CHAPTER 160

A New Design Method of Rubble Mound Structures

Cheong-Ro Ryu[1)] and Toru Sawaragi[2)]

Abstract

A new design method of rubble mound structures with stability and wave control consideration is proposed, by which the reduction of wave reflection and run-up and increase in rubble stability are assured under the given wave conditions. Wave control and stability increasing functions due to change of the slope shape of rubble mound structures are discussed on the basis of the experimental results for regular and irregular waves.

The new design formula developed here considered the allowable percentage of damage and the wave grouping effects on rubble stability using a new assumption of the mean run-sum as an index of the irregular wave force. The run-sum is defined as the energy sum of a run satisfying a critical wave condition and the mean run-sum is the mean of run-sum for a irregular wave train.

1. Introduction

The selection of statistic design wave height in irregular waves is the most important problem in the design of rubble mound structures using the conventional design formulas. The problem mainly occurs due to the irregularities of ocean waves well known as the spectrum shapes and the grouping characteristics etc.. The effects of irregularity on the stability of rubble monnd structures were pointed out by Johnson et al.(1978) and Sawaragi et al.(1984 and 1985) as an important external force index that should be considered in the design. Furthermore, the stability will depend not only on the wave height but also on the slope controling wave breaking conditions which is greatly affected by the interaction of successive waves and wave period.

The destruction process of rubble mound structures shows a tendency to form a stable equilibrium slope under the given condition. This means that the stability increases due to the formation of equilibrium slope. It is desirable to use a berm type composite slope at the initial design stage for the ease of construction, and is important in a optimal design concept with allowable failure ratio.

Ryu(1984) and Ryu et al.(1986) stressed the reflected wave problems for a calm sea and the reduction of run-up correspond to the necessity

---

1) Assistant Professor, Department of Ocean Engineering, National Fisheries University of Pusan, Pusan, 608 Korea.

2) Professor, Department of Civil Engineering, Osaka University, Osaka, Japan.

of construction of lower crown-height in the design of coastal structures with an optimal design concept. Considering those irregularity effects on the stability and the reduction of wave reflection and run-up, Sawaragi et al.(1985) and Ryu et al.(1986) developed the design formula introducing the irregular wave force index such as the mean run-sum, but still the formula has a problem that the design rubble weight must be calculated by every design formula for every slope.

In this paper, the characteristics on the reduction of wave reflection and wave run-up, and stability increasing functions by the change of the slope shape are studied through model tests, and new design formulas are developed for the uniform slopes that can apply an universal formula to every slope and for the optimal composite slope considered wave control and stability increasing functions. The new design formulas have considered the allowable failure ratio and the irregularity effects including wave period and wave grouping effects on the stability of rubble mound structures with the conception of the mean run-sum of the conditional run of modified surf-similarity parameter under the condition of significant wave height.

2. Model experiments

A wave tank of 30m long, 70cm wide, and 95cm height was used in the present experiments. An irregular wave generator is installed at an end of the tank. For all experiments, characteristic dimensions of structures with uniform and composite slopes shown as Fig.1 were selected as specified in Table 1. Wave height is initially set at less than 3cm and then it increase until 100% destruction of rubble mound results.

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:

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

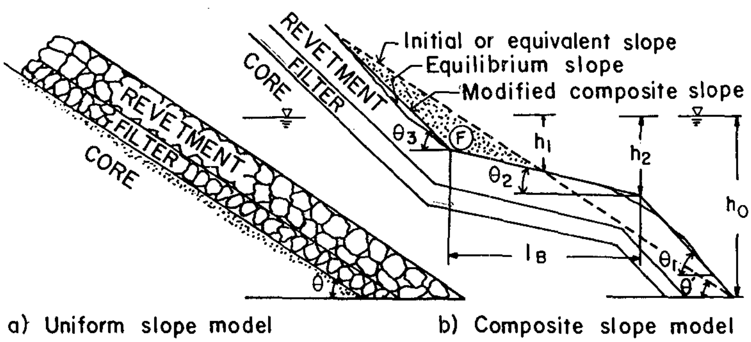

Fig. 1 Schematic diagram of model breakwaters.

Table 1. Experimental conditions

General conditions	Slope conditions							Wave conditions						
	Uniform slopes	Composite slope configurations						Regular wave		Irregular wave				
	θ	h_1 (cm)	l_B (cm)	θ_1	θ_2	θ_3	θ'	T(sec)	H(cm)	case	f_p	$S(f_p)$	m	n
W_a : 20g		5	15	1:1.5	0	1:1.5	1:2.0	0.8	3.0	W.1	1.0	5.0	5	4
l_a : 1.96cm	1:1.5	5*	25*	1:1.5*	0*	1:1.5*	1:2.3*	0.9	5.0	W.2	0.9	5.0	5	4
l_f : 1.0cm		5	30	1:1.5	0	1:1.5	1:2.5	1.0	6.0					
1.5cm		5	35	1:1.5	0	1:1.5	1:2.7	1.1	7.0	W.3	0.6	5.0	5	4
l_c : 0.5cm	1:2.0	10	15	1:1.5	0	1:1.5	1:2.0	1.2	9.0	W.4	1.0	5.0	6	2
		10	25	1:1.5	0	1:1.5	1:2.3	1.4	11.0	W.5	0.8	5.0	6	2
r_a : 2.0 l_a	1:3.0	10	30	1:1.5	0	1:1.5	1:2.5	1.6	13.0					
h_0 : 20cm		10	35	1:1.5	0	1:1.5	1:2.7	1.8	15.0	W.6	0.6	5.0	6	2

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

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 1. 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 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 proposed by Goda et al.(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 shape of all experimental waves were in good agreement with those of expected waves given in Table 1. For the probability distribution of wave heights, periods and surf-similarity parameters, experimental and well-known theoretical results on the statistics of ocean waves coincide fairly well. As for the ascertain of 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($j=1,2,3,\ldots\ldots,\infty$) for the present data, reported field data and predicted results by the well-known stochastic process. The present data is the mean of 200 experimental cases of irregular waves, and field data are referred from the results analyzed by Burcharth(1980) and Rye(1974). The grouping characteristics of experimental waves in the study satisfied also with that of the field data. From these results, the model irregular waves are judged to be 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 with the run-length of higher waves by Johnson et al.(1978). However, the effects of wave period and resonance condition on the stability have been verified as a non-negligible one by Sawaragi et al. (1983), Bruun et al.(1978) and many other researchers. Considering these effects, in the study, a new irregularity parameter is proposed by the comparative study of correlations between the irregularity parameters for various definitions of the run, and its significancy is verified by applying to the representation of the stability of rubble mound structures. The new grouping parameter finally used here is a conditional run of ξ_o^* under the condition of critical wave height H_c same as Sawaragi et al.(1985), where ξ_o^* means the relative surf-similarity parameter

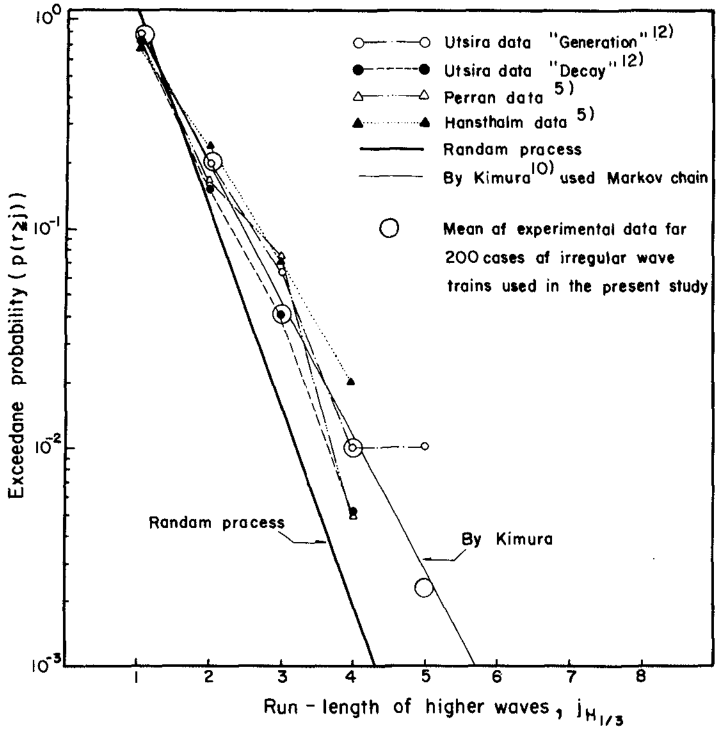

Fig. 2 The occurrence probability of run-length j of higher waves.

such as:

$$\xi_o^* = \frac{\xi}{\xi_o} = \frac{\tan\theta \,/\sqrt{H/L_o}}{\tan\theta \,/\sqrt{(H/L_o)_{max}}} \tag{2}$$

where θ is the slope of the structures, H the wave height, L_o 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 have the linear relationship 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 conception of run-sum is proposed in the study, where the run-sum is defined as the energy-sum of grouped waves. The mean run-sum(E_{sum}) is the mean of run-sum in a irregular wave train expressed as:

$$E_{sum} = \frac{1}{8}\,\rho_w\, g \sum_{k=1}^{\infty} H_k \Big/ \sum_{j=1}^{\infty} N_j \tag{3}$$

where N_j is the numbers of run-length $j(j=1,2,3,\ldots,\infty)$, $H_k(k=1,2,3,\ldots,\infty)$ denote the k-th wave height of group formed waves in a wave trian, ρ_w is the density of sea water and g is the acceleration of gravity. From the author's another study results on the relations among the mean run-sum, the mean run-length and the spectrum peakedness parameter Q_p, we can estimate the E_{sum} by the following equation(Sawaragi et al.(1985)).

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

$$Q_p = \frac{2}{m_o^2}\int_o^\infty f\, S^2(f)\,df \tag{5}$$

where, $m_o = \int_o^\infty S(f)\,df$

4. Effects of the friction coefficient and the slope on the stability

4.1 The effect of friction coefficient

The author(1983,1984), Losada and Gimenez-Gurto(1979), Günbak et al.(1983), Brunn et al.(1976,1978) and Ahrens(1975,1981) pointed out the wave period effects on the stability as a important factor in design. And they reported same tendency of results that the stability number N_s varied with surf-similarity parameter ξ or wave steepness, and the minimum N_s was appeared in the range of $2<\xi<3$. Ryu(1984) suggested the parameter ξ_o^* to present the stability with a universal stability curve for the various slopes instead of ξ because ξ for the minimum point of N_s changed by 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. If this problem can not be improved, the design considering the wave period effects on the stability must be done by using the every stability curve for the every slope. This problem occurs, because the friction coefficient and the slope angle is not included in the stability number indicated as following:

$$N_s = \frac{\gamma_w^{1/3}\, H_{D(\%)}}{(\gamma_r/\gamma_w - 1)\, W_a^{1/3}} \tag{6}$$

where γ_r 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).

To estimate the effect of friction coefficient on the stability, the experiments on the variation of friction coefficient 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 times of experiments for every materials are repeatedly carried out. The friction coefficient f is estimated by the relation of

$$f = \tan\phi \tag{7}$$

where ϕ is the repose angle of materials in the water.

As a results, 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 act upon an armour unit, the variance of the destructive force F according tothe change of the slope can estimate by the following equation:

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

From Eq.(8), the calculated results of nondimensional destructive force F/W_a due to the change of relative slope($\tan\theta/\tan\phi$) are shown as Fig.3. In the figure, the correlation between two parameters has almost linear relation regardless the change of the friction coefficient. From the Fig.3, the effects of the friction coefficient and that of slope angle can estimate by using the linear relation.

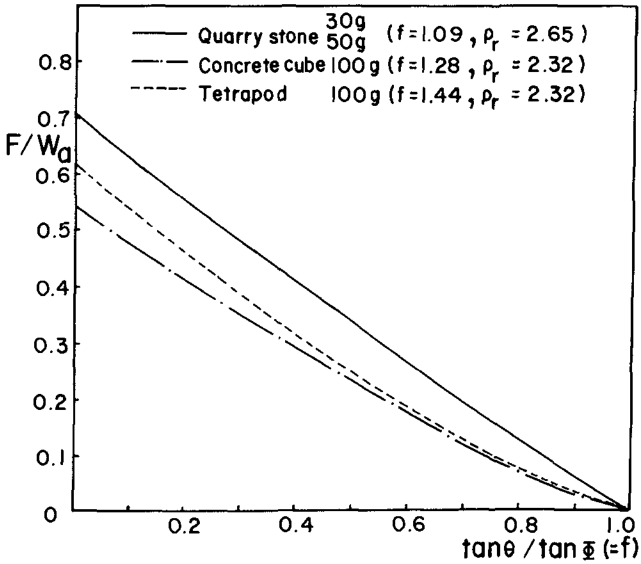

Fig. 3 Variation of the destructive force due to change of slopes.

4.2 The effect of slope angle

The effect of relative slope angle on the stability is considered using the modified stability number N_s' as shown as equation (9) from the linear relation of Fig.3.

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

Fig.4 corresponds to the correlation between modified stability number and ξ_o^* using the results for regular waves. In the figure, the variation of N_s' by ξ_o^* shows a similar curve regardless the difference of initial slopes for the same ratio of destruction. The minimum N_s' is appeared in the range of $1.5<\xi_o^*<2.5$ and the point of ξ_o^* shown the minimum N_s' is shifted to smaller region according to the progress of destruction. The destruction ratio D(%) is defined as:

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

where A_f is the destructed volume of the cover layer and A_o is the destructed volume where the destruction reaches the core layer. The physical meaning of this failure ratio was discussed in the previous work of the author(1983). This shifting of the minimum N_s' is occured caused by the change of the local slope at the destructed area during the formation of equilibrium slope. However, the stability is not correctly presented, the most convenient curves are obtained to consider the wave period effects by using 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).

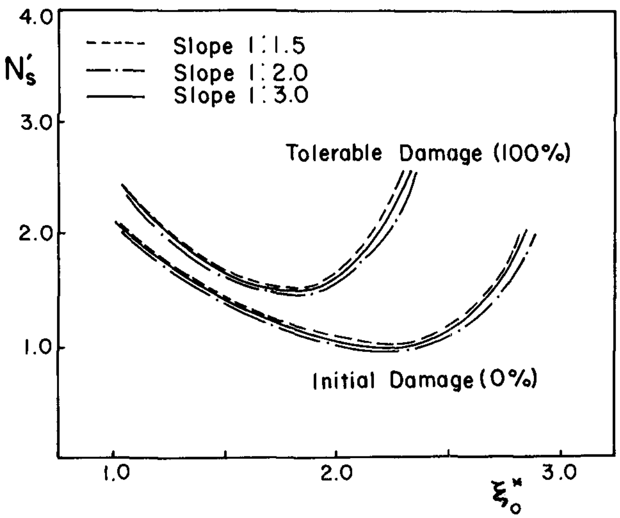

Fig. 4 $N_s' - \xi_o^*$ curves for various slopes.

Since θ_1, θ_2 and θ_3 vary depending on the characteristic 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:

$$\left.\begin{array}{l} 0.4 \le h_1/H \le 0.5 \\ 0.9 \le h_2/H \le 1.1 \end{array}\right\} \qquad (11)$$

$$\frac{l_B}{L_{o\,max}} = 2.075\,\frac{H_{1/3}}{L_{o\,1/3}} + 0.04 \qquad (12)$$

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

From the Eqs.(11) and (12), rubble mound structures with composite slopes are modelled simulating the characteristics of equilibrium slope under the conditions of regular and irregular waves, and experiments on the stability and the hydraulic characteristics such as reflection and run-up on the slope are carried out under the condition of Table 1 as well as for the uniform slopes.

5. Wave control and stability increasing functions of the composite slope

5.1 Stability increasing function

To applicate the characteristics of Fig. 4 to the presentation of stability for irregular waves, normalized the mean run-sum by the characteristic length of armour unit and relative slope. It is assumed as a index of irregular wave force, and the variation of damage ratio is investigated by the index. Fig. 5 shows the results. In the figure, black circles denote the experimental data for the composite slope and white symbols are that for the uniform slopes.

Although the difference of damage ratio for the same irregular wave force index between the uniform and composite slopes is occured 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 reasonable parameter to present the stability of rubble mound structures under the irregular wave condition as well as the regular wave. From the Fig.5, the stability increasing characteristics of the composite slope is also clarified for the irregular wave. 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

If the equilibrium slope is decided and constructed as a shape of rubble mound structures as the initial design stage, the stability is increased more than 50% comparing the stability for the uniform slope, but how is the reflection and the run-up on the slope.

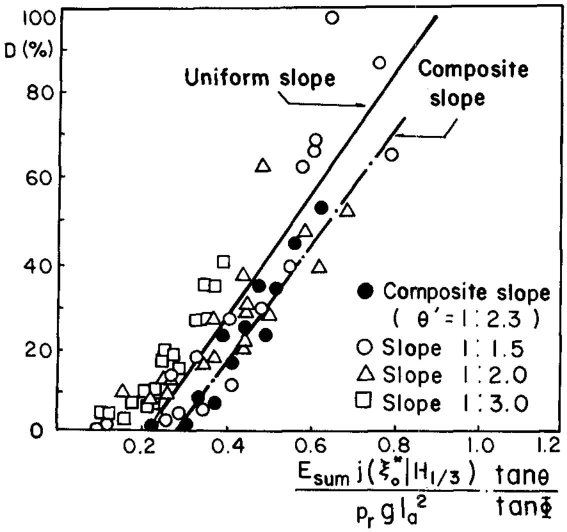

Fig.5 The relations of relative E_{sum} and percentage failure ratio(D%).

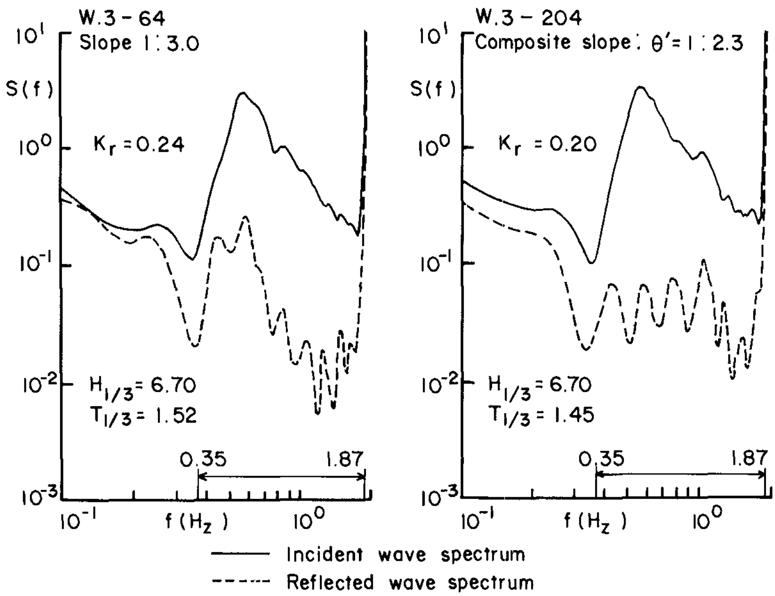

Fig.6 The reflection control function of the composite slope by irregular waves.

The reduction 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 results, it is clarified that the reflection coefficient varies with the berm width l_B and depth h_B, and the minimum value of K_r is always appeared under the condition of $l_B/L_o = 0.2$ 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 decreases about 20-50% comparing the results for the uniform slopes. 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 for regular waves. Fig.6 shows examples of the incident and the reflected wave spectra on an uniform slope and on a composite slope. The reflection coefficient on the composite slope are 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 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 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.

From the wave control and stability increasing functions of the composite slope, the author emphasize the necessity of composite slope for the constructing of the optimal rubble mound structure.

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 Fig.5, the design rubble weight can be estimated by the following steps. The best fitting line for the variance of destruction ratio due to the irregular wave force from the Fig.5 can be derived empirically as:

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

for the uniform slopes,

$$D(\%) = 136.4\left[\frac{E_{sum}\, j(\xi_o^{*}|H_{1/3})}{\rho_r\, g\, l_a^{2}}\,\frac{\tan\theta'}{\tan\phi}\right] - 36.3 \tag{14}$$

for the composite slopes.

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

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

for the uniform slopes,

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

for the composite slopes.

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

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

for the uniform slopes,

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

for the composite slopes.

The design formulas Eq.(17) and Eq.(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, the design rubble weight can be calculated by the only a design formula of Eq.(17). It is an epoch-making advantage comparing the author's previous work (Sawaragi et al. (1985)), and only a design formula that introduced or considered directly the irregularity effects of ocean waves and allowable percentage of damage into the design formula until now. Eq.(18) is a design formula for the composite slope, however, this formula can not use to the all type of composite slopes, only can use for the optimal conditions of composite

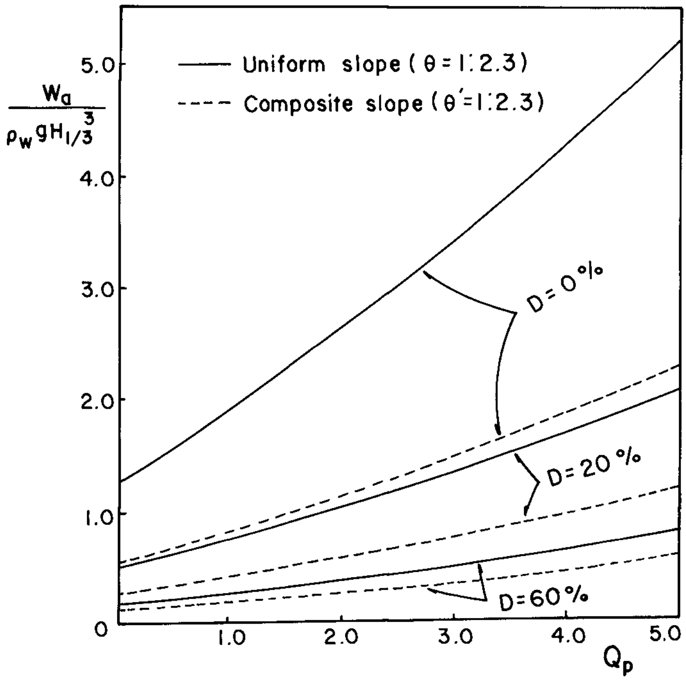

Fig.7 Design example on the variation of ralative design weight according to the spectrum peakedness parameter

slope with stability and wave control consideration such as:

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

where L_p is the wave length for the peak frequency of the energy spectrum. This condition is a optimal condition of the composite slope in the study of the author(1986).

6.2 Design examples

Using the new design formulas Eqs.(17) and (18), the variation of the relative weight of armour unit is calculated according to the spectrum peakedness parameter considered the allowable damage ratio in percent. 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 from the fig., the design weight of armour unit becomes heavier due to the increasing of spectrum peakedness parameter and the rubble weight for composite slope becomes more light than that of uniform slope.

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

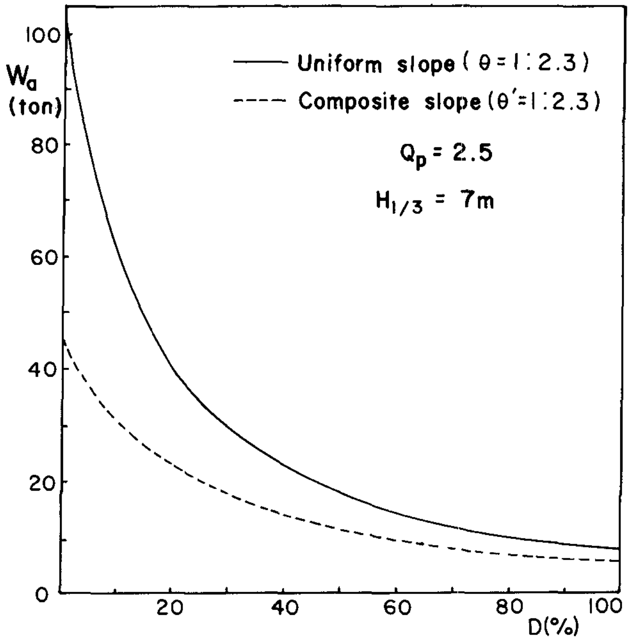

Fig.8 Design example on the variation of design weight according to the allowable failure ratio.

and $H_{1/3}$ =7m for the same slope condition with Fig.7. The design weight becomes heavier according to the decreasing of allowable damage ratio for both slope conditions, and 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 stability increasing and before mentioned 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, and can introduce the allowable damage ratio.

The design formula for the uniform slope structure has an advantage comparing the author's previous work, because the present method calculate by a 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 authors would like to thank to Mr. H. J. Kim and S. K. Kim for their helpful research assistants.

References

1. Ahrens, J.P. and B.L. McCartney (1975): Wave period effect on the stability of riprap, Proc. of the Special Conference on Civil Eng. in the Ocean/III, ASCE, pp.1019 - 1034.
2. Ahrens, J.P. (1981): Design of riprap revetment for protection against wave attack, U.S. Army, Corps of Engineers, C.E.R.C., TP 81-5.
3. Bruun, P. and A.R. Cünbak (1978): Stability of sloping structures in relation to $\xi=\tan\theta/\sqrt{H/L_o}$, Coastal Engineering, 1(4), pp. 287 - 322.
4. Bruun, P. and P. Johannesson (1976): Parameters affecting the stability of rubble mounds, Proc. of ASCE, WW2, pp.141 - 164.
5. Burcharth, H.F. (1980): A comparison of nature waves and model waves with special reference to wave grouping, Proc. of 17th International Conference on Coastal Eng.,2993 - 3009.
6. Goda, Y. and Y. Suzuki (1976): Estimation of incident and reflected waves in random wave experiments, Proc. 15th International Conf. on Coastal Eng., pp. 828 - 845.
7. Günbak, A. and N. Merzi (1983): Effect of wave period on the stability of rubble mound breakwaters, Proc. 8th International Harbor Cong. pp. 3.15 - 3.20.
8. Hudson, T.Y. (1959): Laboratory investigation of rubble mound breakwaters, Proc. of ASCE, WW3, pp. 93 - 121.
10. Kimura, A. (1976): Random wave simulation in a laboratory wave tank, Proc. 15th International Conf. on Coastal Eng., pp.368 -387.
11. Losada, M.A. and L. Cimenez-Curto (1979): The joint effect of the wave height and period on the stability of rubble mound breakwaters using Iribarren's number, Coastal Engineering, 3, pp. 77 -96.
12. Rye, H. (1974): Wave group formation among storm waves, Proc. 14th International Conf. on Coastal Engineering, pp.164 - 183.
13. Ryu, C.R. (1984): A study on the hydraulic optimal design of the rubble mound breakwaters, Thesis of Doctor of Eng., Osaka University, 165p. (in Japaneae)
14. Ryu, C.R. and T. Sawaragi (1986): Wave control functions and design principles of composite slope rubble mound structures, Coastal Eng. in Japan, Vol. 29, (to be published).
15. Sawaragi, T., C.R. Ryu and K. Iwata (1983): Consideration of the destruction mechanism of rubble mound breakwaters due to the resonance phenomena, Proc. 8th I.H.C., pp.3.197 - 3.208.
16. Sawaragi, T., C.R. Ryu and M.kusumi (1984): Destruction characteristics of rubble mound breakwaters by irregular waves, Proc. 31th Japanese Confenence on Coastal Eng., pp.562 - 566 (in Japanese).
17. Sawaragi, T., C.R. Ryu and M. Kusumi (1985): Destruction mechanism and design of rubble mound structures by irregular waves, Coastal Engineering in Japan, Vol. 28, PP. 173 - 189.