CHAPTER 100

NEW WAVE PRESSURE FORMULAE FOR COMPOSITE BREAKWATERS

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ABSTRACT

A proposal is made for new wave pressure formulae, which can be applied for the whole ranges of wave action from non-breaking to postbreaking waves with smooth transition between them. The design wave height is specified as the maximum wave height possible at the site of breakwater.

The new formulae as well as the existing formulae of Hiroi, Sainflou, and Minikin have been calibrated with the cases of 21 slidings and 13 nonslidings of the upright sections of prototype breakwaters. The calibration establishes that the new formulae are the most accurate ones.

I. INTRODUCTION

Japan is currently constructing breakwaters with the rate of 30,000 to 40,000 m per year to protect her commercial ports, which count one thousand in total number. Most of these breakwaters are built with concrete caissons rested upon low rubble mounds, thus being subject to wave pressures upon vertical faces. They are designed mostly for the so-called breaking wave condition because their construction sites are relatively shallow in comparison with the design wave heights. In recent years, however, breakwaters have been extended to the water depths of 15 to 20 m or more, and the so-called standing wave condition has become applicable. Figure 1 is such an example, showing a plan of Kashima Port and its breakwaters. A question arises in the design of such breakwaters: i.e., how the discontinuity in the calculated wave pressure at the transition from breaking to standing wave conditions should be treated, because the discontinuity inevitably appears somewhere in the long stretch of a breakwater extending from the shallow to relatively deep waters.

As every harbor engineer knows, existing wave pressure formulae are for either the breaking wave condition or the standing wave condition except for Ito's formula [1] to the author's knowledge. And at the threshold between the breaking to standing waves, the calculated wave pressure decreases suddenly. The standard method of wave pressure calculation in Japan is no exception. As shown in Fig. 2, the wave height for the calculation of pressure is specified as the significant height, $H_{1/3}$. When the water depth above the mound of a breakwater, d, is less than $2H_{1/3}$, the formula by Hiroi [2] for breaking waves is employed; this gives $p_b = 1.5 w_o H$. Otherwise, the simplified Sainflou's formulae [3] with a partial application of Hiroi's pressure around the still water level are used. The application

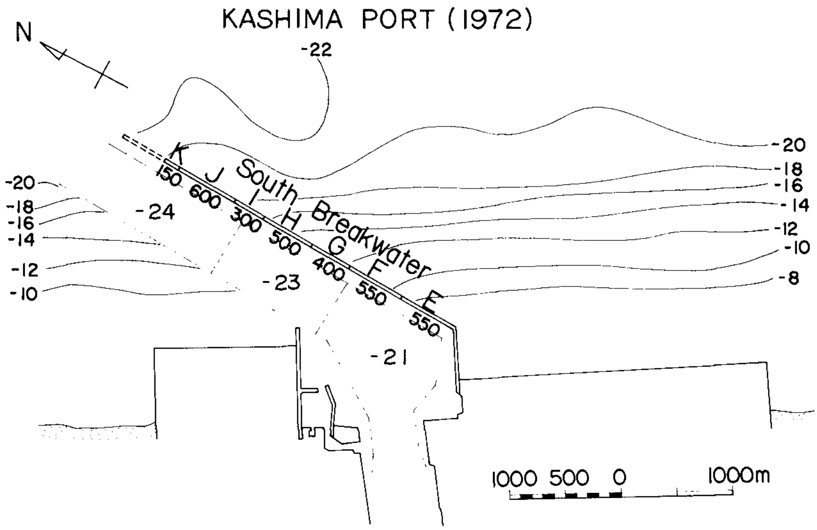


Fig. 1 Plan of Kashima Port

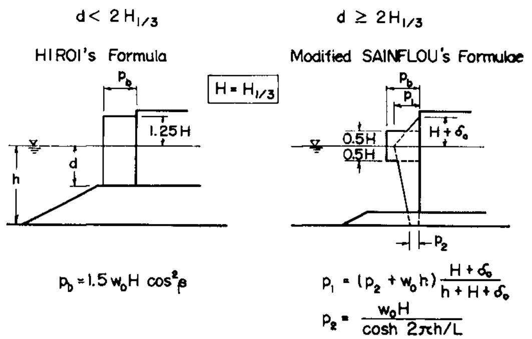


Fig. 2 Standard wave pressure formulae in Japan

of p_b in the range of $\pm 0.5\,H$ has been conceived partly for a possible partial breaking of wind waves and partly for the precaution against the attack of individual waves higher than $H_{1/3}$. Even with the partial application of p_b, a decrease of about 30 per cent in the total wave pressure is observed across the condition of $d = 2\,H_{1/3}$. The employment of Minikin's formulae [4] instead of Hiroi's formula will cause a larger change in the wave pressure across the threshold water depth.

The rationality of engineers calls for a smooth transition of wave pressure from breaking to standing waves, which will yield a gradual variation in the design width of breakwater caissons in a layout such as shown in Fig. 1. The present paper answers for the call by proposing new wave pressure formulae which cover the ranges of standing, breaking, and postbreaking waves with a single expression [5]. The new formulae are also characterized with the employment of the maximum wave height, H_{max}, at the design site. This reflects the design principle that an upright section of breakwater must withstand the attack of largest wave thrust expected. The followings are the details of the new formulae and the result of their calibration with 34 case studies of the performance of prototype breakwaters during high seas around the coasts of Japan.

II. FORMULATION OF THE NEW METHOD OF WAVE PRESSURE ESTIMATION

Wave pressures exerted upon a vertical wall is a complicated phenomenon, and the presence of a rubble mound makes the problem more difficult. Many investigations, theoretical, experimental, and of field observations, have been undertaken to clarify the phenomenon of wave pressures, and they have yielded a number of wave pressure formulae. But most of previous studies set their objects on either breaking waves or standing waves. These studies provide no solution for the question of a smooth transition from breaking to standing waves. By this reason, the author conducted his own experiments which covered the ranges of standing, breaking, and postbreaking waves by the increase of incident wave heights at several preselected wave periods (see [6,7] for details).

Figure 3 is an example of the distribution of maximum and minimum wave pressures along a vertical wall. The bottom of test flume had the gradient of 1 on 100, and the flume was gradually narrowed from the width of 80 cm to that of 50 cm over the distance of 18.5 m so as to secure the condition of postbreaking waves at a fixed test section. Figure 3 shows that wave pressures are almost proportional to the incident wave heights without exhibiting impulsive breaking wave pressures of high intensities.
Though the occurrence of impulsive breaking wave pressure is much feared in the design of vertical-faced breakwaters, it is realized only when wave conditions and dimensions of breakwater satisfy a set of certain requirements. It is a rare phenomenon for prototype breakwaters,which are built on the sea bed of gentle gradient and subject to the action of irregular waves with medium to large wave steepness (cf. Mitsuyasu's experiments [8]). Furthermore, it can be proved that the finite forward momentum of breaking wave front limits the wave load effective for sliding of an upright section resting on the nonlinear vibration system of the rubble mound and foundation to the mean pressure of about $(2 \sim 3)\, w_o H_b$, even if the exerting wave pressure itself may amount to $10\, w_o H_b$ or more [9].

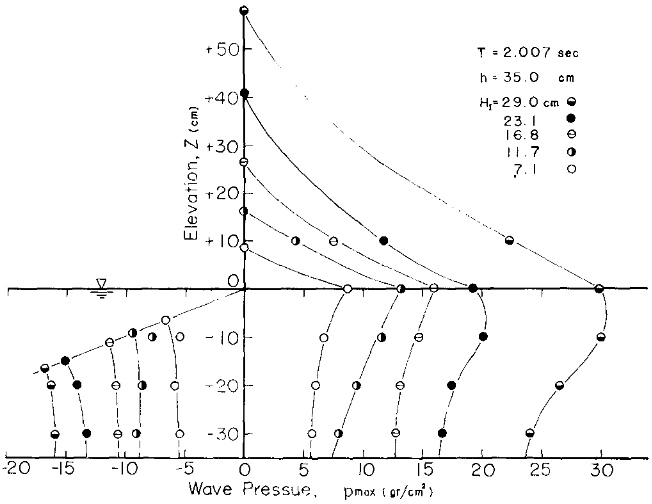


Fig. 3 Distribution of maximum and minimum wave pressures along a vertical wall

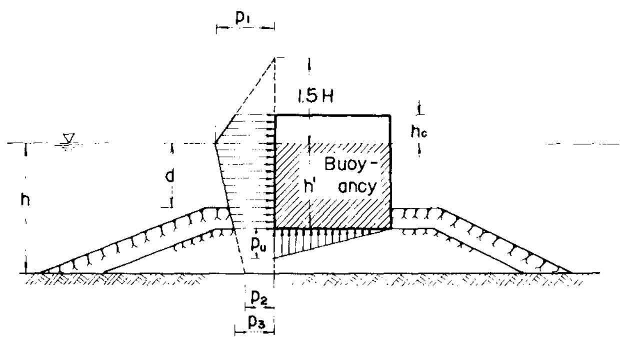


Fig. 4 Distribution of design wave pressure

The results of the experiments and the estimation of effective wave load as well as the requirement of simplicity in the application of formulae have brought the author to propose the distribution of design wave pressure as shown in Fig. 4. It has the largest intensity of p_1 at the still water level, and extends to the elevation of 1.5 H above the still water level. The imaginary wave pressure of p_2 at the elevation of sea bottom is proportional to p_1. The distribution is given by a straight line connecting p_1 and p_2, and another between p_1 and $p = 0$ at $z = 1.5\,H$. The wave overtopping due to a low crest height of breakwater is assumed to exercise no effect on the distribution and intensity of wave pressures for the sake of simplicity.

The intensities of wave pressures, p_1, p_2, and p_3, are calculated with the following formulae:

$$p_1 = w_o H_D (\alpha_1 + \alpha_2 \cos^2\beta) \tag{1}$$

$$p_2 = \frac{p_1}{\cosh 2\pi h/L} \tag{2}$$

$$p_3 = \alpha_3 p_1 \tag{3}$$

where,

$$\alpha_1 = 0.6 + \frac{1}{2}\left(\frac{4\,h/L}{\sinh 4\pi h/L}\right)^2 \tag{4}$$

$$\alpha_2 = \min\left\{\frac{h_b - d}{3\,h_b}\left(\frac{H_D}{d}\right)^2,\ \frac{2\,d}{H_D}\right\} \tag{5}$$

$$\alpha_3 = 1 - \frac{h'}{h}\left[1 - \frac{1}{\cosh 2\pi h/L}\right] \tag{6}$$

H_D : design wave height (see Eq. 8 hereafter)
w_o : specific weight of sea water
L : wavelength of design wave
$\min\{a,b\}$: smaller one of a or b
β : angle of wave approach
h_b : water depth at which the breaker height is to be evaluated (see Eq. 10).

The pressure factors of α_1 and α_2 have been determined empirically on the basis of experimental data and the calibration of new formulae with the case studies of prototype breakwaters. Comparison of the pressure intensity p_1 with experimental data is shown in Fig. 5 for a vertical wall without a rubble mound and in Fig. 6 for a vertical wall rested on a rubble mound. The pressure intensities by experiments are not the raw data of measurements but the results of calculation from the total pressures using the distribution of Fig. 4. The pressure intensity calculated with the above formulae is shown with dash-dot lines. The curves in full lines in Fig. 5 represent the theoretical values of finite amplitude standing waves of fourth order approximation [10,11]. For a vertical wall without a rubble mound, the factor of α_2 is almost nil, and the nondimensional wave pressure of p_1/w_oH_D is regarded constant. The effect of a rubble mound on wave pressure is represented with the factor of α_2, which increases parabolically with the relative wave height of H_I/h. The second term in the expression

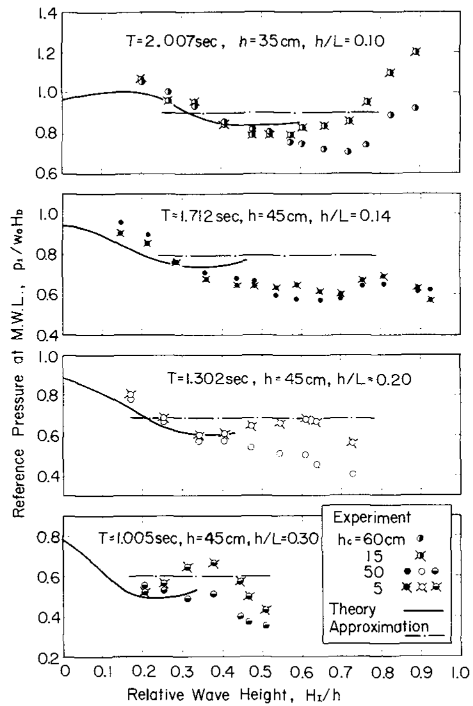


Fig. 5 Pressure intensity of p_1 on a vertical wall without a rubble mound

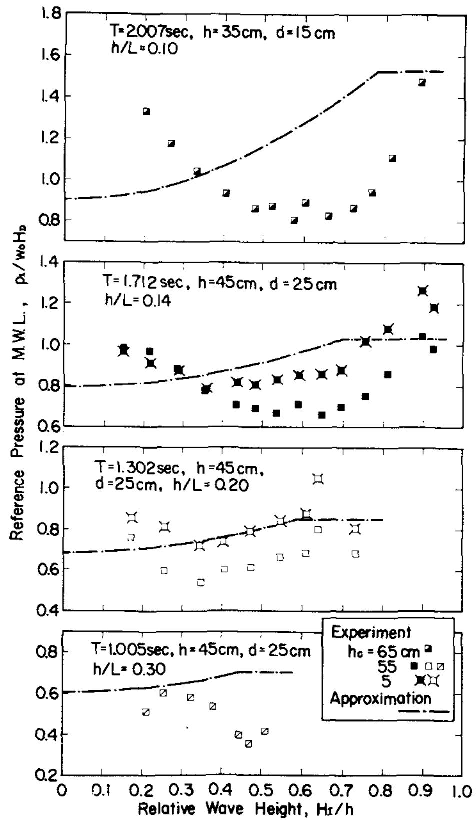


Fig. 6 Pressure intensity of p_1 on a vertical wall rested on a rubble mound

for α_2, or $2d/H_D$, is so incorporated to avoid the excessive increase of α_2 for $d \to 0$. (This term is not employed in Fig. 6, where α_2 is taken constant for postbreaking waves.) Though the agreement between the formulae and the laboratory data is not excellent, the difference may be disregarded for the purpose of general formulation of wave pressure estimation.

For the calculation of uplift pressure acting beneath the bottom of upright section, the triangular distribution is assumed irrespective of wave overtopping. The intensity of toe uplift is given by:

$$p_u = \alpha_1 \alpha_3 w_o H_D \qquad (7)$$

The omission of α_2 term is the consequence of expectation that the part of the wave pressure represented with α_2 is of short duration and will not contribute much to the total uplift. The buoyancy is calculated for the volume of upright section beneath the still water level even if its crest is low enough to cause wave overtopping. This method of buoyancy and uplift calculation, which is a departure from the standard method in Japan, was first proposed by Ito [1].

The design wave height, H_D, is the highest wave height expected under the given wave condition. It is the smaller one of $1.8 H_{1/3}$ or H_b, which is the limiting breaker height. The height of H_b is to be estimated not at the site but at the place in the distance of $5 H_{1/3}$ toward the offshore from the breakwater. That is,

$$H_D = H_{max} = \min\{1.8 H_{1/3}, H_b\} \qquad (8)$$

$$H_b = 0.17 L_o \{1 - \exp[-1.5 \frac{h_b}{L_o}(1 + 15 \tan^{4/3}\theta)]\} \qquad (9)$$

$$h_b = h + 5 H_{1/3} \tan\theta \qquad (10)$$

where L_o is the deepwater wavelength of $gT^2/2\pi$ and $\tan\theta$ denotes the mean gradient of sea bottom. Equation 9 for H_b is an empirical formulation of the breaker index prepared by the author [12], based on the compilation of laboratory data from various sources. The value of H_b/h_b by Eq. 9 is governed by both the gradient of sea bottom and the relative water depth of h_b/L_o as shown in Fig. 7.

The period of design wave is T_{max}, which can be taken as the same with $T_{1/3}$ on the basis of the statistical analysis of a number of surface wave records [13].

III. CALIBRATION WITH PROTOTYPE BREAKWATERS

The new method has been tested with the data of slidings of model breakwaters by regular and irregular waves, and it has succeeded in predicting their slidings [5]. The real test of any wave pressure formula, however, is the one with the data of the performance of prototype breakwaters during heavy seas. The analysis of slidings alone is not sufficient, but the analysis of nonsliding cases should also accompany the former for cross-examination of the accuracy of proposed formulae.

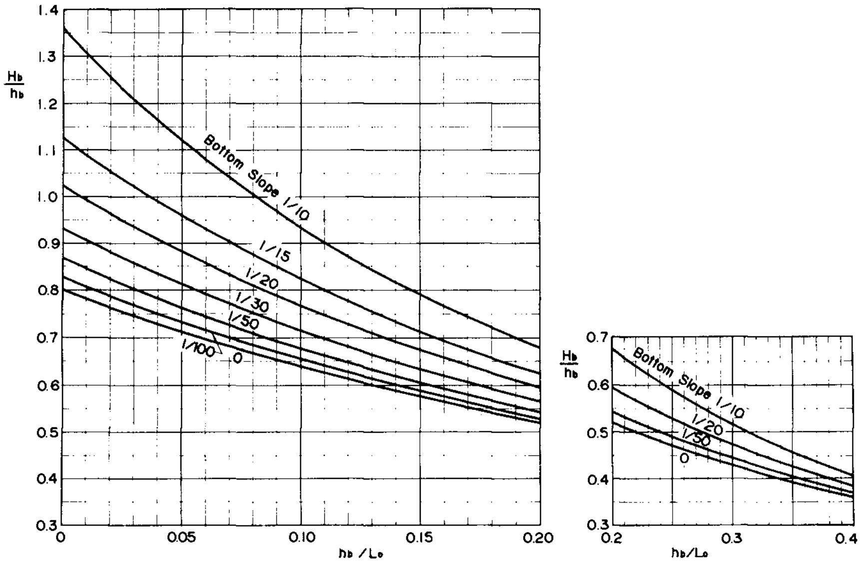


Fig. 7 diagram of limiting breaker height

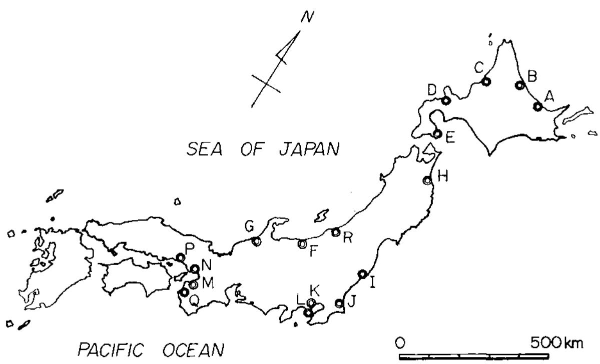
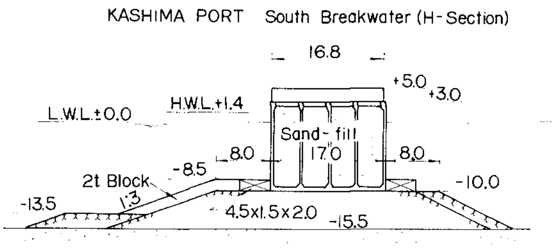


Fig. 8 Location map of the ports for the study of breakwater stability

Being the country with the longest stretch of composite breakwaters in the world, Japan can provide a long list of the performance of composite breakwaters during heavy seas. Slidings of upright sections, however, are relatively few in spite of large quantities of breakwater construction. The author searched for the cases of slidings of breakwaters in various reports and documents, and then investigated these cases if there might be breakwaters in the neighborhood which withstood the same storms without damage. Reportings of the observation of largest waves in record often provided excellent data of nonsliding cases. Existence of a large number of wave observation stations at the offices of harbor construction around Japan was most helpful in estimating the magnitude of high seas. For the cases in the past before modern wave observations became available, various techniques of wave hindcasting have been employed. In total, 21 cases of slidings and 13 cases of nonslidings at 17 ports shown in Table 1 and Fig. 8 have been selected for the calibration of wave pressure formulae.

An example of analysis is the case of the south breakwater (H-section) of Kashima Port, the cross section of which is shown in Fig. 9. When a storm passed by the port on January 9, 1972, twelve caissons of the H-section were slided by 0.2 to 1.8 m by the waves of $H_{1/3}$ = 6.5 m and $T_{1/3}$ = 14 sec (estimated values). The total wave pressures were estimated by four formulae: i.e., the standard method in Japan, the new method, Sainflou's formulae with $H_D = H_{max}$, and Minikin's formulae with $H_D = H_{max}$. The calculated pressures varied from 150.6 t/m by the standard method to 523.7 t/m by Minikin's formulae. With the weight of caisson being 278.9 t/m at the condition of full submergence, the pressures yield the safety factor of 1.11 to 0.33 against the sliding. (The full buoyancy for all the volume without the uplift pressure was assumed in the application of Sainflou's formulae, while the calculation same as the new method was applied for Minikin's formulae.) Since the caissons of the H-section actually slided, the safety factor of 1.11 by the standard method in Japan is contradictory with the reality. On the other hand, the safety factor of 0.33 by Minikin's formulae is considered too small, judging from the fact that nearly two-thirds of the caissons remained at their original positions.

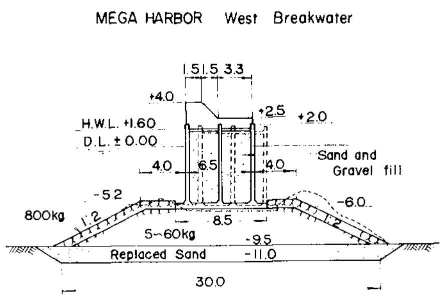
Another example is the case of the west breakwater of Mega Harbor, Himeji Port, shown in Fig. 10. When the typhoon No. 6420 approached the area, a part of the breakwater near the tip was without crown concrete, while the middle part was with cap concrete to the elevation of +2.0 m and the part near the bend to the jetty which connected the breakwater and the shore was just after completion. After the passage of the estimated waves of $H_{1/3}$ = 3.6 m and $T_{1/3}$ = 6.8 sec, the part without crown concrete was found in sliding by 0.17 to 1.12 m, whereas the other parts were almost unslided. The analysis of the safety factor against sliding with the four formulae has resulted in the values less than 1.0 for both the sliding and nonsliding. This may have been caused by an overestimation of wave heights or by some other factors. But attention is called for the difference between the safety factors of sliding and nonsliding cases. Though the non-sliding case should show the safety factor higher than the sliding case, the wave pressure formulae except for the new method fail to satisfy the expectation.



Sliding : $H_{1/3}$= 6.5m, $T_{1/3}$= 14.0 sec

Formulae	H_D (m)	P (ton/m)	S.F. against sliding
Standard	6.5	150.6	1.11
New	11.7	185.7	0.84
Sainflou	11.7	183.3	0.91
Minikin	11.7	523.7	0.33

Fig. 9 Case study of breakwater stability (1)



$H_{1/3}$ = 3.6 m, $T_{1/3}$ = 6.8 sec

Formulae	H_D (m)	Sliding		Nonsliding	
		P(t/m)	S.F.	P(t/m)	S.F.
Standard	3.6	44.6	0.78	55.8	0.74
New	6.5	36.1	0.75	45.7	0.82
Sainflou	6.5	46.2	0.74	58.4	0.71
Minikin	6.5	173.3	0.15	311.0	0.12

Fig. 10 Case study of breakwater stability (2)

Table 1. List of Breakwaters under Examination

No	Port	Breakwater (section)	(1)	(2)	Extent of damage if any	
					(3)	(4)
A	Abashiri	North B. (T)	1927	'27.12 - '28.1	about 4	43 (all)
B-1	Monbetsu	East B.	c*	'65.1.9	about 3	7 among 20
B-2	do	North B.	1930	do	[0]	none
C-1	Rumoi	South B. (B)	c	'21.11.7	0.5	5 (all)
C-2	do	do (A)	c	'21.12.4	[0]	none
D	Iwanai	West B. (C)	c	'65.12.15	0.3	2 among 6
E-1	Todohokke	East B. (H)	c	'59.9.27	about 1.5	3
E-2	do	do (G)	1955	'60.10.22	[0]	none
F-1	Himekawa	West B. (B)	1969	'70.2.1	5.4	3
F-2	do	do (E)	1972	'72.12.1	[0]	none
G-1	Kanazawa	West B. (C)	c	'67.12.15	0.4	9 among 10
G-2	do	do (E)	c	'70.2.1	[0]	none
H-1	Hachinohe	North B.(8th)	c	'66.12.15	3.7	6 among 8
H-2	do	do (10th)	c	'71.1.17	1.4	28 among 30
I-1	Onahama	2nd West B.(A)	c	'70.2.1	0.9	8 among 13
I-2	do	1st West B.(E)	1969	'71.4.29	about 0.6	7 among 120
I-3	do	do	1969	'70.2.1	[0]	none
I-4	do	2nd West B.(A)	1971	'71.4.29	[0]	none
J-1	Kashima	South B. (H)	1970	'72.1.9	about 1	12 among 33
J-2	do	do (J)	c	do	about 0.5	8 among 40
J-3	do	do (K)	c	'72.12.25	1.4	10 (all)
J-4	do	do (G)	1970	'72.1.9	[0]	none
J-5	do	do (I)	1971	do	[0]	none
J-6	do	do (J)	1972	'72.12.25	[0]	none
K-1	Yokohama	Kanagawa B.	c	'38.9.1	about 1	10 (all)
K-2	do	North B.	1935	'49.8.31	[0]	none
L	Kurihama	F_2-Seawall	c	'58.1.27	about 5	6 (all)
M	Kaizuka	West B.	1960	'61.9.16	about 8	9 among 10
N	Kobe	3rd B.	1937	'64.9.25	0.05	2
P-1	Mega	West B. (C)	c	'64.9.25	0.4	11 among 13
P-2	do	do (A)	1964	do	[0]	none
Q-1	Wakayama	Secondary B.	c	'64.9.25	0.4	5 (all)
Q-2	do	West B.	1961	'65.9.10	about 0.6	19 among 86
R	Niigata	West B. (F)	c	'70.2.1	[0]	none

Note:

* : "c" stands for "under construction."
(1) : Year of construction
(2) : Date of occurrence of high seas
(3) : Root-mean-square distance of sliding (m)
(4) : Numbers of slided caissons.

Table 2. Analysis of the Slidings of Casissons of Composite Breakwaters

No	Port	Water level (m)	Waves H_{max} (m)	$H_{1/3}$ (m)	$T_{1/3}$ (s)	β (°)	Dimensions of Breakwater (1) (m)	(2) —	(3) (m)	(4) (m)	(5) (m)	(6) (m)	(7) (t/m)	Safety Factor against sliding (a)	(b)	(c)	(d)	No
A	Abashiri	+1.5	8.7	5.2	9.0	0	-10.9	1/50	-5.0	-7.3	+2.4	11.5	142.6	1.11	0.84	1.04	0.28	A
B	Monbetsu	+1.6	7.8	4.5	8.0	40	-10.0	1/50	-6.0	-7.5	+1.0	9.0	77.3	1.34	0.70	0.76	0.37	B
C-1	Rumoi	+1.5	8.5	4.7	9.0	0	-14.5	1/200	-5.4	-7.3	+0.6	10.6	109.0	1.14	0.78	1.03	0.55	C-1
D	Iwanai	+1.0	8.1	4.5	9.0	0	-12.0	1/100	-7.0	-8.5	+0.5	11.5	113.9	1.09	0.83	0.98	0.37	D
E-1	Todohokke	+1.3	5.5	4.0	9.7	0	-5.5	1/50	-2.1	-5.0	+4.2	7.5	67.6	0.71	0.57	0.80	0.44	E-1
F-1	Himekawa	+0.8	11.3	6.3	11.1	0	-8.8	1/7	-4.5	-6.0	+4.5	15.0	177.9	1.04	0.47	0.86	0.28	F-1
G-1	Kanazawa	+0.5	6.7	5.0	9.0	0	-9.0	1/100	-3.0	-4.5	+1.0	15.0	94.8	1.34	0.65	1.47	0.52	G-1
H-1	Hachinohe	+1.5	6.0	5.0	10.0	10	-6.5	1/120	-2.5	-4.5	+2.5	10.5	80.9	0.93	0.76	1.11	0.46	H-1
H-2	do	+1.5	7.6	6.1	11.5	10	-8.8	1/150	-4.5	-6.5	+3.5	16.0	175.6	1.16	1.13	1.32	0.53	H-2
I-1	Onahama	+1.4	9.9	5.5	10.0	0	-17.0	1/500	-7.0	-8.5	+2.5	15.0	203.4	1.31	1.00	0.99	0.36	I-1
I-2	do	+1.4	9.8	6.0	11.0	14	-13.0	1/500	-6.0	-7.0	+5.0	13.0	195.9	1.12	0.95	1.14	0.33	I-2
J-1	Kashima	+1.4	11.7	6.5	14.0	0	-15.5	1/100	-8.5	-10.0	+5.0	17.0	278.9	1.11	0.84	0.91	0.33	J-1
J-2	do	+1.4	11.7	6.5	14.0	0	-19.0	1/100	-12.5	-14.0	+3.8	17.0	333.1	1.49	0.91	0.94	0.29	J-2
J-3	do	+1.4	11.7	6.5	14.0	0	-21.0	1/100	-12.5	-14.0	+3.8	17.0	333.1	1.49	0.96	0.94	0.31	J-3
K-1	Yokohama	+2.1	5.8	3.2	6.0	0	-11.7	1/500	-6.5	-6.5	+2.2	9.0	43.3	0.98	0.71	0.62	0.09	K-1
L	Kurihama	+2.0	5.7	3.5	8.5	0	-6.0	1/100	-4.5	-4.5	+2.5	8.0	63.2	1.00	0.87	0.94	0.21	L
M	Kaizuka	+3.0	4.7	3.0	5.5	25	-5.5	1/100	-1.5	-3.0	+3.5	5.0	32.8	0.80	0.79	0.73	0.17	M
N	Kobe	+3.2	5.9	3.3	6.0	0	-11.2	1/500	-4.9	-4.9	+3.0	6.9	49.1	1.19	0.88	0.75	0.16	N
P-1	Mega	+2.0	6.5	3.6	6.8	0	-9.5	1/500	-5.2	-6.0	+2.0	6.5	57.8	0.78	0.74	0.75	0.15	P-1
Q-1	Wakayama	+2.8	9.2	6.0	11.0	0	-10.0	1/100	-6.1	-8.1	+2.4	14.0	167.9	1.04	0.81	1.05	0.41	Q-1
Q-2	do	+2.0	8.6	6.3	12.0	0	-9.4	1/100	-5.0	-7.0	+5.0	12.0	162.5	0.84	0.80	0.92	0.34	Q-2

Note: See Table 3 for the explanation of (1) (7) and (a) (d).

Table 3. Analysis of the Nonslidings of Breakwater Caissons under High Seas

No	Port	Water level (m)	Waves H_{max} (m)	$H_{1/3}$ (m)	$T_{1/3}$ (s)	β (°)	Dimensions of Breakwater (1) (m)	(2) —	(3) (m)	(4) (m)	(5) (m)	(6) (m)	(7) (t/m)	Safety Factor against sliding (a)	(b)	(c)	(d)	No
B-2	Monbetsu	+1.6	6.7	4.5	8.0	30	-7.5	1/40	-6.0	-7.5	+3.6	10.5	137.4	1.19	1.10	0.97	0.24	B-2
C-2	Rumoi	+1.5	9.5	5.8	9.0	0	-13.9	1/200	-13.9	-7.3	+1.7	10.6	122.6	1.25	0.94	0.89	0.10	C-2
E-2	Todohokke	+1.3	5.3	5.0	10.0	0	-5.5	1/80	-4.0	-5.0	+4.2	9.0	79.7	0.67	0.95	0.98	0.35	E-2
F-2	Himekawa	+0.6	10.8	6.0	11.0	0	-16.0	1/7	-8.5	-10.0	+5.0	16.5	277.9	1.20	1.06	1.04	0.35	F-2
G-2	Kanazawa	+0.6	7.7	7.2	11.4	15	-9.5	1/100	-5.0	-6.5	+4.0	15.0	181.4	1.00	1.23	1.32	0.63	G-2
I-3	Onahama	+1.4	9.4	5.5	10.0	20	-13.0	1/500	-6.0	-7.0	+5.0	13.0	195.9	1.31	1.10	1.05	0.37	I-3
I-4	do	+1.4	10.8	6.0	11.0	20	-17.0	1/500	-7.0	-8.5	+5.0	15.0	247.9	1.35	1.01	1.02	0.39	I-4
J-4	Kashima	+1.4	11.5	6.5	14.0	0	-14.0	1/100	-8.5	-10.0	+5.0	17.0	278.9	1.11	0.87	0.93	0.32	J-4
J-5	do	+1.4	11.7	6.5	14.0	0	-17.5	1/100	-10.5	-12.0	+5.0	17.0	319.5	1.12	0.92	0.94	0.32	J-5
J-6	do	+1.4	11.7	6.5	14.0	0	-19.0	1/100	-12.5	-14.0	+5.0	17.0	358.8	1.50	1.04	0.96	0.32	J-6
K-2	Yokohama	+2.2	6.3	3.5	7.3	0	-11.7	1/500	-5.2	-6.5	+3.5	9.0	89.7	1.43	1.06	0.94	0.20	K-2
P-2	Mega	+2.0	6.5	3.6	6.8	0	-9.5	1/500	-5.2	-6.0	+4.0	6.5	69.2	0.74	0.82	0.71	0.12	P-2
R	Niigata	+0.7	10.1	7.1	12.5	17	-13.0	1/100	-8.0	-9.5	+4.0	16.0	241.6	1.07	1.00	1.04	0.37	R

Note:
(1) Elevation of sea bottom below the datum
(2) Gradient of sea bottom
(3) Elevation of the crest of armour stones or blocks of rubble mound below the datum
(4) Elevation of the base of caisson below the datum
(5) Elevation of the brest of breakwater above the datum
(6) Width of caisson at its base
(7) Submerged weight of caisson

(a) by the standard formulae in Japan
(b) by the new formulae
(c) by Sainflou's formulae with H_{max}
(d) by Minikin's formulae with H_{max}

The results of similar analysis for the remaining cases of slidings and nonslidings are summarized in Tables 2 and 3. The safety factors against sliding calculated with the four formulae are shown in Figs. 11 to 13 with open circles for nonsliding cases and closed circles for sliding cases. Figure 11 is the comparison of the standard formulae with the new method, while Figs. 12 and 13 exhibit the safety factors by Sainflou's and Minikin's formulae, respectively. If the open circles for nonslidings are found all above the line of S.F. = 1.0 while the closed circles for slidings are all below that line, the formulae under examination can be judged most accurate. From this point of view, the standard formulae in Japan do not perform well as evidenced by the existence of nonsliding data below the line of 1.0 (E and P) and that of sliding data above the line of 1.0 (B, G, I, J, and others). On the contrary, the new formulae produce only a minor intermixing of sliding and nonsliding data, and a line of boundary can be drawn around S.F. = 0.95.

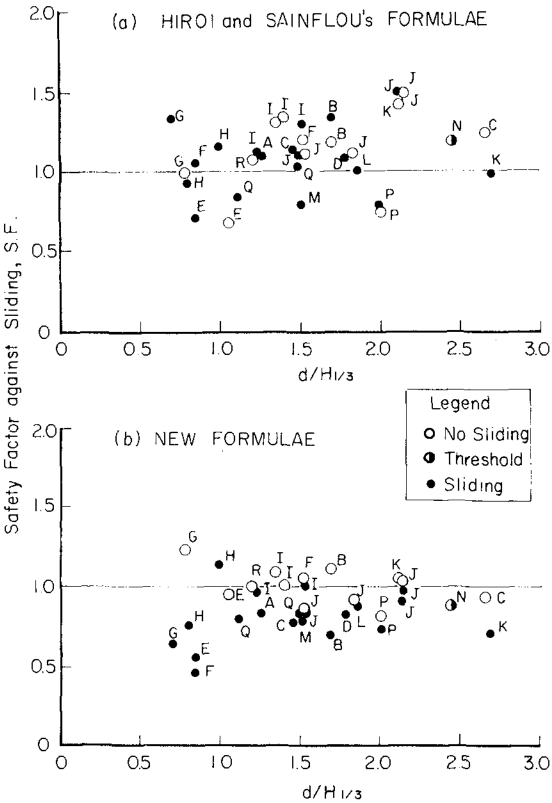


Fig. 11 Safety factors against sliding by the standard formulae and the new ones

The performance of Sainflou's formulae in Fig. 12 is fairly well, even though most of the breakwaters under study do not satisfy the so-called standing wave condition. A close examination of Fig. 12 reveals however that several nonsliding data have the safety factors smaller than those of sliding data at the same ports as in the cases of C, G, I, and P. The result with Minikin's formulae is poor with a total mixing of sliding and nonsliding data at very low level of safety factor. This indicates that Minikin's formulae predict wave pressures far larger than actual values. The employment of $H_{1/3}$ instead of H_{max} increases the absolute values of safety factor, but the extent of mixing of data is not improved. The formula proposed by Ito [1] for composite breakwaters was also examined, but the result was inferior to the new formulae.

The results of the analysis of the performance of prototype breakwaters establish that the new wave pressure formulae are the best ones for practical calculation of wave pressures upon composite breakwaters and for the analysis of the stability of upright sections against wave actions.

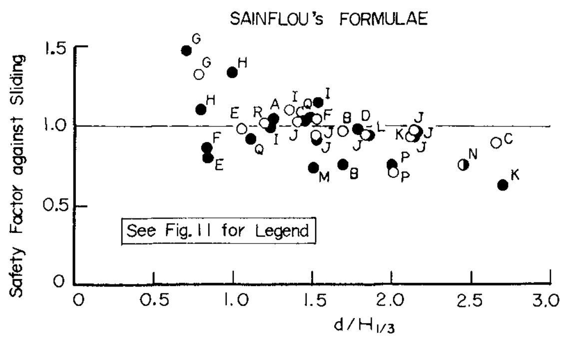


Fig. 12 Safety factor against sliding by Sainflou's formulae

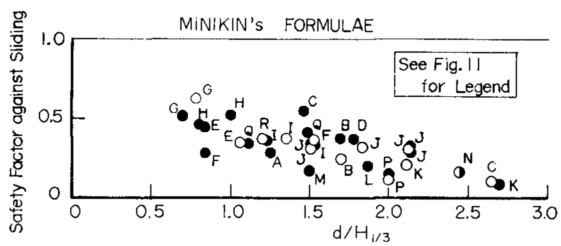


Fig. 13 Safety factor against sliding by Minikin's formulae

IV. CALCULATION OF THE WIDTH OF BREAKWATER CAISSON

The new wave pressure formulae are applied for the calculation of the width of breakwater caisson for illustration. The following assumptions are made:

gradient of sea bottom	: $\tan\theta = 1/100$
crest height of breakwater	: $h_c = 0.6\,(H_{1/3})_o$
crest height of caisson	: $h_c' = 1.0$ m
specific weight of caisson with sand filling	: $\gamma = 2.1$ t/m
specific weight of crown concrete	: $\gamma = 2.3$ t/m
thickness of armour stones	: $h' - d = 1.5$ m
safety factor against sliding	: S.F. = 1.2
frictional coefficient between caisson and rubble mound	: $\mu = 0.6$

The significant wave height at the site of breakwater is taken same as the deepwater value of $(H_{1/3})_o$ or $0.65\,h$ if the former exceeds the latter. The limitation of $(H_{1/3})_{max} = 0.65\,h$ is due to the results of wave observation around the coasts of Japan.

The results of calculation are shown in Figs. 14 to 16. The first figure exhibits the effect of the height of rubble mound on the caisson width. As the rubble mound becomes thick, a larger width of caisson is required as the result of the increase in wave pressure expressed with the factor of α_2. The effect of wave height is shown in Fig. 15. The wave period is so selected that the deepwater wave steepness will be in the range of $(H_{1/3})_o / L_o = 0.03 \sim 0.04$. In the shallow waters, the caisson width does not vary much with the increase in wave height, because

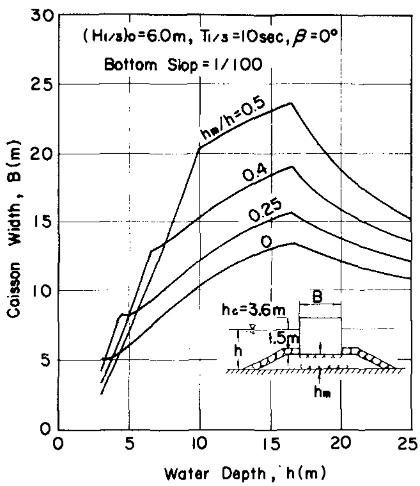


Fig. 14 Effect of rubble mound on the width of caisson

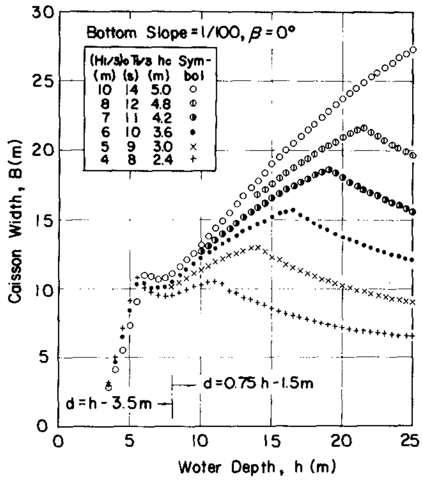


Fig. 15 Effect of wave height on the width of caisson

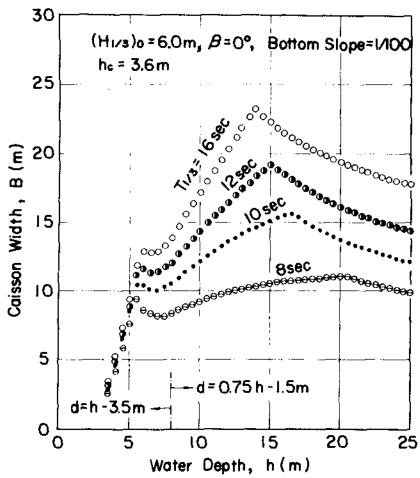


Fig. 16 Effect of wave period on the width of caisson

H_{max} is limited by the water depth. As the water becomes deep, however, the magnitude of wave height directly affects the caisson width. Therefore, the selection of design wave height in relatively deep waters is crucial for the safety of breakwater caissons.

The new wave pressure formulae indicate a strong influence of wave period on the wave pressure as shown in Fig. 16, where the wave period is increased from $T_{1/3}$ = 8 to 16 sec while the wave height is fixed at $(H_{1/3})_o$ = 6.0 m. In the waters deeper than h = 6.0 m, the caisson width is almost proportional to the wave period. The effect of wave period on the wave pressure is due to the two reasons. The first is that the breaker height increases as the wave period becomes long as indicated in Fig. 7. The second is that the intensity of wave pressure increases with the increase in wave period as represented with the factors of α_1 and α_3. Thus, the swells with long periods are more dangerous than wind waves with the same heights.

V. SUMMARIES

The new wave pressure formulae have the following characteristics:

(1) The design wave height is specified as the maximum wave height possible at the site of breakwater.

(2) The changes of wave pressures from standing through breaking to postbreaking waves are smooth, being calculated with a single expression.

(3) The uplift pressure is applied irrespective of the occurrence of wave overtopping, while the buoyancy is calculated for the volume of upright section beneath the still water level.

(4) The effects of the wave period and the gradient of sea bottom are incorporated in the estimation of wave pressure.

The new formulae have been calibrated with the cases of 21 slidings and 13 nonslidings of the upright sections of prototype breakwaters in Japan. The calibration establishes that the new formulae are the most accurate ones among various wave pressure formulae. With the new formulae, engineers will be able to design composite breakwaters under any wave condition with the consistent principles.

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