安定性과 波浪制御機能을 考慮한 捨石構造物의 새로운 設計法

A New Design Method of Rubble Mound Structures with Stability and Wave Control Consideration

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要 旨

構造物의 安定性과 波浪制御機能을 考慮한 새로운 捨石構造物 設計法을 提案하였다. 이 設計法은 임의의 波浪條件下에서 波浪의 反射 및 run-up을 低減시키고, 安定性을 增大시키는 設計概念을 가졌다. 設計式은 許容破壞率과 安定性에 대한 波群特性의 影響을 導入하여 유도하였으며, 이에는 海洋波의 平均 run-sum이라는 새로운 外力概念이 使用되었다. 마지막으로 이 새로운 設計法에 의한 一様斷面과 複合斷面 捨石構造物 設計例로부터 이 設計法의 有用性과 利點을 檢證하였다.

Abstract

A new design method of rubble mound structures that includes the considerations of stability and wave control is proposed. Using the method, design of structures that reduce the wave reflection and run-up and increase the rubble stability is assured under the given wave conditions.

The new design formula is developed so that the allowable prcentage of damage and the wave grouping effects on rubble stability are also considered in design. For this a new definition of the mean run-sum is made. Finally, the new method is applied for the design of uniform and composite slope rubble mound structures and the significant advantages are found.

1. Introduction

In the conventional design process for rubble mound structures under the irregular wave conditions, one of the most important problems is the selection of statistic design wave. The irregular characteristics of ocean waves such as the spectrum shapes and the definition of wave grouping play an important role in the design process. Thus the effects of irregularity on the stability of rubble mound structures have been intensively studied by the several authors(Johnson et al., 1978; Sawaragi et al., 1984, 1985). They regarded the irregularities as an important external force index that should be considered in the design. Further-

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more, the stability will depend not only on the wave heights but also on the slopes of the structure that control the wave breaking conditions. These conditions are greatly affected by the interaction of successive waves and wave period.

Under the specific conditions, tendency of forming stable equilibrium slope can be usually observed in the destruction process of rubble mound structures. This means that the stability can be increased if the equilibrium slope is initially formed. Therefore it is desirable at the initial design stage to use a berm-type composite slope which is equivalent to the equilibrium slope. This is also important in an optimal design concept with allowable failure ratio.

Ryu(1984) and Ryu and Sawaragi(1986) have already emphasized the necessity of the reductions of reflected waves and run-up for a calm sea and construction of lower crownheight coastal structures. Considering those effects of irregularity on the stability and the reduction of wave reflection and run-up, Sawaragi et al.(1985) and Ryu and Sawaragi(1986) developed the design formula introducing the irregular wave force index such as the mean run-sum. However it still has a problem that the design rubble weights must be calculated through the different design formulae for each slope.

In this paper, the reduction characteristics of wave reflection and run-up and stability increasing functions are studied through model tests with the various slope shapes. As a result, new design formulae are derived that can generally be applied for universal uniform slope and the optimal composite slopes. They account for both wave control and stability increasing functions. The allowable failure ratio and the effects of irregularity are also considered in them.

2. Model Experiments

A wave tank of 30m long, 70cm wide, and 95cm deep was used in the present experiments. An irregular wave generator is installed at an end of the tank. Fig. 1 shows characteristic dimensions of structures with uniform and composite slopes and the experimental conditions are listed in Table 1 and Table 2. Initial wave height is less than 3cm and then increases until 100% destruction of rubble mound results for each wave period.

For the test of the sensitivity of rubble mound structures to the wave grouping and other irregularity parameters, an irregular wave simulation technique of the impulse response function method is used. It is basically the same as the method of Kimura(1976) and the spectrum shape of irregular waves can be arbitrarily controlled. The frequency spectrum of ocean waves is normally expressed as:

$$S(f)=S(f_p)\left(\frac{f}{f_p}\right)^{-m}\exp\left[\frac{m}{n}\left\{1-\left(\frac{f}{f_p}\right)^{-n}\right\}\right] \quad (1)$$

where f_p is the peak frequency; m and n are the constants which specify the spectrum shape. The combinations of peak frequency and shape parameters used in this study are also listed in Table 2. Using these basic irregular waves, 200 cases of stability experiments were carried out, and the destruction process and the wave motions on the slope are pictured

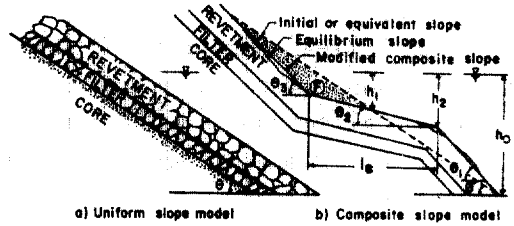

Fig. 1. Schematic diagram of model breakwaters.

Table 1. Experimental structure conditions

General conditions	Slope conditions						
	Uniform slope	Composite slope					
	θ	h_1(cm)	l_B(cm)	θ_1	θ_2	θ_3	θ'
W_a : 20g		5	15	1 : 1.5	0	1 : 1.5	1 : 2.0
l_a : 1.96cm	1 : 1.5	5*	25*	1 : 1.5*	0*	1 : 1.5*	1 : 2.3*
l_f : 1.0cm~1.5cm		5	30	1 : 1.5	0	1 : 1.5	1 : 2.5
		5	35	1 : 1.5	0	1 : 1.5	1 : 2.7
l_c : 0.5cm	1 : 2.0	10	15	1 : 1.5	0	1 : 1.5	1 : 2.0
		10	25	1 : 1.5	0	1 : 1.5	1 : 2.3
r_a : $2.0l_a$	1 : 3.0	10	30	1 : 1.5	0	1 : 1.5	1 : 2.5
h_0 : 20cm		10	35	1 : 1.5	0	1 : 1.5	1 : 2.7

W_a : weight of revetment rubble, l_a : characteristic length of revetment rubble,
l_f : characteristic length of filter layer rubble, l_c : characteristic length of core layer rubble,
r_a : revetment thickness, h_0 : water depth at the toe of breakwater,
* : the composite slope configuration examined under the irregular waves.

Table 2. Experimental wave conditions

Regular wave		Irregular wave				
T(sec)	H(cm)	case	f_p	$S(f_p)$	m	n
0.8	3.0	$W.1$	1.0	5.0	5	4
0.9	5.0	$W.2$	0.9	5.0	5	4
1.0	6.0	$W.3$	0.6	5.0	5	4
1.1	7.0	$W.4$	1.0	5.0	6	2
1.2	9.0	$W.5$	0.8	5.0	6	2
1.4	11.0	$W.6$	0.6	5.0	6	2
1.6	13.0					
1.8	15.0					

and analyzed by 16mm high speed cine camera (50 frame/sec) and analyzer.

For the measurement of the water surface elevation in the presence of the breakwaters, the capacity type wave gauges were located at 2cm intervals along them. The reflection coefficient was estimated by the Healy's method under the regular wave conditions, while the two point measuring method(Goda and Suzuki, 1976) was used to obtain the reflection coefficient under the irregular wave conditions. The wave run-up and run-down were measured by a run-up meter set on the slope surface. After digitizing all the data of water surface elevation recorded in an analog data recorder, the individual wave analysis and the spectral analysis methods were applied for the investigation of hydraulic characteristics on the slopes.

3. Characteristics of Irregular Waves

3.1 Reliability of generated waves

Spectrum shapes of all generated experimental waves were in good agreement with those of expected waves as given in Table 2. For the probability distribution of wave heights, periods, and surf-similarity parameters, both experimental and generally-known theoretical results are fairly coincident. As for the statistical reliability of generated waves and related discussions, readers are referred to Ryu(1984) or Sawaragi et al.(1985).

Fig. 2 shows the occurrence probability of run-length of higher waves $j(j=1, 2, 3, \cdots, \infty)$ for the present data, previously reported field data, and stochastically predicted data. The present data is the mean of 200 experiments with irregular waves, and the field data of Burcharth(1980) and Rye(1974) are used in the figure. The grouping characteristics of present

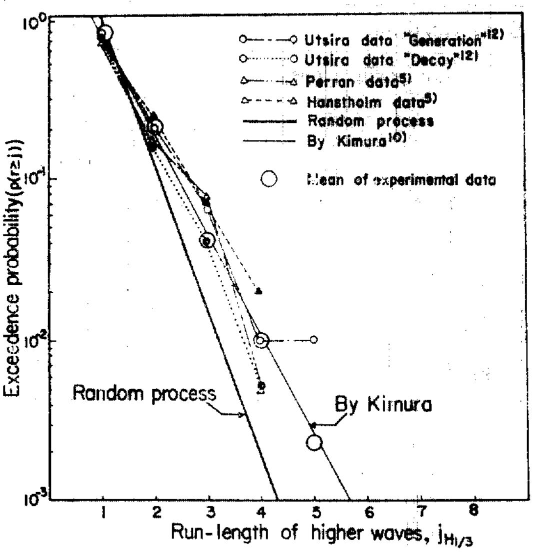

Fig. 2. The occurrence probability of run-length j of higher waves.

experimental waves are also satisfactorily coincident with that of the field data. Thus it is concluded that the model irregular waves are satisfactorily simulated the ocean wave trains.

3.2 Mean run-length and mean run-sum

The effects of grouping waves on the stability of coastal structures were considered through the run-length of higher waves by Johnson et al.(1978). Nontheless, the effects of wave period and resonance condition on the stability are significant as well(Sawaragi et al., 1983; Bruun et al., 1978). Therefore both effects must be taken into consideration. However irregularity parameter may vary depending on the definitions of the run. Thus a unified definition of irregularity parameter is needed. This can be achieved through the comparative study of correlations between differently defined irregularity parameters. For this end a new irregularity parameter whose significancy is verified by applying to the representation of the stability of rubble mound structures is proposed. The new grouping parameter used here in is a conditional run of ξ_0^* under the condition of critical wave height H_c (Sawaragi et al., 1985). Here ξ_0^* is the relative surf-similarity parameter:

$$\xi_0^* = \frac{\xi}{\xi_0} = \frac{\tan\theta/\sqrt{H/L_0}}{\tan\theta/\sqrt{(H/L_0)_{max}}} \qquad (2)$$

In Eq. (2), θ is the slope of the structures, H the wave height, L_0 the deep sea wave length, and subscript max the maximum wave. As for the detail discussions of the run, readers are referred to Sawaragi et al.(1985).

Mean run-length of the run has the linear relations with the spectrum peakedness parameter Q_p. As the run-length is only the number of group formed waves, it is difficult to introduce the run-length into the design formula as a external force parameter. Hence, a new notion of run-sum must be used. In the present study the run-sum is defined as the energy-sum of grouped waves. Thus the mean run-sum(E_{sum}), i.e., the mean of run-sum in an irregular wave train is expressed as:

$$E_{sum} = \frac{1}{8}\rho_w g \sum_{k=1}^{\infty} H_k^2 / \sum_{j=1}^{\infty} N_j \qquad (3)$$

where N_j is the numbers of run-length $j(j=1, 2, 3, \cdots, \infty)$, $H_k(k=1, 2, 3, \cdots, \infty)$ denote the k-th wave height of group formed waves in a wave train, ρ_w is the density of sea water and g is the acceleration of gravity. It is noted that the E_{sum} can be empirically estimated by the following equation(Sawaragi et al., 1985).

$$E_{sum}\ j(\xi_0^*/H_c) = \rho_w g H_c^3 (0.04Q_p + 0.13), \text{ for } H_c = H_{1/3} \qquad (4)$$

$$Q_p = \frac{2}{m_0^2}\int_0^{\infty} f S^2(f)\,df \qquad (5)$$

where, $m_0 = \int_0^{\infty} S(f)\,df$

4. Effects of the Friction Coefficient and the Slope on the Stability

4.1 The effect of friction coefficient

Many researchers(Sawaragi et al.,1983, 1984;

Losada and Gimenez-Curto, 1979; Günbak and Merzi, 1983; Brunn and Günbak, 1976, 1978; Ahrens, 1975, 1981) pointed out that the wave period was an important design factor for stability. In the sense of tendency their research results can be summarized as: the stability number N_s varies with surf-similarity praameter ξ or wave steepness, and the minimum N_s appears in the range of $2<\xi<3$. Ryu(1984) suggested the use of the parameter $\xi_0{}^*$, rather than ξ, to represent the stability with a universal stability curve for the various slopes because ξ for the minium point of N_s changed mainly due to the slope angle.

However, the value of stability number including minimum N_s varies with the change of the initial slope and damage ratio.

$$N_s=\frac{\gamma_r{}^{1/3}H_D(\%)}{(\gamma_r/\gamma_w-1)W_a{}^{1/3}}=(K_D\cot\theta)^{1/3} \quad (6)$$

where γ_a and γ_w are specific weights of rubbles and water, $H_D(\%)$ is the design wave height for the failure ratio in percent, W_a the weight of rubble unit, θ the slope angle, and K_D the stability coefficient in the Hudson's formula (1959). Unless the equation for N_s stated otherwise, the design of structure with the wave period effects on the stability must be done under individual slope-base. That is, the stability curve corresponding to the specific slope must be used in the design process. This kind of difficulty occurs whenever the friction coefficient and the slope angle are not included in the stability number.

To estimate the effect of friction coefficient on the stability, a series of experiments was carried out for various armour materials such as quarry stone(W_a=30g, 50g), concrete cube (W_a=100g), and tetrapod(W_a=100g). 50 experiments are carried out for each material. The friction coefficient f is then estimated as

$$f=\tan\phi \quad (7)$$

where ϕ is the repose angle of materials in

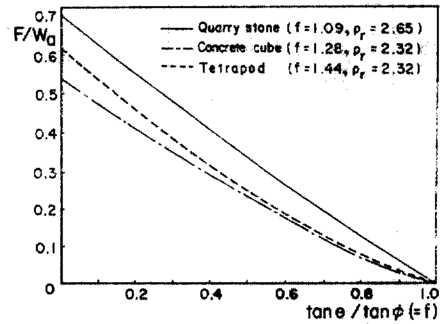

Fig. 3. Variation of the destructive force due to change of slope.

the water.

As a result, obtained the mean friction coefficients of 1.09, 1.28 and 1.42 for the quarry stone, concrete cube and tetrapod respectively, and the standard deviation was 0.1 for all materials.

On the other hand, when the hydrodynamic force acts upon an armour unit, the variance of the destructive force F according to the change of the slope can be estimated by the following equation:

$$F=\left(1-\frac{\rho_w}{\rho_r}\right)W_a(f\cos\theta-\sin\theta) \quad (8)$$

Using Eq. (8), the nondimensional destructive force F/W_a due to the change of relative slope ($\tan\theta/\tan\phi$) is calculated and plotted in Fig. 3. In the figure, the correlation between two parameters has almost linear regardless the change of the friction coefficient. From the experimental results, it is found that the effects of the friction coefficient and the slope angle can be reasonably approximated through the linear relation.

4.2 The effect of slope angle

For the effect of relative slope angle on the stability, a modified stability number N_s' of Eq. (9) is derived from the linear relation of Fig. 3.

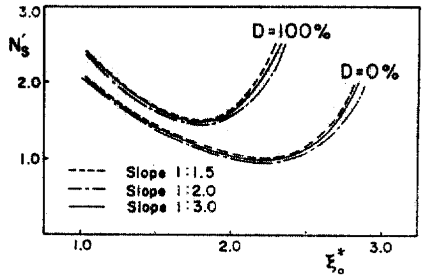

Fig. 4. N_s'-ξ_0^* curves for various slopes.

$$N_s' = N_s \frac{\tan\theta}{\tan\phi} \tag{9}$$

Fig. 4 shows the relation between the modified stablity number N_s' and the relative surf-similarity parameter ξ_0^* in case of regular waves. In the figure, the variation of N_s' shows similar curve for the same ratio of destruction regardless the value of initial slopes. The minimum N_s' appears in the range of $1.5 < \xi_0^* < 2.5$ and associated with the minimum N_s' is shifted to smaller region of ξ_0^* as the damage ratio increases. Here the destruction ratio $D(\%)$ is defined as:

$$D = \left(\frac{A_f}{A_0}\right) \times 100 \tag{10}$$

where A_f is the destructed volume of the cover layer and A_0 is its destructed volume when the destruction reaches the core layer. For the discussion of physical meaning of the damage ratio, Sawaragi et al(1983) is referred. The reason for shifting of the minimum N_s' is the change of the local slope at the destructed area during the formation of equilibrium slope. However, the stability is not correctly represented. The curves convenient for the design purpose should be obtained to consider the wave period effects. This can be done when we use the monochromatic wave conception such as the design wave height and period.

4. 3 Equilibrium slope

The characteristics of equilibrium slope formation due to severe waves are important in designing rubble mound structures. For the quantitative discussion of the equilibrium slope, its characteristic length is defined as shown in Fig. 1(b).

Since θ_1, θ_2 and θ_3 vary depending on the characteristic berm depth and width of equilibrium slope, it is necessary first to discuss the width and the depth of berm in an equilibrium slope. Ryu and Sawaragi(1986) discussed these, and as a result, the berm depth and the berm width of equilibrium slopes are formulated as:

$$\left.\begin{array}{l} 0.4 \leq h_1/H \leq 0.5 \\ 0.9 \leq h_2/H \leq 1.1 \end{array}\right\} \tag{11}$$

$$\frac{l_B}{L_{0max}} = 2.075 \frac{H_{1/3}}{L_{01/3}} + 0.04 \tag{12}$$

where h_1 and h_2 are the minimum and maximum water depths of the berm, H the incident wave height for the regular wave, l_B the berm width; L_0 the deep sea wave length, and the subscripts max and 1/3 denote the maximum wave and the significant wave respectively.

Under the conditions of both regular and irregular waves, rubble mound structures with composite slopes are modelled using the Eqs. (11) and (12). Then the characteristics of equilibrium slope are represented. A series of experiments for the stability and the hydraulic characteristics such as reflection and run-up on the slope are performed under the conditions of Table 1 and 2.

5. Wave Control and Stability Increasing Functions of the Composite Slope

5. 1 Stability increasing function

To apply the characteristics of Fig. 4 for the presentation of stability under irregular waves, the mean run-sum is normalized using the characteristic length of armour unit and relative slope. Then it is used as an index of

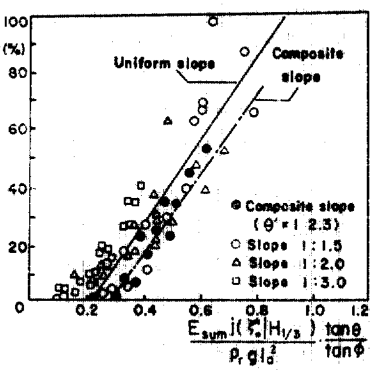

Fig. 5. The relations of relative E_{sum} and percentage failure ratio($D\%$).

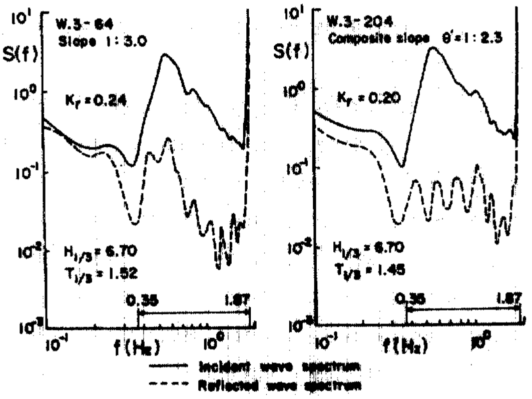

Fig. 6. The reflection control function of the composite slope by irregular waves.

irregular wave force, and the variation of damage ratio due to this index is investigated. The results are shown in Fig. 5. In the figure, black circles denote the experimental data for the composite slope and white symbols for the uniform slopes.

Although the difference of damage ratio for the same irregular wave force index is occured between the uniform and composite slopes caused by the stability increasing functions on the composite slope, the significant difference is not appeared due to the variation of the slope for the uniform slopes. It means that the index is a very useful and resonable parameter to present the stability of rubble mound structures under the irregular wave condition as well as the regular wave. From Fig. 5, the stability increasing characterisitcs of the composite slope is also clarified for the irregular wave. If the equilibrium slope is determined and constructed as a shape of rubble mound structures at the initial design stage, the stability increases more than 50% compared with the case of the uniform slope. As for the detail discussions of stability increasing mechanism under the regular wave condition, readers are referred to Ryu and Sawaragi(1986).

5.2 Wave control function

The reductions of reflection coefficient and run-up are very important for a calm sea and a low crest height respectively. Ryu and Sawaragi(1986) discussed the wave control functions according to the change of relative depth and width of the berm of composite slope. As a result, it is shown that the reflection coefficient varies with the berm width l_B and depth h_B, and the minimum value of K_r always appears under the condition of $l_B/L_0 \fallingdotseq 0.2$ and $H/2 < h_B < H$. If the optimum composite slope is selected, the reflection coefficient decreases more than 50%, the run-up height decreases about 10% and the run-down height decrease about 20~50% compared with the uniform slope cases. As for the detail discussion of the reduction mechanisms, readers are referred to Ryu and Sawaragi(1986).

The reflection characteristics of irregular waves on the uniform and composite slopes are now investigated to examine the applicability of the results to the regular wave conditions. Fig. 6 shows examples of the incident and the reflected wave spectra on an uniform and composite slopes. The reflection coefficient on the composite slope is smaller than that on the uniform slopes. It is noted, however, that the equivalent slope of the composite slope is 1/2.3, which is steeper than the uniform slope of 1/3.

As is seen from the spectrum of reflected waves, the predominent frequency can not be identified in case of the composite slope while it appears in case of the uniform slope, being almost the same as that of incident wave spectrum. From the fact that the composite slopes causes the scattered frequency spectral density of reflected waves, it can be stated that the reflection control function of composite slopes for the irregular waves is basically same as for the regular waves.

6. A New Design Method for the Irregular Waves

6.1 A new design formula

From the correlation between the mean run-sum and spectrum peakedness parameter and the results shown in Fig. 5, the design rubble weight can be estimated by the following steps. From Fig. 5, the best fitting line for the variance of damage ratio due to the irregular wave force can be derived empirically as:

$$D(\%)=153.8\left[\frac{E_{sum}j(\xi_0^*|H_{1/3})}{\rho_r g l_a^2}\ \frac{\tan\theta}{\tan\phi}\right]-30.1 \tag{13}$$

for the uniform slopes,

$$D(\%)=136.4\left[\frac{E_{sum}j(\xi_0^*|H_{1/3})}{\rho_r g l_a^2}\ \frac{\tan\theta}{\tan\phi}\right]-3.63 \tag{14}$$

for the composite slopes.

where θ' is the equivalent slope of the composite slope, and $j(\xi_0^*|H_{1/3})$ denotes the conditional run of ξ_0^* under the condition of significant wave height. To estimate the design weight of a rubble units, Eqs. (13) and (14) are transformed as:

$$W_a=\rho_r g l_a^3 =\left[\frac{153.8E_{sum}j(\xi_0^*|H_{1/3})}{(\rho_r g)^{1/3}(D+30.1)}\ \frac{\tan\theta}{\tan\phi}\right]^{3/2} \tag{15}$$

for the uniform slopes,

$$W_a=\left[\frac{136.4E_{sum}j(\xi_0^*|H_{1/3})}{(\rho_r g)^{1/3}(D+36.3)}\ \frac{\tan\theta}{\tan\phi}\right]^{3/2} \tag{16}$$

for the composite slopes.

From the Eqs. (15), (16) and (4), with the spectrum peakedness parameter Q_p which is directly calculated in the spectrum analysis, we can easily derive the design formula as follows:

$$W_a=\left[\frac{\rho_w g(6.15Q_p+20.0)}{(\rho_r g)^{1/3}(D+30.1)}\ \frac{\tan\theta}{\tan\phi}\right]^{3/2} H_{1/3}^3 \tag{17}$$

for the uniform slopes,

$$W_a\doteqdot\left[\frac{\rho_w g(5.46Q_p+17.73)}{(\rho_r g)^{1/3}(D+36.3)}\ \frac{\tan\theta'}{\tan\phi}\right]^{3/2} H_{1/3}^3 \tag{18}$$

for the composite slopes.

The design formulas of Eqs. (17) and (18) reflect the allowable percentage of damage and the wave grouping effects on the rubble stability using a new conception of the mean run-sum as an index of the irregular wave force. For the uniform slopes, only Eq. (17) is used to calculate the design rubble weight. It has an epoch-making advantage compared with tedious method of previous work(Sawaragi et al., 1985). Until now, this is an only one that has directly introduced or considered the irregularity effects of ocean waves and allowable percentage of damage into the design formula. Eq. (18) is a design formula for the composite slope. However, it can not be used to all types of composite slopes, but can be used for the optimal conditions of composite slope with stability and wave control consideration such as:

$$\left.\begin{array}{l} l_B\doteqdot 0.25L_p \\ 0.4H_{1/3}<h_B<0.7H_{1/3} \end{array}\right\} \tag{19}$$

where L_p is the wave length for the peak frequency of the energy spectrum. In fact, this is an optimal condition of the composite slope in the authors' previous study (Ryu and Sawaragi, 1986).

6.2 Design examples

Using the new design formulas of Eqs. (17) and (18), the variation of the relative weight of armour unit is calculated according to the spectrum peakedness parameter considering the allowable damage ratio. Fig. 7 is one example for the uniform slope of 1/2.3 and for the equivalent composite slope of 1/2.3. As can be seen in the figure, the design weight of armour unit becomes heavier as the spectrum peakedness parameter increases and the rubble weight for composite slope becomes lighter than that of uniform slope.

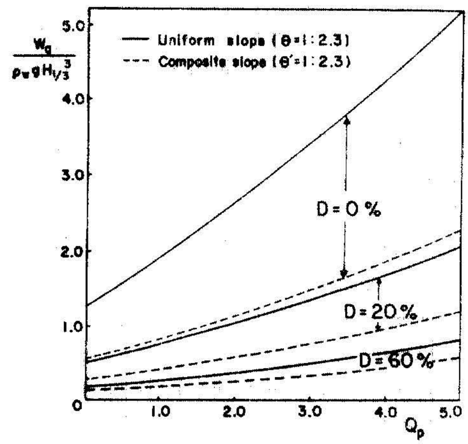

Fig. 7. Design example on the variation of relative design weight according to the spectrum peakedness parameter.

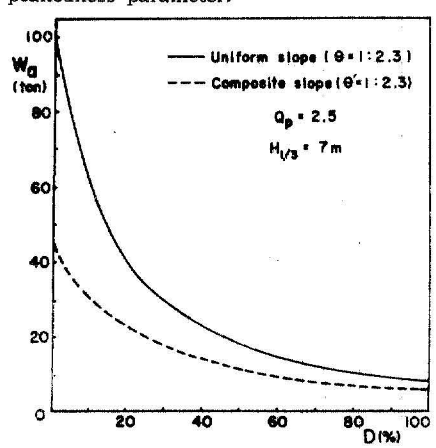

Fig. 8. Design example on the variation of design weight according to the allowable failure ratio.

Fig. 8 shows the variation of the design weight due to the change of the allowable damage under the design wave conditions of $Q_p=2.5$ and $H_{1/3}=7$m for the same slope condition with Fig. 7. For both slope conditions, the design weight becomes heavier as the allowable damage ratio decreases. The design weight for the composite slope is half or less of that for the uniform slope in the initial damage($D<10\%$). Considering this stabiltiy increasing and forementioned wave control functions, the optimal design concept using the rubble mound with the composite slope should be applied to the design of coastal structures.

7. Conclusions

A new design method is developed based on the results of stability experiments under the regular and irregular wave conditions. The new design formula reflects the effects of spectrum shape, breaking conditions, wave period and wave grouping characteristics on the stability. The allowable damage ratio is also introduced in it.

The new design formula for the uniform slope structure is more advantageous than others, since the present method uses only one formula for every slope. The importance of the design of composite slope structures is emphasized through the comparative studies of the hydraulic characteristics and the rubble stability on the uniform and composite slopes. A new method of designing composite slope structures with optimum berm width and depth is proposed, by which the reduction of wave reflection and run-up and increase in rubble stability are assured under the given wave conditions.

Acknowledgements

The financial support of the Korea Science and Engineering Foundation for this research is greatly acknowledged. The author would like to thank to Mr. H.J. Kim and S.K. Kim for their helpful research assistants.

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(接受 : 1987. 6. 8)