CHAPTER 105

Reliability Analysis of Composite Breakwaters Protected with Armor Blocks

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Abstract

The subject of this present rcsearch is to study probabilistic design of caissons of composite breakwaters, rubble-mounds and foundations protected with armor blocks. Concerning caissons, sliding, rocking and overturning motions are considered to be the failure modes. Wave forces acting on a caisson can be calculated using Goda's formulas(1974). The stability of the rubble-mound is estimated by means of Bishop's method while geotechnical problems employ a simplified equation as a reliability function in accordance with the Japanese Standards for Coastal Structures.

Since distribution for occurrence for each failure mode is unknown, Monte Carlo simulation was applied to calculate the risk of each failure. The probability of geotechnical failure was greatest among the failure modes but the rubble-mound was not in danger of collapse.

Introduction

In Japan, breakwaters of the type shown in Fig.1 are the most commonly designed and constructed. They consist of caisson, concrete cap, rubble-mound and armor blocks. Such a breakwater is referred to as "a composite breakwater protected with armor blocks". It has several advan-

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tages, i.e.
(1)The construction period is short.
(2)A large sea area within the breakwater is conserved.
(3)The crest elevation is low.
(4)Ships are easily moored inside the upright section.
(5)etc.

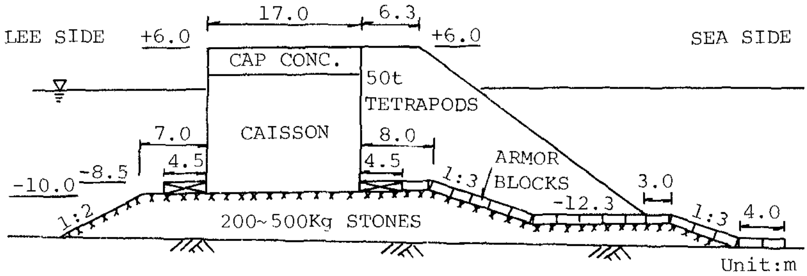

Fig.1 Typical Composite Breakwater Protected with Armor Blocks

In recent years, breakwaters have been constructed in deeper and deeper areas so the armor blocks and upright sections receive large wave forces. As a result, blocks receive damage and caissons slide or overturn. In addition, foundations receive such high pressure from caissons that they collapse.

Engineers in Japan design composite breakwaters protected with armor blocks using a deterministic method introducing a safety factor. The method is based on experiences from construction of breakwaters over many years. However if a new type structure is designed or constructed or conventional breakwaters are placed in a deep sea area where no structure has existed before, the deterministic or safety factor approach is not easily applicable.

Recently, new methodology based on probabilistic and statistical theory has been the subject of study for application to coastal structures e.g. Burcharth(1985), Van der Meer et al(1987), Mizumura et al(1988) and PIANC(1990). In this new design method, ranks of safety between breakwaters can be balanced and structures designed taking into account economical conditions (Yamamoto et al(1988)).

In this paper, the authors applied the probabilistic method to a composite breakwater protected with armor blocks taking wave breaking and retaining wave after breaking into account.

Reliability Analysis

If the external force is represented by S and the resistant force by R in a certain failure mode, the reliability function Z is given by eq.(1).

$$Z = R - S \tag{1}$$

In this case, as Z is greater than 0, failure does not occur. However, if Z is less than 0, failure does occur. In eq.(1) the variables R and S are generally uncertain or probabilistic ones therefore Z is also a probabilistic variable. So if the probabilistic density function of Z is known, the probability where the failure occurs can be calculated as follows;

$$P = \int_{-\infty}^{0} f(Z)dZ \tag{2}$$

in which f(Z) is the probabilistic function of probabilistic variable Z.

If m failure modes exist for a breakwater, the total failure probability for the breakwater is obtained as eq.(3).

$$P_f = \text{Prob}\{(Z_1<0) \cup (Z_2<0) \cup \cdots\cdots \cup (Z_m<0)\} \tag{3}$$

Assuming all m failure modes are independent of each other, the probability for the risk is simply rewritten by eq.(4).

$$P_f = 1 - \prod_{i=1}^{m}\{(1 - P_{fi})\} \tag{4}$$

Concerning the failure modes for the composite breakwater protected with armor blocks as shown in Fig.1, the main modes are (1)movement of armor blocks, (2)slide of the upright section, (3)overturning of the upright section, (4)slide of the rubble-mound and (5)collapse of the foundation that is geotechnical instability shown in Fig.2.

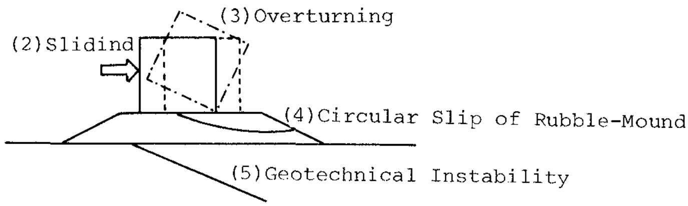

Fig.2 Failure Mode for Composite Breakwater protected with Armor Blocks

The block movement for these failure modes was already discussed by Mizumura et al(1988) and Yamamoto et al (1988) in a previous ICCE in Spain. Therefore in this paper the authors dealt with (2)-(5) as the failure modes of the composite breakwater. The failure modes above were selected as representative, however because the probability density functions of the failure occurrence, f(Z) was not unknown, Monte Carlo simulation was conducted to calculate the functions.

Wave Forces Acting on Caisson Wall

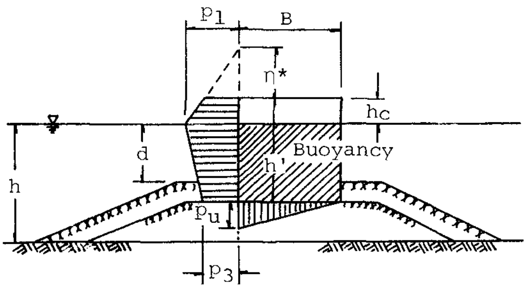

Fig.3 Distribution of Wave Pressure

Goda(1974) proposed formulas representing wave forces acting an upright section of a composite breakwater without armor protection. Tanimoto et al(1976) modified Goda's formulas to apply a composite breakwater protected with armor blocks introducing a wave pressure reducing ratio due to armor blocks.

Fig.3 shows a sketch of a wave pressure distribution along the upright section. Wave pressures acting on the upright section are expressed as below (Tanimoto et al(1976));

$$p_1 = \lambda \alpha_1 w_o H \tag{5}$$

$$p_3 = p_u = \lambda \alpha_1 \alpha_3 w_o H \tag{6}$$

$$\eta^* = 1.5\lambda H \tag{7}$$

where λ is the wave pressure reducing ratio due to the armor block protection, and

$$\left.\begin{aligned} \alpha_1 &= 0.6 + \frac{1}{2}\left[\frac{4\pi h/L}{\sinh(4\pi h/L)}\right]^2 \\ \alpha_3 &= 1 - \frac{h'}{h}\left[1 - \frac{1}{\cosh(2\pi h/L)}\right] \end{aligned}\right\} \tag{8}$$

λ refers to the ratio of the wave force in the case of protection as opposed to that without protection. When λ equals to 1, no protection exists in front of the caisson. In the case where λ is less than 1, that indicates that there is the armor block protection. The value of λ is taken to be 0.8 in the case of the composite breakwater protected with armor units according to the Japanese Standards of design of breakwaters based on hydraulic model tests. The total horizontal wave force acting on the vertical wall, up-lift force and moment around the heel of the caisson are obtained by eq.(9)-(12).

The horizontal wave force;

$$F = \frac{1}{2}(p_1 + p_3)h' + \frac{1}{2}(p_1 + p_4)h_c^* \quad (9)$$

The moment around the corner of the base due to the horizontal wave force;

$$M_p = \frac{1}{2}(2p_1 + p_3)h'^2 + \frac{1}{2}(p_1 + p_4)h'h_c^{*2}$$

$$+ \frac{1}{6}(p_1 + 2p_4)h_c^{*2} \quad (10)$$

in which

$$p_4 = \begin{cases} p_1\ (1 - h_c/\eta^*) & : \eta \geq h_c \\ 0 & : \eta^* < h_c \end{cases}$$

$$h_c^* = \min(\eta, h_c)$$

min(a,b) : smaller of a and b.

The total up-lift force:

$$U = \frac{1}{2} p_u B \quad (11)$$

The moment around the hcel due to the up-lift force:

$$M_u = \frac{2}{3} UB \quad (12)$$

Reliability Functions for Each Failure Mode

When the wave forces F and U operate on the caisson, the resistant force against slide is due to friction represented by eq.(13) and the resistant moment is given by eq.(14).

$$F_r = \mu(W - U) \quad (13)$$

$$M_r = Wt \quad (14)$$

in which W is the weight of the upright section in water, μ indicates a frictional coefficient betwcen the caisson base and thc surface of the rubble-mound and t refers to the distance between the caisson heel and the centroid of

the upright section.
So the reliability functions for slide and overturning of the upright section are expressed by

$$Z_s = \mu \ (W - U) \ F \tag{15}$$

and

$$Z_o = Wt - M_u - M_p \tag{16}$$

respectively.

Concerning the collapse of rubble-mound slope, the mound slope is assumed to be destroyed by a circular arc of which the center is located at point O as shown in Fig.4.

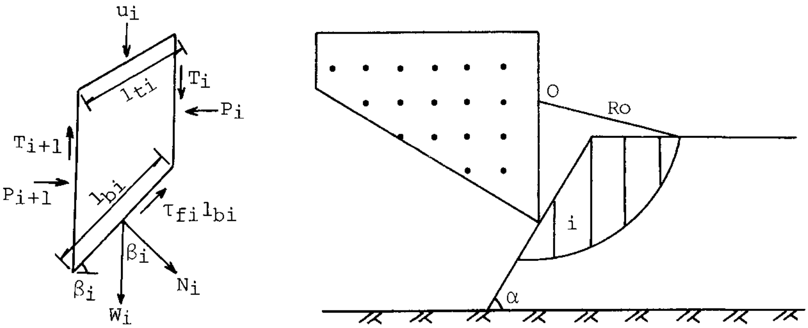

Fig.4 Selection of Center of Circular Arc

The computational method is as proposed by Bishop. The forces exerted on the i-th soil element are described in Fig.4. Assuming that $T_i = T_{i+1}$ and $P_i = P_{i+1}$, the sliding and resistant moment with respect to point O are written as;

$$M_s = \sum_{i=1}^{n} (W_i + u_i) \ Ro \ \sin\beta_i \tag{17}$$

$$M_b = \sum_{i=1}^{n} \tau_{fi} \ l_{bi} \ Ro \tag{18}$$

in which W_i is the weight of the i-th soil element, β_i is an angle of the sliding arc of the i-th soil element to the horizontal, Ro refers to the radius of the circular arc, τ_{fi} indicates the shear stress exerted on the circular arc of the i-th soil element, l_{bi} is the length of the circular arc and u_i is the vertical force due to wave action. If the normal force exerted on the circular arc is given by N_i, the shear stress becomes

$$\tau_{fi} = \frac{N_i}{l_{bi}} \tan\phi \tag{19}$$

where ϕ is the friction angle. The balance of the vertical forces exerted on the i-th element gives the normal force as eq.(20).

$$N_i = \frac{W_i + u_i}{\cos\beta_i + \tan\phi \sin\beta_i / F_s} \tag{20}$$

F_s is obtained as follows.

$$F_s = \frac{1}{\sum_{i=1}^{n}(W_i+u_i)\sin\beta_i} \sum_{i=1}^{n} \frac{(W_i+u_i)\tan\phi}{\cos\beta_i+\tan\phi\sin\beta_i/F_s} \tag{21}$$

Therefore the reliability function is given by

$$Z_b = M_b - M_s \tag{22}$$

This equation is tantamount to F_s. If $Z_b<0$ or $F_s<1$, the rubble-mound slope collapses. F_s should be obtained by iteration in eq.(21).

Concerning the geotechnical instability, the bearing capacity of the foundation engineering for eccentric inclined load. However in this study a simplified technique was employed in order to examine the magnitude of the heel pressure. According to Goda(1985), the largest bearing pressure at the heel P_e is obtained as below;

$$P_e = \begin{cases} \dfrac{2W_e}{3t_e} & : \ t_e \le \dfrac{1}{3} \\ \dfrac{2W_e}{B}\left(2 - 3\dfrac{t_e}{B}\right) & : \ t_e > \dfrac{1}{3}B \end{cases} \tag{23}$$

in which $t_e=M_e/W_e$, $M_e=Wt-M_u-M_p$ and $W_e=W-U$. If the allowance for heel pressure is τ, the reliability function for the geotechnical stability is represented by the following equation.

$$Z_g = \tau - P_e \tag{24}$$

The value of τ is usually taken to be 40ton/m^2-50ton/m^2.

Wave Transformation from Offshore to Breakwater Site

The wave force acting on the upright section can be obtained from eqs.(5)-(12) if the wave height at the breakwater is given. The distribution of the wave height in deep water can be regarded as Rayleigh distribution. However, that at the breakwater in shallow water or the surf zone is generally not known. Goda(1975) proposed the numerical method to calculate the wave height of random waves in the surf zone. In this study, the method was

applied to obtain the wave height distribution at the breakwater site. This method is as follows;
A random wave train of which the height distribution is given in deep water is separated into individual waves defined by the zero-up crossing and deformation of each individual wave due to shoaling or breaking is estimated applying monochromatic wave theory including non-linearity. Goda obtained the wave height distribution in the surf zone taking retaining waves after breaking and surf beat into consideration.

Risk Analysis and Illustrative Examples

In this study, failure probability for the composite breakwater protected with armor blocks was calculated on the basis of the following assumptions.
(1)Uncertain variables are wave heights in deep water, wave periods, frictional coefficients(μ) and wave force reducing ratio(λ). Other variables are deterministic ones.
(2)The distribution of wave heights follows the Rayleigh distribution in deep water. The distribution of wave period of the 2nd power is also Rayleigh distribution.
(3)A frictional coefficient between the caisson base and rubble-mound is distributed in accordance with the normal distribution of which the mean value is 0.57 and the standard deviation is 0.05. This is decided under a condition where 10cm sliding distance of the upright section is allowed (Toyama(1985)).
(4)The wave force reducing ratio is uniformly distributed from 0.4 to 1.0 with reference to the results of hydraulic model tests conducted by Tanimoto et al(1976).
The procedure for risk analysis was as follows;
The distribution of wave height at the site of the breakwater was calculated employing the numerical method described previously. Next, we sampled a wave height coupled with a wave period from their distribution at the breakwater site. In this sampling, the distribution of the wave periods was assumed not to change at the site of the breakwater. The wave force and moment acting on the upright section could be calculated substituting the sampled wave height and period into eq.(5)-(12). We judged whether the failure for each mode occurred or not utilizing the reliability function. This procedure was repeated 5000 times (Monte Carlo simulation). Eventually, the failure probability for each failure mode could be estimated. Total risk of the breakwater is obtained by eq.(25).

$$
\begin{aligned}
P = {} & Ps + Po + Pb + Pg \\
& - PsPo - PoPb - PbPg - PgPs \\
& + PsPoPb + PoPbPg + PbPgPs + PgPsPo
\end{aligned}
$$

$$- PsPoPbPg \qquad (25)$$

in which P:the total failure probability, Ps:the failure probability of slide, Po:the failure probability of overturn, Pb:the failure probability of mound slip and Pg:the failure probability of geotechnical instability.
In the computation, the value 45 ton/m^2 was taken as the limitation of the heel pressure.
Fig.5 shows the model of the breakwater used in this analysis.

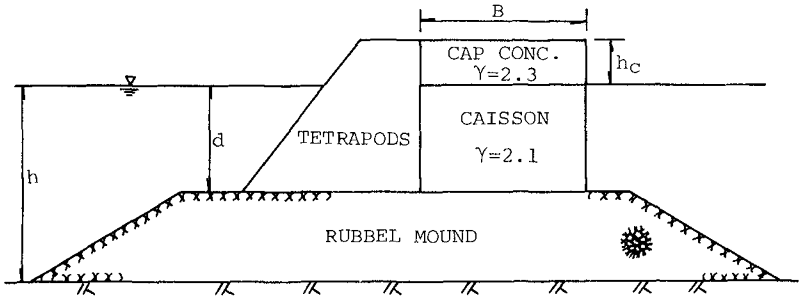

Fig.5 Model of Breakwater

Fig.6 shows variations of the total and each failure probability versus the equivalent deep water wave heights when the wave period(T) is 8.0sec, the water depth(h) is 6.0m, the caisson width(B) is 4.0m, the clearance(a distance between the still water and the top elevation of the upright section h_c) is 2.0m and the water depth on the mound(d)is 4.0m.
The value of the individual and total failure probabilities increases the wave height becomes larger. The risk to the rubble-mound slope is not seen in the figure because that the probability was zero through all computation cases in our simulation. In other words, the rubble-mound is strongest against wave force. In the range where the wave height is smaller than 5m, the geotechnical and total

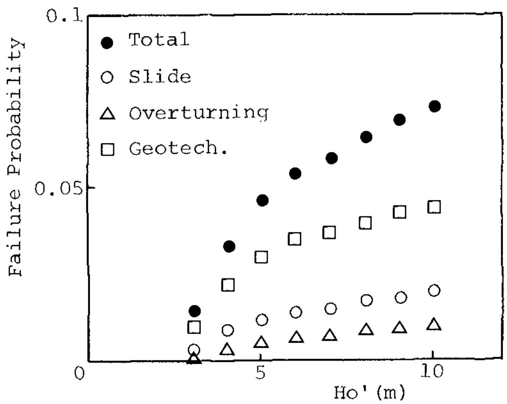

Fig.6 Effect of Wave Height on Failure Probability

failure probabilities increase significantly. However when the wave height exceeds 5m, the increment of this risk decreases. This is one reason why the number of the incident wave breaking between the offshore and the site of the breakwater increases when Ho' becomes larger than 5m. Among the failure modes, geotechnical risk is the greatest problem followed by slide of the upright section. Therefore if an incident wave height became large, the foundation would collapse before the upright section slide or overturned.

Change in the failure probability versus the wave period is shown in Fig.7 in the case where the equivalent deep water wave height(Ho') is fixed at 6.0m, h=6.0m, B=4.0m, h_c=2.0m and d=4.0m.

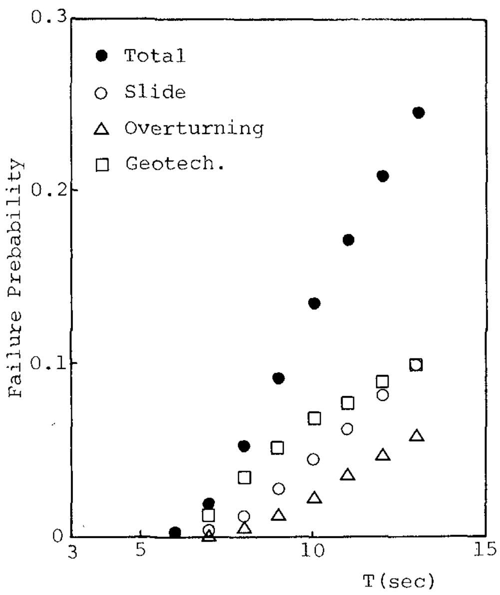

Fig.7 Effect of Wave Period on Failure Probability

The failure probability linearly increases as the wave period becomes long. In this case, the risk for geotechnical instability is greatest, but that for slide of the upright section becomes the same as the risk for geotechnical when the wave period increases. As the wave length becomes large, α_1 and α_3 in eq.(8) increase, therefore the wave force also become large. In addition, when h/Lo (Lo:a wave length in deep water) is small, waves tend not to break. So the wave force becomes large and the failure probability increases as the wave period is large.

Fig.8 indicates the influences of the water depth on a breakwater when Ho'=6.0m, T=8.0sec, B=4.0m,h_c=2.0m and d=4.0m. As the water depth increases, each as well as the total failure probability increases. When the water depth is large, wave heights hitting the breakwater becomes large and their wave lengths also increase. The wave forces acting on the breakwater therefore increase because the number of breaking waves decreases and because of the effects of wave length from the formulae for wave forces. In this case also, in the range where the water depth is greater than 8m, the difference between the risk for foundation collapse and caisson slide is close.

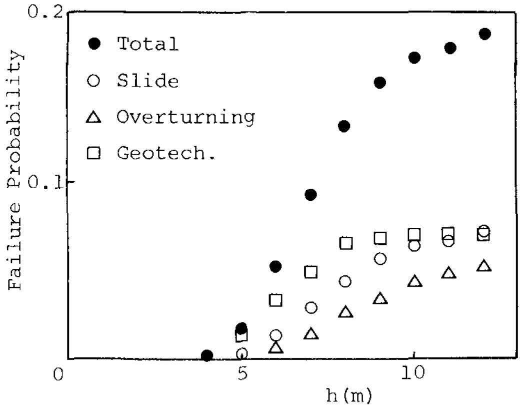

Fig.8 Effect of Water Depth on Frailure Probability

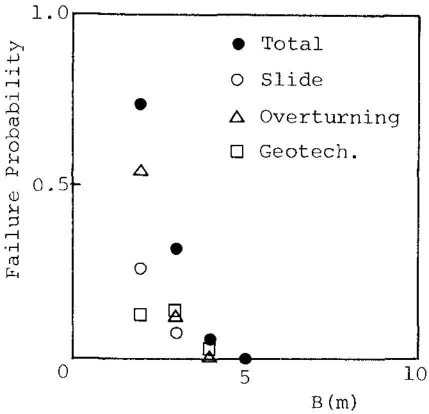

Fig.9 Effect of Upright Section Width on Failure Probability

The relationship between the width of the upright section and failure probability is shown in Fig.9 in the case of Ho'=6.0m, T=8.0sec, h=6.0m, h_c=2.0m and d=4.0m. When the caisson width is narrow, the risk of overturn is greatest. From this figure, it can be seen that in order to reduce risk increase of the width of the upright section is the best way. In particular, the wide caisson effect to decrease the risk of overturning.

Fig.10 shows the variation of the failure probability as the gradient of the sea bottom slope is changed from 0.01(1/100) to 0.1(1/10). The failure probability increases as the sea bottom becomes steeper. The breaking wave height becomes large if the wave period and water depth are fixed. As a result, the incident wave height becomes is so large that the great wave force can lead the breakwater to collapse. Failure probabilities of slide and overturn increase with the steepness of the sea bottom. Geotechnical risk increases until the gradient of the sea bottom slope is 0.08. This is the plateau and is less than the probability for the slide and overturn when the gradient is greater than 0.08.

From Fig.6 - Fig.10, the wave period and gradient of the sea bottom affect the failure probability more significantly than the wave height. The longer the wave period and the steeper the sea bottom, the greater the failure probability for the composite breakwater protected with armor blocks is. In most cases, the probability of geotechnical instability is larger than the slide and overturn of the upright section of the composite breakwater protected with armor blocks.

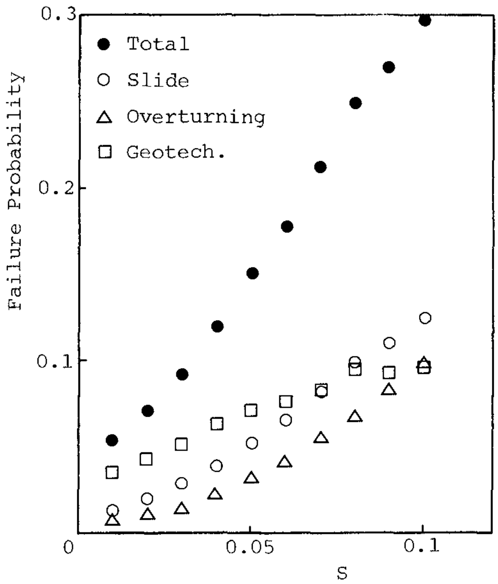

Fig.10 Effect of Gradient of Sea Bottom Slope on Failure Probability

Comparison with Risk for Armor Protection

Mizumura et al(1988) calculated the failure probability for displacement of armor blocks whose weights were obtained using the Hudson formula under the conditions where Ho'=4.0m with all other previous conditions remaining the same.

Fig.11 is a comparison of the failure probability of an upright section with that of motion from armor blocks as obtained by Mizumura et al(1988) versus the wave height and period respectively.

In the figure, the solid lines show the failure probability of the composite breakwater protected with armor blocks designed with a safety factor of one unit concerning slide of the upright section.

The failure probability of the armor units is greater than that of the composite breakwater in the total range of wave height. When the wave period is less than approximately 15sec, the risk to the armor blocks is also greater than that of the composite breakwater protected with armor blocks. This implies that when high waves attack a composite breakwater protected with armor blocks, armor

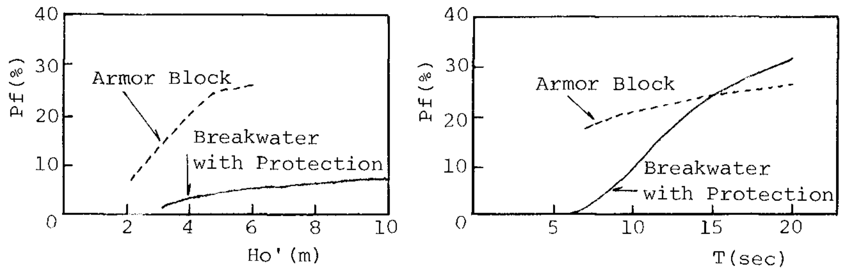

Fig.11 Comparison of Failure Probability of Protection with Upright Section

units collapse sooner than the upright section. However if armor blocks move sooner than the composite breakwater with protection, the width of the protection decreases, the wave force acting on the upright section through the protection increases (Hattori et al). Therefore the risk for motion of the upright section or geotechnical instability of the composite breakwater would become larger. It is therefore necessary to take into account the correlation between the armor blocks and upright section of the breakwater protected with armor blocks.

Conclusions

The following conclusions have been derived from this study.

1.Wave transformation, i.e. wave shoaling, breaking and retaining wave energy after breaking are acceptable for inclusion in simulations from offshore to the site of a breakwater.

2.Wave height is not as significant as wave period, water depth, caisson width and gradient of the sea bottom slope in failure probability of a composite breakwater protected with armor blocks. In particular, it can be seen that wave period is most significant.

3.In order to decrease risk to a breakwater, increases of caisson width proved effective because failure probability is reduced remarkably as the caisson width increases.

4.Geotechnical failure is potentially more dangerous than other failure modes.

5.Comparing the failure probability of armor unit displacement with that of the composite breakwater protected with armor blocks, the former is greater than the latter even when the wave height is increased. The risk of the former is less than that of the latter in the case of short period waves, while when the wave period becomes longer, the former is greater than the latter. However correlation between collapse of the upright section and blocks should be taken into consideration in the case of a composite breakwater protected with armor blocks.

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