CHAPTER 59

PRESSURES BY BREAKING WAVES ON COMPOSITE-TYPE BREAKWATERS

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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 comparatively 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 pres-
sures at various elevations of the vertical wall of the breakwater
do not occur at the same time. In general, there are slight re-
tardations 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

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

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

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

    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

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

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

$$P = \int_0^H p_y \cdot dy = \frac{1}{3} p_{max} \cdot H \quad . \qquad\qquad (4)$$

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

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

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 break-
waters 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 proto-
type 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 break-water 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 neces-
sary 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 measure- ment of standing wave pressure	For the measure- ment 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 per-
fect 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 ra-
tioes 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 $0.40 \lesssim h_1/h_2 < 0.75$.

(3) The value of $\alpha = p_{max}/w_0 H$, in which w_0 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

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

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

$$\beta = \sqrt{6.0\alpha + 26.0} - 5.0 \quad , \tag{8}$$

in which $1.0 \lesssim \alpha \lesssim 5.0$. 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 $(1.0 \text{ to } 1.3)H$ 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

$$p_{max} = \alpha w_0 H \qquad \text{at the still water surface,}$$

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

(c) The resultant of the maximum simultaneous pressures is

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

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

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.
    Comparisons of the pressures calculated by Eqs. 7, 8, and 9 with those obtained by the experiments are shown in Figs. 13 to 16 (a), (b), and (c). Figures (a) in these figures show pressures very close to those of standing waves, and figures (b) and (c) pressures 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 pressures in the experiments, P_e, and that computed by Eq. 9, P_{cal}, that is P_e/P_{cal}, are 0.81 to 1.07. In the other numerous comparisons the value of P_e/P_{cal} was mostly 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 pressures exerted on a composite-type breakwater which the writer could have for recent fifteen years is the field measurement 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 periods 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/m^2, but the mean sea level when $p_{max}=11$t/m^{2}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}=4.5m, 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$1.5 for
the both cases, waves are assumed to break in front of the break-
water. From Fig.6, α=3.0 for the both cases, then β=1.63 by Eq.8,
and γ=0.50 from Table 3. The full line shown in Fig. 17 is the
curve of vertical distribution of the maximum simultaneous pressures
calculated by Eq. 7 by using the values of α, β, and γ. The calcu-
lated values show to be comparatively close to the measured. The
calculated values of p$_{max}$ are 12.4t/m^2 for H=4.0m, and 13.9t/m^2 for
H=4.5m, while the design engineers of the breakwater were assumed
P$_{max}$=14 to 15t/m^2. 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 $h_1/h_2 \gtrless 0.75$, and $h_2/H \gtrless 1.80$, 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 $0.40 \lesssim h_1/h_2 < 0.75$, breaking waves or standing waves are
formed in front of the vertical walls of the composite-type break-
waters, 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 $1 \lesssim \alpha \lesssim$
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 inten-
sities 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.

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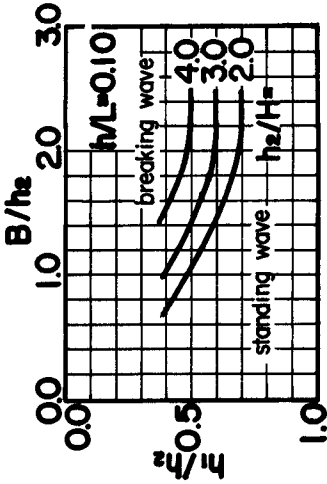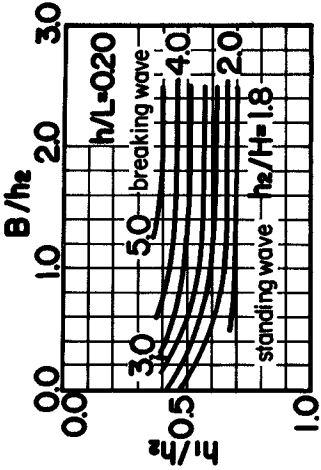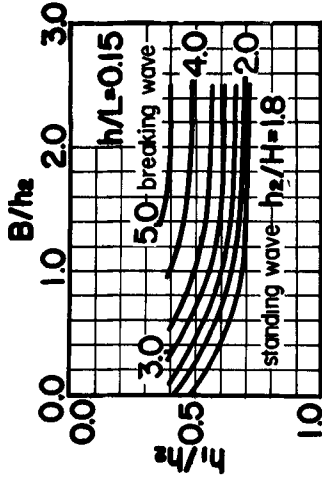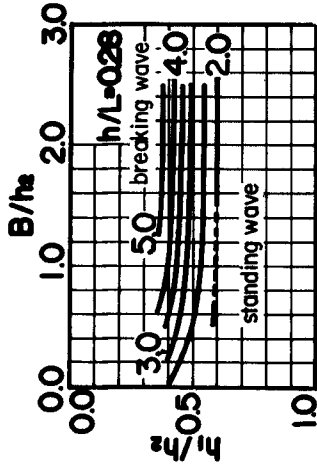
FIGS. 2 to 5.— RANGES OF BREAKING AND STANDING WAVES. THE UPPER PART OF THE LINE SHOWS THE RANGE
OF BREAKING WAVES AND THE LOWER PART BELOW THE LINE THE RANGE OF STANDING WAVES

FIG. 9.

FIG. 10.

FIG. 7.

FIG. 8.

FIG. 1.- WAVE CHANNEL WITH
WIND BLOWER
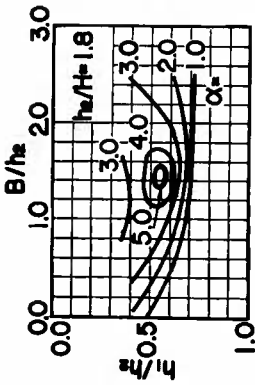
FIG. 6.
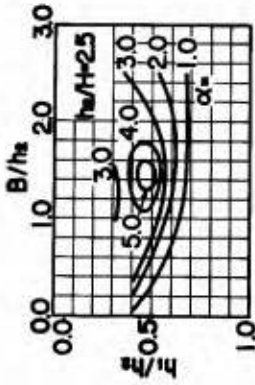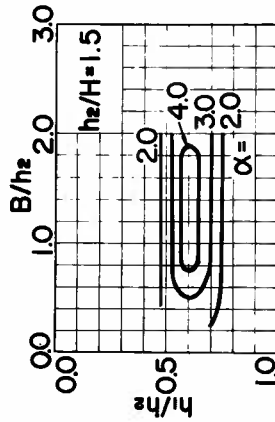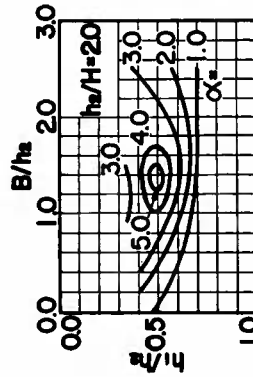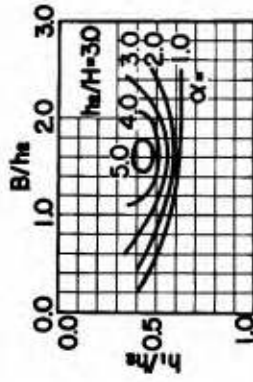
FIGS. 6 to 10.- VALUE OF $\alpha = p_{max}/w_0 H$
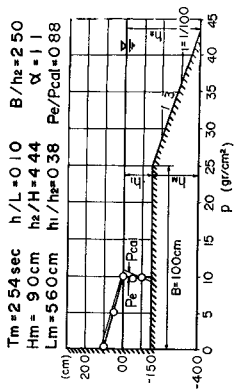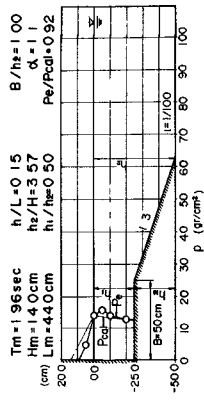
FIG. 15 (a).
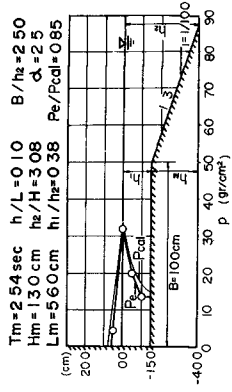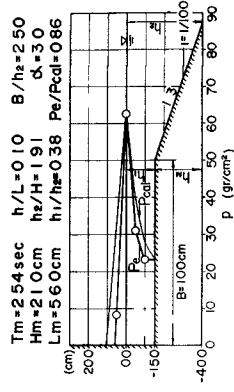
FIG. 15 (b).

FIG. 15 (c).

FIG. 14 (a).

FIG. 14 (b).

FIG. 14 (c).

FIG. 13 (a).

FIG. 13 (b).
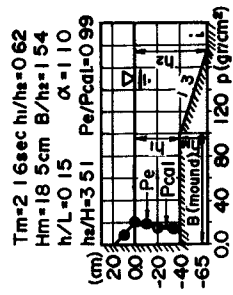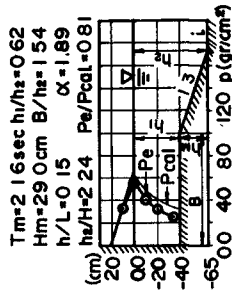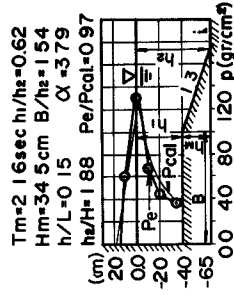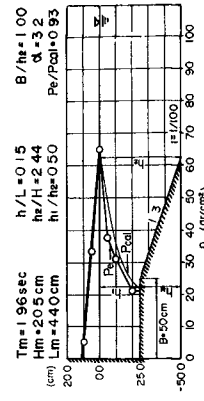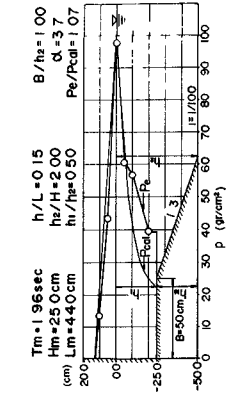
FIG. 13 (c).

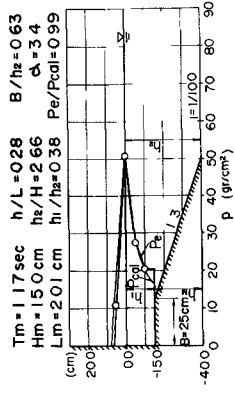FIGS. 13 to 16.- COMPARISONS BETWEEN EXPERIMENTAL AND CALCULATED PRESSURES
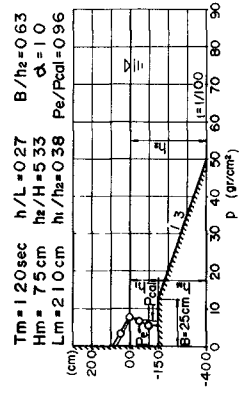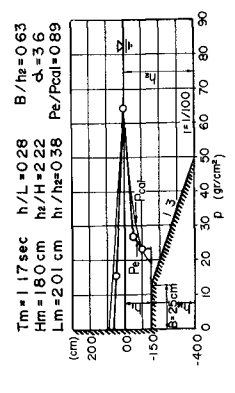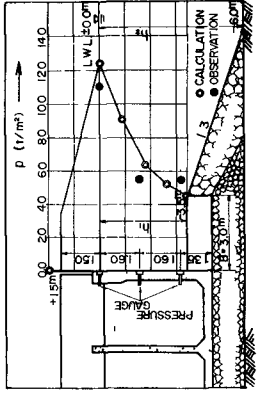
FIG. 17.— CROSS-SECTION OF THE BREAKWATER AT THE HARBOR OF HABORO AND COMPARISON BETWEEN MEASURED AND CALCULATED PRESSURES

FIG. 12.— RELATION BETWEEN α AND γ
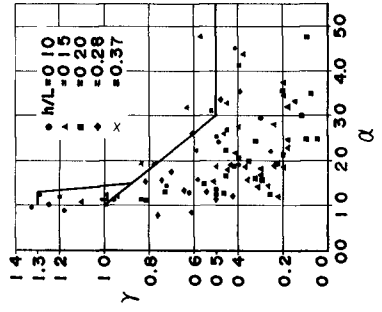
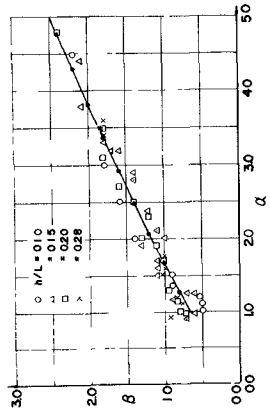FIG. 11.— RELATION BETWEEN α AND β

FIG. 16 (a).

FIG. 16 (b).

FIG. 16 (c).

$T_m = 1.20$ sec $h/L = 0.27$ $B/h_2 = 0.63$
$H_m = 75$ cm $h_2/H = 5.33$ $\alpha = 1.0$
$L_m = 210$ cm $h_1/h_2 = 0.38$ $P_e/P_{cal} = 0.96$

$T_m = 1.17$ sec $h/L = 0.28$ $B/h_2 = 0.63$
$H_m = 150$ cm $h_2/H = 2.66$ $\alpha = 3.4$
$L_m = 201$ cm $h_1/h_2 = 0.38$ $P_e/P_{cal} = 0.99$

$T_m = 1.17$ sec $h/L = 0.28$ $B/h_2 = 0.63$
$H_m = 180$ cm $h_2/H = 2.22$ $\alpha = 3.6$
$L_m = 201$ cm $h_1/h_2 = 0.38$ $P_e/P_{cal} = 0.89$