Horizontal loading experiments on reinforced gravity type breakwater with steel walls

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ABSTRACT

In the 2011 off the Pacific coast of Tohoku Earthquake, especially coastal areas of the Pacific Ocean suffered extensive damage by tsunami. Then, there were needs to develop reinforcement methods for existing breakwaters against tsunami force. The authors proposed a method to reinforce a breakwater against tsunami with a steel wall behind the breakwater. This report presents the experimental results of the breakwater reinforcing method. In the proposed method, steel piles are inserted behind the breakwater and rubbles are filled between the caisson and the piles. The model caisson failed in sliding mode without this improvement method. When it was reinforced with steel walls and filling, the horizontal resistance of the model caisson improved effectively. The failure mode of the caisson changed from sliding mode to ground failure mode. By analyzing the bending moment of the steel wall, the stress distribution in the wall from the caisson via filling and the ground was obtained and failure surface of the ground can be estimated.

Keywords: breakwater, steel wall, tsunami, lateral resistance of pile

1. INTRODUCTION

In the 2011 off the Pacific coast of Tohoku Earthquake, especially coastal areas of the Pacific Ocean suffered extensive damage by tsunami. One of the main reasons of the damage to gravity type breakwaters was from extensive horizontal force by water level difference in two sides of the caisson (Arikawa et al., 2012). Then, there were needs to develop reinforcement methods for existing breakwaters against tsunami force.

A research to improve the gravity type breakwaters against horizontal forces such as wave forces with backfilling has been conducted. It has been recognized that passive resistance of the backfilling made the caissons more stable against horizontal forces (Kikuchi et al., 2011).

In this paper, the authors proposed a method to reinforce a caisson against tsunami with a steel wall behind the breakwater. In the proposed method, steel walls are inserted into the ground behind the caisson and rubbles are filled between the caisson and the walls as shown in Fig. 1. Passive resistance of the ground can effectively be used with steel wall as the horizontal forces acting on the caisson can effectively be transferred to the steel wall by the fill.

A series of model experiments was conducted. In the experiments, the distance between the caisson and the steel walls, filling or non-filling of the space between them were selected as parameters. The experiments were conducted under dry sand condition. The model caisson was statically and horizontally loaded. This paper discusses the effect of this method on improvement of horizontal resistance of the caisson and the mechanism of horizontal resistance improvement by the method.

Fig. 1. Schematic diagram of the reinforced gravity type breakwater with steel walls.

2. EXPERIMENT OUTLINE

The models used in the experiment were designed at 1/60 scale. The similarity rules used in this model was
proposed by Iai (Iai, 1988) and given in Table 1. The schematic diagram of the model set up is shown in Fig. 2. A steel sand box of 400 mm long, 800 mm high and 1580 mm wide was used. Dry silica sand #5 (D50=0.591 mm, G=2.647, e=1.072, εmin=0.689) was used for the model ground. The model ground was prepared with 80% of relative density by air pluviation method. The height of the model ground was 450 mm. The model caisson was 380 mm long, 300 mm high and 300 mm wide. Its weight was controlled to 752 N. Average contact pressure to the ground was 8.4 kN/m². The steel wall was designed considering the prototype steel pipe pile of 61000 mm × 112 mm. The steel wall is 380 mm long, 380 mm high and 3.2 mm thick with EI=5.6 × 10⁶ Nmm²/m. Embedment length of the steel wall was set to 280 mm. Strain gauges were attached to the steel wall to measure the bending moment of the wall (Kikuchi and Mizutani, 2003). Horizontal force was applied to the caisson with a loading rod. Friction between the rod and the caisson was reduced by attaching a Teflon sheet to the caisson. Friction between sand box wall and sand was reduced by grease and rubber membrane. Friction between sand and bottom of the caisson was controlled by an attached sand paper #150.

Table 2 shows the experiment cases in this series. The parameters considered in this series of experiments were with or without steel wall, fill or non-fill (i.e., without fill) when steel wall was used and distance between the caisson and the steel wall. Height of the fill was fixed at 50 mm.

Table 2. The details of experimental cases.

| Distance between the caisson and the steel wall | without fill | with fill |
|-----------------------------------------------|--------------|----------|
| (without steel wall)                          | Case-N-N     | Case-S-20|
| 20 mm                                         | Case-N-20    | Case-S-50|
| 50 mm                                         | Case-N-50    | Case-S-50|
| 100 mm                                        | Case-N-100   | Case-S-100|

3. EXPERIMENTAL RESULTS

3.1 Resistance and displacement of the caisson

Fig. 3 shows the relationship between horizontal load and displacement of the center of the caisson. In Case-N-N (i.e., no steel wall and no fill), horizontal load reached maximum within a small displacement. After that, the load remained same with displacement. This failure mode is considered as sliding failure mode. On contrast, in Case-N-50, relationship between load and displacement is almost the same as in Case-N-N up to the maximum load of Case-N-N, however, the load in Case-N-50 increased gradually with increase of the displacement. In Case-S-50, increment of horizontal load was much larger than that of Case-N-50. That is to say, resistance of caisson against horizontal load significantly increased with steel wall behind the caisson and filling between the wall and the caisson.

![Fig. 2. Schematic diagram of the experimental model set up.](image)

![Fig. 3. The relationship between horizontal load and horizontal displacement of the caisson.](image)

Horizontal load was applied to the caisson at the height of 195 mm from the bottom of the caisson. The loading was applied as displacement controlled with a loading rate of 0.6 mm/min. Load of the jack, displacement of the jack, displacement of the caisson, strains of the steel wall were measured during the loading.

Table 1. Similarity ratio (Iai, 1988).

| Physical parameter (λ=60) | model/prototype |
|---------------------------|-----------------|
| Density                   | 1               |
| Acceleration              | 1               |
| Ground strain             | 1/λ ¹/₂         |
| Time                      | 1/λ ³/₄         |
| Length                    | 1/λ             |
| Effective stress          | 1/λ             |
| Ground displacement       | 1/λ ⁴/₂         |
| Bending moment            | 1/λ             |
| Flexural rigidity         | 1/λ ⁷/₂         |
the caisson increased with steel wall insertion. Additional filling between the caisson and the steel wall improved the horizontal resistance of the caisson. In this case, distance between the caisson and the wall pile affected the resistance characteristics of the caisson.

Fig. 4 shows the relationship between horizontal load and rotational angle of the caisson. In Case-N-N, rotational angle of the caisson increased without increasing horizontal load as shown in Fig. 4. In cases with steel wall and fill, tendency of the rotation against the horizontal load was not affected by the distance between the caisson and the steel wall. In these cases, the rotational angle increased with increase of the load. In the case of without fill, the relationship between rotational angle and load was between that of without steel wall and that of with wall and fill.

Fig. 5 shows the relationship between the horizontal displacement of the center of the caisson and rotational angle of the caisson. The rotation of the caisson was larger at the same horizontal displacement with steel wall and fill than without wall or fill.

Fig. 6 shows the relationship between settlement of the back-toe of the caisson and the horizontal displacement of the center of the caisson. The settlement of the back-toe of the caisson was large in the cases with smaller distance between the wall and the caisson such as Case-S-20, Case-S-50, etc. As horizontal loads were larger compared to other cases in the same horizontal displacement in these cases, bearing failure mode of deformation of the ground could be observed.

Fig. 7 shows the ground deformation observed at 50 mm of the displacements of the center of the caisson in Case-N-N and Case-S-50. In Case-S-50, the ground beneath the caisson deformed in large area. On contrast, in Case-N-N, quite small deformation of the ground was observed. As shown in Fig. 7, failure mode of the
caisson changed from sliding mode to bearing failure mode with the steel wall and filling.

### 3.2 Deformation of the steel wall

Fig. 8 shows the bending moment distributions of Case-S-50 and Case-N-50. In these cases, distance between the caisson and the wall were 50 mm. Horizontal loads to the caisson in Case-S-50 were 1024, 1280, 1319 N at 20, 40, 60 mm of displacements of the center of the caisson, respectively. On contrast, horizontal loads to the caisson in Case-N-50 were 507, 630, 787 N at 20, 40, 60 mm of displacements of the center of the caisson, respectively.

In Case-N-50, deflection of the ground surface of the wall was 2 mm and the bending moment was observed at 20 mm of the displacement of the center of the caisson. At that time, horizontal load to the caisson was 100 N larger than that of Case-N-N. It means that even at that time, ground beneath the caisson deformed and the effect of steel wall made the resistance of the caisson to increase. With displacement of the caisson, bending moments of the wall increased and the resistance of the caisson increased due to the existing of the wall.

In Case-S-50, bending moments of the wall was large within small displacement of the caisson, and bending moments were observed above the ground surface. The fill between the caisson and the wall effectively transferred the load from the caisson to the wall.

In all the experiments, deflection of the wall at the ground surface in Case-S-50 was larger than that of Case-N-50 at the same displacement of the center of the caisson. It means that the wall could use the passive resistance of the ground more effectively in Case-S-50 than in Case-N-50.

Fig. 9 shows the bending moment distribution with the distance between the caisson and the wall. Fig. 9 shows that bending moments decrease when the distance between the caisson and the wall increases. Especially, as shown in Fig. 3, the horizontal load of Case-S-20 at 40 mm displacement of the center of the caisson was almost the same as those of Case-S-50 and Case-S-100 at 60 mm displacement of the center of the caisson. Even the horizontal loads were the same, the bending moments observed in the wall increases when the distance between the caisson and the wall was decreased.

As shown in Fig. 3, when the distance between the caisson and the wall increases, the resistance of the caisson decreases.

From these reasons, the best distance between the caisson and the wall, in which the caisson resists large horizontal load with smaller flexural rigidity wall, can be obtained.

### 4. ESTIMATION OF THE HORIZONTAL LOAD APPLIED TO THE STEEL WALL

Fig. 10 shows the subgrade reaction distribution which is given from the second order differentiation from the bending moment distribution shown in Figs. 8 and 9. The subgrade reaction shown in Fig. 10 is increment of the subgrade reaction from the initial subgrade reaction acting to the wall. These subgrade reactions are differential earth pressures acting on both sides of the wall.

One of the subgrade reaction models used in the horizontal resistance of the pile given in Eq. (1) is Kubo's equation (Shinohara, T. and Kubo, K., 1961).

\[
p = k_s \cdot x \cdot y^{0.5}
\]

Where \( p \) is subgrade reaction (kN/m²), \( k_s \) is coefficient of subgrade reaction (kN/m³), \( x \) is depth from the ground surface (m) and \( y \) is deflection at each depth \( x \) (m).

The subgrade reaction considered in Eq. (1) is the differential earth pressures acting on both sides of the wall. However, usually change in earth pressure in the back side of the pile is very small compared to the front
side of the pile. From this reason, Eq. (1) can be used to calculate change in the earth pressure of the front side of the wall.

![Deflection distribution from bending moment distribution](image1)

**Fig. 10** Calculated subgrade reaction distribution from bending moment distribution (Case-S-50).

Fig. 11 shows the deflections of the wall in Case-S-50. These deflections were calculated from bending moment distribution and boundary conditions measured at the wall top. The subgrade reaction from the ground when the wall deflects shown in Fig. 11 is assumed to be calculated from Eq. (1). The subgrade reaction shown in Fig. 10 is assumed to show the difference in subgrade reaction of the stress from the caisson and the subgrade reaction produced from the deflection of the wall.

![Deflection distribution from bending moment distribution](image2)

**Fig. 11** Deflection distribution calculated from bending moment distribution and boundary conditions (Case-S-50).

Fig. 12 shows the estimated stress from the effective outer stress in this experiment. The estimated stress was calculated from above assumption. In Fig. 12, positive stress shows the stress from the caisson side of the wall. The wall was stressed from the caisson side in shallow part of the wall. However, it was stressed from opposite side of the caisson in deep part of the wall.

Stress transferred by the fill was merely changed from 10 mm to 40 mm of the displacement of the center of the caisson. On contrast, stress from the caisson side greatly increased with the displacement of the center of the caisson. Stress acting on the wall was almost 0 at around -220 mm depth. This depth it seems to be a part of slip failure surface.

![Stress acting on the steel wall from caisson side ground](image3)

**Fig. 12** Deflection distribution calculated from bending moment distribution and boundary conditions (Case-S-50).

5. CONCLUSIONS

Considering the damages on breakwaters by tsunami, static horizontal loading experiment on caisson improved by steel wall and filling was conducted under dry state condition. The series of experiments showed the failure mode of the caisson changed from sliding failure to bearing failure mode when it is improved by steel wall and fill behind the caisson. By this improved method, horizontal resistance of the caisson increases with displacement of the caisson.

Finally, the stress actin on the steel wall from the caisson was estimated with the change in subgrade reaction around the ground.

ACKNOWLEDGEMENTS

The authors acknowledge Mr. Mizuno, a previous graduate student in Tokyo University of Science, Japan, and Mr. Oikawa, an engineer of Nippon Steel Sumitomo Metal Corporation, Japan, for their efforts in conducting experiments and analysis.

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