WAVE FORCE AND STABILITY OF ARMOR UNITS FOR COMPOSITE BREAKWATERS

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Abstract

The stability of armor units for composite breakwaters was studied by two- and three-dimensional model tests and prototype failure analyses. The wave force on armor blocks was cleared for different relative mound depths and berm widths. The stable weight of armor blocks is proposed in which the block shape factor is used as a parameter. The necessary thickness of foot-protection blocks is formulated as a function of the relative mound depth for the breakwater trunk and head.

Introduction

Rubble mounds for composite breakwaters are usually protected by armor blocks and foot-protection blocks. These blocks are conventionally designed according to the knowledge obtained through past experience. However, due to a recent increase in the construction of breakwaters at deeper locations with a higher design wave height, a number of cases have occurred in which such experience-based methods are no longer effective. In addition, the armor stability for three-dimensional conditions, such as oblique incident waves and wave action around breakwater heads, remains unknown and damage under such conditions has been increasing.

In this study, the stability of armor blocks and foot-protection blocks was

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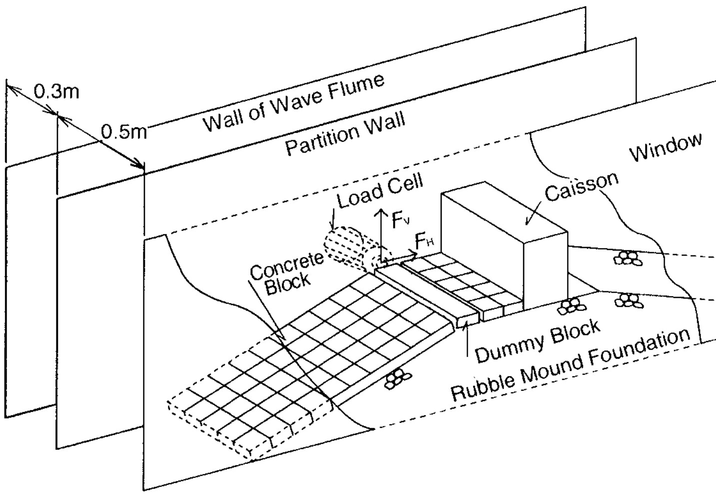

Figure 1 Breakwater model for measuring wave forces

examined by two- and three-dimensional model tests. From these results, methods to calculate the necessary armor units for composite breakwaters are proposed, and their applicability for practical design is verified by data analyses of previous damage.

Wave forces on armor blocks

A two-dimensional wave flume (24m × 0.8m × 1.0m) was divided into two parts. The horizontal and vertical wave forces acting on the dummy block (48cm × 10cm ×

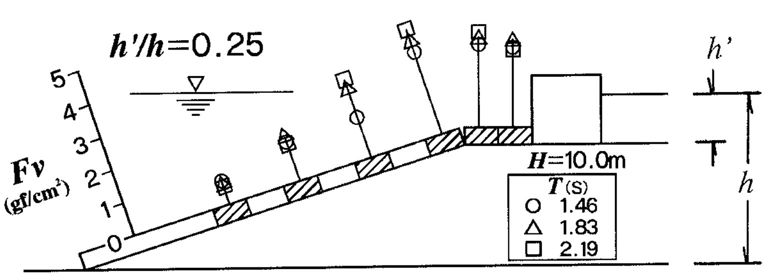

Figure 2 Wave force distribution

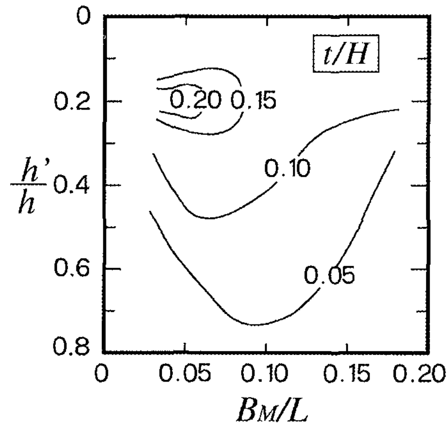

Figure 3 Contour diagram for wave force acting on blocks

5cm) were obtained using a load cell (**Figure 1**). The structural conditions were: water depth h was set at 50 cm and the depth of the mound h' and mound berm width B_M were altered. The wave force experiment was made with regular waves and the wave conditions were altered using three types of wave period, T, and a wave height, H , of 2 to 14 cm. In addition to the wave forces, the water levels above the dummy block were also measured

Figure 2 shows the vertical wave force at each location of the mound when the relative mound depth h'/h was 0.25. The wave force reached its maximum at the mound shoulder and decreased as the water depth increased. The wave force at the front of the caisson was slightly smaller compared with that of the shoulder.

Figure 3 shows the vertical wave forcc acting on the block at various values of relative berm width B_M/L (L: wave length for the water depth h) and relative mound depth h'/h, which was obtained by dividing t, or the required thickness of blocks to resist the vertical wave force, by the wave height H. The maximum wave force acting on the block was at h'/h = 0.2, with the corresponding required block thickness t/H being about 0.2. Under the usual mound conditions of h'/h = 0.6 to 0.8, the maximum wave force occurs when B_M/L is at or around 0.1.

Stability Formula for Armor Blocks

The following Hudson's formula gives the stable weight of an armor block,

$$W = \frac{\gamma_d H_{1/3}^3}{N_S^3 (S_r - 1)^3} \tag{1}$$

where, $H_{1/3}$ is the significant wave height needed for designing, γ_d is the unit weight of the block, and S_r is the relative density of concrete in the sea water. The stability number N_S was formulated by Tanimoto et.al.(1982) and was extended for armor stones by Kimura et.al.(1994). For armor blocks, the following equations were modified to separate the block shape factor and mound shape conditions of composite breakwaters,

$$N_S = N_{SO} \cdot \max\{1.0, A\frac{(1-\kappa)}{\kappa^{1/2}}\frac{h'}{H_{1/3}} + \exp[-0.9\frac{(1-\kappa)^2}{\kappa^{1/2}}\frac{h'}{H_{1/3}}]\} \tag{2}$$

where, h' is the water depth of the mound foundation, N_{SO} is the standard stability number of each armor unit, and is determined by stability tests for the high mound conditions. The coefficient A was decided from the results of the stability tests. The non-dimensional flow parameter κ is expressed as,

$$\kappa = \begin{cases} \dfrac{4\pi h'/L'}{\sinh 4\pi h'/L'} \cdot \sin^2 kB_M & (B_M/L' < 0.15\ ) \\ \dfrac{4\pi h'/L'}{\sinh 4\pi h'/L'}\{2\sin^2(0.15\cdot 2\pi) - \sin^2(2\pi B_M/L')\} & (0.15 < B_M/L' < 0.25) \end{cases} \tag{3}$$

where L' is the wave length of the design significant wave period where the water depth is h'.

Two-dimensional model tests for armor blocks

The stability tests were made for three types of blocks, A, B and C, using irregular waves. The number of waves was 500 and the stability number, N_S, was calculated by using the critical stability weight corresponding to the damage ratio of 1%. The relative mound depth, h'/h, was altered within the range of 0.25 to 0.75. The high

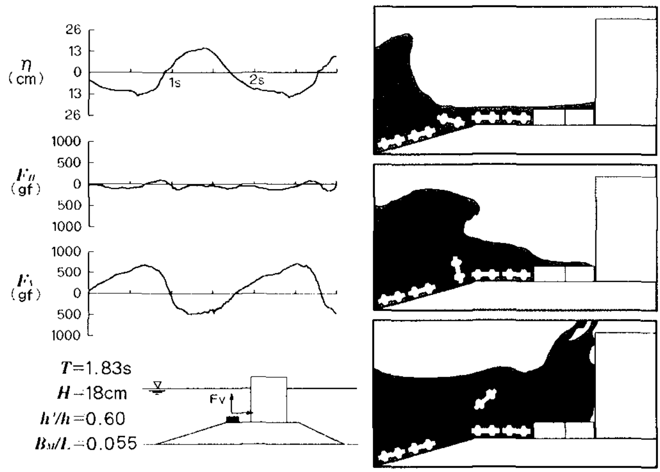

Figure 4 Wave action and block motion

mound condition of h'/h= 0.2 represents the hardest condition for the stability of the armor block, and the stability number at this time was defined as the critical stability number N_{SO}.

Figure 4 shows the motion of blocks together with time-series describing wave forces obtained by load cells when the wave period T was 1.83 s, wave height H was 18 cm, h'/h was 0.6 and B_M/L' was 0.055. The vertical wave force, F_V, was larger than the horizontal wave force, F_H. Considering the block motion, the peak of the vertical wave force coincided with the time when the block was uplifted. The uplifted block was then rolled by the wave-induced flow. Predominance of the vertical wave force is characteristic of the wave force acting on armor blocks. After the uplift motion, the blocks are overturned by the wave-induced flow. The stability number N_S was obtained for blocks A, B and C under various structural and wave conditions. The coefficient A in Eq.(2) was found to be 0.525.

Figure 5 shows the relationship between $h'/H_{1/3}$ and the dimensionless stability number N_S/N_{SO}, when κ is from 0.06 to 0.40. The values of N_S/N_{SO} for blocks A, B

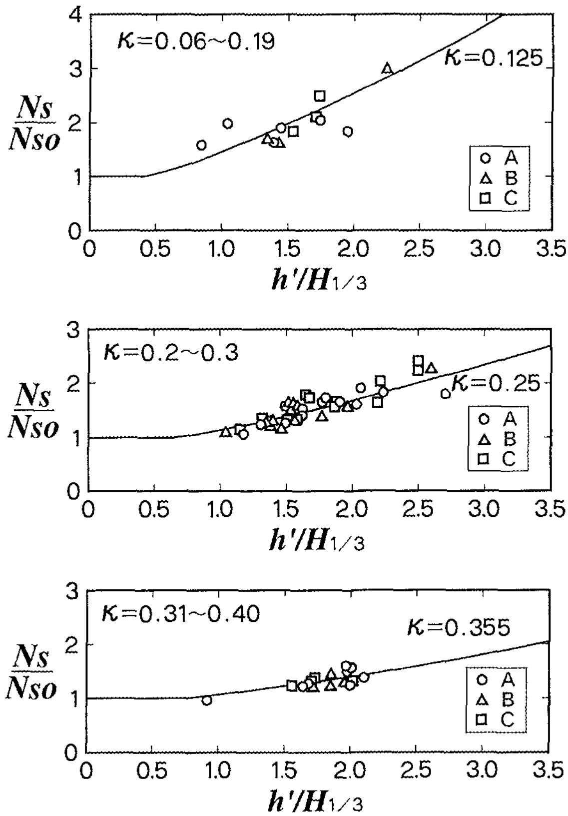

Figure 5 Stability number

and C are indicated by different marks, but each mark corresponds well with the calculated values (solid line), thus verifying the calculation method. N_{SO} was found to be 2.0 for all blocks used.

Three-dimensional model tests for armor blocks

The three-dimensional test was made in a wave basin 33 m wide and 20 m long.

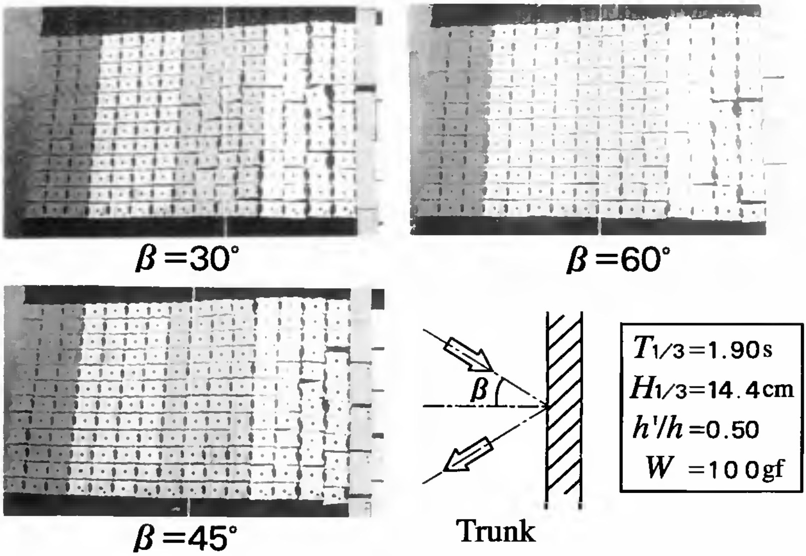

Photo 1 Damage patterns for oblique wave attack

By changing the breakwater alignment, the wave incident angle was altered in four cases to 0, 30, 45 and 60deg. The length of the island breakwater was 8.4 m for normal incidence and 7.2 m for oblique incidence. The water depth was set constant at 46.8 cm and the mound depth was altered. Long-crested irregular waves were used for the stability test with a D-block of three weights (66gf, 100gf and 140gf). The mound condition of h'/h was set at 0.5, the wave period $T_{1/3}$ was constant at 1.90s and wave height $H_{1/3}$ was varied in a range of 5 to 25 cm.

Photo 1 shows the movement of blocks under oblique incident conditions of $H_{1/3}$=14.4cm, $T_{1/3}$=1.90s, with β = 30, 45 and 60 deg. The damage of armor blocks occurred mostly at the berm of the mound for oblique wave conditions. This compares with normal incident conditions when damage starts from the slope of the mound.

Figure 6 shows the effect of the incident wave angle for a stable block weight for the wave condition of $T_{1/3}$=1.90s and $H_{1/3}$=18.0cm. The stable weight for oblique waves was smaller than that for normal incidence.

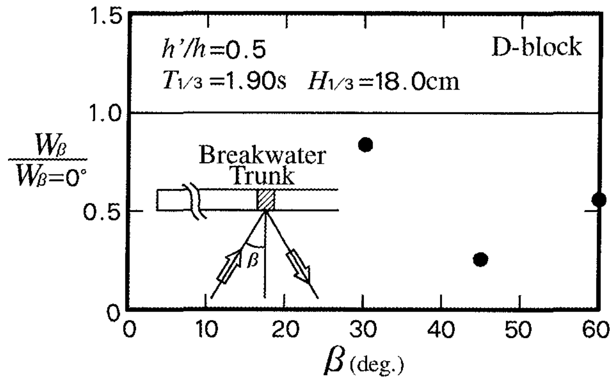

Figure 6 Armor block stability for oblique wave attack

Required thickness of foot-protection blocks

The two-dimensional experiments were made to identify the stability of foot-protection blocks using a wave flume (28 m × 0.6 m × 1.0 m) and irregular waves. Water depth h was constant at 46.8 cm, and three different mound depth h' were used. The foot-protection blocks used were of a uniform plane shape (10cm in length × 5cm in width), with their thickness t varied between 1.6cm, 2.4cm and 3.2cm. The specific gravity of the block model mortar was adjusted in accordance with the specific gravity of seawater. The foot-protection blocks were laid side by side in two rows, with the longer side facing the upright section.

The three-dimensional experiments were made using the wave basin mentioned previously. The model's cross section was the same shape as in the two-dimensional experiments. Using four different incident angles β (0, 30, 45 and 60 deg.), the breakwater trunk and head were analyzed for their respective required thicknesses. The breakwater head, in particular, was checked both forward and backward in relation to the direction of the waves.

Table 1 shows the standard dimensions of foot-protection blocks commonly used in Japan. The thickness of foot-protection block t was formulated as a function of relative mound height

Table 1 Standard dimension of foot-protection blocks

Required Thickness t (m)	Size of F-P Block $l \times b \times t$ (m) (m) (m)
~0.8	2.5×1.5×0.8
~1.0	3.0×2.5×1.0
~1.2	4.0×2.5×1.2
~1.4	5.0×2.5×1.4
~1.6	5.0×2.5×1.6
~1.8	5.0×2.5×1.8
~2.0	5.0×2.5×2.0
~2.2	5.0×2.5×2.2

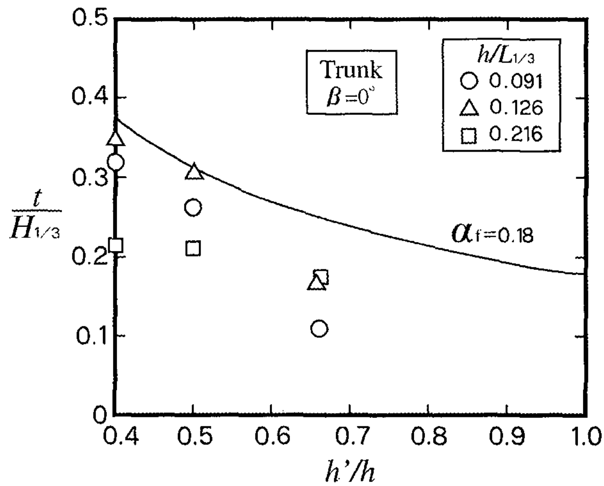

Figure 7 Necessary thickness for trunk section

h'/h as follows;.

$$t/H_{1/3} = a_f(h'/h)^{-0.787} \qquad (h'/h \geq 0.4\ ) \qquad (4)$$

Here a_f is a parameter showing the differences in breakwater trunk and head.

Figure 7 shows the results of the stability test at the breakwater trunk. In this figure, the horizontal axis represents the relative mound depth, h'/h, and the vertical axis represents the thickness of the foot-protection block, t, which was made non-dimensional by the wave height $H_{1/3}$. The necessary thickness was greater when the period was shorter. By focusing on the condition of the safe side, the necessary thickness was formulated as shown with the dotted line in this figure (a_f = 0.18) for the breakwater trunk.

Figure 8 shows the effect of incident wave angles. In the case of oblique wave attack at the breakwater trunk, relative mound depth, h'/h, was limited to 0.5. When the oblique incident angle was within 60 deg, the difference in necessary thickness was smaller compared with the normal incident condition.

Figure 9 shows the results of the experiments on the breakwater head. Unlike

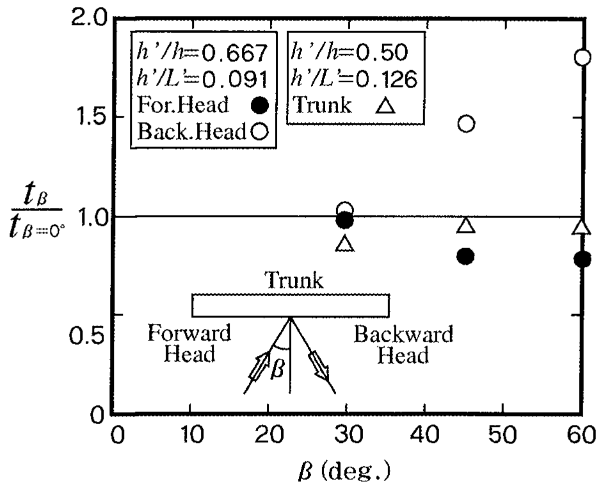

Figure 8 Effect of oblique wave attack

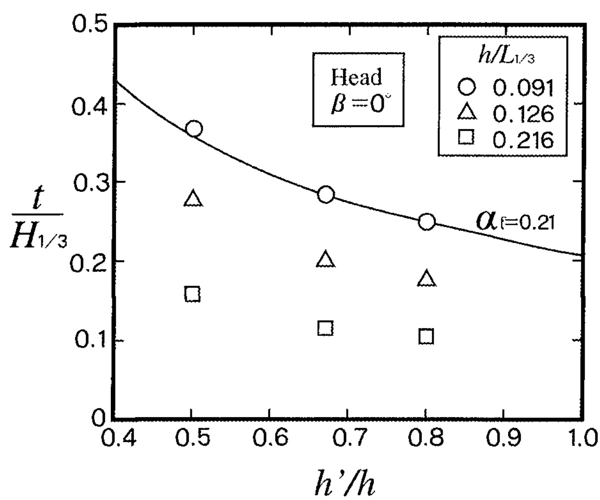

Figure 9 Necessary thickness for head section

the breakwater trunk, the required thickness increased as the wave period increased, which is due to local flows generated at the breakwater head. From the values on the safe side of the experiment range, a_f for the breakwater head corresponds to 0.21. According to **Figure 8**, which shows the influence of the direction of waves when the relative mound depth $h'/h = 0.667$ at the breakwater head, the required thickness at the forward head is maintained at about the same level by the incident angle β, while at the backward head the required thickness increases when β is 45 deg. or more. This is because of the rapid flow around the corner and the increasing standing wave height at the backward head of the island breakwaters.

Table 2 Prototype failures of foot-protection block

Name of Port	Year and Mon.	Head or Trunk	Structural Conditions		Storm Wave Conditions			F-P Block	
			h (m)	h' (m)	$H_{1/3}$ (m)	$T_{1/3}$ (s)	β (deg.)	t_p (m)	W (tf)
A	66.01	H	8.0	5.5	5.8	-	5.0	1.5	21.6
B	67.03	T	16.5	10.0	5.5	-	18.0	1.5	13.8
C	70.01	H	11.5	9.5	6.8	13.0	17.5	1.5	38.8
D	70.01	H	9.0	6.0	5.5	14.1	0.0	1.5	21.6
E	70.01	H	17.0	8.0	5.0	8.1	0.0	1.0	17.3
F	70.01	T	8.0	6.5	6.8	13.0	17.5	1.5	38.8
G	71.08	H	14.0	6.7	4.5	-	-	1.0	-
H	71.01	H	15.5	8.5	4.6	11.0	0.0	1.0	11.5
I	71.09	H	5.7	4.0	4.7	15.0	19.0	1.0	23.0
J	72.01	T	15.5	10.0	7.0	14.0	0.0	1.5	31.1
K	72.02	T	6.0	4.0	5.8	15.0	19.0	1.0	23.0
L	76.10	H	19.0	14.0	7.5	12.3	31.0	1.5	38.8
M	76.10	T	15.5	13.5	7.1	13.6	31.0	1.5	38.8
N	78.01	H	11.0	8.5	6.2	-	7.0	1.5	41.1
O	78.01	H	17.5	11.0	6.3	-	0.0	1.5	41.1
P	78.01	H	17.0	11.0	6.3	-	0.0	1.5	41.1
Q	79.12	H	18.0	11.8	10.8	-	26.3	2.0	47.6
R	79.12	H	18.0	11.8	10.8	-	26.3	1.5	46.6
S	80.01	H	16.5	13.0	7.0	-	15.0	1.5	41.4
T	80.09	T	11.5	8.5	5.6	10.0	14.0	1.2	34.5
U	80.10	T	8.0	5.0	5.7	13.0	45.7	1.5	31.1
V	80.10	H	13.7	8.5	5.9	9.4	-	1.0	-
W	81.08	H	24.0	16.5	9.2	11.0	0.0	1.5	41.4
X	85.01	H	11.6	8.0	5.6	10.1	66.0	1.2	27.6

Analysis of disaster damage regarding foot-protection blocks

Table 2 summarizing the disasters (1966 to 1991) related to destruction of foot-protection blocks of composite breakwaters in Japan. In 18 cases out of 25, breakwater heads experienced disaster damage, this indicating that breakwater heads suffer more damage than breakwater trunks. The details show that foot-protection blocks on the breakwater trunk experience less displacement. In this case, the damage was limited merely to foot-protection block displacement. In contrast, on the breakwater head, in many cases, foot-protection block displacement is followed by rubble foundation scouring. The incident wave angle β in most disasters was within 30° .

Figure 10 shows a typical example of the disaster condition at the breakwater head in S-Port. The details of the storm wave are assumed to be: $H_{1/3}$ =7.0 m, $T_{1/3}$ =13.5 s, incident wave angle β =15 deg. Foot-protection blocks on the harbor side (40 tf) were scattered, and 150 m^3 of material at the corner of the rubble mound foundation (0.2～0.3tf) below the caisson was scoured. When the wave incidence was almost normal, the damage started from the harbor side corner of the head caisson.

In X-Port, the island breakwater tail suffered greatest damage from the β =66° of oblique incident waves (**Figure 11**). The wave condition was: $H_{1/3}$ =5.6m, $T_{1/3}$ =10.1s. The damage was concentrated at the breakwater tail, and the foot-protection blocks (28tf) were displaced and partly broken by the shock of the displacement. Such damage to the breakwater head was caused by the rapid flow around corners. It corresponds well with the numerical calculation results shown by Kimura et al. (1994) in terms of the flow speed around the mound. The required thickness expected for the foot-protection block to prevent disaster damage in **Table 2** is given by Eq.(4).

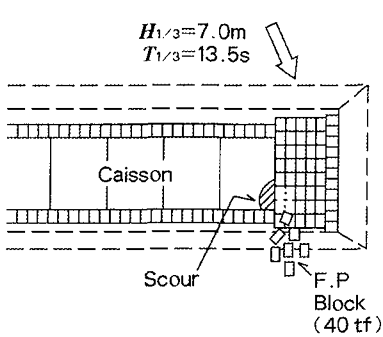

Figure 10 Damage at S-Port

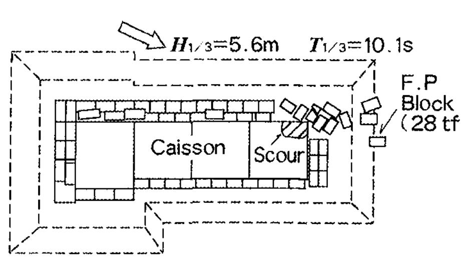

Figure 11 Damage at X-Port

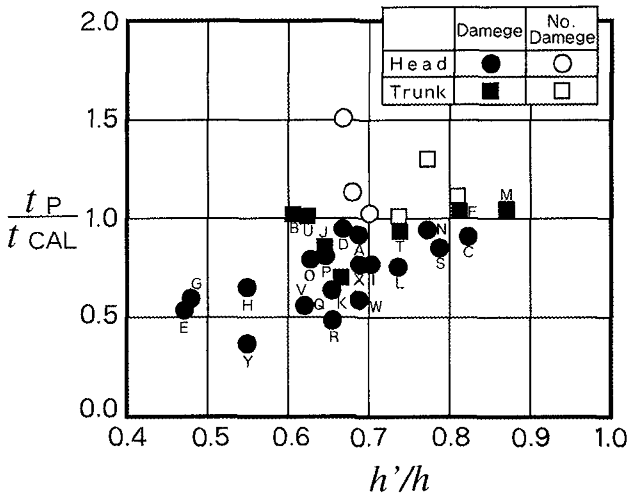

Figure 12 Comparison with prototype failures

Figure 12 shows on the axis of the ordinates that the ratio of required thickness of foot-protection blocks used t_p to t_{cal}, which contrasts with the relative mound depth h'/h on the axis of the abscissa. Most of the damage to foot-protection blocks occurred under the condition of t_p/t_{cal} =1 on both the breakwater head and trunk, which verifies the adequacy of the calculation method.

Conclusions

The calculation method of the stable weight for composite breakwater armor blocks and the required thickness of foot-protection blocks are discussed. The major conclusions are as follows;

- Armor blocks:

・A calculation method for the stability number using a block form factor as a parameter is proposed.

・ For oblique incident waves, the weight required for stability is likely to be less than for normal incident waves.

・ The stability number for wide mound berm conditions is formulated.

- Foot-protection blocks:

· The required thickness for breakwater head and trunk are formulated by using the ratio of the relative mound depth as a major parameter.

· The wave direction effect on the breakwater trunk was found to be small..

· The necessary thickness at the breakwater head needs to be increased depending on the wave direction.

· The adequacy of the required thickness calculation method was verified from field damage data.

Acknowledgments

We would like to thank Dr. Shigeo Takahashi, Port and Harbour Research Institute, Ministry of Transport, Japan, for his valuable overall suggestions concerning this study. The great contribution of Mr. Takanobu Suzuki of Hokkaido Development Bureau and Mr. Yoshikazu Doi of North Japan Port Consultants is also deeply appreciated.

References

Kimura, K., S. Takahashi and K. Tanimoto (1994): Stability of Rubble Mound Foundations of Composite Breakwaters under Oblique Wave Attack, Proc. of 24th ICCE, pp. 1227-1240.

Tanimoto, K., T. Yagtu and Y. Goda (1982): Irregular Wave Tests for Composite Breakwaters Foundations, Proc. of 18th ICCE, pp.2144-2163.