Local parameter for armor weight estimation

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ABSTRACT

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A new surf parameter called the 'wave action slope' is introduced to represent local wave conditions in shallow waters by employing local values of wave length or wave celerity and wave height. The use of wave theory on a flat bed of depth at the front of the breakwater might be considered far superior to the simple adoption of the deep water wave length for characterizing surfing waves at a shoaling depth. The wave action slope is formed by the product of the breakwater slope and the ratio of the celerity to the wave height. The optimum or minimum weight of the armor unit is related to the wave action slopes, which are employed for developing new empirical equations. Analyzing the laboratory data of van der Meer revealed that the armor weights are closely related to the wave action slope. By using the new surf parameter given with local parameter values, we suggest a simple but accurate equation to estimate the breakwater armor weight. Several empirical equations are applied to cases of failure experienced along the eastern coast of Korea. The values given by the Hudson equation are satisfactory for 26 of the 57 total failure cases. The number of satisfactory cases decreases by using recent empirical equations; of the 57 failure cases, the number of satisfactory cases yielded is 17 for the van der Meer/de Jong equation and 13 for the equation of the present study considering local parameters. The reduced number of satisfactory cases may indicate improvement of the new empirical equations over the Hudson equation.

ADDITIONAL INDEX WORDS: *breakwater, armor block optimum weight, wave action slope, local value of wave celerity.*

INTRODUCTION

When waves approach a coast, the movement of wave particles transforms from elliptic to horizontal, and the energy transport velocity (group velocity) continues to decrease to near zero. The waves then increase in height to a certain limit until the stability of wave formation is lost. The type of breaking wave is a major factor that determines the wave forces affecting beaches and coastal structures.

A number of methodologies are available for determining armor stone requirements for coastal structures subject to wave attack. Commonly used equations for determining armor weight are given by Iribarren (1950), Hudson (1959), van der Meer (1987, 1999), de Jong (1996) and van Gent (2003). And many more laboratory studies (Thompson and Shuttler , 1975; Carver, 1982; Hedar, 1986; Ryu. 1987; among others) was also conducted for various armor blocks.

Recently, Yoo *et al*. (2001a, b, 2003) suggested a new method that uses the second-order wave action slope given with local values as a major variable for determining armor weight. The Hudson formula is well known because of its simplicity and wide coverage of various armor blocks. In recent years, however, there has been an increasing demand for more reliable design formulas for estimating armor block weight.

In this study, we compared the characteristics of these well-used formulas and the improved Yoo equation by employing the local effect of wave conditions and nonlinearity in shallow waters. The improved equation employs the second-order wave action slope with local values of wave celerity and wave height, considering various nonlinear aspects of wave formation, including the reflection factor, wave-breaking condition associated with local Iribarren number, percolation, number of incident waves or impact duration, degree of damage, and set-up, particularly in very shallow waters. The final equation is expressed by a simple form for use in engineering. The accuracy of this equation was confirmed by testing it with laboratory data. The newly improved equation was tested against field data of failure cases experienced along the eastern coast of Korea. All equations were tested against the same data collected from the Korean east coast.

EXISTING FORMULAS

Hudson Equation

The non-dimensional physical coefficient η (ratio of weight) and δ (stability factor) are introduced as follows:

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Table 1. Hudson's parameter K_D, Yoo's parameter α, and e_R at each armor stone unit (for a two-layer system).

Armor unit	K_D		α	e_R
	Non breaking wave	***Breaking wave***		***(Breaking wave)***
Riprap	2.4	1.2	1.33	2
Riprap (> 3 layers)	3.2	1.6	1.0	2
Quarry stone	4.0	2.0	0.8	2
TTP	8.0	7.0	0.4	1.14
Dolos	31.8	15.8	0.15	2
T-bar	10.0	7.0	0.32	1.43

$$\eta = \frac{W}{\gamma H^3} \tag{1}$$

$$\delta = \frac{H}{(s-1)\phi_n} \tag{2}$$

$$W = \gamma \phi_n^3 \tag{3}$$

$$\eta = \delta^{-3}(s-1)^{-3} \tag{4}$$

where W is the armor weight, γ is the specific weight of the armor block, s is the specific gravity of the armor block, and ϕ_n is the nominal diameter defined by Equation (3).

The Hudson equation is expressed as follows:

$$W = \frac{\gamma H^3}{K_D(s-1)^3 R} \quad : \quad \eta = \frac{S}{(s-1)^3 K_D} \tag{5}$$

where H is the wave height; R is the ratio of the horizontal to vertical lengths of the breakwater slope ($\cot\theta$); $S = R^{-1} = \sin\theta$; and K_D is the empirical constant, which is primarily associated with the type of armor block (Table 1). The Hudson equation indicates that the weight ratio is solely dependent on the breakwater slope and type of armor block. The influence of wave period and random characteristics of wave motion are not considered.

Van der Meer Equation

Various researchers have found that the optimum weight of the armor block is also significantly influenced by the wave period, percolation of slope, and formation of irregular waves. Van der Meer (1987) employed the inshore Iribarren number and suggested a set of two equations depending on the condition determined by this number. The van der Meer equation is represented as follows:

$$\delta = 6.2P^{0.18}D_N I_i^{-0.5} \qquad (0.4 < I_i < 2.5) \tag{6a}$$

$$\delta = P^{-0.13}D_N I_i^{P}\sqrt{R} \qquad (2.5 < I_i) \tag{6b}$$

where P is the coefficient associated with percolation, $D_N = \left(D/\sqrt{N}\right)^{0.2}$, N is the number of incident waves, D is the degree of damage defined by the ratio of the damage area to the projected area of the armor unit, $I_i = S/\sqrt{M_i}$, the wave steepness $M_i = H/L_0$, and L_0 is the wave length at deep water. The number of incident waves for a field is normally between 1000 and 3000, and the degree of damage is between 1 and 3. The percolation factor P ranges from 0.1 (impermeable) to 0.6 (permeable).

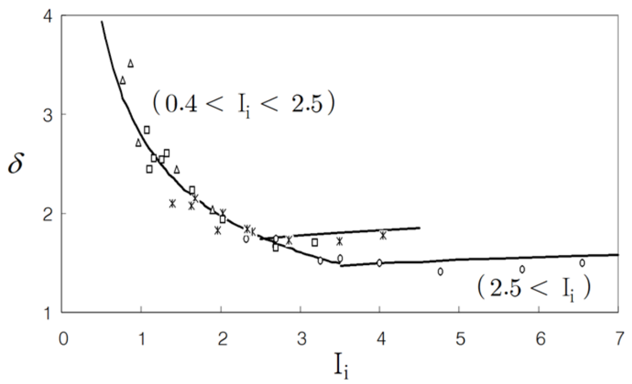

Figure 1. δ vs I_i (P = 0.1 (impermeable), N = 3000), van der Meer (1988).

The armor weights measured in the laboratory by van der Meer (1988) are related to the Iribarren number I_i (Figure 1). Equation (6) has a different formation depending on the characteristics of the breaking waves: Equation (6a) is applied to plunging waves, and Equation (6b) to surging waves. In the van der Meer formula, the wave steepness M_i is one of the most important factors.

Van der Meer (1999) has conducted experiments of tetrapod (TTP) blocks at slope R = 1.5 mainly for the case of surging waves. Furthermore, De Jong (1996) has conducted re-analyses for plunging waves. The armor weight equation for TTP blocks is suggested as follows:

$$\delta = (8.6E_N + 3.94)M_i^{0.2} \qquad (I_i < I_c) \tag{7a}$$

$$\delta = (3.8E_N + 0.85)M_i^{-0.2} \qquad (I_c < I_i) \tag{7b}$$

where $E_N = (E/\sqrt{N})^{0.5}$ and E is the ratio of moving length to block length. Damage can be serious when E is greater than 0.1. The critical Iribarren number (I_c) for Equation (7) and (8) is given by $I_c = 5.4S^{0.63}$. Equation (7) can be transformed into a general form by using the Iribarren number and slope R as follows:

$$\delta = (7.3E_N + 3.35)I_i^{-0.4} \qquad (I_i < I_c) \tag{8a}$$

$$\delta = (3.8E_N + 0.85)I_i^{0.4}R^{0.4} \qquad (I_c < I_i) \tag{8b}$$

We assume that $E_N = 0.043 (E = 0.1, N = 3000)$ and $K_D = 7.0$ for TTP blocks.

The van der Meer equation considers the effects of wave breaking, wave steepness, percolation, and number of incident waves, whereas the Hudson equation represents all effects with a single parameter. Therefore, the van der Meer equation is substantially more advanced than the Hudson equation. However, the van der Meer equation employs the deep water wave length in estimating the Iribarren number. Hence, as the equation is assumed not to consider the local effects near the breakwaters sufficiently, it may be further refined if local parameters such as

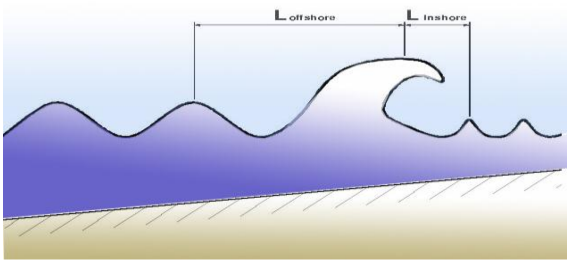

Figure 2. Wave deformation on a beach slope.

wave length and wave height are included in estimating surf parameters.

NEW FORMULA

Local Second-Order Wave Action Slope

Estimating, measuring, or even defining the wave length at a sloping bed in shallow waters may be highly difficult. However, the wave length at the water depth of a local point given by any wave theory, assuming the sloping bed is flat, may be far superior to the deep water wave length for estimating the wave steepness at a local point (Figure 2). Iribarren adopted the square root of wave steepness for his non-dimensional physical number. Direct use of wave steepness also results in a non-dimensional physical number, which yields a new form of the Iribarren number. On the other hand, a new surf parameter is formulated by introducing the Froude number, which is the ratio of the wave celerity to the group velocity of wave height. The wave action slope is the product of the beach slope S and the Froude number associated with wave celerity c and wave height H, as suggested by Yoo *et al*. (2001b).

The second-order wave action slope S_Y is represented as follows:

$$S_Y = F_H^2 S \tag{9}$$

where $F_H = c/\sqrt{gH}$; *c* is the local wave celerity, possibly given by nonlinear wave theory in shallow waters; and *H* is the local wave height at the breakwaters. The offshore wave action slope S_{Yo} uses the deep water values of wave height H_o and wave celerity c_o. Although the inshore wave action slope S_{Yi} uses the local value of wave height, it still uses the deep water value of wave celerity. On the other hand, the local wave action slope takes the local values of both wave height and wave celerity.

Yoo *et al.* (2001) suggested a new empirical formula that uses the local second-order wave action slope (S_Y) as follows:

$$\eta = (s-1)^{-3} C_\eta S_Y \tag{10}$$

with the following parameters.

$C_\eta = \alpha\, e_R R_\varsigma g_p M_L N_S$

α, coefficient for the type of armor unit (in Table 1)

$e_R = 1 \sim 2$, reflection factor (in Table 1)

$R_\varsigma = 0.03$, for rubble-mound breakwaters

$g_p = (1-0.48P)^3$, function of permeability

$M_L = 0.45 + 0.45 I_L - 0.09 I_L^{\,2}$, wave breaking coefficient

$I_L = \dfrac{S}{\sqrt{H_b / L_b}}$, local Iribarren number

H_b, L_b, wave height and length at breaking (local) point

$N_S = (\sqrt{N}/D)^{0.6} = D_N^{\,-3}$

$S_Y = F_H^{\,2} S$, wave action slope ($F_H = c/\sqrt{gH}$)

$c = \sigma / k$, wave celerity given by the proper wave theory

S_Y considers the breakwater slope and the wave celerity and wave height at the breakwater point. Although it is closely related to the Iribarren number, it considers the local values of wave celerity and wave height. As the local values of all parameters are included in the estimation of the second-order wave action slope, knowledge of local conditions is important for its proper use in estimating the armor block weight. Thus, all information is considered important, including the nonlinearity of wave formation, breaking condition, and set-up, particularly in very shallow waters. The breaking condition is reflected by the parameter e_R, as presented in Table 1. When wave breaking occurs, e_R is replaced by 1.

Van der Meer considered the effects of wave breaking associated with the inshore Iribarren number I_i. To improve the accuracy of the empirical equation, the local Iribarren number has been employed to define the condition of wave breaking, which increases the endurable weight of the armor block by the rate of wave breaking coefficient M_L. The distribution of M_L as a function of I_L is shown in Figure 3. M_L is related to I_L, and the empirical equation obtained by regression analysis is expressed as follows:

$$M_L = 0.45 + 0.45 I_L - 0.09 I_L^{\,2} \tag{11}$$

To compute the wave celerity at a local position, the dispersion relation should be provided. The dispersion relation for linear small-amplitude waves is given as follows:

$$\sigma^2 = gk \tanh kh \tag{12}$$

where σ is the angular frequency ($\sigma = 2\pi / T$) and k is the wave number ($k = 2\pi / L$).

Kirby and Dalrymple (1986) derived a nonlinear dispersion relation extended to shallow water areas for finite amplitude waves as follows:

$$\sigma^2 = gk\left[1 + (ak)^2 f_1 \tanh^5 kh\right] \tanh\left(kh + akf_2\right) \tag{13}$$

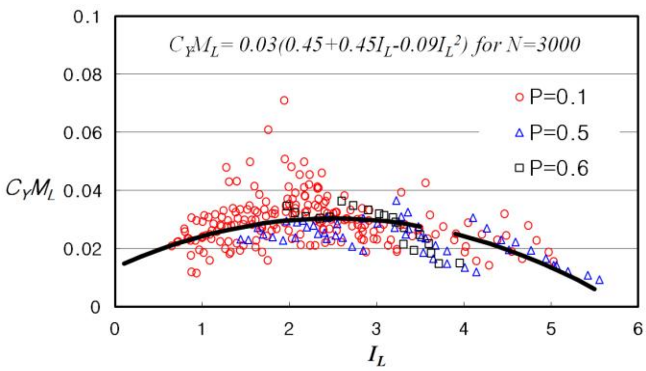

Figure 3. $C_Y M_L$ vs. I_L for quarry stone data from van der Meer (1988).

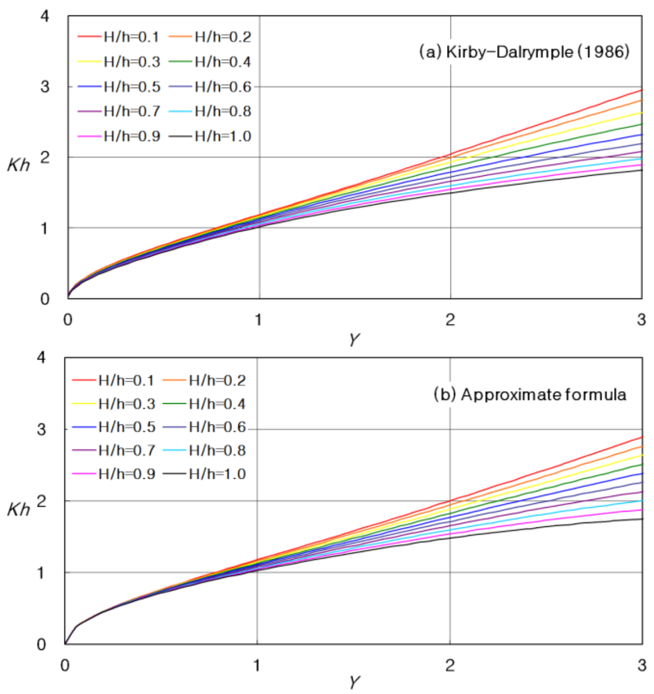

Figure 4. Comparison between the nonlinear dispersion relation of Kirby and Dalrymple (1986) and the new approximate formulas.

where $f_1 = \dfrac{\cosh 4kh + 8 - 2\tanh^2 kh}{8\sinh^4 kh}$ and $f_2 = \left(\dfrac{kh}{\sinh kh}\right)^4$.

To simply estimate the new dispersion relation of nonlinear finite-amplitude waves (Equation 13), an approximate formula is developed as follows:

$$kh = \begin{cases} (1-0.14Y_H)\sqrt{Y}\,(1-0.167Y)^{-1} & (0<Y\le 1) \\ (1-0.14Y_H)(0.9Y+0.3) & (1<Y\le 3) \end{cases} \qquad (14)$$

where $Y_H = \dfrac{\sigma^2 H}{g}$ and H is the wave height. The comparison between Equation (13) and Equation (14) is shown in Figure 4, which demonstrates the degree of accuracy of the approximate equation for the dispersion relation of finite-amplitude waves.

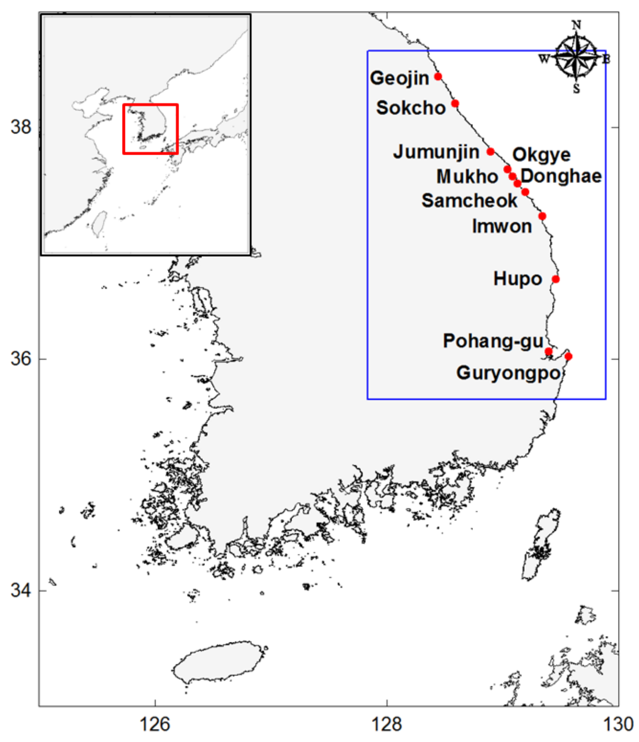

Figure 5. Location of damaged breakwaters.

ANALYSIS OF DAMAGED BREAKWATERS IN THE EASTERN COAST OF KOREA

We have analyzed the status of 119 breakwaters at 12 principal harbors in the eastern coast of the Korean peninsula (Figure 5). Of the 119 TTP armor breakwater sites, 59 had serious damage. The information on damaged breakwaters (Jung, 2000) is presented in Table 2. Armor block weights are re-estimated by using several formulas presented in this study. We assumed N = 2000, D = 1, and P = 0.1(impermeable), for the design parameters of the TTP armor breakwater. This assumption would represent enhanced design conditions for armor weight estimation. The armor block weight has been commonly estimated on the basis of the Hudson formula. The Hudson equation yields 31 unsatisfactory cases (NG: no good), which gives an underestimate of 54%(31/57). This means that 54% of the breakwaters are damaged in their lifetime (normally 50 years) owing to the shortage of weight in comparison with the computation. The remaining 46% should not be damaged if the Hudson formula were correct.

Re-estimation of the results yields 70% (40/57) NG by the van der Meer–de Jong equation and 77% (44/57) by the Yoo equation. These results showed significant improvement (increase in accuracy by more than 16% compared with the Hudson equation).

In particular, when the water is relatively deep, the Hudson equation yields small values of armor weight (OK: satisfactory) while the Yoo equation shows instability (NG: Nos. 3, 6, 13, 15, 17, 23, 29, 93, 94, 101, 105, 113, 115, 116, 117). As the water depth is sufficient, the influence of wave steepness becomes significant. Hence, the armor weight is estimated to be larger than the weight estimated using the Yoo equation (11).

On the other hand, 11 failure sites are judged safe by using both the Hudson and Yoo equations (Nos. 16, 23, 24, 25, 26, 32, 65, 106, 108, 111, 119). The water depths of six sites are shallower than 5 m (Nos. 23, 24, 32, 65, 108, 119), and the slopes of two sites are extremely mild (Nos. 32, 65). The other sites do not show any unique features. Among the failure cases, both the M–J (van der Meer/de Jong) and Y–Y (the present equation) equations indicate NG at six sites (denoted by ∆ in Table 2), whereas the Hudson equation yields OK for these sites. This implies that the effects of wave steepness or wave period are important at these sites. For three failure sites, the present equation (10) still shows OK whereas the M–J equation indicates NG (denoted by □). In these cases, spilling-type wave breaking may occur because the breakwater slope is very mild. When this occurs, the pressure becomes negative and draws the armor blocks upwards; hence, the breakwater may be easily broken. Therefore, an extremely mild slope is not considered to be an economic measure as the type of breakwater. Among the failure cases, the M–J equation still indicates OK at eight sites whereas the present equation (10) indicates NG (as indicated by ◇ in the table).

Table 2. Application of empirical equations for armor weight to cases of failure in the eastern coast of Korea

Port	NO.	h (m)	H_s (m)	T_p (s)	R	Applied weight(t)	Hudson	Decision	V. D. Meer	Decision	Yoo	Decision	Remark
Guryongpo	3	10.0	5.5	12.0	1: 3.0	12.5	9.7	OK	23.1	NG	14.3	NG	∆
	6	11.5	6.1	12.0	1:2.0	20.0	19.9	OK	29.6	NG	32.4	NG	∆
	7	11.5	6.1	12.0	1:1.5	25.0	26.5	NG	29.6	NG	42.1	NG	O
Pohang-gu	9	4.5	5.0	11.0	1:1.3	10.0	16.9	NG	15.9	NG	14.9	NG	O
	12	6.5	4.5	11.0	1:1.3	10.0	12.3	NG	10.9	NG	15.0	NG	O
	13	7.4	4.5	11.0	1:2.0	12.5	8.0	OK	12.8	NG	11.8	OK	□
	15	7.7	5.2	11.0	1:2.0	12.5	12.3	OK	18.2	NG	16.5	NG	∆
	16	8.0	5.2	11.0	1:2.0	20.0	12.3	OK	18.2	OK	17.0	OK	×
	17	6.3	4.2	11.0	1:2.0	8.0	6.5	OK	10.9	NG	8.9	NG	∆
Hupo	23	4.0	2.6	12.0	1:3.0	5.0	1.0	OK	1.4	OK	1.4	OK	×
	24	2.5	3.1	12.0	1:1.5	5.0	3.5	OK	2.6	OK	2.9	OK	×
	25	5.6	3.1	12.0	1:2.0	5.0	2.6	OK	2.6	OK	4.3	OK	×
	26	5.3	3.1	12.0	1:2.0	5.0	2.6	OK	2.6	OK	4.1	OK	×
	28	13.5	4.2	12.0	1:1.5	12.5	8.7	OK	7.7	OK	19.2	NG	◇
	29	16.5	4.2	12.0	1:1.5	12.5	8.7	OK	7.7	OK	21.5	NG	◇
Imwon	32	3.5	7.4	12.0	1:10.0	12.5	7.1	OK	47.0	NG	2.4	OK	□
	33	11.0	8.4	12.0	1:2.0	20.0	51.9	NG	63.8	NG	61.0	NG	O
Samcheok	39	7.0	7.4	12.0	1:2.0	12.5	35.5	NG	47.0	NG	32.6	NG	O
	40	7.0	7.4	12.0	1:2.0	12.5	35.5	NG	47.0	NG	32.6	NG	O
	44	10.0	7.4	12.0	1:2.0	12.5	35.5	NG	47.0	NG	43.4	NG	O
	45	9.0	7.4	12.0	1:2.0	12.5	35.5	NG	47.0	NG	39.8	NG	O
	46	9.0	7.5	12.0	1:2.0	20.0	37.0	NG	48.6	NG	41.0	NG	O
	47	9.0	7.4	12.0	1:2.0	20.0	35.5	NG	47.0	NG	39.8	NG	O
	48	9.8	7.5	12.0	1:2.0	20.0	37.0	NG	48.6	NG	43.9	NG	O
	51	4.0	4.0	12.0	1:2.0	3.0	5.6	NG	6.4	NG	5.6	NG	O
	52	7.0	4.0	12.0	1:2.0	3.0	5.6	NG	6.4	NG	8.8	NG	O
	53	8.0	4.5	12.0	1:2.0	5.0	8.0	NG	9.8	NG	12.7	NG	O
	54	7.0	4.9	12.0	1:1.5	12.5	13.7	NG	13.4	NG	17.7	NG	O
Donghae	56	8.0	7.4	12.0	1:1.5	40.0	47.4	NG	47.0	NG	49.5	NG	O
	57	9.0	7.4	12.0	1:1.5	40.0	47.4	NG	47.0	NG	54.1	NG	O
	61	6.0	8.0	12.0	1:1.3	20.0	69.0	NG	56.7	NG	54.5	NG	O
	62	9.0	6.6	12.0	1:2.0	12.5	25.2	NG	35.7	NG	31.3	NG	O
Mukho	65	3.0	4.5	12.0	1:4.0	5.0	4.0	OK	14.3	NG	2.3	OK	□
	66	3.0	4.5	12.0	1:1.3	5.0	12.3	NG	9.8	NG	8.7	NG	O
	67	4.0	5.8	12.0	1:1.5	5.0	22.8	NG	26.2	NG	17.1	NG	O
	68	4.0	5.7	12.0	1:1.5	20.0	21.6	NG	23.0	NG	16.4	OK	▲
	74	8.0	7.8	12.0	1:2.0	20.0	41.6	NG	53.4	NG	40.5	NG	O
	75	11.0	7.5	12.0	1:2.0	20.0	37.0	NG	48.6	NG	48.1	NG	O
	76	11.6	7.5	12.0	1:2.0	20.0	37.0	NG	48.6	NG	50.2	NG	O
	81	8.0	4.5	12.0	1:3.0	5.0	5.3	NG	14.3	NG	7.9	NG	O
	82	5.0	4.5	12.0	1:1.5	8.0	10.6	NG	9.8	NG	11.3	NG	O
Okgye	93	15.5	6.1	12.0	1:1.3	32.0	30.6	OK	29.4	OK	54.3	NG	◇
	94	13.5	6.1	12.0	1:1.3	32.0	30.6	OK	29.4	OK	50.0	NG	◇
Jumunjin	101	8.0	3.6	12.0	1:3.0	5.0	2.7	OK	8.3	NG	5.1	NG	∆
	102	9.0	5.1	12.0	1:2.0	5.0	11.6	NG	19.3	NG	18.2	NG	O
	103	13.0	7.4	12.0	1:2.0	12.5	35.5	NG	47.0	NG	53.5	NG	O
	104	8.5	5.4	12.0	1:1.3	12.5	21.2	NG	18.9	NG	27.4	NG	O
	105	8.5	4.7	12.0	1:2.0	12.5	9.1	OK	15.8	NG	14.6	NG	∆
	106	9.0	4.7	12.0	1:2.0	20.0	9.1	OK	15.8	OK	15.3	OK	×
Sokcho	108	5.0	4.0	12.0	1:1.3	12.5	8.6	OK	6.4	OK	9.1	OK	×
	109	13.0	6.5	12.0	1:2.0	20.0	24.1	NG	34.5	NG	40.8	NG	O
	111	13.0	3.0	12.0	1:1.3	12.5	3.6	OK	2.3	OK	5.0	OK	×
Geojin	113	8.0	3.2	12.0	1:2.0	5.0	2.9	OK	2.9	OK	6.1	NG	◇
	115	8.0	4.2	12.0	1:1.5	12.5	8.7	OK	7.7	OK	13.6	NG	◇
	116	8.0	4.2	12.0	1:1.5	12.5	8.7	OK	7.7	OK	13.6	NG	◇
	117	8.0	5.2	12.0	1:1.5	20.0	16.4	OK	16.5	OK	22.4	NG	◇
	119	5.0	1.5	12.0	1:1.5	3.0	0.4	OK	0.2	OK	0.5	OK	×
NG/total							**31/57 (54.4)**		**40/57**		**44/57**		
Reliability (%)							**54.4**		**70.2**		**77.2**		

Notes: ×, Hudson (OK), VDM (OK), Yoo (OK); ◇, Hudson (OK), VDM (OK), Yoo (NG); ∆, Hudson (OK), VDM (NG), Yoo (NG); O, Hudson (NG), VDM (NG), Yoo (NG); □, Hudson (OK), VDM (NG), Yoo (OK); ▲, Hudson (NG), VDM (OK), Yoo (OK).

This shows the importance of local effects at the breakwater point, where many shoaling effects such as nonlinear wave formation, wave breaking, and wave set-up may occur. Particularly when the water depth is between 8 and 15 m at a wave period of 11 or 12s, the shoaling effects become significant such that larger values of armor block weight result by using the present equation.

All equations still yield OK at nine sites (as indicated by × in Table 2). At several of these sites, the estimated design wave heights are considered of poor quality. The design wave heights and water depth are relatively small in these cases; thus, particular attention should be given to site visit and precise measurement of depth contour. At only one site (No. 68), the Hudson and M–J equations yield NG whereas the Y–Y equation yields OK (denoted by ▲). The water depth at this site is relatively shallow, and the breakwater slope is relatively steep. The shoaling effects might be more significant than we have considered here. The effects of wave reflection are surmised to be dominant at this site.

CONCLUSION

In this study, a new surf parameter called the "wave action slope" is introduced to represent the local wave conditions in shallow waters by employing local values of wave length and wave height. The use of linear wave theory on a flat bed of the depth at the front of the breakwater may be far superior to the simple adoption of the deep water wave length for characterizing surfing waves at a shoaling depth. The wave action slope is formed by the product of the breakwater slope and the ratio of the celerity to the wave height. The optimum or minimum weight of the armor unit is related to the wave action slope, which is employed for developing the new empirical equation. After analyzing the laboratory data of van der Meer (1988), we found the armor weights to be closely related to the wave action slope. By using the new surf parameter, we suggested a simple but accurate equation for estimating breakwater armor weight.

Several empirical equations are applied to cases of failure experienced along the Korean East Coast. The values given by the Hudson equation are satisfactory for 26 of the 57 total failure cases. The number of satisfactory cases decreases by using recent empirical equations. Among the 57 cases, the van der Meer–de Jong and Yoo equations yield 17 and 13 satisfactory cases, respectively. The decrease in the number of satisfactory cases may indicate improvement of the new empirical equations over the Hudson equation.

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