

CHAPTER 101

INVESTIGATIONS OF WAVE-PRESSURE FORMULAS DUE TO DAMAGES OF BREAKWATERS

by

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and

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ABSTRACT

The wave-pressure formulas derived in our laboratory have been verified by the investigations of the slide of the vertical walls of composite-type breakwaters due to severe waves during heavy storms which have attacked numerous harbors in Japan since 1959. Some of the examples are shown in this paper.

INTRODUCTION

Formulas to predict the values of the maximum simultaneous pressures exerted by breaking waves and partial breaking waves on the vertical walls of composite-type breakwaters of various shapes have been derived by one of the authors since 1960⁽¹⁾, ⁽²⁾. Those wave-pressure formulas have been proven by model experiments⁽³⁾ of 1/20 and 1/10 scales which were conducted in a large wave tank with a length of 60 m, a width of 2 m, and a depth of 2 m, and partly compared with wave pressures measured at the Harbor of Haboro in the Japan Sea in 1957 and 1958⁽²⁾.

Since 1959 several typhoons and severe storms have hit a number of breakwaters in Japan in the Pacific Ocean and the Japan Sea to cause damages, and most of the damages were the slide of the vertical walls of composite-type breakwaters.

Although there were no measurement of wave pressures exerted by

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storm waves on the vertical walls of the composite-type breakwaters which were slid during the storms, the maximum simultaneous pressures which would have been exerted during the storms on the vertical walls were calculated by the wave-pressure formulas derived in our laboratory, and those maximum pressures calculated could explain the reason of the slide of the vertical walls in most of the breakwaters damaged. Some of the examples including composite-type breakwaters with low and wide rubble-mounds as well as high and narrow rubble-mounds are presented herein.

DESIGN CRITERIA FOR COMPOSITE-TYPE BREAKWATERS

A breakwater is generally designed by the characteristics of the design wave, the topography of the sea bottom, the soil condition of the sea bottom, and the topography of the harbor. The main parts of the breakwater design are the optimum design of the cross-section, determination of the optimum length and orientation of the proposed breakwater. Only the basic problems needed for the optimum design of the cross-section of a composite-type breakwater are described in this paper.

1. Design-Wave Height

What kind of wave heights should be used would be one of the most important problems for the design of breakwaters. The selection of the design-wave height should be different between a composite-type breakwater and a rubble-mound type one. Since a few meters of slide of the vertical wall of a composite-type breakwater generally would threaten the stability of the vertical wall, and the failure of the vertical wall will result in the complete loss of protection, the highest one-tenth-wave height, $H_{1/10}$, should be used for the design of a composite-type breakwater. Even if $H_{1/10}$ is used, and say, $T_{1/10} = 10$ sec, about 14 to 15 higher waves than $H_{1/10}$ would hit the breakwater only for an hour, because the probability of occurrence of waves with larger height than $H_{1/10}$ is about 4 percent. If $H_{1/3}$ is used, and say, $T_{1/3}$ is 10 sec, about 50 waves with larger height than $H_{1/3}$ would attack the breakwater for an hour, and then the breakwater generally could not withstand for more than an hour because it would be very difficult in most of composite-type breakwaters that the factor of safety for the design is taken larger than 1.3 to 1.5.

The design-wave height for a rubble-mound type structure could be

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the significant wave height, $H_{1/3}$, because any failure that may occur due to higher waves in the wave train is progressive and the displacement of some numbers of an individual armor unit will not result in the complete loss of protection.

2. Buoyancy exerted on the Vertical Wall

If H_C defines the height from the design sea level to the crown of the vertical wall, and $\gamma H_{1/10}$ denotes the height from the design sea level to the point at which the maximum simultaneous pressure exerted on the vertical wall becomes zero, the buoyancy should be considered to act on the whole body of the vertical wall when $H_C \leq \gamma H_{1/10}$.

3. Selection of Up-lift Pressures exerted on the Bottom

The up-lift pressures exerted on the bottom of the vertical wall is assumed that the maximum up-lift pressure, $(p_u)_{\max}$, is exerted on the seaward-side edge of the bottom and the intensity of p_u diminishes linearly towards the harbor-side edge of the bottom to become zero there, i. e. the up-lift pressures on the bottom of the vertical wall distribute triangularly.

The value of $(p_u)_{\max}$ is assumed as follows;

- (a) When the cover-concrete-blocks or cover-stones placed at the seaward-side and in front of the vertical wall are not dislocated by waves, $(p_u)_{\max} = 1.0$ to 1.5 t/m^2 .
- (b) When the cover-concrete-blocks or cover-stones are dislocated by waves, $(p_u)_{\max} = 2$ to 3 t/m^2 .

According to the results of the experiments⁽⁴⁾ and our experiences in prototype, the up-lift pressures distribute triangularly on the bottom of the vertical wall of composite-type breakwater in most of the breakwaters, and the values of $(p_u)_{\max}$ depend mainly upon the cover-blocks or cover-stones placed at the seaward-side and in front of the vertical wall, with no direct relation to the intensities of the horizontal wave pressures exerted near the bottom of the vertical wall. Therefore, in the design of a breakwater the careful placement of the sufficiently large cover-concrete blocks or cover-stones is especially required to reduce the uplift pressures exerted on the bottom of the vertical wall.

4. Selection of the Coefficient of Friction between the Vertical Wall and the Rubble-Mound

The value of the coefficient of friction, f , between the vertical

wall and rubble-mound of composite-type breakwater is influenced mainly by the thickness and width of the cover-concrete blocks or cover-stones placed at the harbor-side and sea-side of the vertical wall. It may be decisively stated that the main factors to insure the stability against slide of the vertical wall of a composite-type breakwater are the dead weight and width of the vertical wall and the cover-layers placed at the both sides of the vertical wall.

According to the results of the experiments⁽⁴⁾ and our experiences in prototype, the critical values of the coefficient of friction, f_{cr} , when the vertical wall is about to be slid due to waves should be taken as follows;

(a) When the vertical wall of composite-type breakwater is completed one to two years ago, and the cover-concrete-blocks or cover-stones were dislocated during a storm, $f_{cr} = 0.65$ to 0.70 .

(b) When it passes more than about two years after completion of the vertical wall of a breakwater, and the cover-blocks or cover-stones were dislocated during a storm, $f_{cr} = 0.70$ to 0.75 .

(c) When it passes more than about two years after completion of the vertical wall, and the cover-blocks or cover-stones were not dislocated during a storm, $f_{cr} = 0.80$ to 0.90 .

The value of the coefficient of friction to be used for the design of the breakwaters must be taken to be smaller than the critical values above mentioned, f_{cr} . It is a long-experienced general rule in Japan that f is taken 0.60 for the design.

5. Wave-Pressure Formulas and Their Regions Applicable

The wave-pressure formulas derived in our laboratory for composite-type breakwaters can be divided into two scopes; one group is available to composite-type breakwaters with high rubble-mounds, the criteria for which is the ratio of h_1 and h_2 , h_1/h_2 , to be smaller than 0.40 to 0.50 , and the other group is applicable for those with low rubble-mounds, the criteria for which is 0.40 to $0.50 \leq h_1/h_2 < 0.75$, in which h_1 defines water depth above the top of the rubble-mound and h_2 denotes water depth at the toe of the sea-side slope of the rubble-mound. The formulas belonging to the first group were presented in Ref.(1) and those to the second group were described in Ref.(2).

When the rubble-mounds of composite-type breakwaters are so low that $h_1/h_2 \geq 0.75$, and $h_2/H \geq 1.80$, in which H defines the design wave

height, standing waves are always formed in front of the breakwaters, regardless of the top-width of the rubble-mound, B. The pressures exerted by perfect and partial standing waves on vertical walls can be obtained with sufficient accuracy for design purposes by the use of formulas (5), (6)

VERIFICATION OF WAVE-PRESSURE FORMULAS DUE TO DAMAGES OF BREAKWATERS

1. The Port of Kashima

This port, the plan of which is shown in Fig. 1, is one of the biggest industrial ports in Japan, and was recently built on a long straight sandy coast directly exposed to the Pacific Ocean.

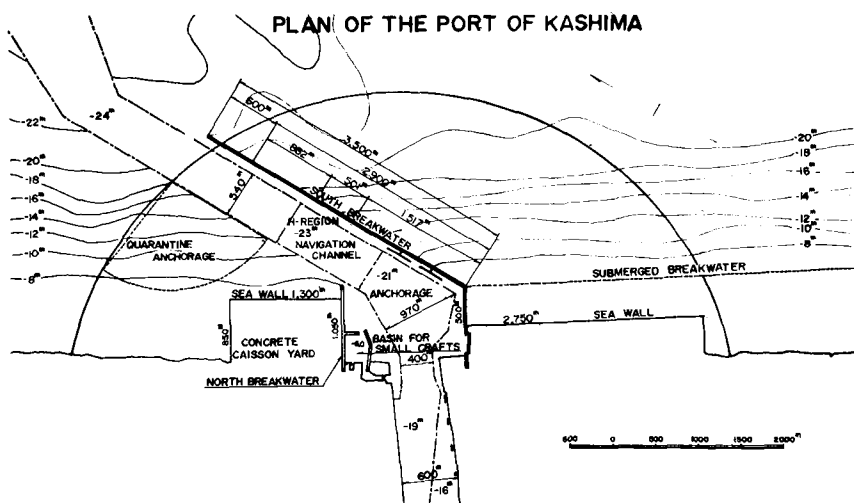


Fig. 1. - THE PLAN OF THE PORT OF KASHIMA

2,900 m in length of the South Breakwater located from a depth of water about 9 m below the Datum Line (D.L.) to a water depth of about 20 m below D.L. was severely hit by storm waves of January in 1970, the wave characteristics during which were hindcast $H_{1/10} = 10$ m and $T_{1/10} = 10$ sec to 12 sec, and the vertical wall of reinforced concrete caisson, shown in Fig. 2, were slid by 0.24 m to 1.81 m. Since the water depths during the storm, $h_1 = 9.9$ m, $h_2 = 16.9$ m, and $h = 18$ m, which defines a

water depth about three wave lengths offshore from the breakwater, at H.W.L. (D.L. + 1.40 m),

$$h_1/h_2 = 0.59, h/L = 18/117 = 0.15 \text{ to } 18/146 = 0.12, h_2/H = 16.9/10 = 1.7, h_1/H = 1.0, \text{ and } H/L = 0.085 \text{ to } 0.068,$$

it is known that the waves broke in front of the vertical wall of the breakwater, and the maximum simultaneous pressures exerted by the breaking waves is obtained by the formula⁽²⁾

$$\begin{aligned} P &= \alpha w_0 H \left(h_1 \frac{\tanh \beta}{\beta} + \frac{1}{2} \gamma H \right) \\ &= 1.7 \times 1.03 \times 10 \left(9.9 \times \frac{\tanh 1.0}{1.0} + \frac{1}{2} \times 0.83 \times 10 \right) \\ &= 204 \text{ t/m.} \end{aligned} \quad \left. \vphantom{\begin{aligned} P &= \alpha w_0 H \left(h_1 \frac{\tanh \beta}{\beta} + \frac{1}{2} \gamma H \right) \\ &= 1.7 \times 1.03 \times 10 \left(9.9 \times \frac{\tanh 1.0}{1.0} + \frac{1}{2} \times 0.83 \times 10 \right) \\ &= 204 \text{ t/m.} \end{aligned}} \right\} (1)$$

SOUTH BREAKWATER, PORT OF KASHIMA H-SECTION

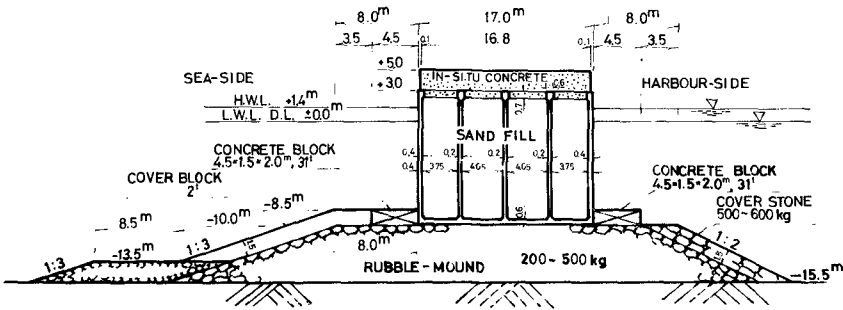


Fig. 2. - CROSS-SECTION OF H-REGION, KASHIMA HARBOR

Subtracting the pressure acting above the crown level of the vertical wall (D.L. + 5.00 m), the net resultant pressure is

$$P = 204 - 23.3 = 181 \text{ t/m.} \quad (2)$$

Since the height from the H.W.L. (D.L. + 1.40 m) to the crown level of the vertical wall (D.L. + 5.00 m), H_c , is 3.60 m, and H_c is much smaller than $\gamma H = 0.83 \times 10 = 8.3$ m, a large amount of wave overtopping is estimated to be caused when the waves hit the breakwater. Therefore, it may be adequate to consider that the breakwater was submerged into water when the waves hit. The dead weight of the vertical wall when submerged

into water is 275 t/m. In view of the fact that the cover concrete blocks placed in front of and at the foot of the vertical wall were dislocated by the waves, the uplift pressure at the sea-side edge of the bottom of the vertical wall, p_u , may be estimated 3 t/m², and the resultant of the uplift pressures on the bottom of the vertical wall is

$$P_u = \frac{1}{2} \times 3 \times 17 = 25.5 \text{ t/m.} \quad (3)$$

Since the breakwater was completed only one year prior to the storm, the critical value of the coefficient of friction, f_{cr} , would be estimated from 0.65 to 0.70, and the resisting force of the vertical wall against slide would be at most

$$\begin{aligned} R &= 0.70 (275 - 25.5) = 175 \text{ t/m, or} \\ R &= 163 \text{ t/m for } f_{cr} = 0.65. \end{aligned} \quad \} (4)$$

The calculation shown above proves that the vertical wall should be slid due to the pressures of the waves, because P is 3 to 10 percent larger than R .

Experiments were performed to know the maximum simultaneous pressures exerted on the vertical wall of the breakwater by various kinds of breaking waves with heights of about 9 m to 12 m and periods of 8 sec to 14 sec by the use of a 1/25-model in a wave channel with a wind blower, 100 m long, 2 m deep, and 1.2 m wide.

The results of the experiment showed that the maximum resultant $P_e = 193$ t/m of the maximum simultaneous pressures was exerted by a wave with $H = 9.6$ m and $T = 8$ sec when the sea level was D.L. ± 0 m, as shown in Fig. 3, and $P_e = 140$ t/m by the same wave when the sea level was D.L. + 1.40 m.

The resultants of the maximum simultaneous pressures exerted by the breaker of the same wave are obtained by the formula as follows:

For the sea level of D.L. ± 0 m, $\alpha = 2.4$, $\beta = 1.36$, and $\gamma = 0.65$, hence

$$\begin{aligned} P_{cal} &= 2.4 \times 1.03 \times 9.6 \left(8.5 \times \frac{\tanh 1.36}{1.36} + \frac{1}{2} \times 0.65 \times 9.6 \right) \\ &= 204 \text{ t/m,} \end{aligned} \quad (5)$$

and for the sea level of D.L. + 1.40 m, $\alpha = 1.5$, $\beta = 0.90$, and $\gamma = 0.88$, hence

$$\begin{aligned} P_{cal} &= 1.5 \times 1.03 \times 9.6 \left(9.9 \times \frac{\tanh 0.90}{0.90} + \frac{1}{2} \times 0.88 \times 9.6 \right) \\ &= 180 \text{ t/m.} \end{aligned} \quad (6)$$

Subtracting the pressures acting above the crown of the vertical wall from the resultants, $P_{cal} = 202 \text{ t/m}$ for the sea level of D.L. $\pm 0 \text{ m}$, which corresponds to $P_e = 193 \text{ t/m}$, and for the sea level of D.L. $+ 1.40 \text{ m}$ $P_{cal} = 157 \text{ t/m}$, which corresponds to $P_e = 140 \text{ t/m}$. The experimental and calculated values of the resultant of the maximum simultaneous pressures, $P_e = 193 \text{ t/m}$ and $P_{cal} = 202 \text{ t/m}$, may be said to be in a fairly good agreement, and about 10 to 16 percent larger than $R = 175 \text{ t/m}$.

If Hiroi's formula is used to obtain the maximum resultant pressure,

$$P = 1.5 w_0 H_1 / 10 (h_1 + H_C) = 1.5 \times 1.03 \times 10 (9.9 + 3.6) = 209 \text{ t/m.} \quad (7)$$

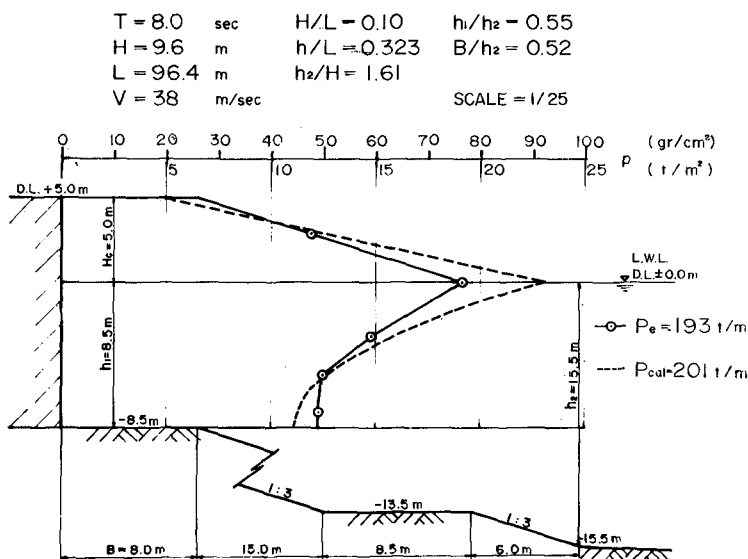


Fig. 3. - P_e AND P_{cal} -CURVES OF H-SECTION, KASHIMA HARBOR

This value is very close to P of Eq. 1, and this case proves that Hiroi's formula is in some cases to be adequate to be used for obtaining the maximum resultant pressure when there is a large overtopping of waves over the vertical wall of composite-type breakwater, as it has been proven in prototype and experiments in Japan⁽¹⁾.

Minikin's formula is quite inapplicable to such a case, showing $P_{cal} = 529$ to 437 t/m at H.W.L., and $P_{cal} = 464$ to 383 t/m at L.W.L. for the same waves with $H_{1/10} = 10$ m, $T_{1/10} = 10$ sec to 12 sec.

2. The Port of Hachinohe

A part, 318 m long, of the 1,400 m long breakwater which is located at depths about 6 m to 9 m below L.W.L. (D.L. + 0.30 m), as shown in Fig. 4, was severely damaged and slid 6 m at maximum by storm waves of January, 1971. The slid part of the breakwater is located at a water depth of about 8.5 m below L.W.L., as shown in Fig. 5, and the storm waves offshore from the breakwater were hindcast $H_{1/10} = 7.8$ m to 8.0 m and $T_{1/10} = 8$ sec to 12 sec from wave data recorded at a water depth of 10 m in the vicinity of the breakwater during the storm.

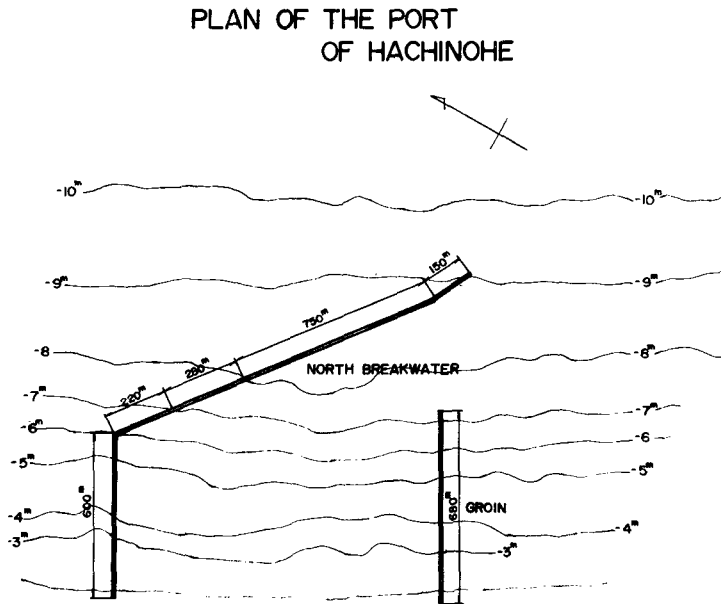


Fig. 4. - PLAN OF THE PORT OF HACHINOHE

wall will be

$$P_u = \frac{1}{2} \times 3.0 \times 16 = 24 \text{ t/m.} \quad (9)$$

Since the value of f_{cr} may be assumed 0.65 to 0.70 from the lapse of only about two years after completion of the breakwater, the resisting force of the vertical wall against slide would be at most

$$R = 0.7 (176 - 24) \approx 106 \text{ t/m.} \quad (10)$$

The results of the calculation of Eqs. 8 and 10 that the value of P is 32 percent larger than R may be said to prove a large amount of slide of the vertical wall.

In order to investigate the maximum simultaneous pressures which would be exerted by the highest waves that are possible to be generated offshore the breakwater at the sea level of L.W.L., experiments were conducted by using a 1/25-model in the 100-meter long wave channel. The maximum resultant pressures, P_e , measured in the experiments were as follows:

At the tidal level of L.W.L. (D.L. + 0.3 m)

$$H = 8.3 \text{ m, } T = 8 \text{ sec, } P_e = 147 \text{ t/m, } P_{cal} = 144 \text{ t/m,}$$

$$H = 7.1 \text{ m, } T = 9 \text{ sec, } P_e = 103 \text{ t/m, } P_{cal} = 116 \text{ t/m,}$$

$$H = 8.2 \text{ m, } T = 9 \text{ sec, } P_e = 165 \text{ t/m, } P_{cal} = 142 \text{ t/m,}$$

$$H = 7.4 \text{ m, } T = 10 \text{ sec, } P_e = 108 \text{ t/m, } P_{cal} = 113 \text{ t/m,}$$

$$H = 8.3 \text{ m, } T = 10 \text{ sec, } P_e = 139 \text{ t/m, } P_{cal} = 142 \text{ t/m.}$$

The results of the experiments showed:

(1) Very large waves with $H \approx 8 \text{ m}$ to 8.3 m and $T = 8 \text{ sec}$ to 10 sec , the steepnesses of which were as steep as 0.077 to 0.10, could be generated within at least several wave-lengths offshore from the breakwater due to the superposition of large reflecting waves from the breakwater on incoming waves.

(2) Those very steep waves severely broke against the breakwater, exerting large resultants of pressures of about 140 t/m to 165 t/m on the vertical wall of the breakwater.

The values of the resultants of the maximum simultaneous pressures calculated and measured in the experiments, P_{cal} and P_e , are in a good agreement, and they may be stated to prove well the severe damages of the breakwater.

The wave-records measured at a water depth of 10 m below L.W.L. offshore from a breakwater located at the east district of the Harbor of

Hachinohe showed $H_{1/3} = 6.05$ m and $T_{1/3} = 11.5$ sec, although the wave recorder did not act during the severest hours of the storm. Therefore, it would be estimated that the $H_{1/10}$ during the severest hours of the storm was larger than approximately 7.8 m, and the $H_{\max}$ was about 10 m which was nearly equal to the depth of water where the wave recorder was located. As was mentioned above, such extremely high and steep waves were also generated offshore from the breakwater in the 1/25-scale model experiment.

If Hiroi's formula is used, $P = 98.9$ t/m for the same wave, which is a little smaller than R. Minikin's formula is quite inapplicable to such a case.

3. The Port of Himeji

The Port of Himeji is situated about 60 km west of the Port of Kobe, and located on the northern coast of the Seto Inland Sea. This harbor was hit on the 24th of August and 25th of September in 1964 by typhoons, and especially the latter one caused damages to the breakwaters of the harbor.

A wave recorder of pressure-type set on the bottom of the sea with a water depth of 12 m below D.L. ± 0 m recorded well waves during the August-24th typhoon, and the significant wave was $H_{1/3} = 3.50$ m and $T_{1/3} = 6.1$ sec. The ratios of $H_{\max}/H_{1/3}$ and $H_{1/10}/H_{1/3}$ were 1.43 and 1.29, respectively. The significant wave at the site of the wave recorder estimated by SMB-method from the winds measured during the typhoon was $H_{1/3} = 3.2$ m and $T_{1/3} = 6.6$ sec.

Although the wave recorder did not act well during the September-25th typhoon, the waves during the typhoon could be estimated sufficiently by SMB-method from the winds measured during the typhoon and by the use of the wave data obtained during the August-24th typhoon as mentioned above. The waves thus estimated at the site of the wave recorder were as follows:

$$H_{1/3} = 3.50 \times \frac{3.50}{3.20} = 3.80 \text{ m}, \quad T_{1/3} = 7.0 \text{ sec},$$

$$H_{1/10} = 1.29 \times 3.8 = 4.90 \text{ m} \approx 5.0 \text{ m}, \quad T_{1/10} = 7.5 \text{ sec}.$$

The breakwater, the cross-section of which is shown in Fig. 6, was under construction when the typhoon hit the harbor, and some portion of the breakwater was not completed, being left the parapet-wall from D.L. + 2.50 m to D.L. + 4.00 m to be constructed, and the other portion of

the breakwater was just completed up to the crown of the parapet-wall of D.L. + 4.00 m.

WEST BREAKWATER, MEGA HARBOUR, PORT OF HIMEJI

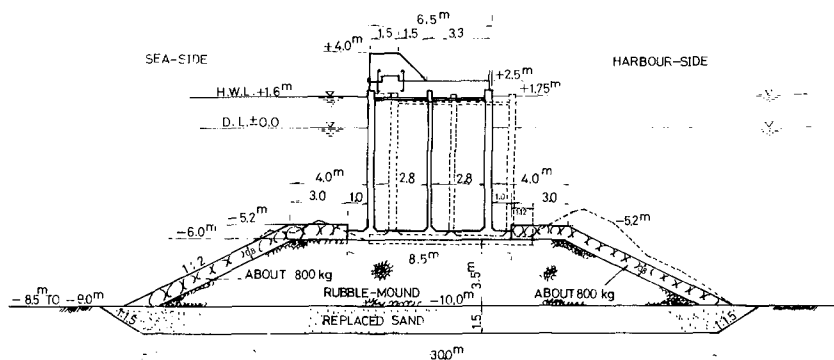


Fig. 6. - CROSS-SECTION OF THE BREAKWATER, HIMEJI HARBOR

The completed part of the breakwater could withstand with no damages against the waves, but the uncompleted portion was slid about one meter toward the harbor-side, as shown in Fig. 6 by a broken line, and somewhat subsided.

Although the highest tidal level during the typhoon was D.L. + 2.00 m, it was estimated that the largest wave pressures during the typhoon would have been exerted on the breakwaters at a tidal level of about D.L. $\pm$ 0 m, because the crown levels of the uncompleted and the completed breakwaters were so low as D.L. + 2.50 m and D.L. + 4.00 m, respectively, against $H_{1/10} = 5.0$ m.

Since the depth of water, h , at the tidal level was about 10 m at the sea about three times of the length of the incoming wave with a period of $T_{1/10} = 7.5$ sec from the breakwater, the steepness of the incoming wave is $H_{1/10}/L_{1/10} = 5.0/65 = 0.078$. The deep-water wave was hindcast by SMB-method to be $T_0 = 7.8$ sec and $L_0 = 95$ m, and then the height of the deep-water wave was calculated $H_0 = 5.4$ m from $H/H_0 = 0.93$ for $h/L_0 = 10/95 = 0.11$. The depth of water at the breaking point of the incoming wave, h_b , was obtained about 7.3 m from $h_b/H_0 = 1.35$ on an

average for $H_0/L_0 \approx 0.058$.

Judging from the ratios of $h_1/h_b = 5.2/7.3 = 0.70$, $h_2/h_b = 8.5/7.3 \approx 1.2$ to $9.5/7.3 = 1.3$, and $H_{1/10}/L_{1/10} = 0.078$, the incoming wave may decisively be estimated to have broken on the rubble-mounds of the breakwaters. Therefore, the resultant of the maximum simultaneous pressures⁽²⁾ exerted on the vertical walls by the breaking wave, P , is obtained as follows:

From the values of $h_2/H_{1/10} = 8.5/5.0 = 1.7$, $B/h_2 = 4.0/8.5 = 0.47$, $h_1/h_2 = 0.55$, $\alpha = 1.5$ is obtained⁽²⁾, then $\beta = 1.1$ and $\gamma = 0.88$ are obtained. Finally P is obtained by

$$P = 1.5 \times 1.03 \times 5.0 \left(5.2 \times \frac{\tanh 1.1}{1.1} + \frac{1}{2} \times 0.88 \times 5.0 \right) \\ = 7.73 (3.78 + 2.20) = 46.23 \text{ t/m.} \quad (11)$$

Subtracting the pressures acting above the crown level of the breakwater (D.L. + 2.50 m), the net resultant of the maximum pressures which would have acted on the vertical wall of the uncompleted breakwater is

$$P = 46.23 - 3.17 \approx 43.06 \approx 43.1 \text{ t/m.} \quad (12)$$

Judging from the fact that the breakwater was under construction when the typhoon hit the harbor, and the thickness of the cover layer of rubble at the seaward-side foot of the vertical wall was only 0.80 m without any cover concrete block, the up-lift pressure at the seaward-side edge of the bottom of the vertical wall may be estimated at least $p_u = 3 \text{ t/m}^2$, and the resultant of the up-lift pressure on the bottom is

$$P_u = \frac{1}{2} \times 3.0 \times 8.5 = 12.8 \text{ t/m.} \quad (13)$$

The critical value of the coefficient of friction against slide between the vertical wall and the rubble-mound may be estimated $f_{cr} = 0.70$ at most, because the breakwater was under construction and the thickness of the cover layer of rubble was only 0.80 m. Since $H_c = 2.50 \text{ m}$ was smaller than $\gamma H_{1/10} = 0.88 \times 5.0 = 4.4 \text{ m}$, the breakwater should be considered to have been completely under water when the $H_{1/10}$ -wave hit it. Therefore, the resisting force of the vertical wall against slide would be at most

$$R = 0.70 (62.1 - 12.8) = 34.5 \text{ t/m,} \quad (14)$$

in which 62.1 t/m denotes the dead weight in water of the vertical wall of the uncompleted breakwater.

The result of the calculation that the value of P is about 25 per-cent larger than R may be said to prove the slide of the vertical wall.

The resisting force of the vertical wall of the completed break-water against slide would be

$$R = 0.70 (79.5 - 12.8) = 46.7 \text{ t/m}, \quad (15)$$

in which 79.5 t/m denotes the dead weight in water of the vertical wall of the completed breakwater.

Since $\gamma H_1/10 = 4.4 \text{ m}$ is a little larger than $H_c = 4.0 \text{ m}$ for the completed breakwater, decrease in the maximum simultaneous pressures due to overtopping is 0.14 t/m, hence the net resultant pressure is

$$P = 46.23 - 0.14 = 46.09 \approx 46.1 \text{ t/m}. \quad (16)$$

From Eqs. 15 and 16, the value of R is a little bit larger than P . This may be stated to show that the completed portion of the breakwater would have been near the critical state of the stability of the vertical wall.

Although Minikin's formula is inapplicable to this case, if it is used for reference, the resultant of the maximum wave pressures is

$$\begin{aligned} P &= \frac{1}{3} P_{\max} H + \frac{1}{2} w_0 H \left(\frac{H}{4} + h_1 \right) \\ &= \frac{1}{3} \{ 102.4 \times 5.2 \left(1 + \frac{5.2}{9.5} \right) \times 0.078 \} \times 5.0 + \frac{1}{2} \times 1.03 \times 5 \\ &\quad \times \left(\frac{5}{4} + 5.2 \right) \\ &= 106 + 16.6 = 122.6 \text{ t/m}. \end{aligned} \quad (17)$$

(Actually such a difference of hydrostatic pressure between the seaward-side and harbor-side as 16.6 t/m does not exist.)

Since the value of P is much larger than $R = 34.5 \text{ t/m}$ and 46.7 t/m , both the uncompleted and the completed breakwaters would have been slid to a comparatively large distance toward the harbor-side. This is contrary to the fact.

If Hiroi's formula is used, the resultant of the wave pressures on the completed breakwater is

$$P = 1.5 \times 1.03 \times 5.0 \times 9.2 = 71 \text{ t/m}. \quad (18)$$

Since the value of P is larger than $R = 46.7 \text{ t/m}$, the completed break-water must be slid by the wave of $H_1/10 = 5 \text{ m}$. This is also contrary to the fact.

4. The Port of Kada

This harbor is a small commercial and fishery harbor located on the northernmost eastside coast of the Kii Channel which connects with the

Pacific Ocean at the southern end, and protected by small islands from waves coming from the west and by a cape from waves coming from the south. Therefore, a small breakwater of about 67 m in length was constructed to protect the harbor from waves coming in by diffracting the cape from the south west.

This breakwater, which has two kinds of cross-section, as shown in Fig. 7, was hit by storm waves during Isewan Typhoon which was one of the biggest typhoons passing over the Japanese Archipelago and passed about 100 km south of harbor on the 26th of September in 1959.

The middle portion of about 40 m in length of the breakwater, the cross-section of which is shown as B-B section in Fig. 7, was slid toward the harbor-side from 4 cm to 30 cm and subsided from 4 cm to 13 cm at the crown, but the head portion of the breakwater, the cross-section of which is shown as A-A section in Fig. 7, was not slid.

The tidal level of the sea when the waves were the severest during the typhoon was estimated D.L. + 2.10 m. Since the sea-side feet of the vertical walls of the A-A and B-B sections were well covered by a concrete block and rubbles, the up-lift pressures acting on the sea-side edges of the bottoms of the vertical walls are assumed $p_u = 1.0 \text{ t/m}^2$ to 1.5 t/m^2 .

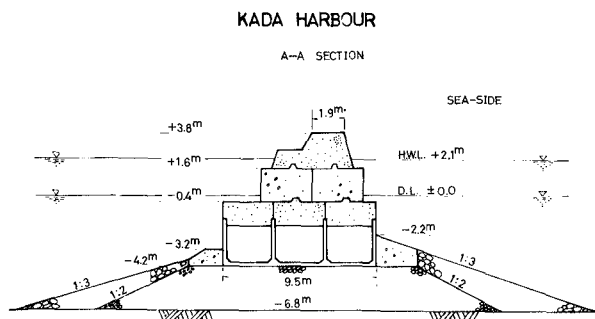


Fig. 7. - CROSS-SECTION OF THE BREAKWATER, KADA HARBOR
(A-A section)

KADA HARBOUR

B-B SECTION

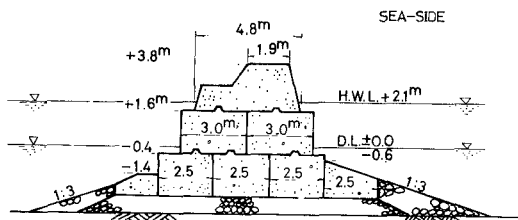


Fig. 7. - CROSS-SECTION OF THE BREAKWATER, KADA HARBOR
(B-B section)

Judging from the fact that the breakwater was constructed more than two years before the typhoon hit it, and the cover-concrete blocks and rubbles placed at the seaward sides of the vertical walls were not dislocated by the waves during the storm, the critical coefficient of friction may assumed $f_{cr} = 0.90$. Therefore, the resisting forces against slide due to the waves would be assumed for the A-A section

$$\left. \begin{aligned} R_A &= 0.90 (63.75 - 4.5) = 53.3 \text{ t/m for } p_u = 1.0 \text{ t/m}^2, \\ \text{and } R_A &= 51.3 \text{ t/m for } p_u = 1.5 \text{ t/m}^2, \\ \text{and for the B-B section} \end{aligned} \right\} (19)$$

$$\left. \begin{aligned} R_B &= 0.90 (49.71 - 3.75) = 41.4 \text{ t/m for } p_u = 1.0 \text{ t/m}^2, \\ \text{and } R_B &= 39.7 \text{ t/m for } p_u = 1.5 \text{ t/m}^2, \end{aligned} \right\} (20)$$

in which the values of 63.75 t/m and 49.71 t/m are the dead weights in water of the A-A and B-B sections, respectively.

The $H_{1/10}$ in Kada Harbor during the severest waves of the typhoon was estimated about 4 m to 4.5 m and $T_{1/10}$ from 10 sec to 12 sec, from the wave data recorded during the typhoon in the Port of Wakayama which is located several kilometers south of the Kada Harbor.

Judging from the fact that the depths of water in front of the vertical walls of the B-B and A-A sections were $h_1 = 2.7$ m and 4.3 m, which were smaller than or nearly equal to the estimated $H_{1/10} = 4.0$ m to 4.5 m, and the seaward slopes of the rubble-mounds of the breakwaters were equally 1:3, and moreover, the rubble-mounds did not have a flat crown in front of the vertical walls, i.e., $B = 0$, it is decisively estimated

that the waves would have broken in a state of plunging breaker just in front of the vertical walls and the vertical distribution of the maximum simultaneous pressures exerted by the breaking waves was the C-type⁽¹⁾. The maximum pressure of the breaking waves is estimated $p_{\max} = 20 \text{ t/m}^2$ to 26 t/m^2 from the values of $h_1 \frac{h_1}{h} \frac{H}{L} \approx 0.040$ to 0.12 , but $p_{\max} = 20 \text{ t/m}^2$ will be adequate for the breakwater which is well protected from severe waves coming in from the Kii Channel⁽¹⁾. Therefore, the resultant of the maximum simultaneous pressures is obtained by

$$P = \frac{1}{2} p_{\max} H_{1/10} = \frac{1}{2} \times 20 \times 4.5 = 45 \text{ t/m}. \quad (21)$$

It may be understood from Eqs. 19 to 21 that the B-B section of the breakwater would be slid only by a short distance by the wave of $H = 4.5 \text{ m}$ and $T = 10 \text{ sec}$ to 12 sec , but that the A-A section, i.e. the breakwater head, would not be slid by the wave.

If Hiroi's formula is used, the limiting wave heights required for the slide of the breakwater are $H = 5.53 \text{ m}$ to 5.75 m for the A-A section, while $H = 5.83 \text{ m}$ to 6.08 m for the B-B section. This means that the A-A section would be slid by smaller waves in height than those which can slide the B-B section. This is quite contrary to the fact.

If Minikin's formula is used, $p_{\max}$ for the A-A section is obtained by

$$p_{\max} = 102.4 \times 4.3 \left(1 + \frac{4.3}{8.9} \right) \frac{4.5}{84} = 35.0 \text{ t/m}^2, \quad (22)$$

and the resultant of the maximum pressure exerted by the wave of 4.5 m in height is

$$\begin{aligned} P &= \frac{1}{3} \times 35.0 \times 4.5 + 1.03 \times \frac{4.5}{2} \left(4.3 + \frac{4.5}{4} \right) \\ &= 52.6 + 5.1 = 57.7 \text{ t/m}, \end{aligned} \quad (23)$$

which is larger than $R_A = 53.3 \text{ t/m}$ of Eq. 19. Therefore, the breakwater head must be slid by the wave. This is contrary to the fact.

If $H_{1/10}$ is estimated 4.0 m , $p_{\max} = 31.2 \text{ t/m}^2$, and $P = 41.6 + 10.9 = 52.5 \text{ t/m}$. This value of P is a little bit smaller than $R_A = 53.3 \text{ t/m}$, and about 27 percent larger than $R_B = 41.4 \text{ t/m}$. This may mean that according to Minikin's formula the B-B section must be slid at a comparatively large distance toward the harbor-side and the A-A section was at a critical state of stability when the wave of $H_{1/10} = 4.0 \text{ m}$ attacked the breakwater. This is a little different from the fact.

CONCLUSIONS

The optimum design of the cross-section of a breakwater is to design such a cross-section as to meet the requirements of the breakwater at the proposed construction site at a minimum of costs. The costs include those for design and construction, and the capitalized costs of repairs of estimated damages due to waves. In general, since a harbor has been requiring a larger and deeper water basin, the cross-section of the breakwater which protects the water basin from the design wave has been becoming larger, and more important and expensive. Therefore, it may be stated common practice through the world for the design of such a breakwater to conduct a three-dimensional model experiment to study the optimum length and orientation of the proposed breakwater, and moreover, a two-dimensional stability model study to determine the optimum cross-section of the breakwater. At the same time the optimum design of the cross-section of a projected breakwater is especially required to know the construction cost of the breakwater at the first stage of the planning of a harbor and to verify the result of a two-dimensional model study.

The optimum design of the cross-section of a projected composite-type breakwater requires the selections of the values or formulas of the five design factors at least mentioned in this paper. Although those values of the design factors have been studied by using numerous model experiments of different scales of 1/10 to 1/25 during more than twenty years in our laboratory, such investigations have also been carried out for practical design purposes by the use of damages of composite-type breakwaters which have been suffered with severe waves during typhoons hitting the Japanese Archipelago since 1959.

It may therefore be stated from those investigations that the values of the design factors and the wave-pressure formulas presented in this paper would be applicable with sufficient reliability for the optimum design of the cross-section of a composite-type breakwater.

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