CHAPTER 115

Optimal Design of Rubble Mound Structures under the Irregular Wave

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Abstract

A design algorithm for the optimal cost effective rubble mound structures is developed considering extreme wave statistics and wave control functions as well as the stability under the irregular wave attack. In the algorithm, the conventional deterministic design method and the optimization technique, SUMT (Sequential Unconstrained Minimization Techniques) are used to evaluate the minimum cross section of the structure.

The applicabilty and design sensitivity of the algorithm is examined using experimental data on stability, run-up / overtopping and reflection under the various irregular waves that have different wave grouping characteristics. The field conditions not only the construction method but also the costs can be introduced in the cost estimation subroutine.

1. Introduction

Cost effectiveness is an important factor in the optimal design concept of coastal structures as well as the stability and wave control functions of the structures are. In the design, uncertainty of extreme waves and/or design waves, allowable damage considered destruction process, and various wave control functions of the structures in irregular wave field should be considered, however, these effects were not introduced systematically in the conventional design process (Hudson, 1959; CERC, 1984; Ryu and Sawaragi, 1986; Smith, 1987; Van der Meer, 1987).

In this study, conventional deterministic design method and a new optimization technique, SUMT are used simultaneously and/or as a dual system to develop a systematic design algorithm for the rubble mound structures considering those effects and cost effectiveness. In the construction of the algorithm, previous experimental works on stability, run up / overtopping and reflection by irregular waves with different wave

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grouping characteristics are applied to the present problem of setting up the modules for design constraints.

To discuss some aspects of the usefulness of the algorithm, we first analyze the reliability of the well known conventional design methods for regular and irregular waves. Next, check the design sensitivity due to the irregularity and the uncertainty of extreme waves, as well as construction field conditions and by introducing design concept of allowable damage and wave control functions.

2. Construction of the Optimal Design Algorithm

The algorithm is constructed with the modules for 1) extreme wave proability analysis, 2) structural stability analysis, 3) considering of wave run up, overtopping and reflection problems for wave-structural interaction analysis as the wave control functions, 4) total cost minimizing during life time, 5) drawing the structural dimensions.

In the construction of each module, the up-to-date research results including experimental results carried out in the study and conventional well known results are used simultaneously to study comparatively. In the design process, the conventional deterministic and the optimal design methods are used to calculate the structral dimensions and the total cost.

Figure 1 shows the flow of the optimal cost effectiveness design process. As shown in the figure, the calculation starts with the defined initial total cost (TCM_{max}), allowable damage steps (N), return period steps (NN). The total cost for every step can be estimated in the every iteration step, and the cross sectional dimensions for the minimum total cost condition is found as the optimal cost effective design.

3. Optimization of the Cross-Section of Structures

3.1 Design Variables

Design variables for rubble mound structures with uniform and composite slopes defined as following equation in relation to Figure 2 and Figure 3.

$$X = (X_1, X_2, X_3, X_4, X_5, X_6, X_7, X_8, X_9, X_{10})^T \quad (1)$$

where $X_1, X_2, \cdots, X_n$ are the characteristc length scales of the structure as shown in Figure 2, 3, X_5 in the uniform slope and X_8 in the composite slope are the specific length of the revetment armor, and h the water depth.

3.2 Application of the Optimization Technique

In the development of design algorithm, to minimize the cross-sectional area efficiently, the conventional deterministic design method is applied to calculate initial design variables, and the optimization technique such as Sequential Unconstrained Minimization Techniques (SUMT) are used shown as Figure 1.

The optimal design concept can be expressed as the finding problem of design variables under the condition of minimizing the objective function and satisfing the contraint functions as;

$$\text{Find} \quad X = (X_1, X_2, \cdots\cdots, X_n)^T \quad (2)$$

$$\text{Minimize} : f(X) \quad (3)$$

$$\text{Subject to} : g_j(X) \le 0, \quad j=1,\ldots\ldots,m \quad (4)$$

$$h_k(X) = 0 \ , \ k = 1, \ \ldots\ldots \ , \ l \quad (5)$$

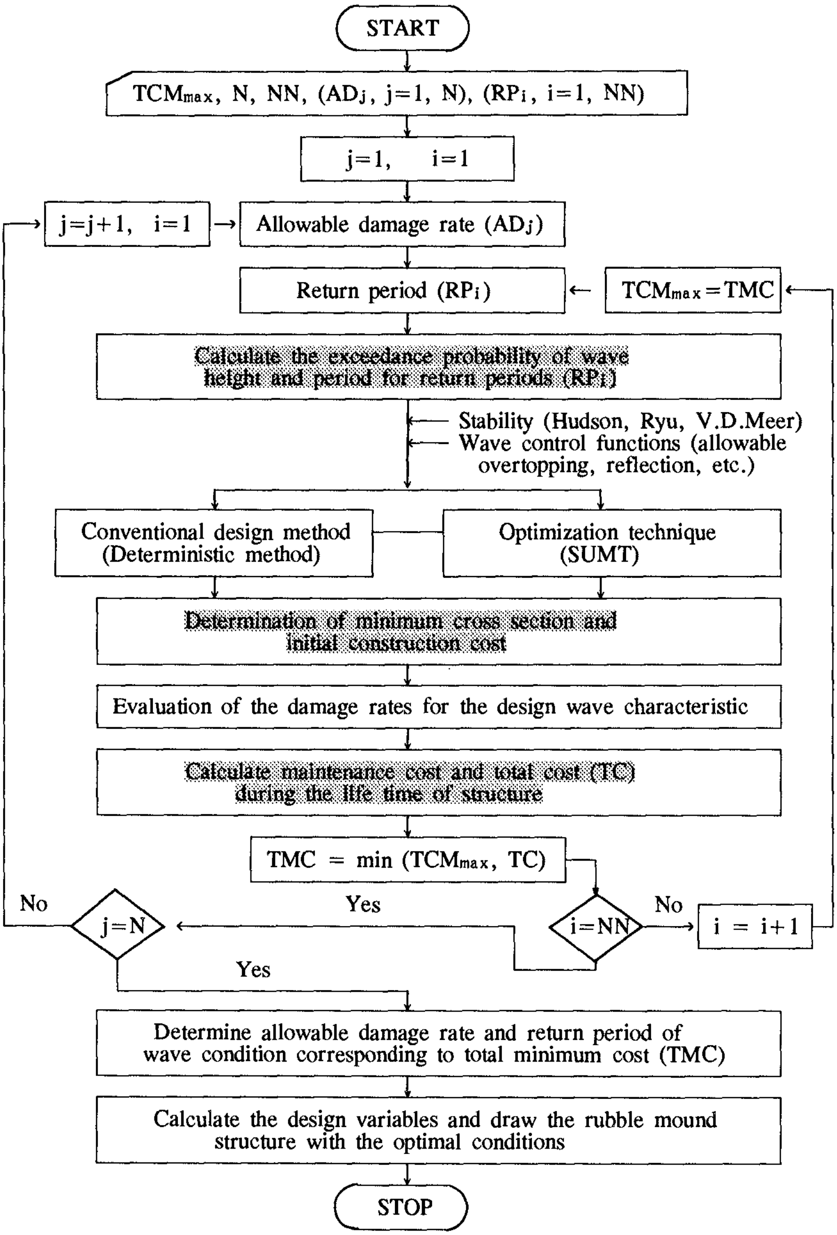

Figure 1. The flow of the optimal design process.

where $f(X)$ is objective function, X design variable vector, $g_j(X)$ the inequality constraints functions, and $h_k(X)$ the equality constraint functions.

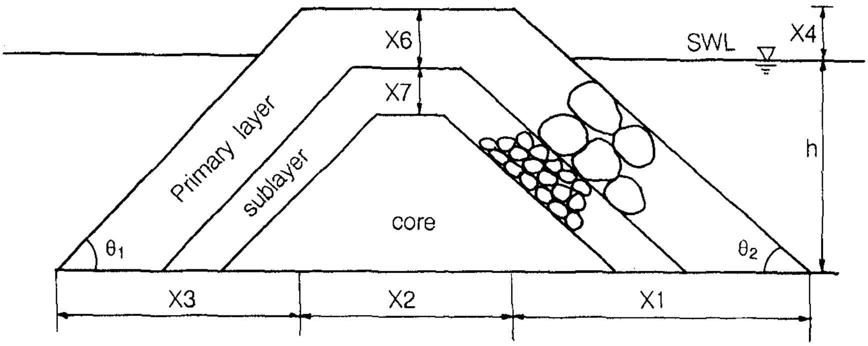

Figure 2. Definition of design variables and sectional area of the rubble mound breakwaters with uniform slope.

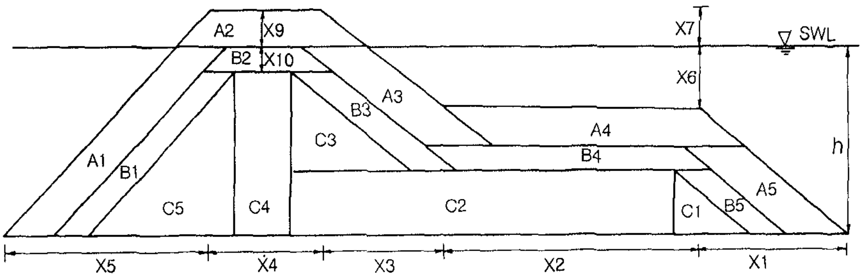

Figure 3. Definition of design variables and sectional area of the rubble mound breakwaters with composite slope.

3.2.1 Objective function

The objective function of cross-sectional area is estimated by following equations for the uniform slope.

$$\begin{aligned} f(X) &= A + B + C \\ &= A1 + A2 + A3 + B1 + B2 + B3 + C1 + C2 + C3 \end{aligned} \tag{6}$$

where,

$$
\begin{aligned}
A1 &= X6 / \sin\theta1 * (h + X4 - X6) \\
A2 &= (X2 + X2 + X6 / \tan\theta1 + X6 / \tan\theta2) * X6/2 \\
A3 &= X6 / \sin\theta2 * (h + X4 - X6) \\
B1 &= X7 / \sin\theta1 * (h + X4 - X6 - X7) \\
B2 &= [(X2 + X6 / \tan\theta1 + X6 / \tan\theta2 - X6 / \sin\theta1 - X6 / \sin\theta2) \\
&\quad + (X2 + X6 / \tan\theta1 + X6 / \tan\theta2 - X6 / \sin\theta1 - X6 / \sin\theta2) \\
&\quad + X7 / \tan\theta1 + X7 / \tan\theta2)] * X7/2 \\
B3 &= X7 / \sin\theta2 * (h + X4 - X6 - X7) \\
C1 &= (h + X4 - X6 - X7) * (h + X4 - X6 - X7) / (2 * \tan\theta1) \\
C2 &= [X2 - (X6 / \sin\theta1 - X6 / \tan\theta1) - (X7 / \sin\theta1 - X7 / \tan\theta1) \\
&\quad - (X6 / \sin\theta2 - X6 / \tan\theta2) - (X7 / \sin\theta2 - X7 / \tan\theta2) \\
&\quad * (h - X6 - X7) \\
C3 &= (h + X4 - X6 - X7) * (H + X4 - X6 - X7) / (2 \tan\theta2)
\end{aligned}
$$

The cross-sectional area for the composite slope can also calculate by the same conception in relation to Figure 3 as

$$
\begin{aligned}
f(X) &= A + B + C \\
&= A1 + A2 + A3 + A4 + A5 \\
&\quad + B1 + B2 + B3 + B4 + B5 \\
&\quad\quad + C1 + C2 + C3 + C4 + C5
\end{aligned} \tag{7}
$$

where A, B and C are the area of cover layer, sublayer and core layer respectively.

3.2.2 Design Constraint Functions

(1) Estimation of Extreme Wave Conditions

Using the relation between the return period (RP) and non-exceedance probability, $P(X \leq x)$ and considering Weibull distribution for the extreme storm wave heights and wave periods for design can be estimated by the following equations.

$$RV = [- \ln\{ 1 - P(X \leq X_{m,n}) \}]^{1/k} \tag{8}$$

$$X = a_0 RV + b_0 \tag{9}$$

where RV: transformed variable for the non-exceedance probability, m: order of data, n: number of data, X: variables (wave height & period), k: parameter of extreme value distribution function, a_0, b_0: regression constants.

The uncertainties of the extreme waves are defined by the reliability analysis, and the design waves with confidence interval can be expressed as

$$H_{1/3} = H_{1/3}' \pm \sigma H_{1/3} \tag{10}$$

$$T_{1/3} = T_{1/3}' \pm \sigma T_{1/3} \tag{11}$$

in which $H_{1/3}$ is the significant wave height, $T_{1/3}$ is the significant wave period, superscript ′ denotes the value from the regression formular equation (2), $\sigma H_{1/3}$ and $\sigma T_{1/3}$ are the uncertainty parameter for extreme wave height and period respectively, and (+) and (-) signals denote the upper and the lower confidence level.

The extreme / design wave characteristics for the return periods $(RP_i;\ i=1,NN)$ can be estimated from the equations (9), (10), (11).

(2) Stabilty Analysis of Rubble Mound Structure

Among the many well known conventional design formulas, Hudson (1959) and CERC (1984) for the regular wave, and Ryu & Sawaragi (1986) and Van der Meer (1987) for the irregular waves are applied for the stability check in the algorithm.

For the uniform slope structures:

$$W_H = \frac{\gamma_r H^3}{K_D \cot\theta\ (\ S_r - 1\ )^3} \tag{12}$$

by Hudson (1959) & CERC (1984)

$$W_R = \left[\frac{\gamma_w(6.15Q_p+20.0)}{\gamma_r^{1/3}(D_R+30.1)}\ \frac{\tan\theta}{\tan\phi}\right]^{3/2} H_{1/3}^3, \tag{13}$$

by Ryu and Sawaragi (1986)

$$W_V = \frac{\xi_z^{3/2}}{\{6.2P^{0.18}(D_V/\sqrt{N})^{0.2}\xi_z^{0.5}\}^3}\ \frac{H_{1/3}^3\gamma_r}{(S_r-1)^3} \tag{14}$$

for plunging wave

$$= \frac{1}{\{P^{-0.13}(D_V/\sqrt{N})^{0.2}(\cot\theta)^{0.5}\xi_z^P\}^3}\ \frac{H_{1/3}^3\gamma_r}{(S_r-1)^3} \tag{15}$$

for surging wave by Van der Meer (1987)

For the composite slope structures:

$$W_R = \left[\frac{\gamma_w(5.46Q_p+17.73)}{\gamma_r^{1/3}(D_R+36.3)}\ \frac{\tan\theta'}{\tan\phi}\right]^{3/2} H_{1/3}^3 \tag{16}$$

by Ryu and Sawaragi (1986)

where W is the rubble unit weight, subscripts H, R and V are the weight by Hudson, Ryu and Sawaragi, and Van der Meer respectively, H the design wave height, K_D the empirical stability coefficient, θ the angle from horizontal of seaward slope of the structure, γ_r the unit weight of armor unit, S_r the specific gravity of armor unit , Q_p the wave spectrum peakedness parameter, $\tan\phi$ the friction coefficient, D_R the allowable damage level by Ryu's definition, D_{n50} the nominal diameter, T_z the average wave period, D_v the damage level by Van der Meer's definition, P the permeability of core, ξ_z the surf-similarity parameter with average wave period, and θ' the equivalent slope of the composite structures.

These stability equations can rearrange to inequality constraint functions for the optimization formulation as

$$g_1(X) = X_5^3\ \gamma_r - [\ \gamma_r\ /\ \cot\theta\ (K_D(S_r - 1)^3)]\ H_{1/3}^3 \geq 0 \tag{12'}$$

$$g_1'(X) = X_5^3\ \gamma_r - \left\{ \frac{\gamma_w(6.15\ Q_P + 20.0)}{\gamma_r^{1/3}\ (D_R + 30.1)} \frac{\tan\theta}{\tan\phi} \right\}^{3/2} H_{1/3}^3 \geq 0 \tag{13'}$$

$$g_1''(X) = X_5^3\ \gamma_r - \frac{\xi_z^{3/2}}{\{6.2P^{0.18}(D_V/\sqrt{N}\ )^{0.2}\xi_z^{0.5}\}^3} \frac{H_{1/3}^3\gamma_r}{(S_r-1)^3} \geq 0 \tag{14'}$$

for plunging wave

$$= X_5^3\ \gamma_r - \frac{1}{\{P^{-0.13}(D_V/\sqrt{N}\ )^{0.2}(\cot\theta)^{0.5}\xi_z{}^P\}^3} \frac{H_{1/3}^3\gamma_r}{(S_r-1)^3} \geq 0 \tag{15'}$$

for surging wave

$$g_1'''(X) = X_8^3\ \gamma_r - \left\{ \frac{\gamma_w(5.46\ Q_P + 17.73)}{\gamma_r^{1/3}\ (D_R + 36.3)} \frac{\tan\theta'}{\tan\phi} \right\}^{3/2} H_{1/3}^3 \geq 0 \tag{16'}$$

(3) Wave Control Functions

In considering with function and purpose of the structres, the following equations are used as the design constraints for wave control functions in relation to wave overtopping and reflection (Takada, 1973; Ryu and Kang, 1990; Ryu, 1984).

$$g_2(X) = AQ - Q \geq 0 \tag{17}$$

$$g_3(X) = AR - K_r \geq 0 \tag{18}$$

$$Q = 0.5a(R_u-H_c)^2(X_0/R_u - \cot\theta\ ) \tag{19}$$

$$a = 7.6\ (\cot\ \theta\ )^{0.73}\ (\ H_0\ /\ L_0\ )^{0.83} \tag{20}$$

$$R_u = [1.17\ \xi_{1/3}/(1 + 0.8\ \xi_{1/3}\ )] \times H_{1/3} \quad \text{, for the uniform slopes} \tag{21}$$

$$K_r = 0.5 \left\{ \frac{(\xi_{1/3} - 2.65\ \tan\theta\ )}{4.3} \right\}^{0.7} \quad \text{, for the uniform slopes} \tag{22}$$

where AQ is the allowable overtopping rate, Q the overtopping rate, AR is the allowable wave reflection and K_r the wave reflection coefficient, R_u is run-up height, $\xi_{1/3}$ is significant surf-similarity parameter, H_c is the crest height, $X_0 = L_0/4$, and L_0 and H_0 are deep sea wave length and height respectively.

For the composite slopes, the same inequality constraints are used as equations (17) and (18), however the run-up and the wave reflection characteristics on the composite slope are considered reffered to the analysis results with phase interaction mechanism on the composite slope face by Ryu et al. (1986) instead of equations (21) and (22).

Crest width, slope stability and other design constraints can also be included from necessity.

4. Optimal Cost Effectiveness

4.1 Initial Construction Cost *(ICC)*

Initial construction cost is very different by field by field, however, the cost changes can be defined to be proportional to the cross-sectional area and material scales. The relation simply can express as following equation.

$$ICC = \alpha_a A + \alpha_b B + \alpha_c C \tag{23}$$

The weight functions α_a, α_b and α_c should be change with the field conditions, construction facilities and techniques, and other a lot of constraints. Where A, B, and C are the unit costs of the cover layer, sublayer and core layer respectively.

4.2 Maintenance Cost

To estimate the maintenance cost, the probability of failure accurance should be considered as denoted as following equation.

$$dF = (1 - (1 - \Delta P))^{\lambda} \tag{24}$$

where dF denotes the probability of failure occurence per a year, ΔP is the exceedance probability of wave conditions, and λ the total number of wave events per a year to destruct. If it can be assumed that the maintenance work will be immediatly made after damage is accured, the total maintenance cost during life time of structures (TMY) is given by

$$TMY = \int_L^U [\alpha_f \ (EDR \times ICC)] \ dRP \tag{25}$$

where dRP is considered interval of the return period, U and L are the upper and lower limit of the return period, EDR is the equivalent damage rate, and α_f the weighting value for repair compare to the initial construction cost. α_f in the equation is one of the sensitive constant in the maintenance cost estimation. It is very changeable parameter by field by field, and the value in the $[\alpha_f \ (EDR \times ICC)]$ means the maintenance cost per a year.

4.3 Total Cost (TC) Minimization

Total cost during the life time (TC) can be written as the sum of initial construction cost (ICC) and total maintenance cost (TMY). The maintenance cost TMY is required every year, and the cost can be changed to the present value considering interest / discount rate and the other effects.

The typical total cost change is shown in Figure 4, and it can be written as

$$TC = ICC + P_{wf} \ (TMY) \tag{26}$$

$$P_{wf} = \frac{(1+i)^n - 1}{i(1+i)^n} \tag{27}$$

where P_{wf} denotes the present value exchange rate, i the discount rate, n the life time of the structure. In the above equation, i is an important and effective parameter in the TC calculation, and it is changeable by the economic conditions.

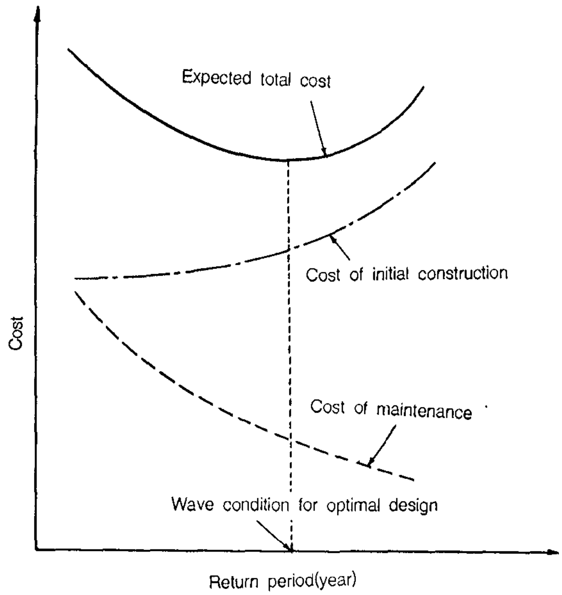

Figure 4. Construction, maintenance, and total costs as functions of wave conditions.

If those effective parameters such as i, α_f, α_a, α_b and α_c can be introduced reasonably in the cost estimation, the minimum cost condition can be found easily by the conception as shown in Figure 4.

5. Application of the Design Algorithm

5.1 Variation of the Cross-Section for Uniform Slope

Using the wave observed data at the east coast of Korean Peninsula, the extreme wave characteristics can be derived as

$$\left.\begin{array}{l} H_{1/3} = 1.01\ RV + 1.93 \\ T_{1/3} = 2.50\ RV + 6.66 \end{array}\right] \qquad (28)$$

where the symbols are same as equations (8) and (9). The unit construction cost and other design constraints are also reffered to G-port in Korea

From the equation (28), extreme / design wave characteristics by return periods can be easily estimated, and can get the cross-sectional dimensions by the algorithm. Figure 5(a) shows the cross-sectional variation characteristics due to the select of return period for the design waves. Figure 5(b) indicates the effect of allowable reflection of waves on the cross-section. The detail dimensions are illustrated in the Figures. From the Figures, it can be remarkably said that the allowable reflection very sensitively affect to the slope angle and total cross-sectional area, and the change of design waves

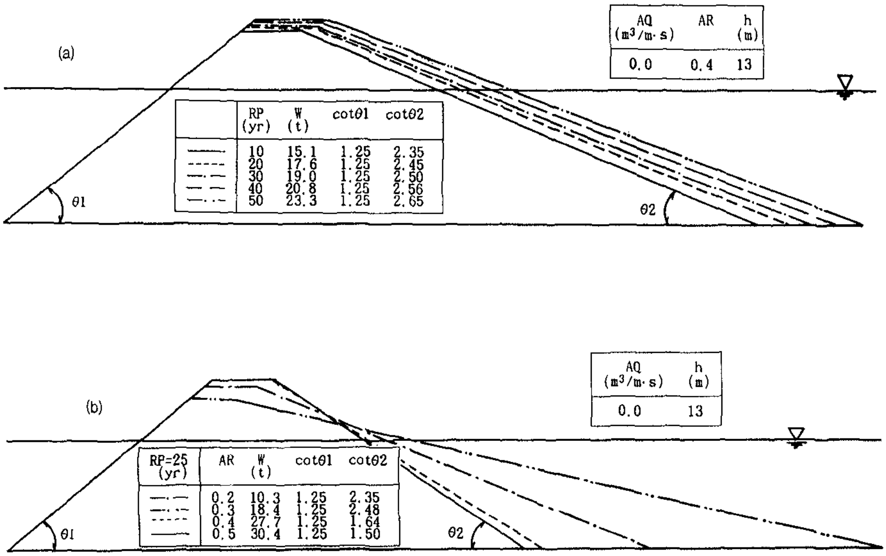

Figure 5. The cross-sectional variation characteristics due to the select of return period for the design waves and the effect of allowable reflection of waves on the cross-section.

affects to the weight of cover layer armor unit. Figure 6 shows the cross-sectional and the crest height changes as well as the rubble weight and the slope angle variations due to the allowable overtopping rate as a function of return period of design waves.

In the calculation the allowable reflection coefficient AR is considered to 0.3, and Ryu and Sawaragi's formula is applied to the stability analysis with allowable damage rate $D_R = 60\%$ and spectral peakedness parameter of irregular wave $Q_P = 2.0$. From the figures it can be concluded that allowable overtopping rate affects sensitively on the change of crest height. As the result the cross-sectional area is also changed by the rate as a function of the return period.

Figure 7 shows the effects of the spectral peakedness parameter of irregular design waves. In the figure, it is also found that the effects can not be neglected in the design as neglected in a lot of conventional design formulas.

5.2 The Cross-Section Changes of Composite Slope

The optimal design results for the composite slope structures are listed in Table 1. In this case, the reflection control functions are considered with the reflection coefficient at boundary 1, berm front, to be less than 0.4, and the overtopping is not allowed. In the table, it can understand that the design variables are sensitively affected by the extreme wave statistics (RP). The berm width and depth are especially sensitive with a small change of design waves. These may will be more changeable if the allowable reflection is considered as a function of the wave statistics with a conception of environmental wave control in a sea area.

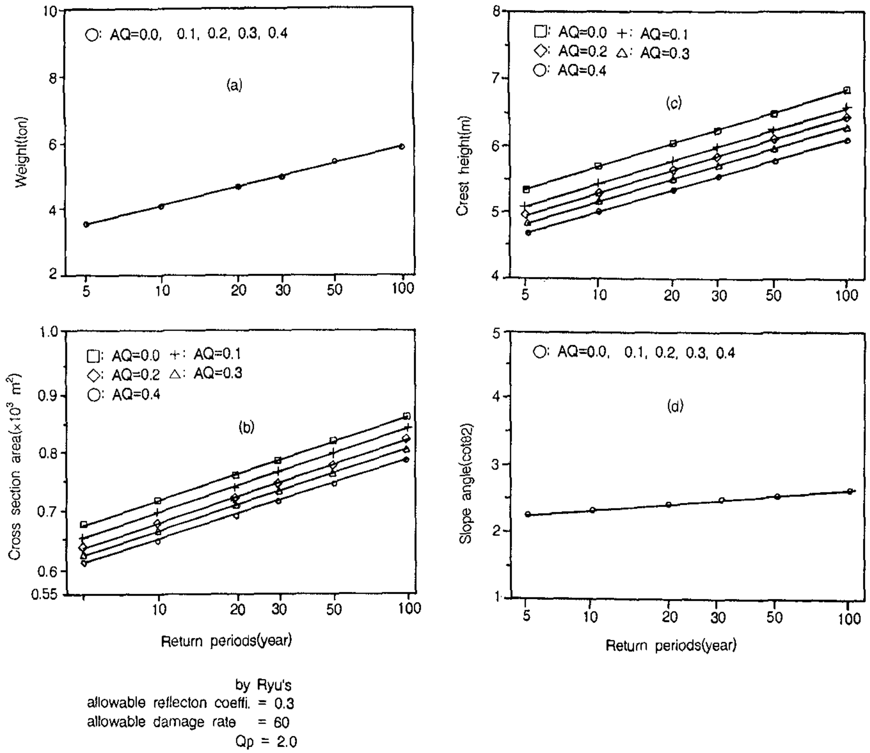

Figure 6. The variations of design variables according to the allowable overtopping rates.

Table 2 shows the results of the design variables change due to the allowable damage. The rubble weight, X8 has a decreasing tendency with increasing of allowable damage D_R, however, we can not find a typical cross sectional area in the table.

From these results, it is emphasized that the algorithm can be applied to the design of composite slope structures.

5.3 Total Cost Variations

Figure 8 is the final results on unit total cost calculation by the algorithm. It changes according to the return period and the other design constraints listed in the figure, and we can easily find the minimum cost point by the design conditions. Although, it is a special application example and the result can be changed by field by field, it can be emphasized that the minimum point take place in some region of medium return periods shorter than 50 years. It means that the optimal design concept is a reasonable design method for the systematic and cost effective coastal developments.

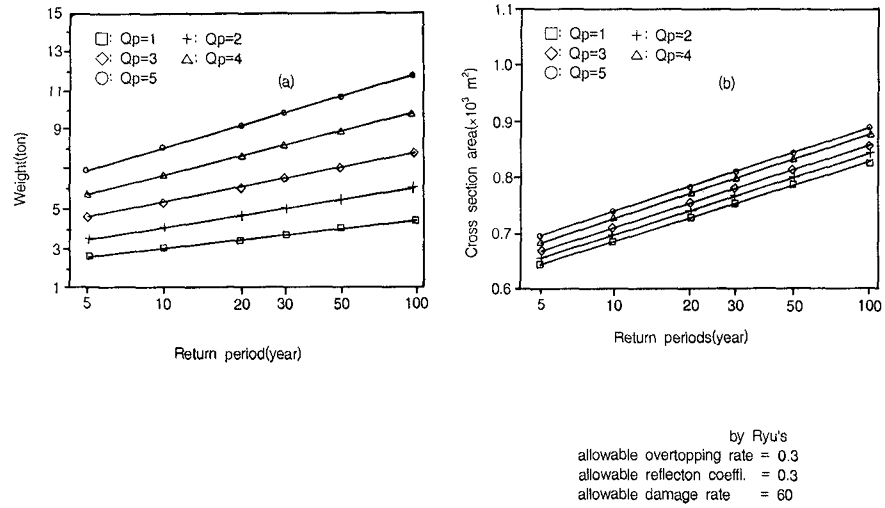

Figure 7. The effects of the spectral peakedness parameter of irregular design waves.

Table 1. Optimal design variables for the composite slope according to the return period of the waves

RP (year)		1	5	10	20	30	40	50	100
$H_{1/3}$ (m)		3.15	3.90	4.23	4.55	4.75	4.88	4.99	5.31
$T_{1/3}$ (sec)		13.17	15.40	16.36	17.32	17.88	18.29	18.59	19.55
Dimensions of the section	X1	18.58	17.69	17.22	16.85	16.61	16.60	16.45	16.13
	X2	54.14	74.03	83.55	93.64	99.79	104.42	107.88	119.31
	X3	8.64	10.32	11.03	12.09	12.09	12.41	12.63	13.18
	X4	2.06	2.30	2.39	2.46	2.51	2.54	2.68	2.62
	X5	30.94	31.49	31.70	31.87	31.99	32.15	32.22	32.23
	X6	3.15	3.90	4.23	4.55	4.75	4.88	4.99	5.31
	X7	2.60	2.97	3.11	3.22	3.30	3.37	3.39	3.46
	X8	0.66	0.68	0.71	0.79	0.81	0.81	0.82	0.84
	X9	1.32	1.47	1.53	1.58	1.61	1.63	1.64	1.68
	X10	0.61	0.68	0.71	0.73	0.75	0.76	0.76	0.78

Table 2. Variation of the design variables according to the allowable damage rate D% for the composite slope structure

D (%)		20	40	60	80	DATA
Dimensions of the section	X1	16.60	16.60	16.60	16.60	$H_{1/3}$ = 4.88 m
	X2	104.42	104.42	104.42	104.42	$T_{1/3}$ = 18.29 sec
	X3	12.41	12.41	12.41	12.41	Q_P = 2.5
	X4	2.54	2.34	2.13	1.98	h = 18 m
	X5	32.15	32.16	32.16	32.16	
	X6	4.88	4.88	4.88	4.88	
	X7	3.37	3.35	3.35	3.35	
	X8	0.81	0.75	0.68	0.64	
	X9	1.64	1.50	1.36	1.27	
	X10	0.76	0.70	0.63	0.59	

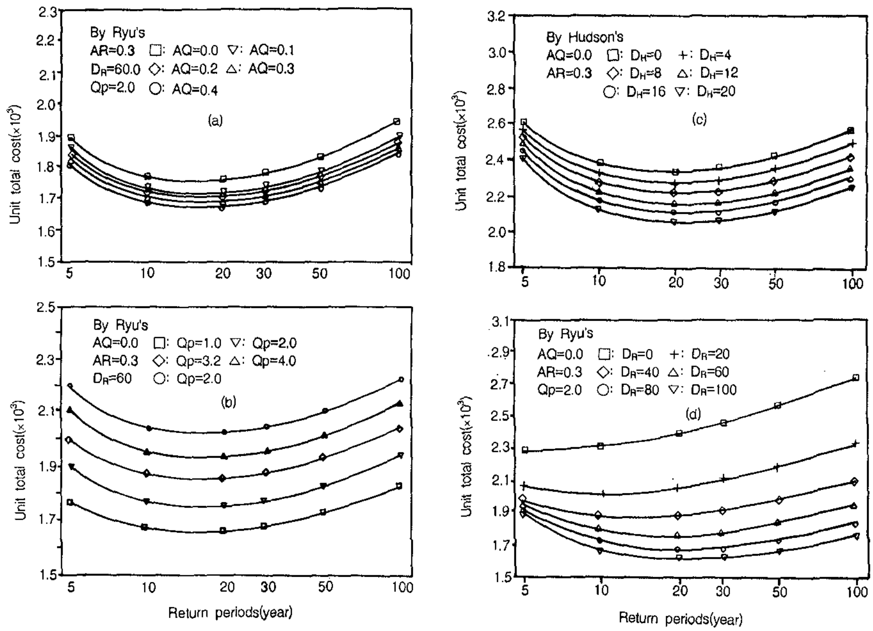

Figure 8. The final results on unit total cost calculation in the algorithm according to the return period and the other design constraints.

6. Conclusions

The conclusions obtained herein are as follows:
A cost-effective design algorithm of rubble mound structure with various slope shapes has been developed considered the uncertainty and irregularity of design waves / the return period of extreme waves, allowable damage (D_R), and wave control functions.

The reliability of the design results can be directly estimated using the algorithm. It means that the design algorithm can overcome the design fault because of the design wave selection problems in the extreme waves. In considering the allowable ranges of the design constraints and irregularity of the design wave, the design sensitivity can easily be checked also by using the algorithm.

In order to examine the applicability of the algorithm, the design sensitivity for the structural dimensions and total costs are analysed and compared with those of conventional methods using design examples. From the results of comparative studies, the algorithm is found to be applicable, and it will be more useful and powerful algorithm for the design of rubble mound structures under the more complex design conditions, design constraints, and cost functions.

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