CHAPTER 59

PRESSURES BY BREAKING WAVES ON COMPOSITE-TYPE BREAKWATERS

S. Nagai

Professor of Hydraulics,

and

T. Otsubo

Assistant,

Department of Civil Engineering,
Faculty of Engineering,
Osaka City University,
Osaka, Japan.

INTRODUCTION

The intensity and vertical distribution of pressures exerted by water waves on the vertical walls of composite-type breakwaters vary in a very wide range from those of high shock pressures exerted by severe breaking waves to those of low pressures similar to hydrostatic pressure due to standing waves, depending upon the shape of the composite-type breakwaters, the characteristics of incoming waves, and the depths of water where the breakwater are located. In this paper are presented the intensity and vertical distribution of pressures exerted by various kinds of breaking waves on the vertical walls of composite-type breakwaters with low and large base-rubble-mounds constructed in comparaticely deep water.

COMPOSITE-TYPE BREAKWATERS WITH HIGH RUBBLE-MOUNDS

Shock pressures exerted by perfect and partial breaking waves on the vertical walls of composite-type breakwaters with high base rubble-mounds were studied in our laboratory from April of 1953 to March of 1959, and the results were published in June, 1960[1]. Those studies were concerned with the intensities of the shock pressures and their vertical distributions on the vertical walls of composite-type breakwaters, which had as high base-rubble-mounds as $h_1/h_2=0$ to 0.50, in which h_1 defined a water depth above the top of the rubble-mounds, and h_2 was a water depth at the toe of the harbour-side slope of the rubble-mounds. The top widths of the base-rubble-mounds, B, were so narrow as zero to two meters, and water depths in front of the vertical walls, h_1, were mostly smaller than 1.3 times the heights of incoming waves, H.

When breaking waves strike on composite-breakwaters, peak pressures at various elevations of the vertical wall of the breakwater do not occur at the same time. In general, there are slight retardations of occurrence in turn towards the wave crest from the bottom of the wall. Therefore the maximum wave pressure inducing the sliding and overturning of the vertical wall is not equal to the resultant of the peak pressures, but equal to the resultant of the simultaneous wave pressures exerting at the same instant as the occurrence of the maximum peak pressure, p_{max}, on the wall. This was proved by numerous experiments(2).

The vertical distributions of the maximum simultaneous pressures exerted by breaking waves on the vertical walls of composite-type breakwaters with high rubble-mounds are divided into three types which were referred to as A, B, and C-types(1).

A-type distribution. The maximum peak pressure on the wall, p_{max}, occurs at or in the vicinity of still water surface, and the pressures acting simultaneously with p_{max} above and below the still water surface decrease quickly in accordance with the following equation

$$p = p_{max} \left(1 - \frac{2y}{H} \right)^2 , \tag{1}$$

in which p_y defines the intensity of the maximum simultaneous pressure at a distance y above and below the still water surface. The resultant, P, of the A-type distribution is

$$P = 2 \int_0^{H/2} p_y \cdot dy = \frac{1}{3} \, p_{max} \cdot H \quad . \tag{2}$$

B-type distribution. p_{max} occurs at the bottom of the vertical wall and the simultaneous pressures above the bottom decrease with increasing elevation in accordance with the following equation

$$p_y = p_{max} \left(1 - \frac{y}{H} \right)^2 , \tag{3}$$

in which p_y denotes the intensity of the maximum simultaneous pressures at an elevation y from the bottom of the wall. The resultant is

$$P = \int_0^{H} p_y \cdot d_y = \frac{1}{3} \, p_{max} \cdot H \quad . \tag{4}$$

C-type distribution. p_{max} occurs at the bottom of the vertical wall, but the run-up along the wall is larger than that of B-type, in accordance with

$$p_y = p_{max} \left(1 - \frac{y}{1.5H}\right)^2 , \tag{5}$$

therefore the resultant is

$$P = \int_0^{1.5H} p_y \, dy = \frac{1}{2} \, p_{max} \cdot H \quad . \tag{6}$$

COMPOSITE-TYPE BREAKWATERS WITH LOW AND LARGE RUBBLE-MOUNDS

1. EXPERIMENTAL EQUIPMENT AND PROCEDURES

Since several years ago in Japan large composite-type breakwaters which have very large and low base rubble-mounds with wide top-widths, B=10 to 20m or more in front of vertical walls have been constructed in seas about 10 to 12m deep. The values of h_1/H in those breakwaters during typhoons are from 1 to 1.8.

Numerous experiments on pressures exerted on the vertical walls of such composite-type breakwaters with low and large base rubble-mounds have been conducted on a model-to-prototype scale of 1:20 since two years ago by the use of a wave channel equipped with a wind blower, 50m long, 1.0m wide, and 1.5m deep, as shown in Fig. 1. The conditions used in the experiments were as follows:

(a) Water depth at the horizontal bottom of the wave channel, h=51 to 81cm, in prototype h_p=10.2 to 16.2m; (the bottom slope of the channel from the toe of the outer slope of a rubble-mound to a 10m-distance is 1/100);

(b) Shape of breakwaters; h_2=20 to 65cm, in prototype h_{2p}=4 to 13m; h_1=10 to 50cm, h_{1p}=2 to 10m; the height of rubble-mounds, h_M=10, 15, 25, and 35cm, in prototype h_{Mp}=2, 3, 5, and 7m; B=0, 12.5, 25, 50, 75 and 100cm, in prototype B_p=0, 2.5, 5, 10, 15 and 20m; h_1/h_2=0.22 to 0.80, and B/h_2=0 to 2.5: Outer slope of the base rubble-mound was constant to be 1:3 in all breakwaters tested.

(c) Characteristics of waves; period T=1.17 to 3.05sec, in prototype T_p=5.2 to 13.6sec; wave height H=5 to 38cm, H_p=1 to 7.6m; wave length L=201 to 801cm, L_p=40 to 162m; h/L=0.08 to 0.37, and h_2/H= 1.3 to 6.2.

The range of the experiments are summerized in Table 1.

TABLE 1.- CHARACTERISTICS OF WAVES AND DIMENSIONS OF BREAKWATERS TESTED

		Model	Prototype
Water depth at the horizontal bottom : h		h_m=51 to 81cm	h_p=10.2 to 16.2m
Wave	Period : T	T_m=1.17 to 3.05sec	T_p=5.2 to 13.6sec
	Height : H	H_m=5 to 38cm	H_p=1.0 to 7.6m
	Length : L	L_m=201 to 801cm	L_p=40.2 to 162m
Breakwater	Water depth at the toe of rubble-mound : h_2	h_{2m}=20 to 65cm	h_{2p}=4.0 to 13.0m
	Water depth on the top of rubble-mound : h_1	h_{1m}=10 to 50cm	h_{1p}2.0 to 10.0m
	Height of rubble-mound : h_M	h_{Mm}=10, 15, 25, 35cm	h_{Mp}=2.0, 3.0, 5.0, 7.0m
	Width of the top of rubble-mound : B	B_m=0, 12.5, 25, 50, 75, 100cm	B_p=0, 2.5, 5.0, 10.0, 15.0, 20.0m
Relation between wave and water depth		h/L=0.08 to 0.37, h_2/H=1.3 to 6.2	
Relation between breakwater and water depth		h_1/h_2=0.22 to 0.80, B/h_2=0.0 to 2.5	

The right column in Table 1 shows numerals in prototype which is on a 1:20-model-to-prototype scale.

The experiments were conducted at an each shape of the composite-type breakwaters, as shown in Table 1, to generate from strong breaking waves to standing waves by varying the depth of water and the characteristics of wave for a given value of relative water depth at the horizontal bottom, such as h/L=0,10, 0.15, 0.20, and 0.28, respectively. The bottom-slope of the wave channel at the toe of the outer slope of base-rubble-mound was 1/100 when $h_2/H \geqq 1.80$, and that was 1/40 when $h_2/H < 1.80$. Since it was necessary to know the ranges of adaptability of the equations to obtain the pressures exerted by breaking waves on the vertical walls of composite-type breakwaters with high as well as low and large rubble-mounds, and by standing waves on vertical-wall breakwaters, respectively, comprehensive experiments were performed over a wide range from composite-type breakwaters with high rubble-mounds to those with extremely low and large mounds similar to a vertical-wall breakwater. In all the experiments six pressure-gages were used simultaneously to measure the maximum simultaneous pressures on the walls, and they were of a same type which have been used in the experiments concerning the pressures of breaking and standing waves. The characteristics of the pressure-gages are shown in Table 2.

TABLE 2.- CHARACTERISTICS OF PRESSURE-GAGES USED

Pressure plate		Phosphor-bronze; 0.2mm thick	
Number of bellows Pitch of bellows		4 to 5 4mm	
Thickness of acrylic plate attached with strain-gages		For the measurement of standing wave pressure	For the measurement of breaking wave
		0.8mm	2.0mm
Frequency of free oscillation	In air	f=140 to 160c/s	f=210 to 160c/s
	In water	f=100 to 110c/s	f=150 to 160c/s
Damping factor		0.02 to 0.03	0.02

2. EXPERIMENTAL RESULTS

(1) When the base-rubble-mounds of a composite-type breakwaters are so low that $h_1/h_2 \geqq 0.75$, and $h_2/H \geqq 1.80$, standing waves are always formed in front of the breakwaters regardless of the top-width of the base-rubble-mound, B. The pressures exerted by perfect or partial standing waves on vertical walls can be obtained with sufficient accuracy for practical design by the use of formulas(3),(4).

(2) When $0.40 \leqq h_1/h_2 < 0.75$, waves formed in front of the vertical walls of composite-type breakwaters vary over a wide range from a perfect breaking wave to a standing wave, depending upon the ratioes of h_1/h_2, B/h_2, h/L, and h_2/H. The ranges of the breaking and standing waves are shown in Figs. 2 to 5.

Generally speaking, as the value of h_1/h_2 becomes smaller towards 0.40, that is, as the height of a base rubble-mound becomes larger, waves become to break severely in front of the breakwater, while when the value of h_1/h_2 approaches to 0.75, there are formed non-breaking waves similar to a standing wave in front of the breakwater. Therefore the intensities of wave pressures vary over a wide range, but p_{max} occurs at or in the vicinity of the still

water surface in almost of the cases when $0.40 \leqq h_1/h_2 < 0.75$.
(3) The value of $\alpha = p_{max}/w_o H$, in which w_o denotes the unit weight of water, varies, depending upon the values of h_1/h_2, B/h_2, and h_2/H, as shown in Figs. 6 to 10.
(4) The vertical distributions of the maximum simultaneous pressures occurring simultaneously with the p_{max}, exerted by breaking or nonbreaking waves on the vertical wall are obtained as follows:

(a) Below the still water surface

The pressure intensity at the point of -z below the still water surface, p_z, is obtained by

$$p_z = p_{max} \cdot \frac{\cosh \beta \left(1 + \dfrac{z}{h_1}\right)}{\cosh \beta} , \qquad (7)$$

in which β is a function of α only, being independent of a relative water depth h/L, and is given by

$$\beta = \sqrt{6.0\alpha + 26.0} - 5.0 , \qquad (8)$$

in which $1.0 \leqq \alpha \leqq 5.0$. The relation between β and α is shown in Fig. 11.

(b) Above the still water surface

The height of a wave crest above the still water surface, $H_c = \gamma H$, on the vertical wall of a composite-type breakwater with high base-rubble-mound is 0.5H for the A-type distribution given in Eq. 1, and that is (1.0 to 1.3)H for a standing wave(3). Therefore, the value of $\gamma = H_c/H$ for a composite-type breakwater with low base-rubble-mound is considered to vary from 0.5 to 1.3, depending upon the values of α and h/L. The values of γ for various values of α and h/L obtained by the experiments are shown in Fig. 12, and as the upper values of these experimental ones, available for practical design purposes, were decided as shown in Table 3.

From numerous experiments the distribution of the pressures above the still water surface can approximately be assumed triangular such as

$$p_{max} = \alpha w_o H \qquad \text{at the still water surface,}$$

and

$$p_z = 0 \qquad \text{at } H_c = \gamma H \text{ above the S.W.L.}$$

The values of γ varies shown in Table 3.

TABLE 3.- VALUES OF γ

$h/L < 0.135$		$0.135 \leqq h/L < 0.35$	
$\alpha \leqq 1.3$	$\gamma = 1.3$	$\alpha < 1.0$	$\gamma = 1.0$
$1.3 < \alpha \leqq 1.5$	$\gamma = -2.13\alpha + 4.06$	$1.0 < \alpha \leqq 3.0$	$\gamma = -0.25\alpha + 1.25$
$1.5 < \alpha \leqq 3.0$	$\gamma = -0.25\alpha + 1.25$	$3.0 < \alpha$	$\gamma = 0.50$
$3.0 < \alpha$	$\gamma = 0.50$		

(c) The resultant of the maximum simultaneous pressurea is

$$P = \int_{-h_1}^{0} p_z \cdot d_z + \frac{1}{2} p_{max} \cdot H_c$$

$$= \alpha \, w_o H \left(h_1 \frac{\tanh \beta}{\beta} + \frac{1}{2} \gamma H \right) . \qquad (9)$$

3. COMPARISONS BETWEEN PRESSURES OBTAINED IN THE EXPERIMENTS AND MEASURED IN PROTOTYPE AND THOSE CALCULATED BY THE EQUATIONS

(1) Comparisons between the Experimental and Calculated Values.

Comparisona of the pressures calculated by Eqs. 7, 8, and 9 with those obtained by the experiments are shown in Figa. 13 to 16 (a), (b), and (c). Figures (a) in these figures show pressures very cloae to those of atanding waves, and figures (b) and (c) pressurea of breaking waves.

According to Figs. 13 to 16, it is known that the values calculated by the equations are in a fairly good agreement with the pressures obtained by the experiments, and the ratioes of the reaultant of pressurea in the experimenta, P_e, and that computed by Eq. 9, P_{cal}, that ia P_e/P_{cal}, are 0.81 to 1.07. In the other numerous comparisons the value of P_e/P_{cal} was moatly 0.85 to 1.0, with very rare cases of $P_e/P_{cal} > 1.0$.

(2) Comparisons between the Measured and Calculated Values.

The only one report on the field measurement of wave pressurea exerted on a composite-type breakwater which the writer could have for recent fifteen years is the field meaaurement which the Hokkaido Development Bureau made in 1957 and 1958 at the Harbor of Haboro in the Japan Sea(5). Fig. 17 shows the cross-section of the breakwater at the Harbor of Haboro and the location of three pressure-gages used for the measurement. The three pressures indicated by a black circle are measured values which would approximately be considered as the maximum simultaneous pressures during the measurement. The three double-circle marks in Fig. 17 denote the pressures calculated by Eqs. 7 and 8, which were obtained by the following calculation;

The heights and preiods of waves during the measurements were $H_{max} \doteqdot 2$ to 4.5m and $T \doteqdot 7$ to 8sec. The period of wave was assumed T= 7.3sec when the pressure gage located at the still water surface recorded $p_{max}=11$ t/m², but the mean sea level when $p_{max}=11$t/m² was not reported. Since the tidal range of the Japan Sea is within 0.5 to 0.6, the calculations of pressures were made for two cases of the mean sea level, D.L. +0m and +0.60m.

(i) For the case of D.L. ±0m sea level

The average depth of water at sea (4 to 6) L (wave length) offshore the breakwater is approximately assumed h=7.0m. Therefore L=55m for T=7.3sec, and $h/L \doteqdot 0.13$. Since $h_1/h_2=3.5/6.0=0.58$, the breakwater is defined as a composite breakwater with a low base-rubble-mound, and $B/h_2=3.0/6.0=0.50$. If H is assumed 4.0m, $h_2/H=1.5$.

(ii) For the case of D.L.+0.60m sea level

The average depth of water h=7.6m, and L=57m, therefore h/L= 0.133. If the maximum wave height is assumed $H_{max}=4.5$m, $h_2/H=6.6/4.5=1.46 \doteqdot 1.5$. $h_1/h_2=0.62$ and $B/h_2=0.46$. Since $h_2/H \doteqdot 1.5$ for the both cases, waves are assumed to break in front of the breakwater. From Fig.6, $\alpha=3.0$ for the both cases, then $\beta=1.63$ by Eq.8, and $\gamma=0.50$ from Table 3. The full line shown in Fig. 17 is the curve of vertical distribution of the maximum simultaneous pressures calculated by Eq. 7 by using the values of α, β, and γ. The calculated values show to be comparatively close to the measured. The calculated values of p_{max} are 12.4t/m² for H=4.0m, and 13.9t/m² for H=4.5m, while the design engineers of the breakwater were assumed $p_{max}=14$ to 15t/m². It may be stated that this assumption of p_{max} (presumably made from their experiences) was quitely adequate.

4. EFFECTS OF OVERTOPPING AND WIND ON WAVE PRESSURE

(1) Effect of Wave Overtopping.

According to the results which have been obtained until now in the experiments of composite-type breakwaters with low and large rubble-mounds, it may be stated in general that as the volume of wave overtopping increases, the intensities of the maximum simultaneous pressures would decrease from the top to the bottom of the vertical wall of the breakwater, and when the difference of height between the top of the vertical wall and the still water level is very small, the intensities of the maximum simultaneous pressures would distribute approximately uniformly, as these trends were seen in the results of experiments of breaking waves on composite-type breakwaters with high rubble-mounds and partial standing waves on vertical walls(4).

(2) Effect of Strong Winds.

It is seen that if strong winds with velocities over about 20m per sec blow over waves, the pressure intensities of breaking waves also increase slightly from the top to the bottom of the vertical wall of composite-type breakwater with low rubble-mound. The studies on the effects of wave overtopping and strong winds are now under way. The detailed report is scheduled to be written in near future.

CONCLUSIONS

From the study on the wave pressures exerted on the vertical walls of composite-type breakwaters with low and large rubble-mounds the followings are concluded.

1. When the base-rubble-mounds of composite-type breakwaters are very low and $h_1/h_2 \geqq 0.75$, and $h_2/H \geqq 1.80$, standing waves are always formed in front of the breakwaters, regardless of the top-width of the rubble-mound, and the pressures exerted by all kinds of standing waves on vertical walls can be obtained by using formulas(3),(4).

2. When $0.40 \leqq h_1/h_2 < 0.75$, breaking waves or standing waves are formed in front of the vertical walls of the composite-type breakwaters, depending upon the values of h_1/h_2, B/h_2, h/L, and h_2/H. The ranges of breaking waves and standing waves are shown in Figs. 2 to 5.

It may be stated that the maximum peak pressure on the vertical walls of the composite breakwaters with low and large rubble-mounds, p_{max}, is always located on the still water surface or on its vicinity, and the value of $\alpha = p_{max}/w_o H$ increases as the value of h_1/h_2 approaches to 0.40, while α decreases as h_1/h_2 approaches to 0.75, being $1 \leqq \alpha \leqq 1.3$ for standing waves.

3. When waves break in front of the vertical wall, the value of α varies, depending upon the values of h_1/h_2, B/h_2, and h_2/H, as shown in Figs. 6 to 10. The vertical distributions of the maximum simultaneous pressures exerted by breaking waves are determined by Eq. 7, and the resultant pressures are obtained by Eq. 9.

4. When the difference of height from the still water surface (or mean sea level during storms) to the top of the vertical wall is so small that there is a large volume of wave overtopping, the intensities of wave pressures on the wall decrease from the top to the bottom, as the volume of wave overtopping increases. The strong winds over waves lead to a slight increase in the intensities of wave pressures on the wall. But since the study on the effects of wave overtopping and strong winds on the pressures of breaking waves on the composite breakwaters is now under way, the detailed report will be written in near future.

REFERENCES

(1) Nagai, S. ; "Shock Pressures exerted by Breaking Waves on Breakwaters", Journal of the Waterways and Harbors Div., ASCE., June, 1960.

(2) Nagai, S. ; "Sliding of Composite-Type Breakwaters by Breaking Waves", Journal of the Waterways and Harbors Div. ASCE., Feb., 1963.

(3) Nagai, S. ; "Pressures of Standing Waves on a Vertical Wall", Journal of the Waterways and Harbors Div., ASCE.(to be published).
(4) Nagai, S. ; "Pressures of Partial Standing Waves", Journal of the Waterways and Harbors Div., ASCE, Aug., 1968.
(5) Kuribayashi, Muraki, and Others ; "On the Pressures exerted on a Breakwater", Proc. of the Fifth Coastal Engineering Conference (Japan), 1958.

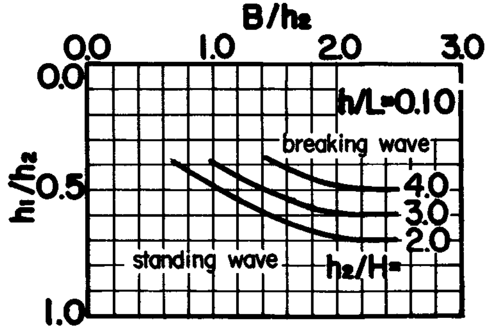


FIG. 2.

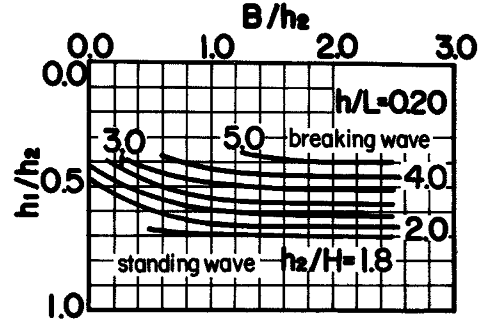


FIG. 4.

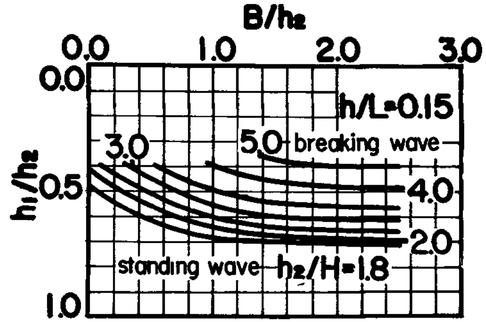


FIG. 3.

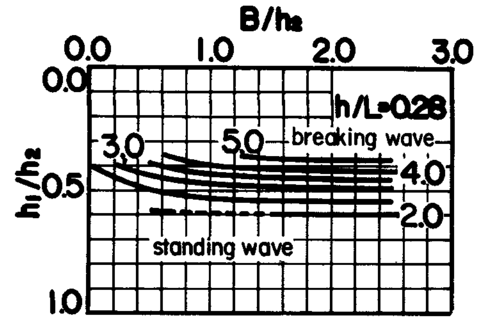


FIG. 5.

FIGS. 2 to 5.- RANGES OF BREAKING AND STANDING WAVES. THE UPPER PART OF THE LINE SHOWS THE RANGE OF BREAKING WAVES AND THE LOWER PART BELOW THE LINE THE RANGE OF STANDING WAVES

FIG. 1.- WAVE CHANNEL WITH WIND BLOWER

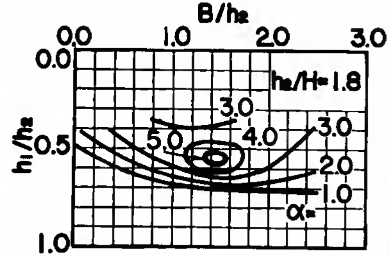


FIG. 7.

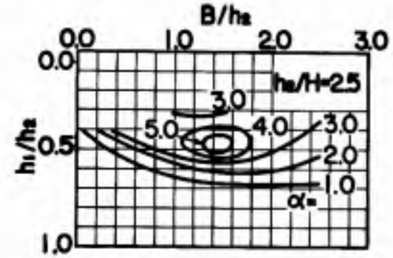


FIG. 9.

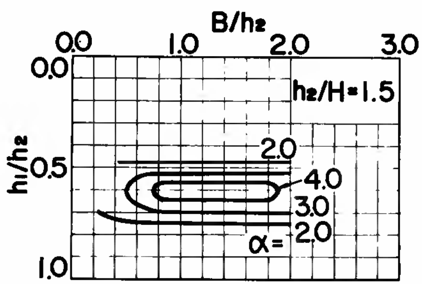


FIG. 6.

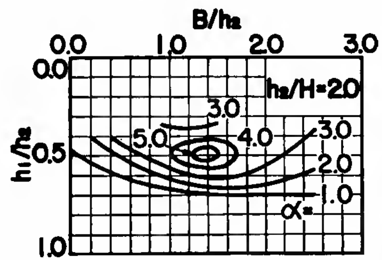


FIG. 8.

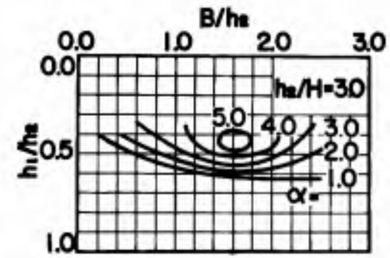


FIG. 10.

FIGS. 6 to 10.- VALUE OF $\alpha = p_{max}/w_0 H$

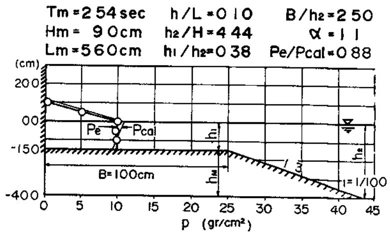
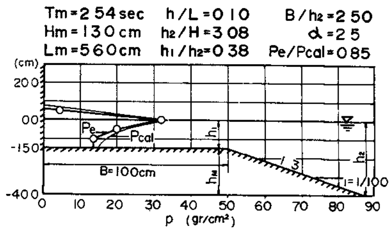


FIG. 13 (a).



(b).

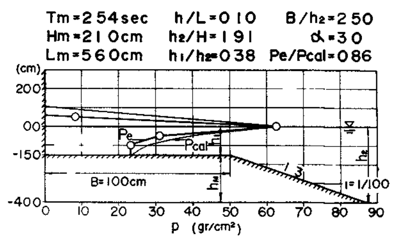


FIG. 13 (c).

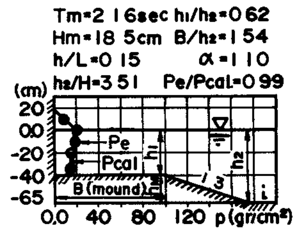
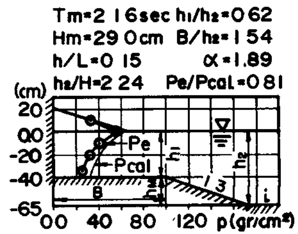


FIG. 14 (a).



(b).

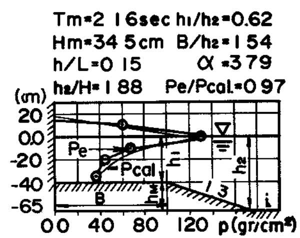


FIG. 14 (c).

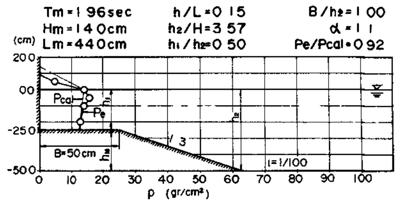
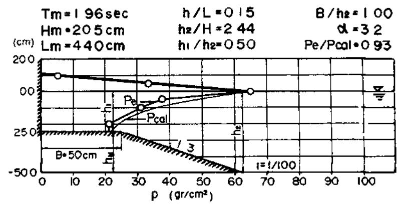


FIG. 15 (a).



(b).

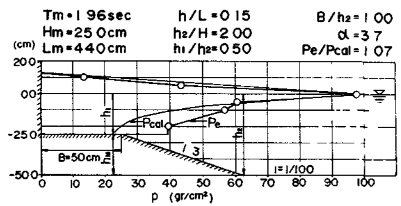


FIG. 15 (c).

FIGS. 13 to 16.- COMPARISONS BETWEEN EXPERIMENTAL AND CALCULATED PRESSURES

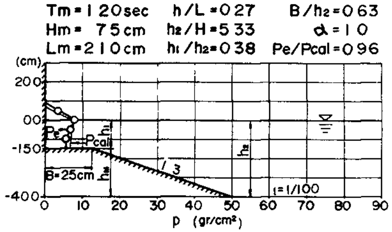
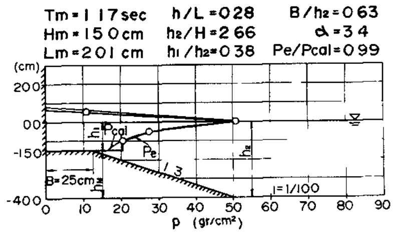


FIG. 16 (a).



(b).

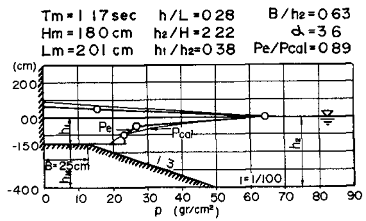


FIG. 16 (c).

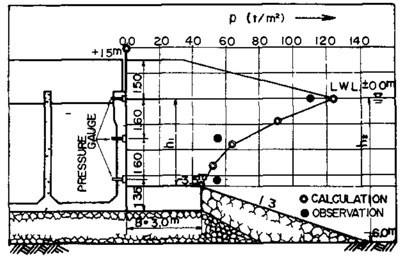


FIG. 17.- CROSS-SECTION OF THE BREAKWATER AT THE HARBOR OF HABORO AND COMPARISON BETWEEN MEASURED AND CALCULATED PRESSURES

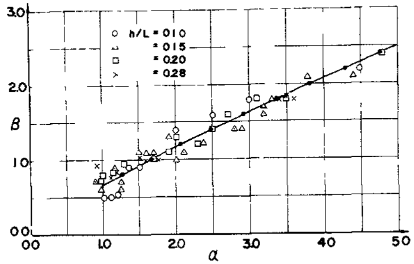


FIG. 11.- RELATION BETWEEN α AND β

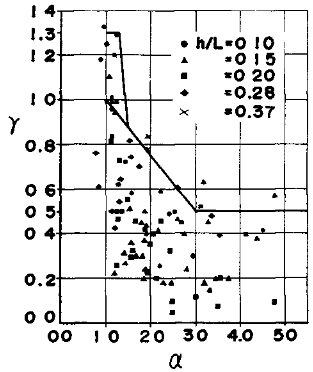


FIG. 12.-RELATION BETWEEN α AND γ