Design and Construction of Seawater Exchange Breakwaters

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Abstract

The establishment of breakwaters in Japanese ports has progressed in recent years and has greatly improved the calmness in the ports. However, the seawater exchange within and outside of the port has greatly decreased. In ports near urban areas, in particular, the inflow of domestic and industrial wastewater, and the accumulation of sludge are causing a variety of environmental problems. Therefore, it is necessary to solve two contradictory technical problems: improving seawater exchange while at the same time improving calmness in the port.

An armored caisson breakwater with intake holes has been developed to conduct external seawater into the port using wave energy. In this paper, the hydraulic characteristics and the practical design of this structure will be presented.

Introduction

In ports near urban areas located in inner bay, the various water environmental problems have occurred due to the inflow of domestic and industrial wastewater, and the accumulation of sludge. It is necessary to urgently improve the water environment in these ports. Therefore, it is required that port structures promote the seawater exchange, which is contradictory to original functions such as structural resistiveness against wave action and the maintenance of the calmness in the port.

An armored caisson breakwater is one of the most popular breakwater structure in Japan. It excels in decreasing reflected and transmitted waves, and in being stable in stormy wave conditions. By 2-D & 3-D hydraulic model tests and field tests, the Civil

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Engineering Research Institute has researched and developed an armored caisson breakwater with intake holes, which enabled the seawater exchange within and outside of the port, while making the most of its excellent hydraulic characteristics (C.E.R.I. et al., 1991-1995, 1997). Fig.1 shows the conceptual drawing of the armored caisson breakwater with intake holes.
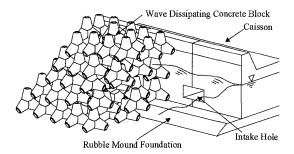

Fig.1 Conceptual Drawing of an Armored Caisson Breakwater with Intake Holes

Experimental Set-up

In the armored caisson breakwater, wave-dissipating blocks installed in front of the breakwater bodies dissipate incident wave energy, and raise the mean water level by wave-setup. The principle of seawater exchange of the armored caisson breakwater with intake holes is the generation of one-way flow into the port through intake holes, using the increase in the mean water level within wave-dissipating blocks (Sarukawa et al., 1993).

Although the seawater exchange is promoted by the increase in the size of intake holes, the wave transmission into the port through intake holes is also increased, and the calmness in the port is decreased. There are also unsolved problems regarding the procedure of design calculation on wave force. Therefore, to confirm the hydraulic functions of the armored caisson breakwater with small or large intake holes, and to establish the structurally resistive design against wave action, 2-D hydraulic model tests were conducted on the following research items.

    1) the wave transmission through intake hole

    2) the increase in the mean water level within wave-dissipating blocks

    3) the intake flow

    4) the wave pressure distribution for the structurally resistive design
        against wave action

    5) the influence of the opening ratio ε of intake holes on the wave
        transmission and the intake flow

Fig.2 shows the experimental models for 2-D hydraulic model tests. The definition of opening ratio ε of intake hole was as follows. The opening ratio ε of intake hole was changed to 16.8, 10.5 and 5.0%.

$$\varepsilon = \frac{\text{an area of cross section of intake holes}}{\substack{\text{an area of underwater cross section of breakwater} \\ \text{including rubble mound foundation}}} \times 100 \ (\%)$$
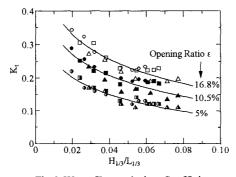

Fig.2 Experimental Models for 2-D Hydraulic Model Tests

Fig.3 Wave Transmission Coefficient

Hydraulic Characteristics
(1) Wave transmission

An understanding of the wave transmission through intake hole is very important, when considering the size and the arrangement of intake holes, and their influence on the utilization of the port in stormy wave conditions. Fig.3 shows the relationship between the wave steepness $H_{1/3}/L_{1/3}$ and the wave transmission coefficient Kt with the opening ratio ε as a parameter. As the wave steepness $H_{1/3}/L_{1/3}$ decreased, and as the opening ratio ε increased, the wave transmission coefficient Kt of the armored caisson breakwater with intake holes increased, on condition that 2-D hydraulic model tests were conducted under condition without the wave transmission by wave overtopping.
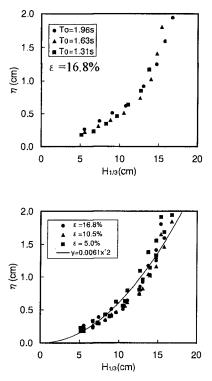
(2) The increase in the mean water level within wave-dissipating blocks

Fig.4 shows the relationship between the wave height $H_{1/3}$ and the increase in the mean water level η_{mean} within wave dissipating blocks, with the opening ratio ε and the wave period $T_{1/3}$ as parameters. The increase in the mean water level η_{mean} within wave-dissipating blocks was not related to the opening ratio ε and the wave period $T_{1/3}$, but was in proportion to the square of the wave height $H_{1/3}$, which was equivalent to the wave energy-loss dissipated within wave-dissipating blocks.

(3) Intake flow

Fig.5 shows the relationship between the increase in the mean water level η_{mean} within wave-dissipating blocks and the mean velocity U_{mean} through intake hole. The mean velocity U_{mean} was in proportion to the dimensionless velocity $(2g\,\eta_{mean})^{0.5}$. "g" is the acceleration of gravity. If the increase in the mean water level η_{mean} within wave dissipating blocks exceeding a certain level does not occur, the portward one-way flow through intake hole does not occur. Similar results were obtained through field tests in

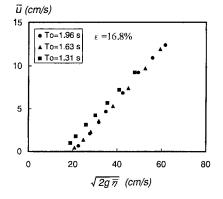Fig. 4 Increase in the Mean Water Level within Wave-dissipating Blocks

Fig.5 Increase in the Mean Water Level within Wave-dissipating Blocks and Mean Velocity through Intake Hole

Urakawa port (Akeda et al.,1996). As shown in Fig.5, it means that the seawater exchange can not be expected from the armored caisson breakwater with intake holes, unless there is the increase in the mean water level exceeding a certain level, in other words, the wave height exceeding a certain level.

(4) Outflow into inner port from intake hole

When using the inside of the armored caisson breakwater with intake holes as a quaywall, it is necessary to understand the influence of the outflow into the inner port from intake hole. Fig.6 shows the damping characteristics of the outflow from intake hole. The damping factor U/U_0 was defined as shown in Fig.6. The outflow from intake hole decreased as the distance from intake hole increased, and almost leveled off when $x/h=1.5-2.5$. When the opening ratio ε was 16.8%, the damping factor U/U_0 was 0.3-0.4 when $x/h=1.5-2.0$.
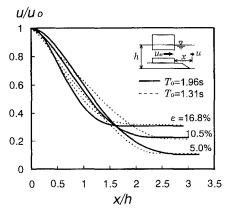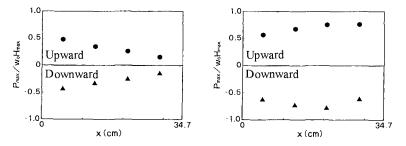

Fig.6 Damping Characteristics of the Outflow

Fig.7 Wave Pressure Distribution (T_0=1.96s, H_0'=21.3cm)

Design wave pressure

(1) Wave pressure distribution

When the intake hole is put on relatively shallow position, the impact wave pressure

acts on the inside of intake hole. Fig.7 shows an example of the wave pressure distribution on the top and the bottom of intake hole. The horizontal axis is the distance from the front-end of intake hole. The vertical axis is the dimensionless wave pressure P_{max}/w_0H_{max}. The left figure is the case when the intake hole is put on relatively deep position. The wave pressure distribution was largest at the front-end of intake hole, then decreased linearly toward the rear-end of intake hole. The upward and downward wave pressure distribution in intake hole were almost symmetrical. The right figure is the case when the intake hole is put on relatively shallow position. The wave pressure distribution was not attenuated toward the rear-end of intake hole.

Fig.8 shows the influence of the crown depth of intake hole on the dimensionless wave pressure P_{max}/w_0H_{max}. The horizontal axis is the dimensionless crown depth of intake hole $d_1/H_{1/3}$. The vertical axis is P_{max}/w_0H_{max}. d_1 was the crown depth of intake hole. When the wave-dissipating blocks were installed in front of the breakwater bodies, P_{max}/w_0H_{max} was almost constant at 0.2-0.4 when $d_1/H_{1/3}$ was larger than 0.2. When $d_1/H_{1/3}$ was smaller than 0.2, the impact wave pressure did not occur even though P_{max}/w_0H_{max} increased. When the wave-dissipating blocks were not installed in front of the breakwater bodies, P_{max}/w_0H_{max} was almost constant at 0.3-0.4 when $d_1/H_{1/3}$ was larger than 0.4. It was observed that P_{max}/w_0H_{max} increased with the decrease in $d_1/H_{1/3}$, and the impact wave pressure occurred when $d_1/H_{1/3}$ was smaller than 0.3.
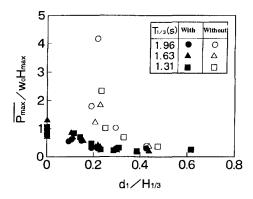

Fig.8 Influence of the Crown Depth of Intake Hole
on the Dimensionless Wave Pressure in Intake Hole

(2) Wave pressure formula

Fig.9 shows the wave pressure distribution used for the structurally resistive design against wave action. The horizontal wave pressure and the uplift pressure acting on the front and the bottom of the breakwater body can be determined on the basis of the modified Goda's formula (Yamamoto et al., 1997). In consideration of the influence of intake hole, the horizontal wave pressure is not acted on the vertical surface of "opening part". The uplift pressure is acted on the top of intake hole, and triangularly

distributed toward the rear-end of intake hole, with an intensity of wave pressure at the front-end of intake hole of p_{u2}. The difference between the horizontal intensity of wave pressure acting on the top and the bottom of intake hole is defined as p_{u2}.
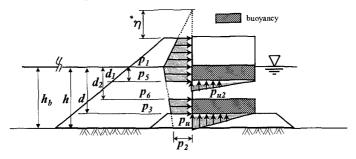

Fig.9 Design Wave Pressure Distribution

The details of the wave pressure formula are as follows:

$$\eta^* = 1.5\lambda H_D$$

$$p_1 = \lambda\alpha_1 w_0 H_D$$

$$p_2 = \frac{p_1}{\cosh(2\pi h / L)}$$

$$p_3 = \alpha_3 p_1$$

$$p_u = \lambda\alpha_1\alpha_3 w_0 H_D$$

$$P_{u2} = p_5 - p_6 = \frac{d_2 - d_1}{h'}(p_1 - p_3)$$

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi h / L}{\sinh(4\pi h / L)}\right]^2$$

$$\alpha_2 = min\left[\frac{h_b - d}{3h_b}\left(\frac{H_D}{d}\right)^2, \frac{2d}{H_D}\right]$$

$$\alpha_3 = 1 - \frac{h'}{h}\left[1 - \frac{1}{\cosh(2\pi h / L)}\right]$$

Where,

$\eta^{\cdot}$: the wave height where the intensity of wave pressure becomes 0 (m)

p_1: the intensity of wave pressure on the still water level (tf/m^2)

p_2: the intensity of wave pressure on the sea bottom (tf/m^2)

p_3: the intensity of wave pressure at the bottom of the upright wall (tf/m^2)

p_5: the intensity of wave pressure on the top of the intake hole (tf/m^2)

p_6: the intensity of wave pressure on the bottom of the intake hole (tf/m^2)

p_u: the intensity of uplift pressure at the front leg of the bottom of the upright wall (tf/m^2)

p_{u2}: the intensity of uplift pressure at the front of the intake hole, $p_{u2}=p_5-p_6$(tf/m^2)

h: the water depth at the front of the upright wall (m)

h_b: the water depth at a distance 5 times longer than the significant wave height from the front of the upright wall to the offshore side (m)
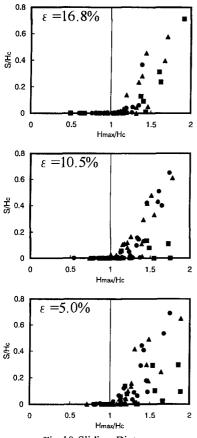
d_1: the water depth at the top of the intake hole (m)

d_2: the water depth at the bottom of the intake hole (m)

w_0: the unit volume weight of sea water (tf/m^3)

H_D: the wave height used for design calculation (m)

L: the wave length used for design calculation at the water depth of h (m)

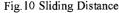λ: the wave pressure reduction rate by covering with wave-dissipating blocks ($\lambda=0.8$)

Fig.10 shows the relationship between the sliding distance and the critical sliding wave height. The horizontal axis is H_{max}/H_c. H_{max} is the maximum wave height. H_c is the critical sliding wave height calculated by the wave pressure formula. The vertical axis is the dimensionless sliding distance S/H_c. S is the sliding distance. Because S/H_c increased dramatically over $H_{max}/H_c=1$ regardless of the opening ratio ε, the above-mentioned wave pressure formula was thought to be reasonable.

Field Test in Urakawa Port

(1) Measuring conditions

In Urakawa port on the Pacific coast of Hokkaido, the armored caisson breakwater with a intake hole with the opening ratio of 5.0% was constructed to conserve the water quality and promote the seawater exchange. A field test on the hydraulic functions of this structure was conducted from October 26 to November 16, in 1995. The following items were observed:

1) the wave condition (wave height, wave period) in front of the breakwater (at 150m offshore, the depth of 8m)

2) the water pressure (at the front-end and the rear-end of intake hole)

3) the intake flow

Fig.10 Sliding Distance

Fig.11 shows the layout of measuring instruments. Fig.12 shows the time histories of the wave conditions in front of the breakwater of $H_{1/3}$, $T_{1/3}$, the difference in the mean water level in front-end and rear-end of intake hole of $\Delta \eta$, and the mean velocity through intake hole of U_{mean}. The difference in the mean water level were calculated on the basis of the difference in the water pressure in front-end and rear-end of intake
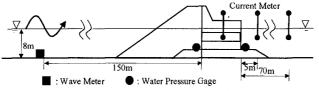

Fig.11 Layout of Measuring Instruments in Field Test

hole. At 8 a.m. on November 8, when the maximum wave ($H_{1/3}$=4.40m, $T_{1/3}$=8.6s) occurred, the difference in the mean water level in front-end and rear-end of intake hole was $\Delta \eta$ =6.9cm, and the mean velocity through intake hole was U_{mean}=46.1cm/s. The observed value was equivalent to the inflow of seawater of about 10,000 tons per hour.
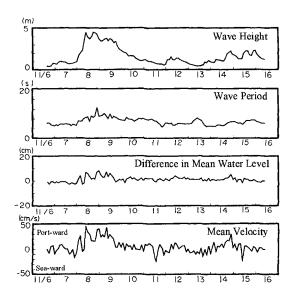

Fig.12 Time Histories of Observation Data

(2) Flow velocity through intake hole

   Fig.13 shows the relationship between the wave height in front of the breakwater $H_{1/3}$ and the difference in the mean water level in front-end and rear-end of intake hole $\Delta \eta$. It was found from 2-D hydraulic model tests that the increase in the mean water level within wave-dissipating blocks was not related to the opening ratio ε or the wave period, but was in proportion to the square of the wave height. In field test, the difference in the mean water level in front-end and rear-end of intake hole $\Delta \eta$ was in proportion to the square of the wave height in front of the breakwater $H_{1/3}$, in case of the long-period waves with the wave period $T_{1/3}$ of 8 sec or longer. In case of the short-period waves with the wave period $T_{1/3}$ of shorter than 8 sec, there was hardly any difference in the mean water level in front-end and rear-end of intake hole due to the small incident wave height.

   Fig.14 shows the relationship between the difference in the mean water level in front-end and rear-end of intake hole $\Delta \eta$ and the mean velocity in intake hole U_{mean}.

In the same way as in 2-D hydraulic model tests, the mean velocity in intake hole U_{mean} was in proportion to the a half power of the difference in the mean water level in front-end and rear-end of intake hole $\Delta \eta$. From the relationship shown in Figs.13 and Fig.14, it was found that the mean velocity in intake hole was in proportion to the wave height in front of the breakwater.

(3) Effect of the long-period oscillation

Fig.15 shows an example of the fluctuation of the mean water level in the port $\Delta \eta$ and the mean velocity in intake hole U_{mean} observed at 8 a.m. of Nov.13. The wave condition were the wave height of $H_{1/3}=0.44m$, and the wave period of $T_{1/3}=8.4sec$. Under relatively calm wave conditions, when the wave period $T_{1/3}$ is 6 to 7 seconds or shorter and the wave height $H_{1/3}$ is 1.5m or smaller, the external force which causes the seawater exchange is thought to be the long-period oscillation in the port with a period
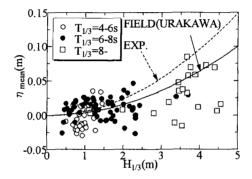

Fig.13 Wave Height in Front of Breakwater and
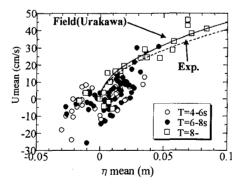Difference in the Mean Water Level in Front-end and Rear-end of Intake Hole

Fig.14 Difference in the Mean Water Level in Fron-tend and Rear-end
of Intake Hole and Mean Velocity in Intake Hole

of over 10 minutes and an amplitude of several centimeters. As shown in Fig.15, an oscillatory flow with a flow velocity amplitude of 20 to 40cm/s was observed, and it was equivalent to a seawater exchange of several hundred tons per hour. From the above results, it was found that long-period oscillation in the port would contribute to seawater exchange within and outside the port under relatively calm wave conditions.
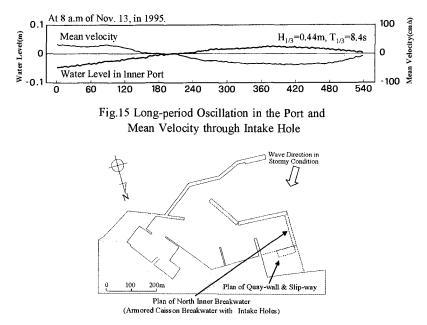

Fig.15 Long-period Oscillation in the Port and
Mean Velocity through Intake Hole

Fig.16 Plan View of Kudoh Fishing Port

Case study on Kudoh Fishing Port

In Kudoh Fishing Port on the Sea of Japan coast of Hokkaido, the construction of an armored caisson breakwater with intake holes with a large opening ratio was planned to improve the calmness in the port. 2-D&3-D hydraulic model tests and numerical analysis were conducted to determine the inflow condition of external seawater and the calmness in the port, to examine the effectiveness of the construction of the armored caisson breakwater with intake holes in Kudoh Fishing Port (Akeda et al.,1998). Fig.16 shows a plan view of the Kudoh Fishing Port. The opening ratio ε was about 16 to 22%, based on the installation depth of the north inner breakwater.

(1) Seawater exchange by tides

If the north inner breakwater is an impermeable structure, the seawater exchange by tides can not be expected, the seawater exchange rate will be decreased to 1/100 to 1/50 of the present (without the north inner breakwater). Although the seawater exchange rate will be decreased to 1/6 to 1/5 of the present if the north inner

breakwater is the armored caisson breakwater with intake holes, there will be no problem concerning water quality in the port.
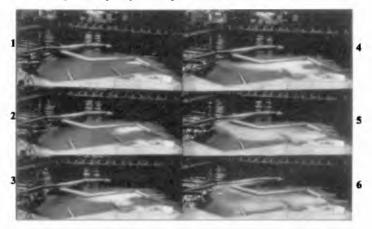

Fig.17 Results of Visualization Experiment on the Inflow ($T_{1/3}$=12.0s, $H_{1/3}$=5.3m)
NO.1 is after 8 min. NO.2 is after 26 min. NO.3 is after 38 min.
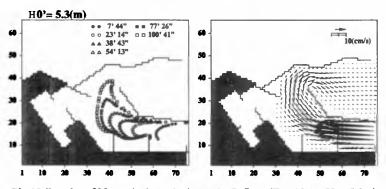NO.4 is after 77 min. NO.5 is after 100 min. NO.6 is after 124 min.

Fig.18 Results of Numerical Analysis on the Inflow ($T_{1/3}$=12.0s, $H_{1/3}$=5.3m)

(2) Seawater exchanges by waves

Fig.17 shows an example of the results of a visualization experiment of the inflow by waves, when the north inner breakwater is the armored caisson breakwater with intake holes. Numbers in the Fig.17 are the field-converted hours after external seawater flowed into the port. Under all experiment conditions, the dye tracer injected into the intake hole flowed into the port without spreading outside the port at all. It was found that the armored caisson breakwater with intake holes were effective in

inducing seawater even with oblique incident waves. The numerical analysis of seawater exchange using a 3-D multi-stratified flow model (Fujihara et al.,1997) was conducted under the same condition as that of the visualization experiment of the inflow of external seawater. Fig.18 shows the numerical analysis result with the same condition as Fig.17. As shown in Fig.18, the tracer distribution and the velocity vector of the inflow external seawater obtained by the numerical analysis almost corresponded with the results of the 2-D hydraulic model tests.

If the north inner breakwater was the armored caisson breakwater with intake holes, it is expected that, in addition to the seawater exchange of 0.5 times/day by tides, the seawater exchange of 3.4 times/day would occur when $T_{1/3}$=12.0s and $H_{1/3}$=5.3m, which were equivalent to wave heights generated several times a year. The seawater exchange of 0.6 times/day would occur when $T_{1/3}$=8.0s and $H_{1/3}$=1.9m, which is equivalent to wave heights generated once or twice a month.

(3) Calmness in the port

Present wave heights (without a north inner breakwater) and future wave heights (with an armored caisson breakwater with intake holes as the north inner breakwater) in the basin for small fishing boat inside of the north inner breakwater were compared. Here, the wave height ratio was defined as the ratio of mean wave height in the basin for small fishing boat to transiting wave height at the position of the north inner breakwater. The wave height ratio was large (0.3 to 0.4) in the basin for small fishing boat inside of the north inner breakwater, due to the direct invasion of waves from the base of the north inner breakwater in the west side of the port. The wave height ratio was particularly high (0.50 to 0.65) in front of the slipway. It was found that, in the future, the wave height ratio would decrease to 0.1 to 0.2 in the basin for small fishing boat and the effect of construction of the north inner breakwater on the calmness in the port would be significant. The rate of effective working days of the quaywall and slipway would also be improved to the required level.

Conclusions

The main results of this research were as follows.

(1)As the wave steepness $H_{1/3}/L_{1/3}$ decreased, and as the opening ratio ε increased, the wave transmission coefficient Kt of the armored caisson breakwater with intake holes increased.

(2)The increase in the mean water level η_{mean} within wave-dissipating blocks was not related to the opening ratio ε and the wave period $T_{1/3}$, but was in proportion to the square of the wave height $H_{1/3}$. The mean velocity U_{mean} through intake hole was in proportion to the dimensionless velocity $(2g\,\eta_{mean})^{0.5}$. The results of field test in Urakawa port were similar to one of 2-D hydraulic model tests.

(3)The horizontal wave pressure and the uplift pressure acting on the front and the bottom of the breakwater body can be determined on the basis of the modified Goda's formula, in consideration of the influence of intake hole.

(4)The amount of seawater exchange could be determined by numerical analysis using

a 3-D multi-stratified flow model.
(5)The validity of the design procedure and the effectiveness of improvement of the seawater exchange could be confirmed through the construction of breakwaters in Setana port (1991-), Urakawa port (1994), Fukushima fishing port (1996-) and Kudoh fishing port (1996-1997).

The armored caisson breakwater with intake holes is excellent in high stability against waves, and can be designed in the same way as a conventional method. Also, it does not require special construction methods and large machinery. The dissemination and the popularization of this technology is expected in the future.

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