Chapter 31

WAVE FORCES AGAINST SEA WALL

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INTRODUCTION

The study concerning the wave forces acting on breakwater has been conducted by numerous scientists and engineers both in field and in laboratory. While few studies have been carried out on the wave forces acting on sea wall which is located inside the surf zone. In this paper are summarized the main results of the experimental studies conducted at the University of Tokyo, Japan, in relation to the subject on the wave forces against a vertical or inclined surface wall located shorewards from the breaking point, and also is proposed an empirical formula of wave pressure distribution on a sea wall on the basis of the experimental data. The computed results obtained by using the above formula are compared with the field data of wave pressure on a vertical wall measured at the Niigata West Coast, Niigata Prefecture, Japan, and also with the experimental data of total wave forces on a vertical wall; the project of the latter is now in progress at the University of Tokyo.

PRESENTATION OF EXPERIMENTAL RESULTS

LABORATORY PROCEDURES

The wave channel which is used for the present studies is 18 m long, 0.6 m high and 0.7 m wide, and a model of sea wall is installed on a gentle uniform slope of 1/15. The face angle of the model is adjustable in a wide range, and six pressure gauges are attached on the surface of model sea wall at different levels to measure simultaneously the time history of pressure working on a sea wall. Fig. 1 shows the sensing element of pressure cell which is mainly used for this experiment, and the block diagram of the recording system, by which the fluctuating phenomena of up to 300 cps can be recorded.

PRELIMINARY ANALYSIS

The wave pressure type is generally speaking classified into three, such as the clapotis type, the shock type and the type due to broken waves. Fig. 2 shows the classified regions of each wave pressure type mentioned above; the wave steepness ratio in deep water H_o/L_o is taken as the abscissa, and the ratio between the water depth at the foot of vertical sea wall

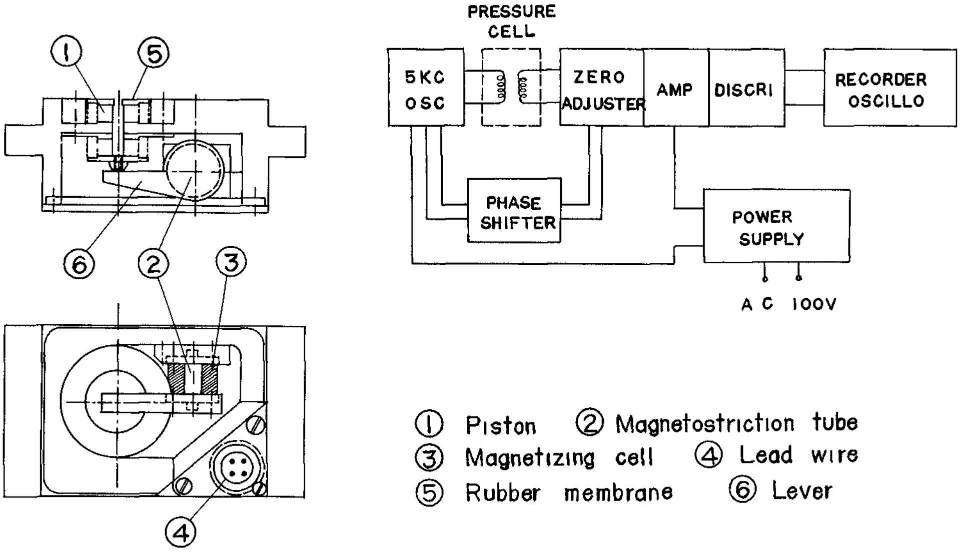


FIG 1 --PRESSURE CELL AND BLOCK DIAGRAM

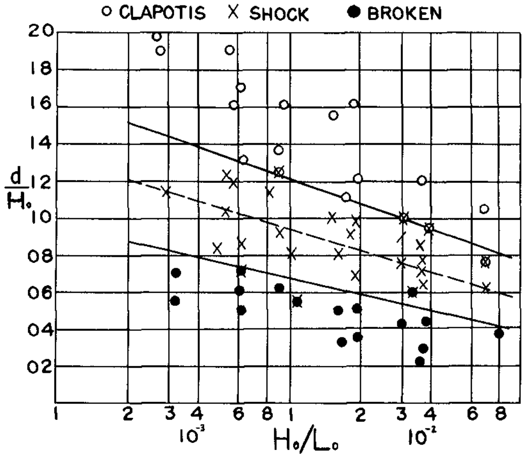


FIG 2 --CLASSIFICATION OF WAVE PRESSURE TYPE d′, and the wave height in deep water H_0 as the ordinate after the expression of Rundgren[1] who has taken the case of 1/9.8 beach slope. The authors are mainly concerned with the region of wave pressure due to the broken waves in this paper.

Several samples of wave height reduction of the broken waves as the decrease of water depth is given in Fig.3, from which it is recognized that the average tendency of these data is well expressed by a straight line. The straight line in this figure is introduced from the following assumption; that is, the broken wave holds its critical height determined by the solitary wave theory[2] at each particular water depth. Fig. 4 is the comparison between the hypothetical curves and the field observation data of wave transformation inside the surf zone at the Niigata West Coast obtained by T. Ijima and others[3]. From these results it is indicated that the above assumption is applicable for our present analysis as a first order approximation, hence

$$H = 0.78d \qquad (1)$$

where H is the wave height at the depth of water d. As the same rule is shown in Fig.5 the comparison on wave celerity inside the surf zone between the experimental data and the calculated ones by using Eq.(2) which is introduced on the basis of the solitary wave theory combining with Eq.(1)

$$c = \sqrt{g(H + d)} = \sqrt{1.78gd} \qquad (2)$$

where c is the wave celerity and g the acceleration of gravity. The agreement is rather good.

PRESSURE DISTRIBUTION ON VERTICAL WALL

According to the careful investigations on the transformation of wave inside the surf zone mentioned above and on the vertical distribution of wave pressure against a vertical sea wall, the following assumptions are introduced:

1) The static pressure works on a vertical wall up to a certain height above still water level h_c, till the bottom of structure.
2) The dynamic pressure distributes simply in a shape of triangle with its maximum pressure intensity at still water level and zero both at the height of h_c above still water level and at bottom.

Fig.6 gives the relationship between the wave pressure intensity at still water level p_1 and the water depth at the foot of sea wall d′, while Fig.7 gives the relationship between the additional wave pressure intensity above the static water pressure at the bottom of vertical wall $p_{-d'}$ and d′. By using these data the following Eqs.(3), (4) and (5) are obtained.

$$h_c = 1.2d' \qquad (3)$$

$$\begin{aligned} p_1 &= f\rho c^2/2 + \rho gh \\ &= (1.78gd')f\rho/2 + 1.2\rho gd' \\ &= (0.89f + 1.2)\rho gd' \qquad (4) \\ f &= 1.8 \qquad (5) \end{aligned}$$

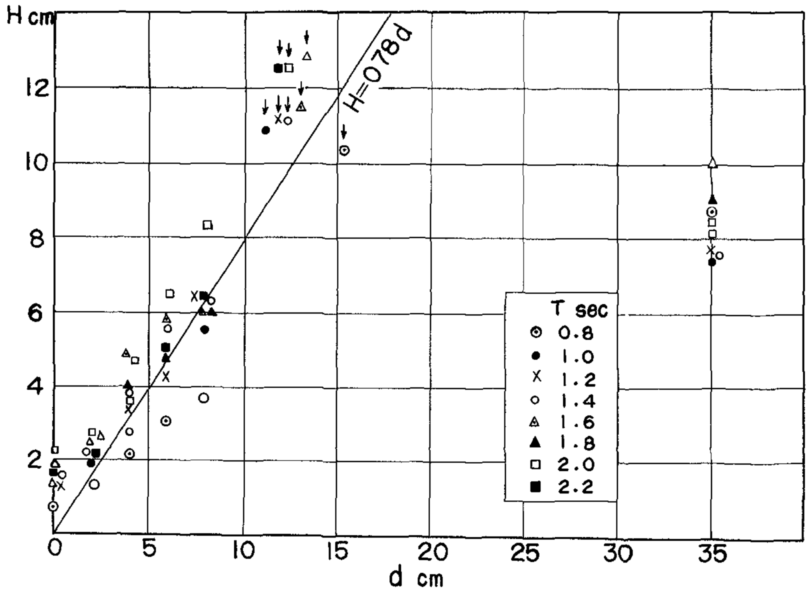


FIG 3 --CHANGE OF WAVE HEIGHT INSIDE SURF ZONE (LABORATORY)

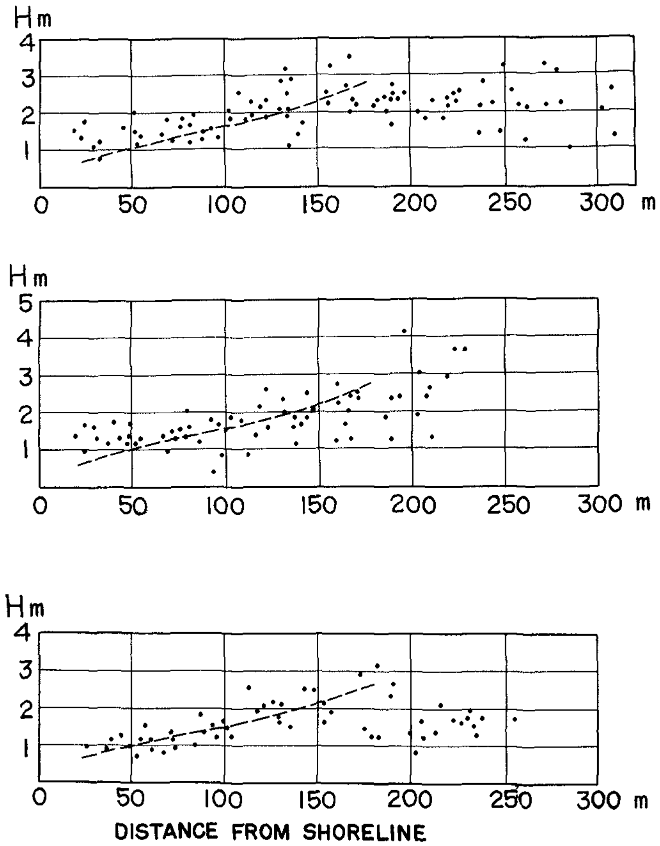


FIG 4 --CHANGE OF WAVE HEIGHT INSIDE SURF ZONE (NIIGATA COAST)

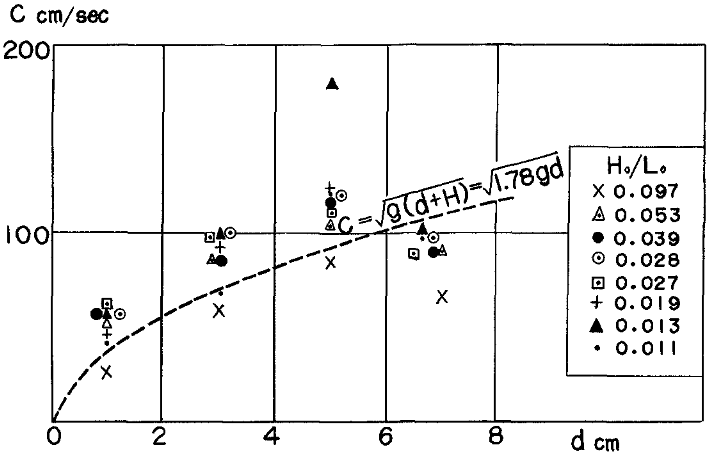
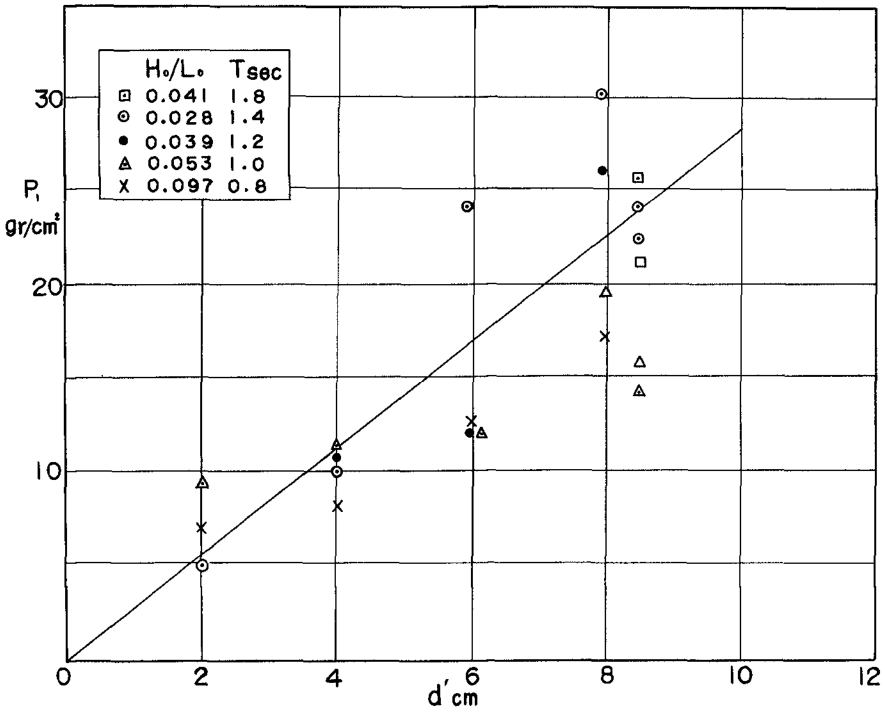


FIG 5 --CHANGE OF WAVE CELERITY INSIDE SURF ZONE (LABORATORY)

FIG 6 --WAVE PRESSURE INTENSITY AT STILL WATER LEVEL



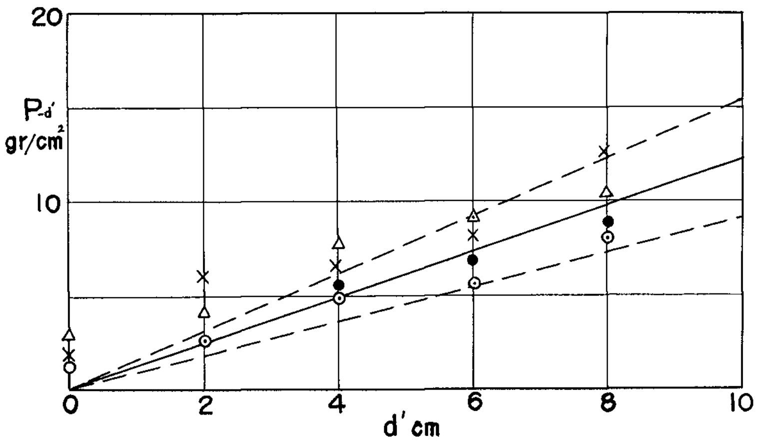


FIG 7 --ADDITIONAL WAVE PRESSURE INTENSITY AT BOTTOM

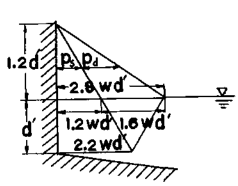


FIG 8

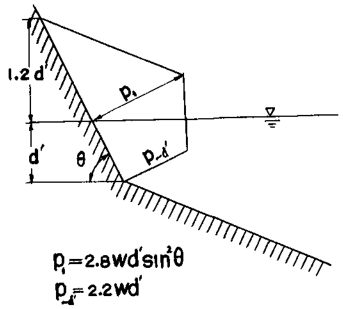


FIG 10

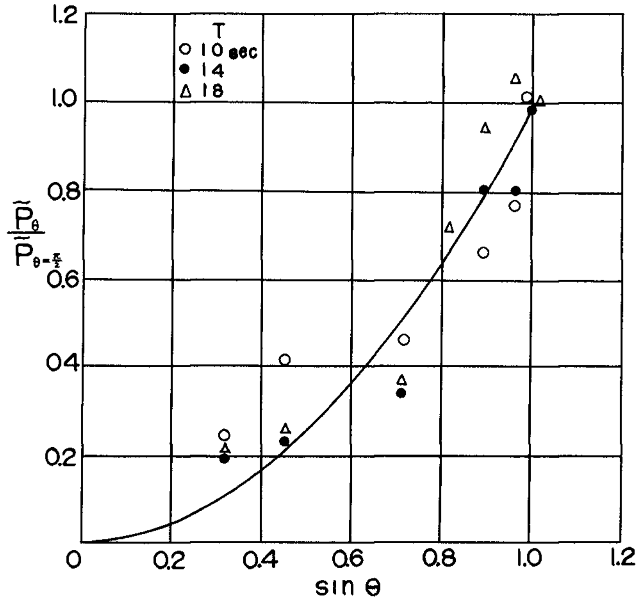


FIG 9 --PRESSURE RATIO AS A FUNCTION OF SLOPE ANGLE where ρ and g are the density of fluid and the acceleration of gravity respectively. Hence the formula which is applicable for the computation of vertical distribution of wave pressure due to broken waves is expressed as shown in Fig.8.

PRESSURE DISTRIBUTION ON SLOPING WALL

The effect of surface slope angle of sea wall on the intensity of wave pressure is demonstrated in Fig.9, where the ordinate is the ratio of pressure intensity on an inclined slope to that on a vertical wall, $\bar{p}_\theta / \bar{p}_{\theta=\frac{\pi}{2}}$, and the abscissa $\sin\theta$, θ being the angle of slope measured from horizontal plane. The curve shown in this figure is[4),5)]

$$\bar{p}_\theta / \bar{p}_{\theta=\frac{\pi}{2}} = \sin^2\theta \qquad (6)$$

Taking into consideration of the above fact, the previous formula is generalized as shown in Fig.10. Figs.11, 12 and 13 show several examples of the comparison between the experimental data and the calculated curves. The agreement seems to be satisfactory from the engineering point of view. But in the case of very shallow water depth at the foot of structure the discrepancy is large as suggested in Fig.3.

PRESENTATION OF FIELD DATA

TEST PROCEDURES

The field observations of wave pressure against sea wall have been conducted by the engineers at the Niigata Prefectural Government under the instruction of the present authors in order to study the scale effect of model investigations conducted at the University of Tokyo. Fig.14 is a diagram showing the position of three spring type pressure gauges attached on a concrete block which is deposited in front of the actual sea wall; the water depth at the foot of the structure is about 1 m. In a few days during the last winter season the authors attached a sensing element which was almost similar to that in the laboratory instead of the spring type pressure gauge in order to check the characteristics of the two kinds of pressure gauges.

STATISTICAL TREATMENTS

Before extending the discussion about the similarity between the laboratory measurements and the field observation data, we have to consider the way of statistical treatments of the irregularities of wave pressure and wave itself.

The cumulative frequency distribution curves of wave period are shown in Fig.15, from which it may be recognized that the occurrance frequency of the apparent wave period is expressed by the log normal distribution curve. On the other hand the frequency distributions of apparent wave height and wave pressure are well expressed by the Rayleigh distribution curves as

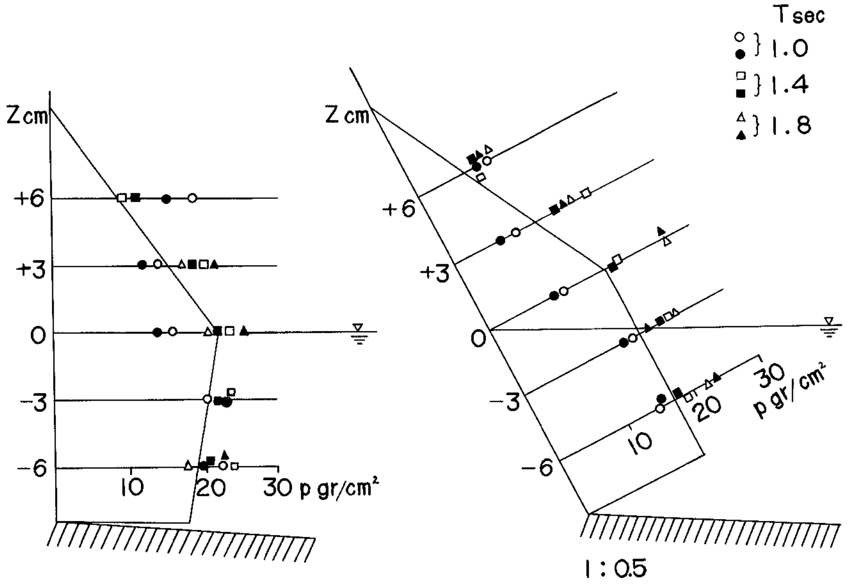
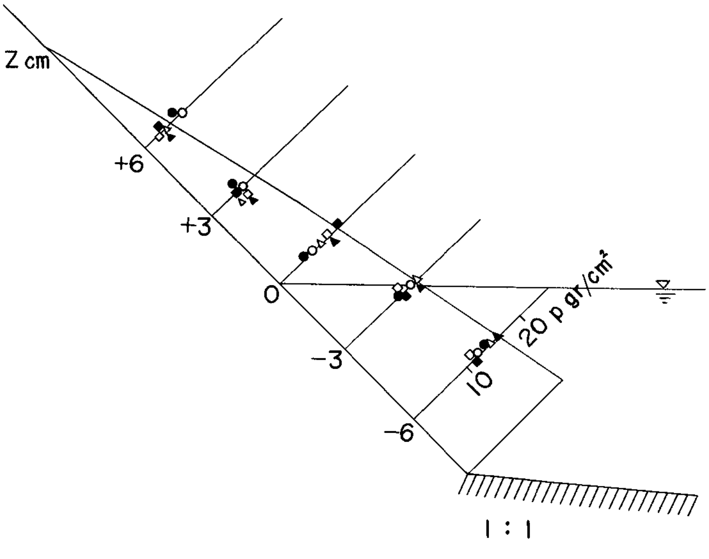

T sec
○ ● 1.0
□ ■ 1.4
△ ▲ 1.8
Z cm
+6
+3
0
-3
-6
10
20
30 p gr/cm²
Z cm
+6
+3
0
-3
-6
10
20
30
p gr/cm²
1 : 0.5

FIG 11


Z cm
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20 p gr/cm²
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FIG 12

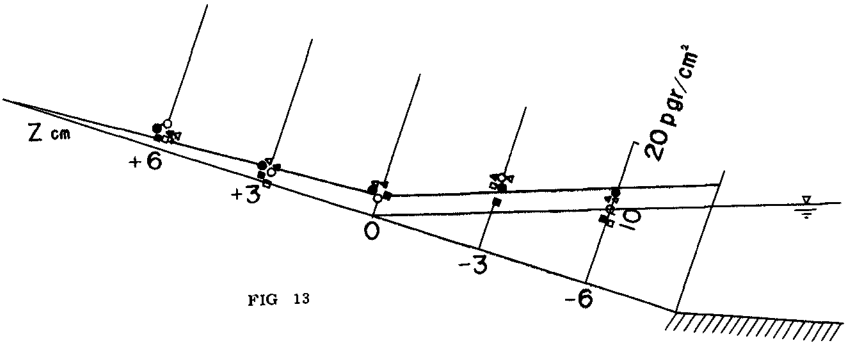

FIG 13

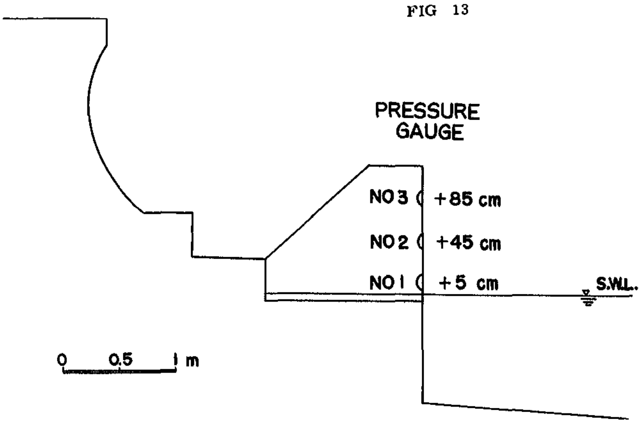

FIG 14 --LOCATION OF WAVE GAUGES (NIIGATA COAST)

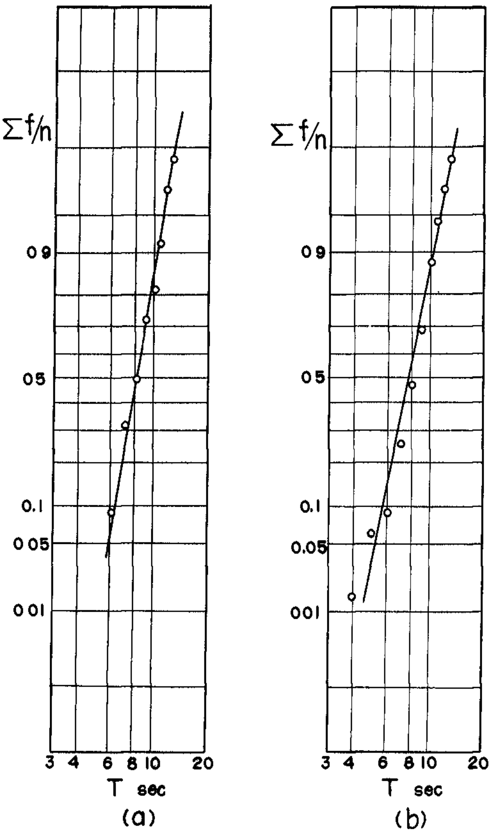

FIG 15 --CUMULATIVE FREQUENCY DISTRIBUTION CURVES OF \ PERIOD shown in Figs.16, 17, 18 and 19. The data of wave pressure and wave characteristics at 12 m water depth are simultaneously recorded for 10 min at 2 hr interval. Considering the above fact the authors reached the conclusion that the mean values of the highest one third of the samples can be used as a statistical measure of wave characteristics and wave pressure intensity.

ANALYSIS OF DATA

In Fig.20 is given one example of the variation of wave characteristics such as the signigicant wave height $H_{1/3}$ and the significant wave period $T_{1/3}$ and of the statistical values of pressure intensity $p_{1/3}$ recorded by Gauges No.1, 2 and 3 (see Fig.14). By using the available data the relationship between the pressure intensity and wave height is obtained as shown in Fig.21. Unfortunately the wave gauges except No.3 have not working well, hence it seems to be difficult to obtain any comprehensive conclusions. The followings are the tentative ones which should be investigated with care by the further observations in field.

1) The pressure intensity reaches its maximum at certain wave height condition which must be determined by the several factors such as the water depth at the foot of structure and the beach slope in front of the structure. Beyond this wave height, the pressure intensity seems to approach gradually to a certain value. The influence of the wave period on the pressure intensity will be the secondary one. The facts mentioned here have been definitely verified in the laboratory.
2) Comparing the asymptotic values of the wave pressure intensity with the ones estimated by using the empirical formula as shown in Fig.22, we recognize that the both values for p_3 have a satisfactory agreement, but the measured value for p_2 is considerably smaller than the expected one. The main reason of the above discrepancy could be found in the followings; (a) the gauge was not in good order, hence the reliability of this data seems to be poor, and (b) the recording system for the field observation can follow only the relatively low frequency phenomena such as 50 cps because the pen recorder was used.

TOTAL WAVE FORCE MEASUREMENT

TEST PROCEDURES

The authors are now doing another series of experimental studies, the aim of which is to determine the absolute value and the acting point of total wave force against sea wall. In this section the preliminary results of newly established experiment will be reviewed briefly in connection to the previous studies. Fig.23 shows the test procedures; that is, the sensing plate (20 cm wide) is supported by two gauge rings, by

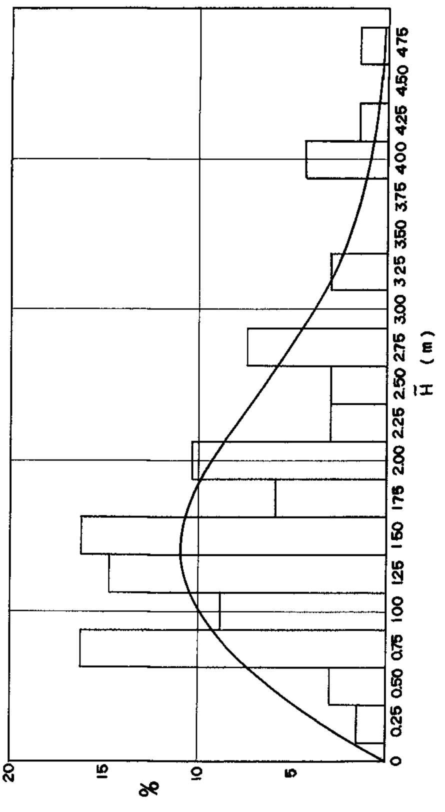

FIG 16 --WAVE HEIGHT DISTRIBUTION

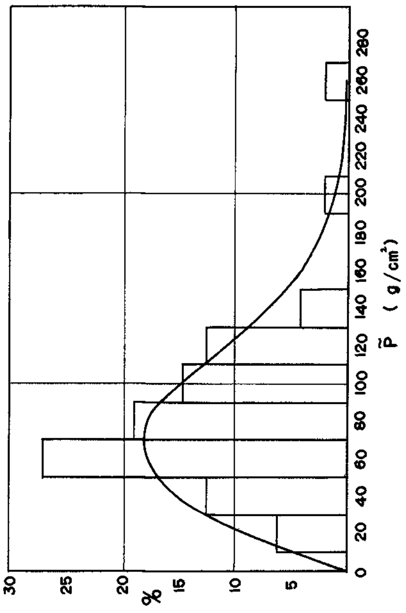

FIG 17 --PRESSURE DISTRIBUTION

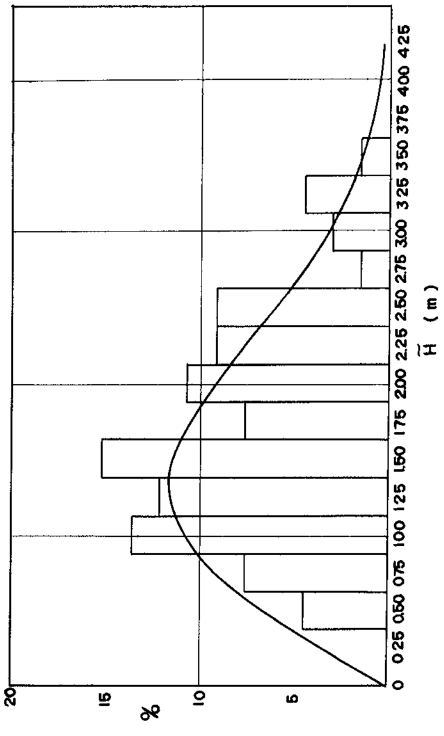

FIG 18 --WAVE HEIGHT DISTRIBUTION

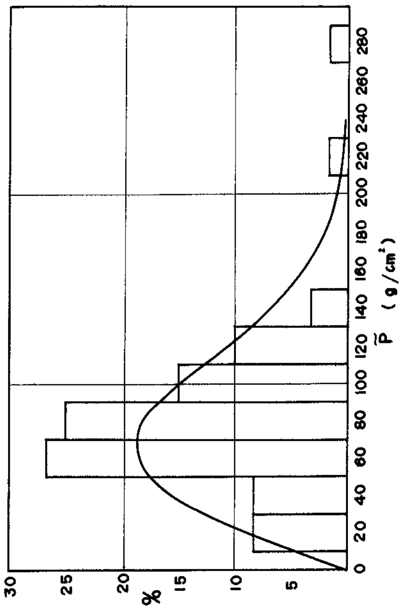

FIG 19 --PRESSURE DISTRIBUTION

Table 1

H cm	T sec	d cm	H /L	d /H	P gr/cm	z * cm	P gr/cm	z * cm
11.9	2.3	6.5	0.015	0.55	125	-2.7	175	-0.8
12.5	2.3		0.015	0.52	119	-2.8		
13.0	2.0		0.021	0.50	196	0.2		
14.0	2.0		0.022	0.47	181	-0.9		
18.3	1.8		0.038	0.36	150	-0.8		
16.3	1.7		0.035	0.40	150	-0.8		
16.1	1.5		0.044	0.40	169	-1.7		
15.9	1.4		0.055	0.41	143	-1.6		
16.7	1.2		0.073	0.39	128	-1.3		
13.8	1.1		0.081	0.47	102	-1.7		
11.6	0.9		0.090	0.56	112	-1.9		

* z, the acting point of total force, is measured upward from still water level.

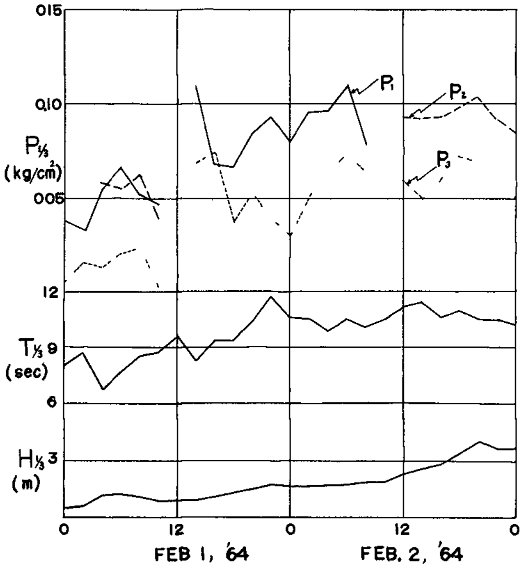

FIG 20 --SAMPLE OF DATA

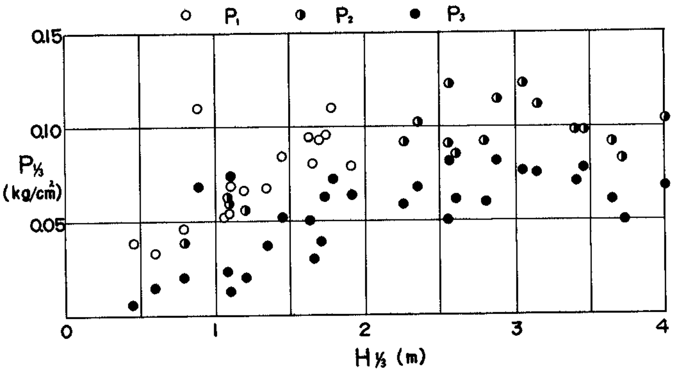

FIG. 21 --PRESSURE VARIATION WITH WAVE HEIGHT

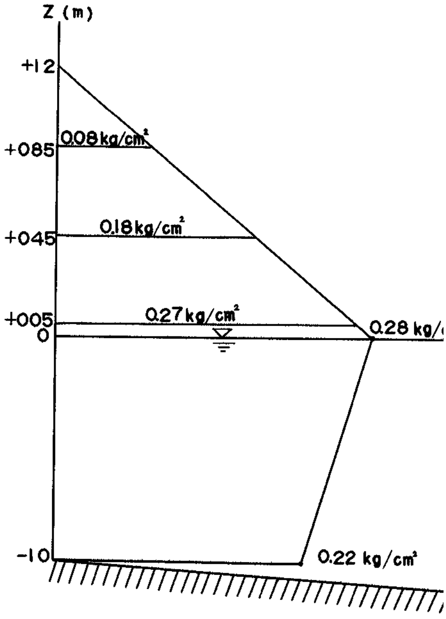

FIG 22 --CALCULATED PRESSURE

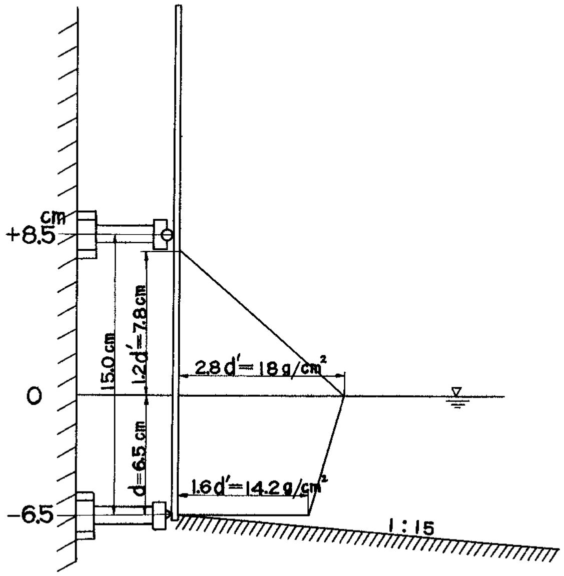

FIG. 23 --TOTAL WAVE FORCE MEASUREMENT

which the fluctuating phenomena of up to 100 cps are measureable.

PRELIMINARY RESULTS

Table 1 gives a part of experimental results, from which it is recognized that the expected conditions of total wave force obtained by using the proposed formula agree fairly well with the measured ones.

ACKNOWLEDGEMENTS

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