at the toe of the slope was the largest. These results showed that using only the wave length parameter L in designs was adequate for scour prevention. There were good results with L/3 from the upright wall of the structure as the effective length of scour prevention works.

4. FIELD TESTING

4-1 Construction of Sand Mound

The tests were conducted in a 16m deep sea area at Tomakomai East Port on the Pacific coast by Tomakomai Port and Harbor Construction Office. Figure 9 shows schematic diagram of sand mound breakwater at Tomakomai Port. Figure 10 shows a model of the breakwater with two 12.5m (L) x 20m (W) x 12m (H) caissons installed on the sand and rubble mound. The construction started in November 1986 and was complete in October of the following year.

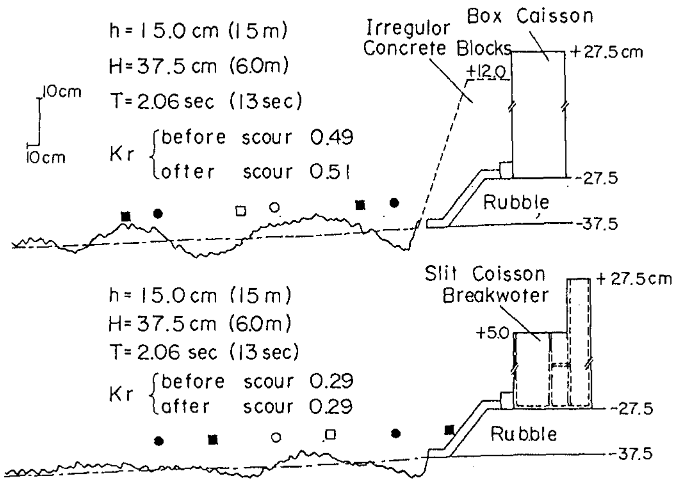


Fig. 7 Scouring location in front of wave-dissipating type breakwater
(Top) Breakwater armoured with wave dissipating concrete blocks
(Bottom) Slit caisson breakwater

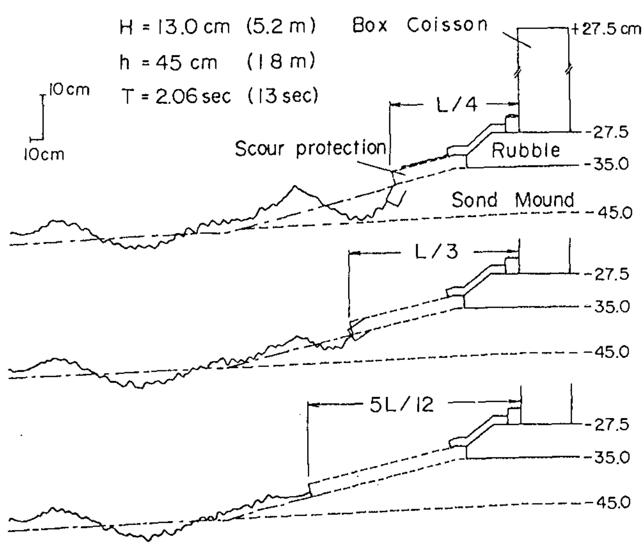


Fig. 8 Scouring shapes in front of breakwater under various length of scour protections.

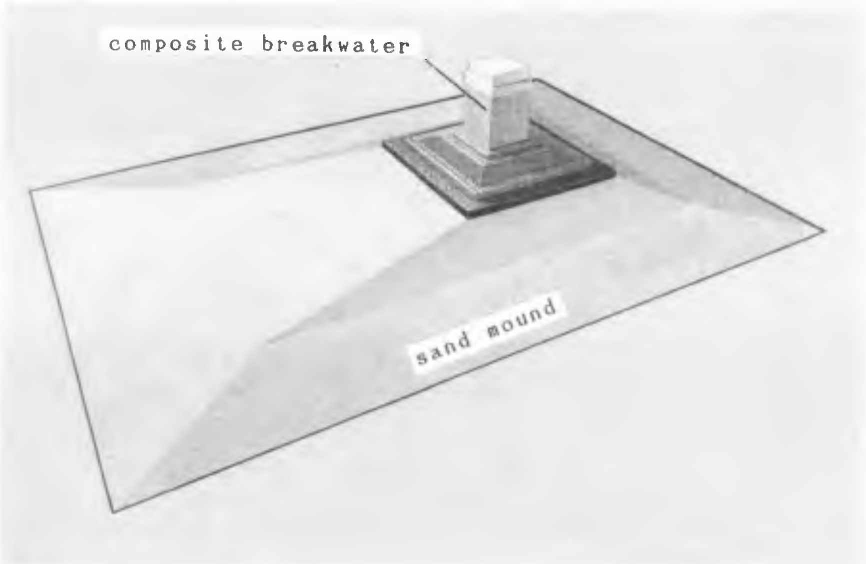


Fig. 9 Schematic diagram of sand mound breakwater for field test at Tomakomai Port

Fig. 10 Test breakwater at Tomakomai Port

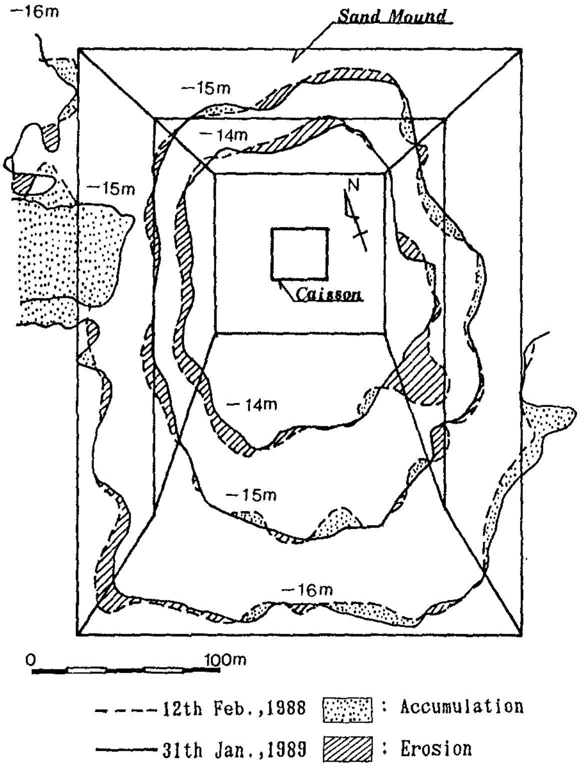


Fig. 11 Comparison of contours

The sand mound used dredged sediment from waterways (the mediam particle size d_{50}=0.8mm) and other fill material (the mediam particle size d_{50}=1.7mm, pump dredged and pile-stored on land for several years during which the fine particles were washed out leaving stones as main component). The materials for the sand mound were transported by bottom door hopper barges installed canvas curtain for dispersion prevention, and approximately 1,000,000 cubic meters were dumped for a maximum mound thickness of 3m.

4-2 Stability Against Scouring

Figure 11 shows sounding surveys conducted between the caissons half a year after the initial installation (February, 1988) together with sounding survey of January 1989. The largest significant waves observed during this time was 4.0m, and the variation in the subbase heights lower than 0.5m, indicating that the sand mound remained stable.

5. SUMMARY

The results obtained from the study may be summarized as follows:
(1) The relation between the slope of the sand mound and the generation of shock pressure breaking waves is affected by the angle of the slope, and also particular slopes of sand mounds and lengths of slope are susceptible to the wave conditions generating shock pressure breaking waves.

(2) The nondimensional ratios: significant wave height (H1/3) to crown depth of rubble mound (d), and slope length of sand mound (l) to wave length related to the significant wave period in Figure 3, and indicates the possibility of shock pressure breaking wave generation.

(3) There were no significant differences in the intensities of wave pressure before and after scouring.

(4) Comparing the scouring of common composite breakwaters with low-reflection type breakwaters shows that the absolute quantities of scouring and deposition are smaller with the low-reflection type, suggesting that the reduction in reflected waves is more effective to control the scouring.

(5) It is recommended that the effective length of scour prevention work is L/3 from the upright wall of the structure.

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