

CHAPTER 89

Stability of Rubble Mound Foundations of Composite Breakwaters under Oblique Wave Attack

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ABSTRACT

The armor stability of composite breakwaters has been investigated by large 3-D model experiments using oblique irregular waves. The characteristics of wave-induced flow near the mound, which directly affect the armor stability, are disclosed theoretically and numerically. The stability number of armor units in breakwater trunk, head and tail sections is formulated for oblique wave angles less than 60°. The applicability of the proposed methods is confirmed by prototype failure data, and the analyses of case studies are shown for some practical conditions.

INTRODUCTION

Rubble mounds of composite breakwaters are usually covered with armor units heavy enough to withstand severe wave actions. Methods to calculate the minimum weight due to irregular waves have been proposed by Tanimoto et al. (1982) and by Wu et al. (1983). However, these techniques apply only for 2-D conditions, such as armor unit stability in breakwater trunks for normal incident waves.

The scouring of rubble mound foundations caused by neglect of the 3-D effects has been one major reason for composite breakwater failures in Japan. According to the 3-D model test data for irregular waves, Ito et al. (1966) suggested that armor damage increases rapidly under oblique wave attack.

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Caissons at breakwater heads and tails were sometimes tilted by the scouring of rubble mound foundations. Conventional design procedures recommend that armor units at breakwater heads be 1.5 times heavier than those in breakwater trunks. However, the effect of incident wave angle and the extent of heavy armoring required are not obvious.

In this paper, the minimum weight of armor units required to withstand oblique incident waves is formulated for breakwater trunk, head and tail sections, based on stability tests, considering peak values of wave-induced water particle velocity on the mound.

ARMOR STABILITY AND WAVE-INDUCED FLOW NEAR MOUND

Stability Number of Armor Units

Figure 1 shows a standard cross section of a composite breakwater with a vertical wall and a rubble mound foundation. Wave forces acting on the vertical wall can be calculated for the breakwater's trunk, head and tail considering wave directions.

Rubble mound foundations have been however designed on the basis of engineer's experience, according to the following formula:

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

where W is the stable weight of armor units (tf); γ_d the unit weight of armor units; S_r the specific gravity of armor units in sea water; $H_{1/3}$ the wave height used in design (m); and N_s the stability number determined by wave factors, mound forms and characteristics of armor units. Equation (1) was developed by Hudson (1959) and has been in general use since Brebner and Donnelly (1962) modified it to estimate the stable weight of rubble mound foundation for vertical walls. Based on experimental results with irregular waves, Tanimoto et al. (1982) proposed the formula below to estimate the stability number of armor

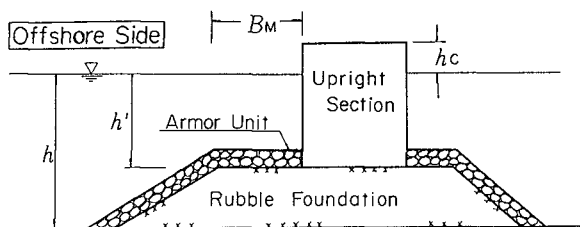


Figure 1 Standard Cross-Section of Composite Breakwater

units (two-layered settlement) against normal incident waves in breakwater trunks:

$$N_s = \max \left\{ 1.8, 1.3 \frac{1-\kappa}{\kappa^{1/3}} \frac{h'}{H_{1/3}} + 1.8 \exp \left[-1.5 \frac{(1-\kappa)^2}{\kappa^{1/3}} \frac{h'}{H_{1/3}} \right] \right\} \quad (2)$$

where κ is non-dimensional flow speed. Peak horizontal speed of water particle motion U near the bottom is defined as;

$$U = \sqrt{\frac{gH^2\kappa}{h'}} \quad (3)$$

where, H is incident wave height, h' is water depth above the rubble mound foundation and g is gravitational acceleration. As for wave forces on armor units, it is assumed that drag forces predominate over inertial forces. For the 3-D conditions, κ is formulated considering peak flow of water particle motions as described below.

In breakwater trunk sections for oblique wave attack, κ is defined by the small amplitude wave theory as follows:

$$\kappa = \kappa_1 \cdot (\kappa_2)_B \quad (4)$$

$$\kappa_1 = \frac{2kh'}{\sinh 2kh'} \quad (5)$$

$$(\kappa_2)_B = \max \{ \alpha_s \sin^2 \beta \cos^2(k/\cos \beta), \cos^2 \beta \sin^2(k/\cos \beta) \} \quad (6)$$

where β is the incident wave angle defined as the angle between the wave direction and the normal line of the breakwater alignment, k is wave number ($= 2\pi/L'$: L' is wave length for depth h'), and l is the distance from the vertical wall, κ_1 is the parameter of relative depth, and $(\kappa_2)_B$ expresses the effect of mound shape and wave direction. The first and second terms of $(\kappa_2)_B$ correspond to the peak flows, components parallel and orthogonal to the alignment, respectively. The slope factor α_s is determined to be 0.45 from the measured data and is multiplied only by the first term of Eq.(6), as the slope effect does not exist along the breakwater alignment.

Figure 2 shows the relationship between $(\kappa_2)_B$ and l/L' for three wave directions; $\beta=0^\circ$, 45° and 60° . For $\beta=0^\circ$, $(\kappa_2)_B$ is determined by the second term of Eq.(6), and armor units with wider berm mounds become more unstable.

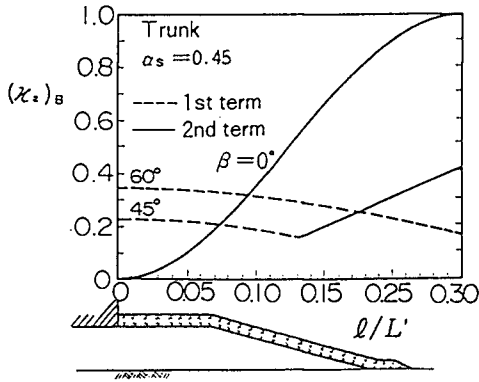


Figure 2 Non-Dimensional Flow Speed in Breakwater Trunk

For oblique incident 60° conditions, $(\kappa_2)_B$ is determined by the first term of Eq.(6) and slightly decreases away from the vertical wall.

Formulae for Breakwater Head and Tail

For breakwater head and tail sections, κ is formulated from the flow speed at the bottom using the small amplitude wave theory as follows;

$$\kappa = \frac{\kappa_1 \alpha_s \tau^2}{4} \quad (7)$$

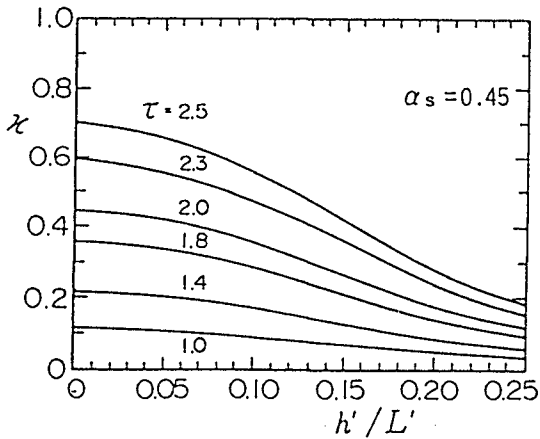


Figure 3 Non-dimensional Flow Speed at Breakwater Head and Tail

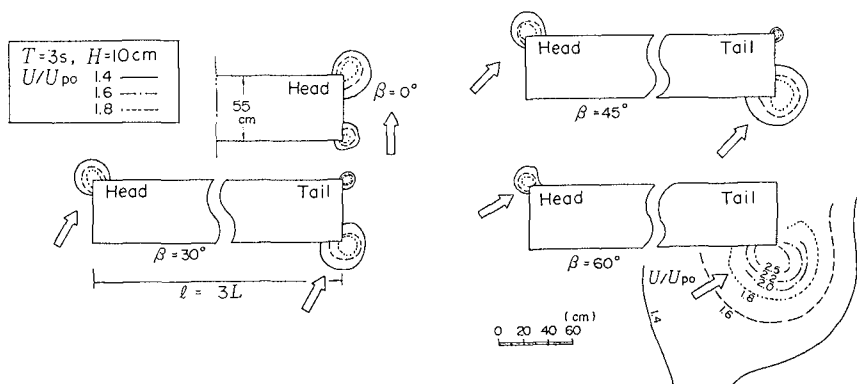


Figure 4 Flow Patterns at Breakwater Head and Tail

where τ is the correction factor for local rapid flow around corners. Figure 3 shows the relationship between κ and the relative water depth h'/L' using τ as a parameter. For longer period conditions with smaller relative depth, flow speed becomes larger.

Figure 4 shows numerical results of the mild slope equations. Flow pattern is analyzed for flat bottom conditions with $h = 30$ cm. The length of breakwater is three times that of incident wave length. The wave condition is $T = 3.0$ s and $H = 10$ cm. The peak speed of rapid flow U is divided by U_{p0} , the peak water particle velocity at bottom in progressive wave conditions without breakwaters. Rapid flow occurs locally around the offshore- and onshore-corners for normal incident conditions. The oblique 60° incidence leads to remarkable

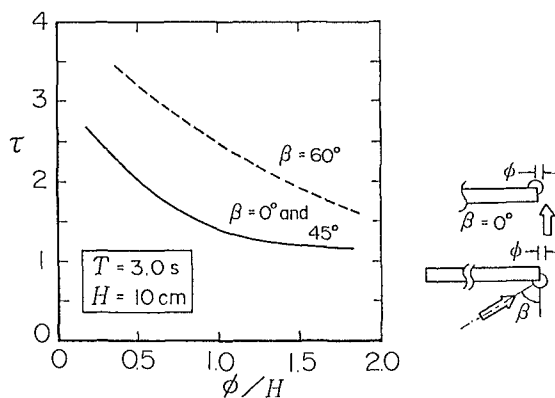


Figure 5 Rapid Flow Area

local flow around corners projecting towards the wave direction at breakwater tails as the standing wave height increases along the breakwater for the wave direction.

Wave-induced flow on the rubble mound is measured by electromagnetic current meters for regular wave conditions. The curved lines in Fig. 5 show the limit of rapid flow areas. Since the stability tests proved that the area to be protected is within the range of $1.0H$ from the corner, $\tau=1.4$ can be used for $\beta=0^\circ$ and $\beta=45^\circ$ conditions, and $\tau=2.5$ can be used for $\beta=60^\circ$ conditions

STABILITY TESTS

Experimental Setup

The tests were carried out in a 50m-long, 20m-wide 3-D wave basin. The basin bottom slopes at a gradient of 1/50, as shown in the lower part of Fig.6 (d). The incident wave angles β were 0° , 45° and 60° , varied by changing the layout of the breakwater model as shown in Fig. 6 (a) ~ (c). The length of the breakwater alignment is more than 2.5 times the wave length for oblique incident conditions.

The standard cross section of the model breakwater was a relatively low mound condition. Water depth h was 65 cm, mound depth h' was 45 cm and mound berm width B_M was 40 cm.

Stability tests of armor units were carried out under irregular wave conditions. The wave range was limited to the non-breaking conditions, $H_{1/3}/h$ is less than 0.35. Three types of wave period ($T_{1/3}=1.64, 2.19$ and 2.92 s) were prepared, and the relative depth h/L was varied from 0.08 to 0.15. The spectrum of the irregular waves was the modified Bretschneider-Mitsuyasu type, and the wave number was about 150 for each experiment.

Standing wave heights in front of vertical wall vary along the breakwater alignment due to diffraction waves. Armor stabilities in the breakwater trunk were examined in front of caissons No. 10 ~ 12 (Fig. 6(a): $\beta=0^\circ$), No.12~14 (Fig. 6 (b): $\beta=45^\circ$) and No.9 (Fig.6 (c): $\beta=60^\circ$), where the standing wave height is approximately twice the incident wave height.

Four types of stones with mean weights of 15, 30, 60 and 100 gf were used as a model of armor units. Physical characteristics of armor stones are shown in Table 1. The wave height was gradually increased without changing the arrangement of armor units.

The inspection area was divided into a 20-cm-square grid, and differently colored armor stones were placed in each square. The number of stones moving out of the grid was counted by visual observation. The damage rate defined as the ratio of moved stones to the total, and maximum damage ratios are used in the following analyses.

Armor Stability at Breakwater Trunk

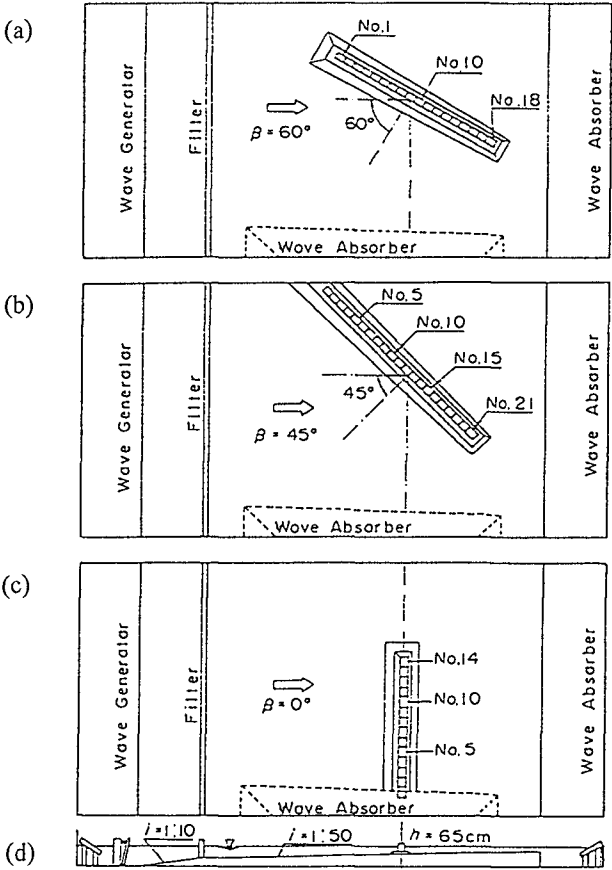


Figure 6 Model Breakwaters in a Three-Dimensional Wave Basin

Table 1 Model Armor Stones

Grade	Weight (gf)		Density (gf/cm ³)
	Average	Standard Div.	
I	15.0	2.05	2.60
II	29.9	3.51	2.59
III	57.3	5.99	2.62
IV	105.5	12.5	2.75

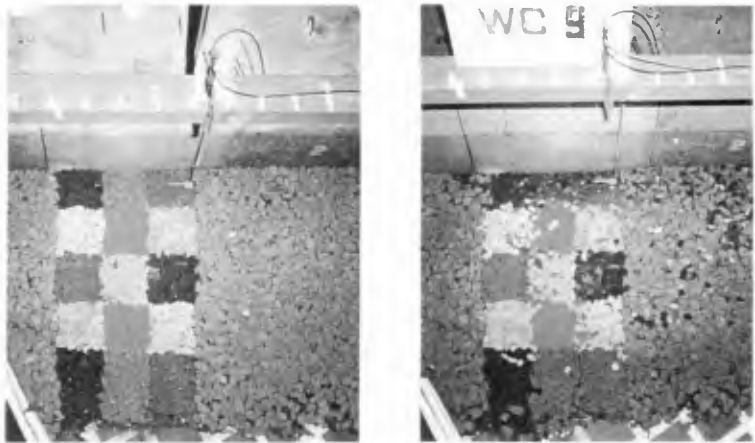


Photo 1 Damage Pattern: Oblique 60° Condition

Photo 1 shows damage patterns for the oblique 60° condition in breakwater trunk. The left part shows the armor conditions before wave action, the right shows them after the wave action of $T_{1/3} = 2.92$ s and $H_{1/3} = 23.0$ cm. Armor units of 15 gf were heavily scoured near the vertical wall by the wave-induced rapid flow along the breakwaters.

Figure 7 shows damage ratio, with the incident wave height on the abscissa. The results for normal and 45° oblique waves are very similar, and the damage does not extend with the increase in the wave height. However, when the wave angle $\beta = 0^\circ$, the damage starts at small wave heights, and then increases remarkably. This is because the non-dimensional flow speed $(\kappa_2)_B$ for $\beta = 60^\circ$ is larger than that for $\beta = 0^\circ$ and $\beta = 45^\circ$ when $l/L' \leq 0.07$ as shown in Fig. 2.

Figure 8 shows the relationship between wave heights and the minimum weights for three wave directions, when $T_{1/3} = 2.92$ s. The circles show the

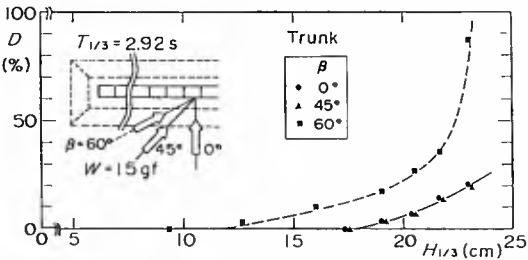


Figure 7 Relationship between Damage and Wave Height

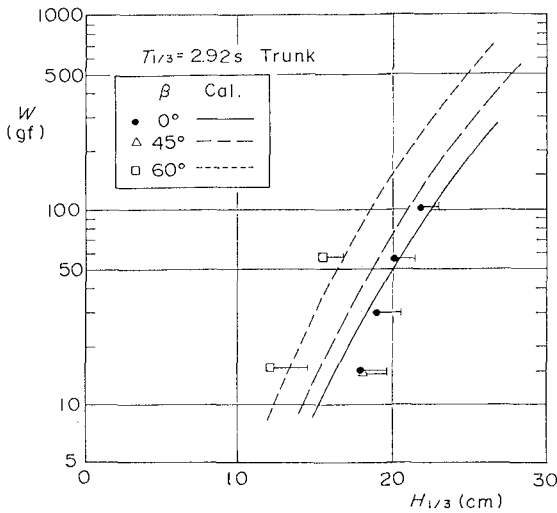


Figure 8 Minimum Armor Weight (Trunk)

limits of damage, with the short, solid horizontal lines indicating the 5% damage range. The curved lines are calculated by the proposed method and agree well with the experimental values. This results confirm the applicability of formulating the wave-induced flow based on the small amplitude wave theory.

Armor Stability at Breakwater Head and Tail

Figure 9 shows scouring patterns at the breakwater head and tail for each of the wave directions with $T_{1/3} = 2.92 \text{ s}$. For the normal incident waves, the damage is larger at the shoreward corner of the caisson. For oblique waves, the scouring occurs at projecting corners of the breakwater tail. These damage

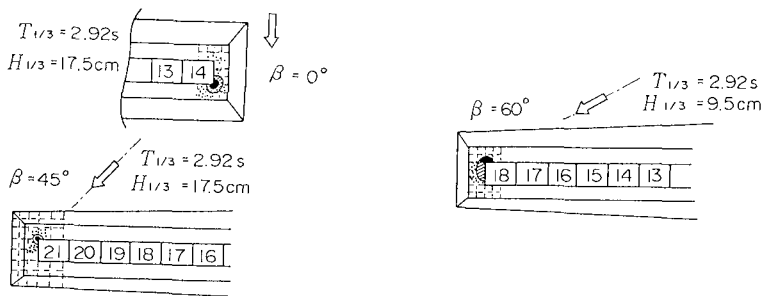


Figure 9 Scouring at Breakwater Head and Tail

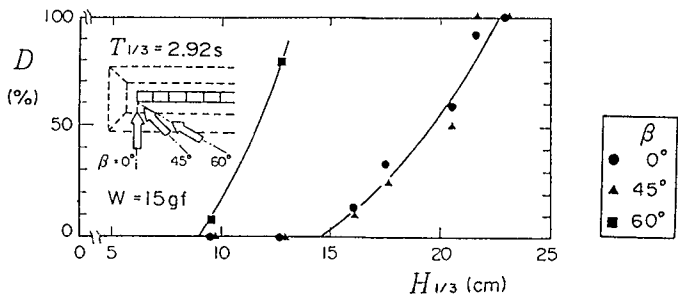


Figure 10 Relationship between Damage and Wave Height

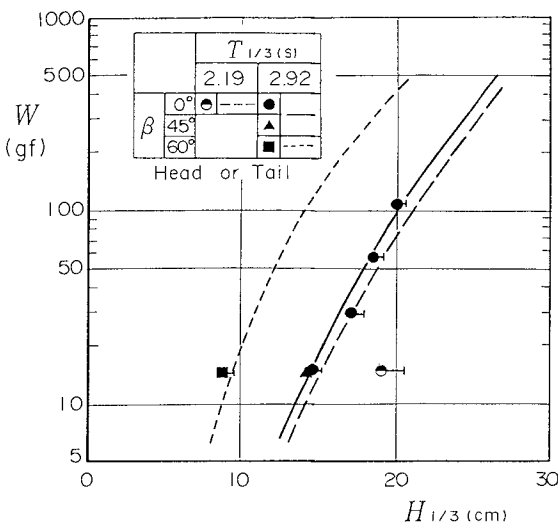


Figure 11 Minimum Armor Weight (Head and Tail)

patterns agree well with the positions where the wave-induced flow speed near the mound is larger as shown in Fig. 4.

Figure 10 shows the damage ratio for breakwater head and tail sections under waves with $T_{1/3} = 2.92$ s. The results for the normal and 45° oblique waves are very similar. With the 60° oblique waves, the damage begins to occur at small wave heights and then increases remarkably.

Figure 11 shows the relationship between the wave height and the minimum armor weight for each wave direction, when $T_{1/3}$ is 2.19 s and 2.92 s. No armor damage was recorded for the wave condition of $T_{1/3} = 1.64$ s. Armor units for

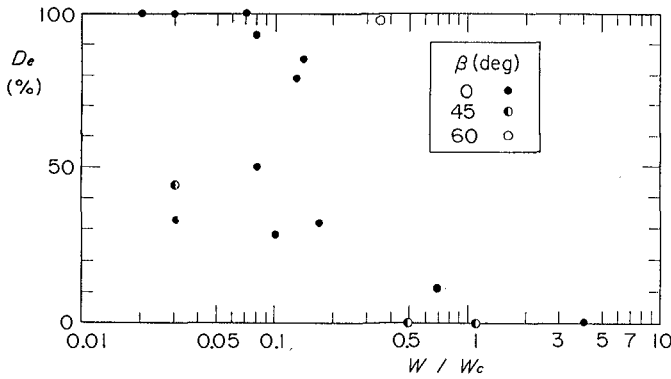


Figure 12 Comparison with Prototype Failures

longer wave periods become more unstable and should be made heavier than those used for shorter wave period.

The curved lines are calculated by the proposed method and agree well with the experimental values. It is shown that the effect of incident wave angles can be expressed by the correction factor τ in the calculation of minimum armor weights.

Comparison with Prototype Failures

Figure 12 compares the prototype armor failures (1965 ~ 1990) and the proposed methods. The x-axis represents the ratio of the mean weight of armor units W to the weight calculated by the proposed method W_c , and the y-axis represents the damage percent of the prototype armor units D_e .

When $W/W_c < 1$, damage is expected, and the applicability of the proposed calculation method is confirmed by the above comparisons.

EXAMPLE OF ARMOR DESIGN

Design Conditions

A composite breakwater, in Fig. 13, is used for the case studies. Design conditions are as follows;

Depth : $h = 13.0$ m, $h' = 9.0$ m

Wave Conditions : $H_{1/3} = 5.0$ m, $T_{1/3} = 13.0$ s

$B_M = 8.0$ m

$\beta = 0^\circ$ and 60°

Armor units : Two layers of stones, $\gamma_d = 2.65$ tf/m³

Armor Units for Breakwater Trunk

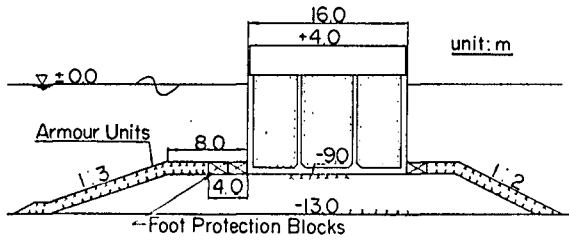


Figure 13 Design Conditions

(1) Normal incident condition

Substituting $h'/L' = 0.076$ for Eq.(5), κ_1 is calculated as follows;

$$\kappa_1 = \frac{2 \times 2 \times 3.14 \times 0.076}{\sinh(2 \times 2 \times 3.14 \times 0.076)} = 0.863$$

For normal incident conditions, the mound shoulder becomes critical for the stability of armor units.

Substituting $l = B_M$ for Eq.(6),

$$(\kappa_2)_B = \sin^2 \left(\frac{2 \times 3.14 \times 8.0}{118} \right) = 0.171$$

$$\kappa = \kappa_1 \cdot (\kappa_2)_B = 0.863 \times 0.171 = 0.148$$

Using Eqs.(1) and (2), the stability number and the necessary minimum weight are calculated as follows:

$$N_s = 1.3 \times \frac{1 - 0.148}{0.148^{1/3}} \times \frac{9.0}{5.0} + 1.8 \exp \left[-1.5 \times \frac{(1 - 0.148)^2}{0.148^{1/3}} \times \frac{9.0}{5.0} \right] = 3.79$$

$$W = \frac{2.65 \times 5.0^3}{3.79^3 \times (2.65/1.03 - 1)^3} = 1.56 \text{ (tf)}$$

(2) Oblique 60° Condition

Armor stability should be checked; at the outside of foot protection blocks ($l = 4.0$ m),

$$\begin{aligned} (\kappa_2)_B &= \max \left\{ 0.45 \sin^2 60 \cos \left(\frac{2 \times 3.14 \times 4.0}{118} \right), \cos^2 60 \sin^2 \left(\frac{2 \times 3.14 \times 4.0}{118} \right) \right\} \\ &= \max \{ 0.322, 0.011 \} = 0.322 \end{aligned}$$

K , N_S and W are calculated as follows:

$$\kappa = 0.278, N_S = 2.80, W = 3.88 \text{ (tf)}$$

For the breakwater trunk, the necessary weight of armor units for oblique 60° waves is 2.5 times that for normal incident conditions.

Armor Units for Breakwater Head and Tail

(1) Normal incident condition

Substituting $\kappa_1 = 0.863$ and $\tau = 1.4$ for Eq.(7), τ is calculated as follows:

$$\kappa = 0.863 \times \frac{0.45 \times 1.4^2}{4} = 0.190$$

Using Eqs.(1) and (2), the stability number and the necessary minimum weight are calculated as follows:

$$N_S = 3.38, W = 2.20 \text{ (tf)}$$

(2) Oblique 60° Condition

Substituting $\tau=2.5$ for Eq.(7), K , N_S and W are calculated for breakwater tails as follows:

$$K=0.607, N_S=2.19, W=8.11 \text{ (tf)}$$

At the breakwater tail, the necessary weight for oblique 60° waves becomes 3.7 times than that for normal incident waves. Under oblique 60° conditions, the tail section requires armor units two times heavier than the trunk section does.

CONCLUSIONS

The characteristics of the wave-induced flow near the mound of composite breakwaters, which directly affect the armor stability, were investigated for 3-D conditions. The results of numerical analyses and physical model tests confirmed that armor damages in prototype failures occurred in the rapid flow areas.

The stability number can be calculated by Eq.(2) for oblique wave conditions of non-dimensional flow speed κ . For breakwater trunk sections, κ is formulated by Eqs.(4) through (6). For breakwater head and tail sections, κ is formulated by Eq.(7), and the area to be heavily protected is within the range of $1.0 H$ from the corner of caissons. The applicability of proposed methods was confirmed by stability model tests and prototype failure analyses.

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