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BEHAVIOR AND HYDRAULIC PERFORMANCES OF COMPOSITE BREAKWATER UNDER HIGHER WAVE THAN DESIGN WAVE

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ABSTRACT: Coastal and harbor structures should be resilient so that the hydraulic performances of the structures will not decreased rapidly at the ultimate state where the incident wave height exceeds the design wave. In this study, the behavior of the caissons of composite breakwaters and the hydraulic performance at the ultimate state are measured through a hydraulic experiment. The influence of the damage to a caisson on the behavior of the adjacent one is also investigated. The authors show that the sliding distance of the caisson for the largest incident wave height in the case where the caissons with the same safety factors are installed in a row is larger than those in the cases where one of the caissons has a smaller safety factor and where one of the caissons is intentionally overturned.

Keywords: caisson, sliding distance, transmission coefficient, composite breakwater, safety factor

INTRODUCTIION

A huge tsunami hit the pacific coasts of the northeastern part of Japan on March 11, 2011. Many coastal and harbor structures were seriously damaged by the tsunami. One of the reasons for the damages to the structures is that the incident wave exceeded the design wave. Coastal and harbor structures should be resilient so that the hydraulic performances of the structures will not decreased rapidly at the ultimate state caused by tsunami as well as extreme wind waves.

Bremner *et al*. (1980) reported that the rubble mound breakwater damaged by a cyclone still functioned effectively as a submerged breakwater and investigated the breakwaters which was designed to be damaged under extreme wave and storm surge conditions. Ahren (1989) investigated the change in the hydraulic performance of the rubble mound breakwater to keep tranquility behind it with the decrease in the crest height. Araki *et al*. (2005) investigated the deformation of the submerged breakwater and the change in the wave height behind it with deformation through a three-dimensional experiment. However, the deformation of breakwaters and the hydraulic performances of them have not been investigated sufficiently.

The purpose of this study is to investigate the behavior of the caissons of composite breakwaters and the hydraulic performance at the ultimate state where the incident wave height exceeds the design wave through a hydraulic experiment. They are discussed further by investigating the influence of the damage to a caisson on the behavior of the adjacent one in the hydraulic experiment.

HYDRAULIC EXPERIMENTS

Outline of Hydraulic Experiments

The hydraulic experiments were conducted in a 25.0 m long and 0.7 m wide wave flume shown in Fig. 1. The composite breakwaters were installed on a fixed flat bed where the water depth was 0.30 m. The caissons were placed on a 10 cm high mound. The main part of the mound consisted of concrete blocks so that the friction between the caisson and the mound would be as uniform as possible. The rest of the mound, *i.e.*, the seaward and landward slope of the mound and the gap between the concrete blocks and the side wall of the wave flume, consisted of crashed stones whose diameter ranged from 2 to 4 cm. In this experiment, the state where the caisson falls down from the crest of the mound was not taken into account. Therefore, the width of the mound on the inside of the harbor was larger than that of usual composite breakwaters in this experiment.

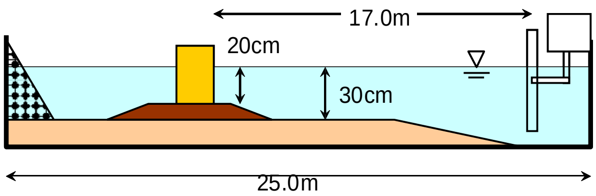

Fig. 1 Experimental setup

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A set of irregular waves was generated by the piston-type wave maker installed at the end of the wave flume. The target spectrum was Bretschneider-Mitsuyasu spectrum. The significant wave period of the irregular wave was $T_{1/3}$ = 1.4 s and 1.7 s. The significant wave height for the irregular wave of $T_{1/3}$ = 1.4 s ranged from 6.2 to 8.1 cm under the condition that the breakwater was not installed. The significant wave height for the irregular wave of $T_{1/3}$ = 1.7 s ranged from 4.8 to 7.3 cm under the same condition. The distance between the wave paddle and the caisson was 17.0 m. In each experimental run, the duration of the incident irregular wave was approximately 250 waves. The sliding distance of the caisson and the water surface elevations in front of and behind the caissons were measured. The sliding distance of the caisson was measured after the incidence of 250 waves. The details of measuring the sliding distance is mentioned later. The surface elevation was measured by capacitance-type wave gauge and recorded at 20 Hz. Fig. 2 shows the set up of the composite breakwater in the wave flume. The incident wave comes from the right hand side of the photo. The left hand side of the photo is the inside of the harbor.

Caisson

The caisson was 17.0 cm long in the direction which is perpendicular to the wave propagating direction. Therefore, four caissons were able to be placed within a 70 cm width wave flume in a row. The crest width of the caisson was 16.0 cm and 26.0 cm, which means that each

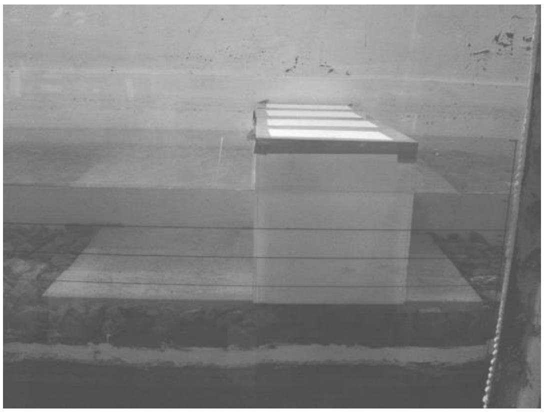


Fig. 2 Caisson and mound in hydraulic experiment

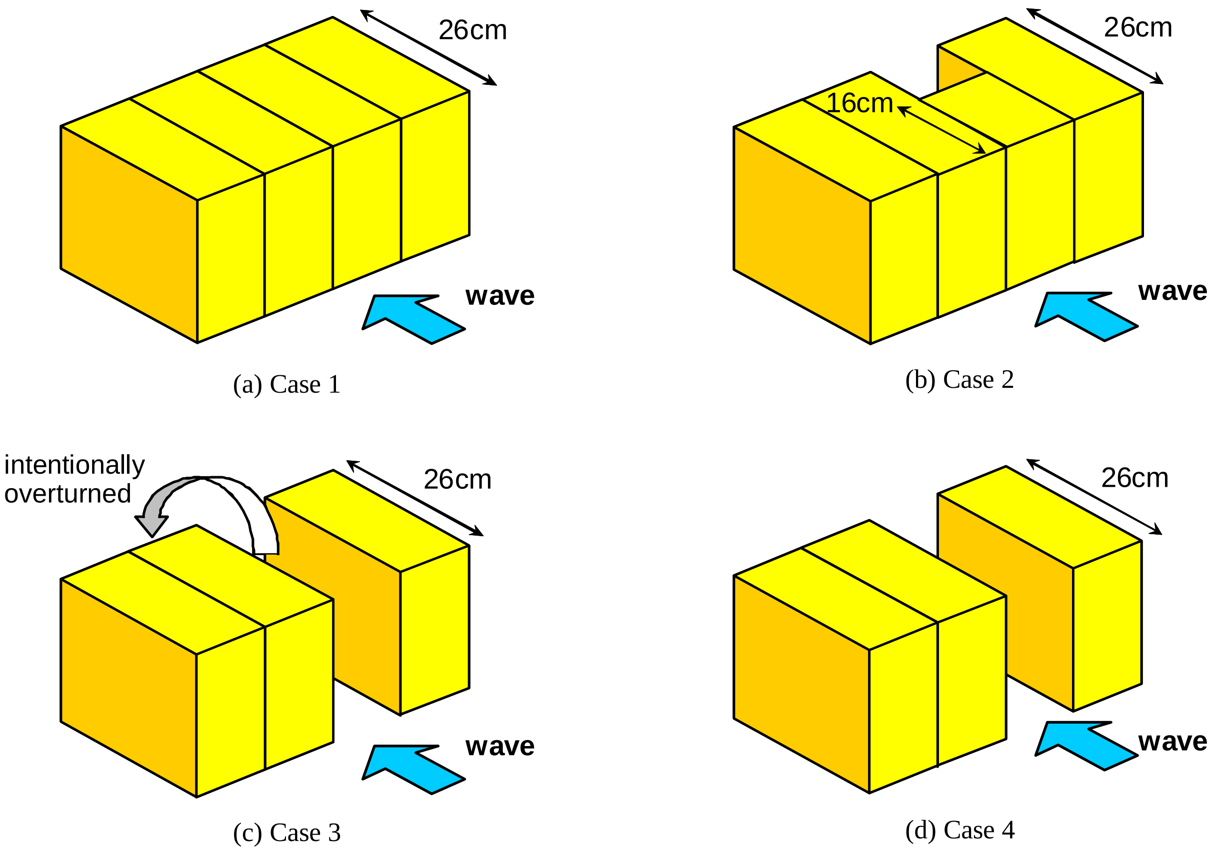


Fig. 3 Combinations of caissons

caisson used in the experiment has a different safety factor. The height of the caisson was 28.0 cm. The caisson was made with polyvinyl chloride. The inside of the caisson was filled with crashed stones and sand.

Both safety factors for sliding and rotating of the caisson with a 26.0 cm crest width were larger than 1.2 for the incident wave of $H_{1/3}$ = 3.0 cm and $T_{1/3}$ = 1.6 s. On the contrary, both safety factors for sliding and rotating of the caisson with a 16.0 cm crest width were larger than 1.0 and smaller than 1.2 for the same incident wave.

In this experiment, the differences in the behavior of the caissons and the tranquility behind the breakwater were investigated under the condition that one caisson with a shorter crest width, *i.e.*, smaller safety factor, was installed in a row of the caissons with a longer crest width. The combinations of the caissons are as follows:

- Case 1

Four caissons with a 26.0 cm crest width are installed. The safety factors for all the caissons are the same.

- Case 2

One caisson with a 16.0 cm crest width and three caissons with a 26.0 cm crest width are installed. The sliding distances of the two kinds of caissons were measured in order to investigate the effect of the inclusion of the caisson with a smaller safety factor.

- Case 3

This is the case where the caisson with a 16.0 cm crest width was assumed to be overturned toward the inside of the harbor. The wave incidence and the measurement started from the state where the caisson was overturned in order to investigate the effect of the intentional overturn of the caisson.

- Case 4

This is the case where one of four caissons in a row is completely disappeared. Therefore, only three caissons with a 26.0 cm crest width are installed. This is one of the worst cases for the harbor tranquility. The result measured in this case is compared with those in other cases.

Fig. 3 shows the rough sketch of the combinations of the caissons.

Friction Coefficient

The friction coefficient between the caisson and the concrete block which was the main part of the mound was measured. The procedure for measuring the friction coefficient is as follows:

(i) Pull a caisson on the concrete block horizontally until the caisson starts to slide.

(ii) The pulling force is measured by spring balance at the moment when the caisson starts to slide.

(iii) The mass of the caisson is changed by putting crashed stones into the caisson.

(iv) Repeat the steps from (i) to (iii).

(v) Find the friction coefficient from the relationship between the pulling force and the normal force to the concrete.

The pulling force was measured twice for each mass of the caisson (It was measured four times for each mass of the caisson in several cases). The measurement was conducted in the air. However, the face of the concrete block was wet by sprinkling water on it. Fig. 4 shows the measurement of the friction coefficient.

Fig. 5 shows the relationship between the pulling force at the moment when the caisson starts to slide and the normal force to the concrete (the product of the mass of the caisson multiplied by gravitational acceleration). The data are not scattered so much because the concrete block was used as a mound in this experiment. The gradient of the solid line indicates the friction coefficient. Therefore, the friction coefficient was 0.55.

Fig. 4 Measurement of friction coefficient

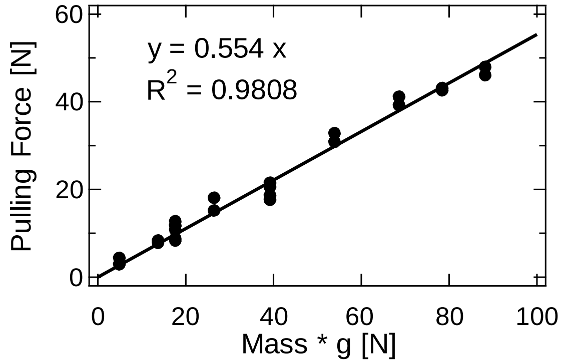


Fig. 5 Relationship between pulling force at the moment when caisson starts to slide and weight of caisson

Measurement of Sliding Distance

The cross-shore distance between the positions of the caisson before and after the wave incidence was measured as the sliding distance. The sliding distances of the caissons next to the side walls of the flume were not measured because the friction between the caisson and the side wall may have the influence on the sliding distance. Therefore, the sliding distances of the two center caissons in the row was measured. The details of the measurement in Cases 1-4 are as follows:

- Case 1

Both two center caissons have the crest width of a 26.0 cm. A larger sliding distance of the two is determined to be the sliding distance in a case.

- Case 2

In this case, one of the two center caissons has the crest width of a 16.0 cm. The caisson with a 16.0 cm crest width slides more than that with a 26.0 cm crest width. However, the purpose of installing the caisson with a 16.0 cm crest width is to prevent adjacent caissons with a 26.0 cm crest width from damaging seriously. Therefore, the sliding distance of the caisson with a 26.0 cm crest width is determined to be the sliding distance in a case.

- Case 3

The sliding distance of the caisson with a 26.0 cm crest width is determined to be the sliding distance in a case as in Case 2.

- Case 4

The sliding distance of the center caisson is determined to be the sliding distance in a case.

In some cases, the caisson rotated around a vertical axis. In those cases, the distance between the offshore sides of the initial and the rotated positions was determined to be the sliding distance in the case.

Fig. 6 Behavior of caissons in Case 1 at the incidence of approximately 250 waves of $H_{1/3}$ = 6.8 cm and $T_{1/3}$ = 1.7 s, taken from the offshore side

EXPERIMENTAL RESULTS

Behavior of Caisson

Case 1

The behavior of the caisson in Case 1 depended on the test conditions. As an example, Fig. 6 shows the positions of the caissons at the incidence of approximately 250 waves of $H_{1/3}$ = 6.8 cm and $T_{1/3}$ = 1.7 s. The photo was taken from the offshore side. The sliding distances of the two center caissons were measured.

Case 2

Under the incident wave whose significant wave height was smaller than 5.4 cm, the caisson with a 16.0 cm crest width slid and the caisson with a 26.0 cm crest width did not slide. Under the incident wave whose significant wave height was larger than 5.4 cm, both caissons slid. As an example, Fig. 7 shows the positions of the caissons at the incidence of approximately 250 waves of $H_{1/3}$ = 6.8 cm and $T_{1/3}$ = 1.7 s. The photo was taken from the inside of the harbor. The sliding distances of the two center caissons were measured.

Case 3

Under the incident wave whose significant wave height was smaller than 5.9 cm, the caisson with a 26.0 cm crest width slid a little. Under the same incident wave, the net sliding distance of the intentionally overturned caisson with a 16.0 cm crest width was very short although it slid offshore and onshore according to the wave crest and trough. Under the incident wave whose significant wave height was larger than 5.9 cm, the sliding distance of the caisson with a 26.0 cm crest width increased. As an example, Fig. 8 shows the positions of the caissons at the incidence of approximately

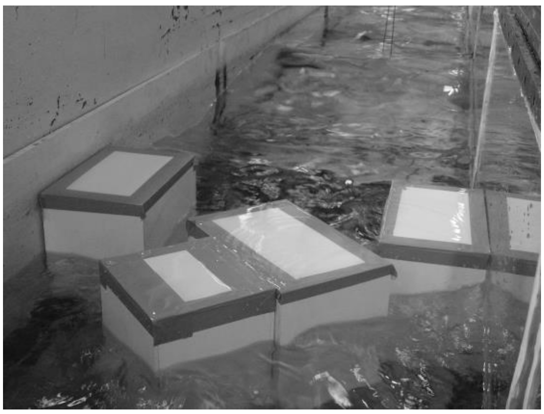


Fig. 7 Behavior of caissons in Case 2 at the incidence of approximately 250 waves of $H_{1/3}$ = 6.8 cm and $T_{1/3}$ = 1.7 s, taken from the inside of the harbor

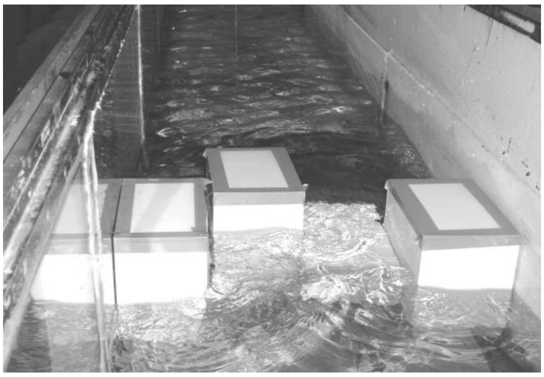


Fig. 8 Behavior of caissons in Case 3 at the incidence of approximately 250 waves of $H_{1/3}$ = 6.8 cm and $T_{1/3}$ = 1.7 s, taken from the offshore side

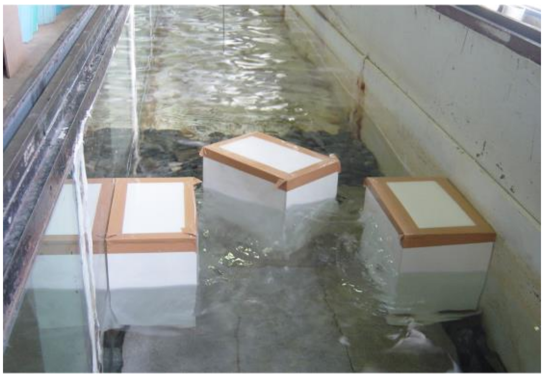
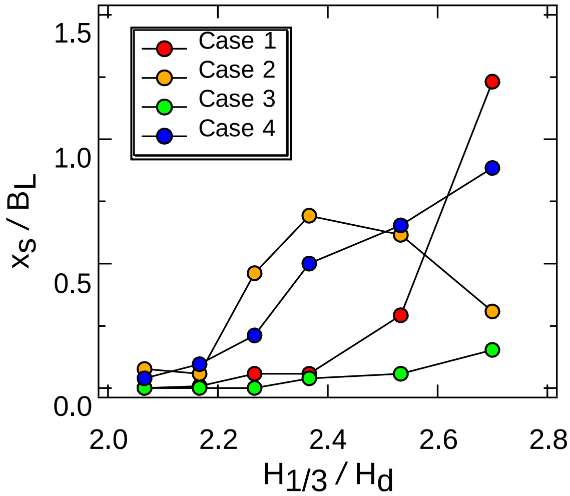
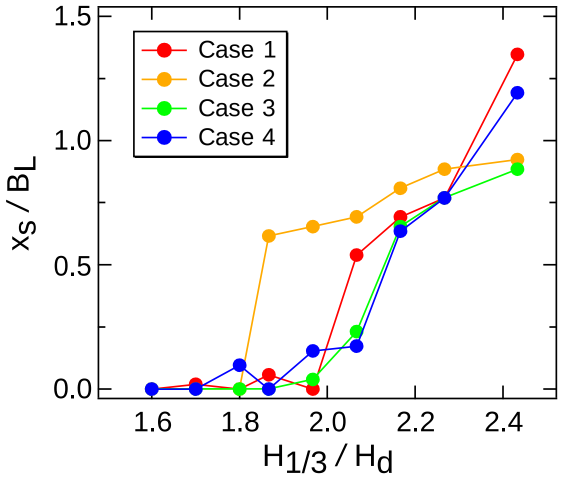


Fig. 9 Behavior of caissons in Case 4 at the incidence of approximately 250 waves of $H_{1/3}$ = 6.8 cm and $T_{1/3}$ = 1.7 s, taken from the offshore side



(a) $T_{1/3}$ = 1.4 s



(b) $T_{1/3}$ = 1.7 s

Fig. 10 Sliding distance of caisson

250 waves of $H_{1/3}$ = 6.8 cm and $T_{1/3}$ = 1.7 s. The photo was taken from the offshore side. The sliding distance of the second caisson from the left on the photo was the sliding distance in this case. The second caisson from the right is an intentionally overturned submerged one.

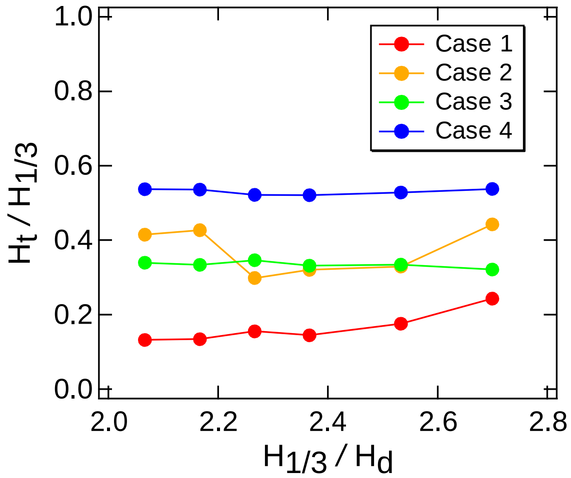
Case 4

In many incident wave conditions in Case 4, the center caisson with a 26.0 cm crest width rotated around a vertical axis. As an example, Fig. 9 shows the positions of the caissons at the incidence of approximately 250 waves of $H_{1/3}$ = 6.8 cm and $T_{1/3}$ = 1.7 s. The photo was taken from the offshore side.

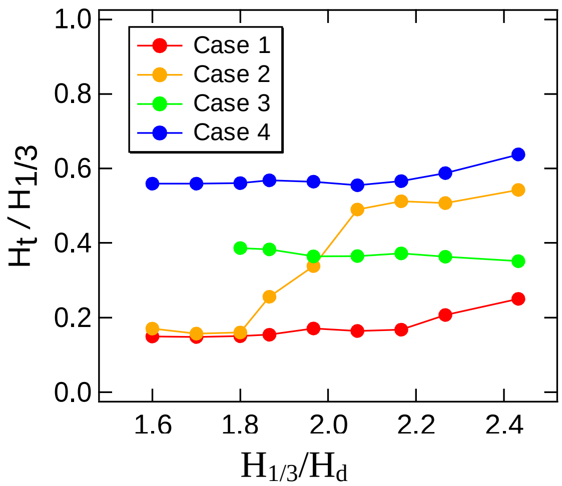
Sliding Distance

Fig. 10 shows the sliding distance of the target caisson in each case. The vertical axis shows the sliding distance of the target caisson mentioned before x_S normalized by the crest width of the caisson B_L (= 26.0 cm). The horizontal axis shows the incident significant wave height $H_{1/3}$ normalized by the design wave height H_d for which both safety factors for sliding and rotating of the caisson with a 26.0 cm crest width are larger than 1.2 (H_d = 3.0 cm).

In Case 2 where the caisson with a 16.0 cm crest width is installed, the sliding distance of the center caisson with a 26.0 cm crest width is the largest in the range of the smaller significant wave height in the figure. However, the sliding distance for the largest incident significant wave height in Case 2 is smaller than that in Case 1 where the breakwater consists of only the caisson with a 26.0 cm crest width. In Case 3 where the caisson with a 16.0 cm crest width is intentionally overturned, the magnitude of the incident significant wave height at which the sliding distance increases rapidly is the similar to that in Case 1. However, the sliding distance of the center caisson with a 26.0 cm crest width for the largest



(a) $T_{1/3}$ = 1.4 s



(b) $T_{1/3}$ = 1.7 s

Fig. 11 Transmission coefficient

incident significant wave height in Case 3 is smaller than that in Case 1.

The characteristics of the sliding distance mentioned above are clear for the conditions of $T_{1/3}$ = 1.7 s shown in Fig. 10(b). Although the characteristics are not so clear for the condition of $T_{1/3}$ = 1.4 s shown in Fig. 10(a), the sliding distances for the largest incident significant wave height in Cases 2 and 3 is smaller than those in Cases 1 and 4, which is the same as the condition of $T_{1/3}$ = 1.7 s.

Transmitted Wave Height

One of the important hydraulic functions of breakwaters is to keep tranquility inside harbors under the high wave conditions. Fig. 11 shows the transmission coefficient in each case. The vertical axis shows the transmission coefficient which is the ratio of the incident to transmitted significant wave heights H_t / $H_{1/3}$ the horizontal axis shows the ratio of the incident significant wave height $H_{1/3}$ to he design wave height H_d (= 3.0 cm).

The transmission coefficient in Case 4 where one of four caissons in a row is completely disappeared is the largest for all the incident significant wave height. The transmission coefficient in Case1 where four caissons with a 26.0 cm crest width are installed is the smallest. The transmission coefficients in Cases 2 and 3 are between the coefficients in Cases 1 and 4. The transmission coefficient in Case 2 fluctuates according to the behavior of the caissons. The variation of the transmission coefficient in Case 3 is small. In Case 3, the overturned caisson resulted in a breakwater like a submerged breakwater. The transmission coefficient in Case 3 is not so large because the incident wave broke on the overturned caisson.

SUMMARY

In this study, the behavior and the hydraulic performances of the caisson breakwater was investigated under the condition that the incident wave height exceeded the design wave. The influence of the damage to a caisson on the behavior of the adjacent one was also investigated. The sliding distance of the caisson for the largest incident wave height in the case where the caissons with the same safety factors were installed in a row (Case 1) was larger than those in the cases where one of the caissons had a smaller safety factor (Case 2) and where one of the caissons was intentionally overturned (Case 3). However, the conditions in this study were very limited. In order to use the caisson with a smaller safety factor for reducing the sliding distance, a more careful and detailed investigation should be done.

REFERENCES

Ahren, J. P. (1989). Stability of reef breakwater. Journal of Waterway, Port, Coastal, and Ocean Engineering, ASCE, Vol. 115, No. 2: 221-234

Araki, S., Niijima, H., Fumoto, H., Miyoshi, H. and Deguchi, I. (2005). Change in transmission coefficient with deformation of submerged breakwater. Proc. of the 15th International Offshore and Polar Engineering Conference, ISOPE, Seoul: Vol. III, 606-611.

Bremner, W., Foster, D. N., Miller, C. A., and Wallace, B. C. (1980). The design concept of dual breakwaters and its application to Townsville, Australia. Proc. of International Conference on Coastal Engineering, ASCE, Sydney: 1898-1908.