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/m^2, and the resultant of the uplift pressures on the bottom of the vertical wall is

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

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

$$\left.\begin{array}{l} R = 0.70 \; (\; 275 - 25.5 \;) = 175 \text{ t/m, or} \\ R = 163 \text{ t/m for } f_{cr} = 0.65. \end{array}\right\} \tag{4}$$

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. $\pm$ 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. $\pm$ 0 m, α = 2.4, β = 1.36, and γ = 0.65, hence

$$P_{cal} = 2.4 \times 1.03 \times 9.6 \; (\; 8.5 \times \frac{\tanh 1.36}{1.36} + \frac{1}{2} \times 0.65 \times 9.6 \;)$$
$$= 204 \text{ t/m,} \tag{5}$$

and for the sea level of D.L. + 1.40 m, α = 1.5, β = 0.90, and γ = 0.88, hence

$$P_{cal} = 1.5 \times 1.03 \times 9.6 \; (\; 9.9 \times \frac{\tanh 0.90}{0.90} + \frac{1}{2} \times 0.88 \times 9.6 \;)$$
$$= 180 \text{ t/m.} \tag{6}$$

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

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

$$P = 1.5\ w_0\ H_{1/10}\ (\ h_1 + H_c\ ) = 1.5 \times 1.03 \times 10\ (\ 9.9 + 3.6\ ) = 209 \text{ t/m.} \qquad (7)$$

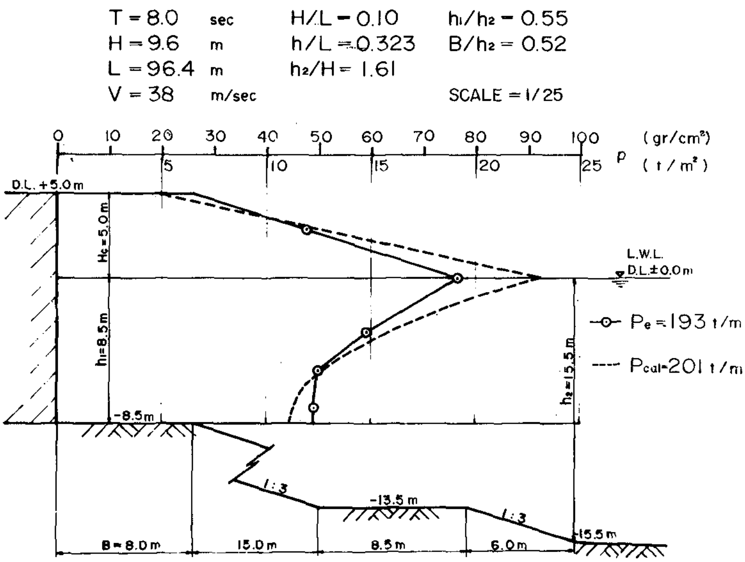

Fig. 3. - P_e AND P_{cal}-CURVES OF H-SECTION, KASHIMA HARBOR

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.

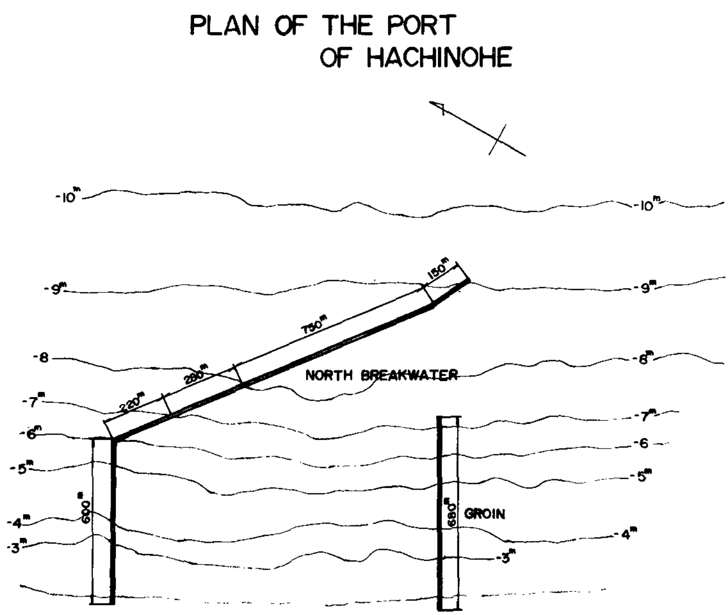

Fig. 4. - PLAN OF THE PORT OF HACHINOHE

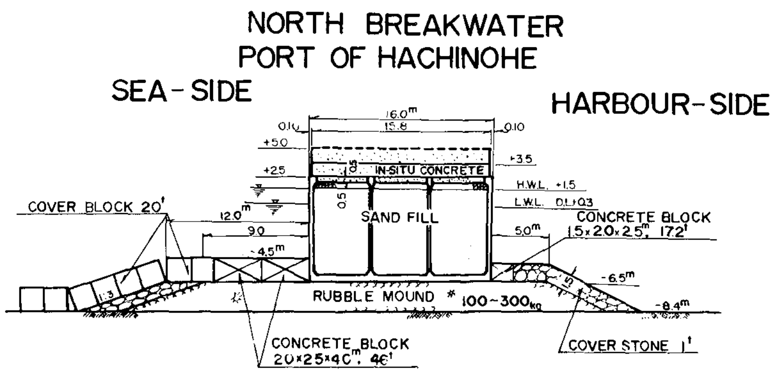

Fig. 5. - CROSS-SECTION OF THE BREAKWATER, HACHINOHE HARBOR

When the tidal level is L.W.L. (D.L. + 0.30 m), h_1 = 4.8 m, h_2 = 8.7 m, hence h_1/h_2 = 0.55, and h_2/H = 1.1 for H = 8.0 m. Therefore, the waves decisively break in front of the breakwater. By using the diagram(2) of α for h_2/H = 1.5, the value of α is assumed α = 4.0, hence β = 2.1 is obtained. Since the values of h/L are approximately 0.19 to 0.14 for waves of $T_{1/10}$ = 8 sec to 10 sec, the value of γ is obtained 0.50. Then the resultant of the maximum simultaneous pressures exerted by the wave of H = 8.0 m and T = 8 sec to 10 sec is obtained

$$P = 4.0 \times 1.03 \times 8.0 \left(4.8 \times \frac{\tanh 2.1}{2.1} + \frac{1}{2} \times 0.50 \times 8.0 \right)$$

$$= 139.3 \cong 140 \text{ t/m.} \tag{8}$$

Since H_c = 3.2 m and γH = 4.0 m, decrease in the pressures due to overtopping, ΔP, is negligible small, and the vertical wall of the breakwater should be considered to have been submerged into water when the waves hit the breakwater.

Judging from the damages of the breakwater that the maximum slide of the vertical wall was 6 m, and the four reinforced concrete caissons suffered some cracks on the walls, it may be assumed that cover-concrete blocks placed at the feet of the vertical walls would probably have been dislocated by the waves prior to occurrence of the slide of the vertical walls, therefore, the uplift pressure at the seaward-side edge of the bottom of the vertical wall may be assumed p_u = 3.0 t/m^2, and the resultant of the uplift pressures exerted on the bottom of the vertical 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

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

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 m, T = 8 sec, P_e = 147 t/m, P_{cal} = 144 t/m,
H = 7.1 m, T = 9 sec, P_e = 103 t/m, P_{cal} = 116 t/m,
H = 8.2 m, T = 9 sec, P_e = 165 t/m, P_{cal} = 142 t/m,
H = 7.4 m, T = 10 sec, P_e = 108 t/m, P_{cal} = 113 t/m,
H = 8.3 m, T = 10 sec, P_e = 139 t/m, P_{cal} = 142 t/m.

The results of the experiments showed:

(1) Very large waves with $H \underset{=}{\sim}$ 8 m to 8.3 m and T = 8 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. $\pm$ 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:

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

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

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

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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

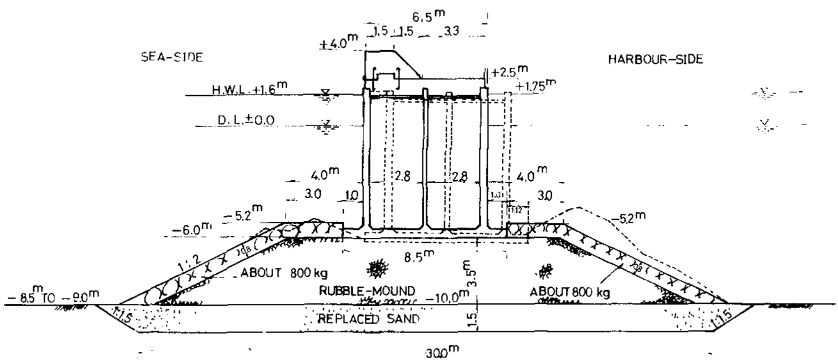

Fig. 6. - CROSS-SECTION OF THE BREAKWATER, HIMEJI HARBOR

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. ± 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 $H_0/L_0 = 0.058$.

Judging from the ratios of $h_1/h_b = 5.2/7.3 = 0.70$, $h_2/h_b = 8.5/7.3 = 1.2$ 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

$$P = 1.5 \times 1.03 \times 5.0 \left(5.2 \times \frac{\tanh 1.1}{1.1} + \frac{1}{2} \times 0.88 \times 5.0 \right)$$

$$= 7.73 (3.78 + 2.20) = 46.23 \text{ t/m}. \tag{11}$$

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

$$P = 46.23 - 3.17 = 43.06 \cong 43.1 \text{ t/m}. \tag{12}$$

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 $p_u = 3$ t/m^2, and the resultant of the up-lift pressure on the bottom is

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

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 $H_c = 2.50$ m was smaller than $\gamma H_{1/10} = 0.88 \times 5.0 = 4.4$ 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

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

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 percent larger than R may be said to prove the slide of the vertical wall.

The resisting force of the vertical wall of the completed breakwater against slide would be

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

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 m is a little larger than H_c = 4.0 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

$$P = 46.23 - 0.14 = 46.09 \cong 46.1\ \text{t/m}. \tag{16}$$

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

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

(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 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

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

Since the value of P is larger than R = 46.7 t/m, the completed breakwater must be slid by the wave of $H_{1/10}$ = 5 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 $p_u = 1.0\ t/m^2$ to $1.5\ t/m^2$.

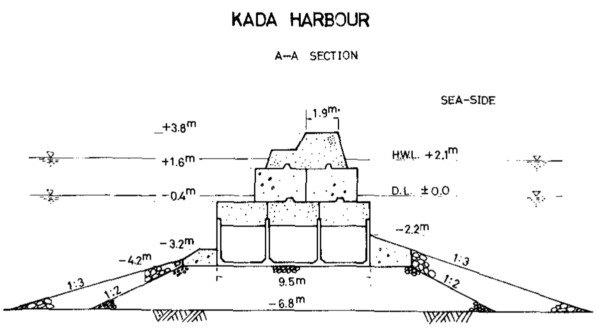

Fig. 7. - CROSS-SECTION OF THE BREAKWATER, KADA HARBOR
(A-A section)

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