

Experimental and FEM Simulation of the Dynamic Response of a Caisson Wall Against Breaking Wave Impulsive Pressures

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Abstract

After presenting a summary of field survey-observed damage incurred over the past 20 years by caisson walls of Japanese breakwaters, this study describes a series of model experiments and three-dimensional FEM simulations performed to evaluate the dynamic response of a caisson wall subjected to breaking wave impulsive pressure. The important roles played by "elastic" and "inertia" soil pressures are subsequently clarified, and good agreement is obtained between measured and calculated results. The proposed calculation method, which applies the Goda pressure formulae in conjunction with the impulsive pressure coefficient α_i , is expected to provide a practical design method against impulsive pressures.

1. INTRODUCTION

Caisson walls seldom fail even when acted upon by impulsive wave pressures that are large relative to normal design wave pressures, *i.e.*, the strain in the wall is reduced by the static and dynamic response of the filling soil and water contained within the caisson chambers. While these responses tend to stabilize a caisson, they have not yet been specifically considered in the design evaluation process. Moreover, if the impulsive pressures are quite large and act at very high frequency, a caisson breakwater can suffer failure (Tanimoto *et al.*, 1975).

Such effects led to the present study whose main objective is to establish a design method against wave pressures, especially impulsive ones, such that wave action failures can be prevented. We consider a caisson wall containing filling sand in the caisson chambers, and focus our attention on its dynamic response, which includes both "inertia" and "elastic" effects, against impulsive wave pressure.

After conducting a field survey of Japanese breakwaters—to determine damage

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that has occurred in caisson walls over the past 20 years—a box-shape model caisson was used to perform a series of experiments in which soil pressures and wall strain were measured, *i.e.*, impulsive pressures were applied after filling the model caisson with sand and water or only with water. We also carried out three-dimensional FEM calculations to simulate the strains generated by impulsive wave pressures, and then compared measured and calculated results such that the applicability of the simulations could be evaluated. Finally, the employed FEM simulation is used to investigate the failure of caisson walls of a prototype built at Mutsu-Ogawara Port.

2. CAISSON WALL FAILURES

Current Design Method of Caisson Wall against Wave Actions

Figure 1 shows a diagram of a breakwater caisson, *i.e.*, a large, reinforced concrete box in which chambers separated by inner walls are filled with sand and water. The outer and inner walls have a thickness of 40–60 and 20–30 cm, respectively. The caisson's front wall is designed against wave pressures, static soil pressure of filling sand, and static water pressure (Fig. 2). The wave pressure distribution at the wave crest is determined by the Goda pressure formulae (Goda, 1974), while that of the wave trough by the relation $0.5 w_0 H$.

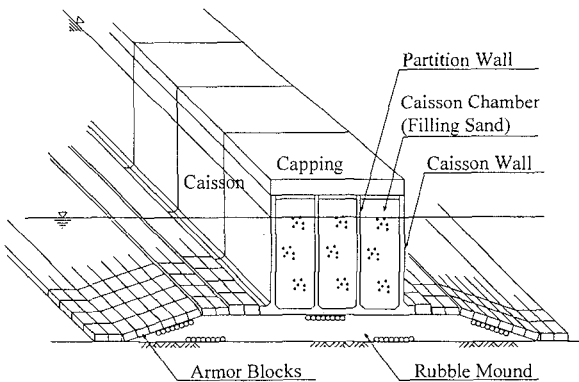


Fig. 1 Diagram of caisson breakwater.

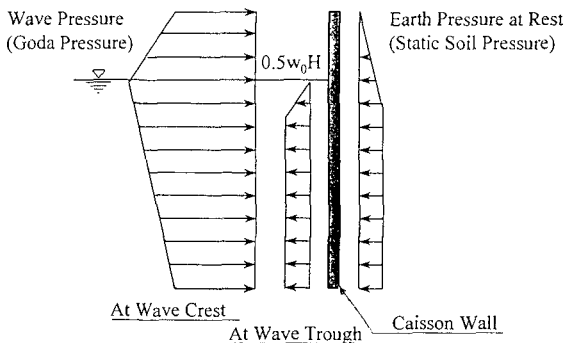


Fig. 2 Current design method.

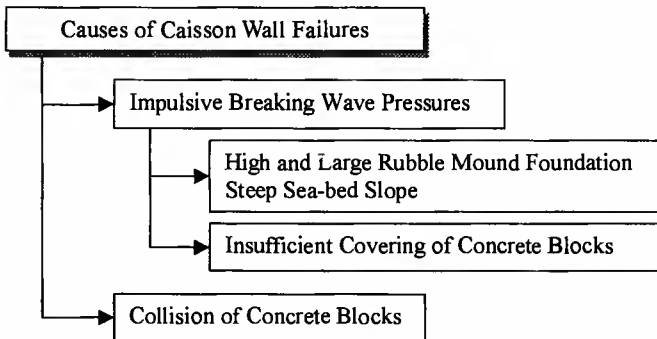
Table 1 Summary of caisson wall failures over past 20 years.

PORT	TYPE OF BREAK WATER	DATE OF FAILURE	FAILURE FEATURES OF CAISSON WALL	CAUSE OF FAILURE
MASHIKE	Composite type	1977/04	Caisson wall damage (cracks) filling sand flow-out	Impulsive wave pressure due to abrupt water depth change
KUSHIRO	Horizontally composite type	1981/08	1m hole on wall	Collision of blocks at the end of covering blocks
ONAHAMA	Composite type	1981/08	Caisson wall damage (cracks) filling sand flow-out	Impulsive wave pressure due to high rubble mound (under construction)
OMAEZAKI	Horizontally composite type	1981/08	1m hole (not penetrated)	Collision of blocks at the end of covering blocks (under construction)
SINNGU	Composite type	1982/09	Caisson wall damage (cracks) filling sand flow-out	Impulsive wave pressure due to abrupt water depth change
A	Horizontally composite type	1987/02	1 caisson almost destroyed	Impulsive wave pressure due to insufficient block-covering (under construction)
B	Horizontally composite type	1987/12	Cracks and 3m hole, and Concrete crown damage	Impulsive wave pressure at the end of covering blocks around breakwater head
KATADOMARI	Composite type	1987/08	1 caisson almost destroyed	Impulsive wave pressure due to the settlement and scattering of covering blocks
UDONO	Composite type	1990/11	Caisson wall damage (Cracks and holes)	Impulsive wave pressure due to high rubble mound caused by sea bottom profile change
KASHIMA	Composite type	1990/11	1 caisson almost destroyed	Impulsive wave pressure due to high rubble mound
MUTSU-OGAWARA	Horizontally composite type	1991/02	1 caisson almost destroyed, 4 caissons damaged	Impulsive wave pressure due to end of covering wave dissipating blocks
OMOTO	Horizontally composite type	1991/02	3 caissons damaged	Impulsive wave pressure due to settlement and scattering of covering blocks
MINAMINOHAMA	Composite type (Jetty)	1991/09	1 caisson almost destroyed	Impulsive wave pressure due to steep sea bottom, Failure of breakwater head caisson
K	Horizontally composite type	1996/09	Holes on walls in several caissons	Collision of covering blocks
H	Horizontally composite type	-1996	Holes and cracks on 18 caissons in 129 caissons	Collision of covering blocks
M	Horizontally composite type	96/8, 97/9	Holes on walls in 3 caissons	Collision of covering blocks
W	Composite type	1997/08	1 caisson almost destroyed,	Impulsive wave pressure due to steep sea bottom, Failure of breakwater head caisson

Caisson Wall Failures in Japan

Table 1 presents a summary of field survey-observed damages incurred over the past 20 years by caisson walls of Japanese breakwaters (Hattori *et al.*, 1984; Miyai *et al.*, 1993), where 12 cases due to impulsive wave pressure are indicated. Although failures at fishery ports are not included, the number of failures is still quite small relative to the total number of caissons in Japanese ports ($> 16,000$). Table 2 summarizes the main reasons for caisson wall failures, with each reason being discussed next using a typical case.

Table 2 Summary of main reasons for caisson wall failures.



Udono Port Case

Photo 1 shows a caisson wall failure at Udono Port, being damage caused by impulsive breaking wave pressures produced due to the presence of a high, large rubble mound foundation. Note that four caissons slid and one suffered caisson wall damage. Such failures seldom occur nowadays because the generation of impulsive pressure due to this type of rubble mound is common knowledge among breakwater design engineers. In this particular failure, the mound was not originally high, but due to substantial movement of a sandbed situated near the breakwater, the water depth in front of the breakwater increased such that the relative height of the mound became high.



Photo 1 Damaged caisson wall.

Mutsu-Ogawara Port Case

Impulsive pressures can also be generated when caisson walls are not sufficiently covered by concrete blocks. This situation especially applies to horizontally composite type breakwaters, where the caisson is covered by concrete blocks that dissipate wave energy. Unfortunately, the covering can sometimes become insufficient due to settlement or scattering of the blocks, or it might be insufficient at

the transition part from the horizontally composite type to ordinary composite type.

Photo 2 shows damage to a breakwater at Mutsu-Ogawara Port following a severe storm equivalent to its design conditions (Hitachi, 1994). Caisson No. 7 was nearly destroyed by impulsive pressures from waves breaking on concrete blocks which were scattered and settled.

Figure 3 shows the breakwater's plane view and cross section. While the upper part of caisson No. 7 was completely destroyed, caisson No. 8 moved only 0.4 m and its wall was not damaged. As caisson No.8 was designed to be located at the transition part, such impulsive pressures were considered in the design. Caisson No. 7 was covered sufficiently by blocks, but due to scattering the transition part was extended such that it reached caisson No. 7. In fact, it had a wall thickness of 70 cm compared to that of No. 7 which was only 45 cm. As will be discussed later, a thicker wall led to differences in resulting damage.



Photo 2 Damaged caisson No. 7.

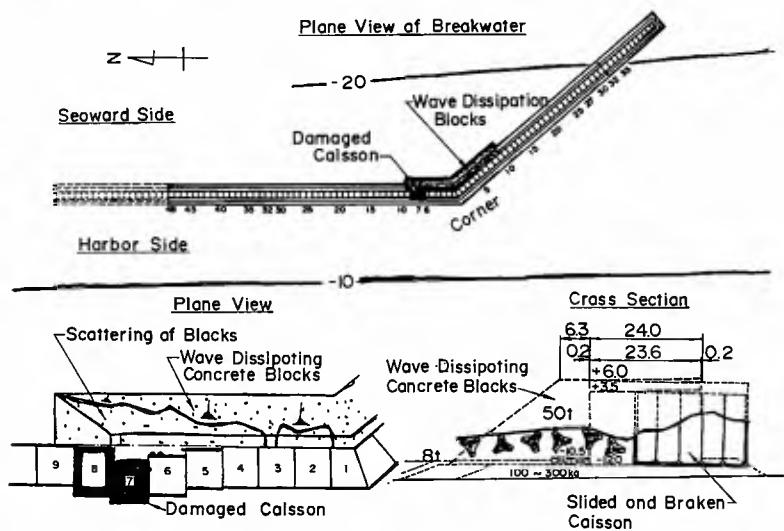


Fig. 3 Plain view and cross section of the breakwater.

Collision with Concrete Blocks

Photo 3 shows a horizontally composite type breakwater whose caisson wall suffered damage due to concrete blocks colliding with it. When a breakwater is located in rough seas, the blocks are usually large and can hit the wall making holes due to shear failure of the wall. Since impulsive breaking wave pressures are typically responsible for producing damage, here we focus our attention on this type of failure and do not consider in detail those due to collisions with concrete blocks.

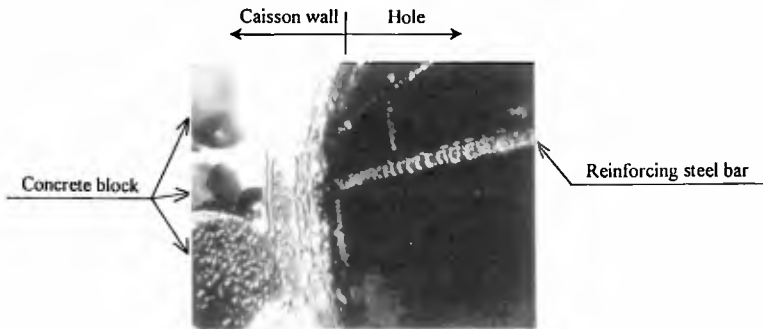


Photo 3 Damaged caisson.

3. MODEL EXPERIMENTS

Experimental Setup

Figure 4 shows a cross section of the box-shape model caisson and locations of instrumentation for measuring pressure, strain, and acceleration. The walls are made from acrylic plates, with the front wall having a thickness of either 10 or 15 mm. Regular waves were applied. The shallow water concrete blocks situated directly in front of the caisson generated wave breaking such that impulsive wave pressures impact the front wall. Note that in order to clearly see the effect of soil pressures due to wall acceleration and the resultant wall deformation, we deliberately used a single

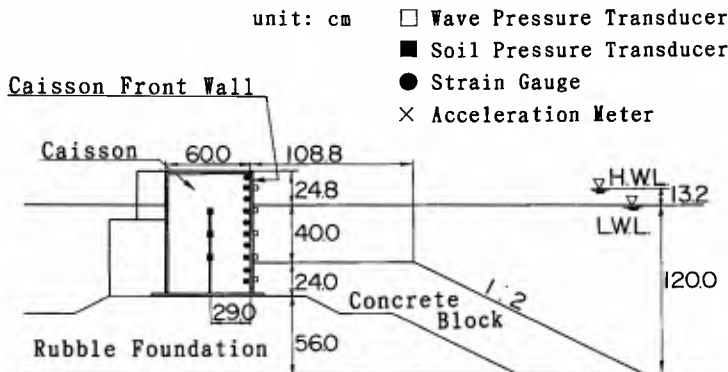


Fig. 4 Experimental setup and instrument locations.

caisson model in which the width of the wall is quite long compared to that of a standard caisson.

The natural frequency of the model caisson wall was measured to be 38.5 Hz in air but only 6.3 Hz in water with filling sand present; values that are nearly the same as theoretical ones. The reduction in frequency is due to the added mass effect of water and sand surrounding the wall.

Elastic Soil Pressure and Inertia Soil Pressure

We consider two types of internal soil pressure:

- (1) elastic soil pressure due to soil acceleration and
- (2) inertia soil pressure due to soil deflection.

The former is mainly discussed based on experiments using non-breaking waves, while the latter using impulsive breaking waves. Static soil pressure, *i.e.*, earth pressure at rest, is also considered.

Caisson Wall Strain Against Non-Breaking Waves

Figure 5 shows the strain and internal soil pressure generated in a sand-filled caisson when a non-breaking wave hits the front wall. Since the internal soil pressure is nearly proportional to wall displacement, we term this as "elastic" soil pressure.

The wave pressure and elastic soil pressure are compared in Fig. 6, where the elastic soil pressure shows a peak value $\approx 70\%$ of the standing wave pressure. This elastic soil pressure assuredly reduces the strain significantly, and therefore resultant pressure ($p-q$) acting on the wall is greatly reduced.

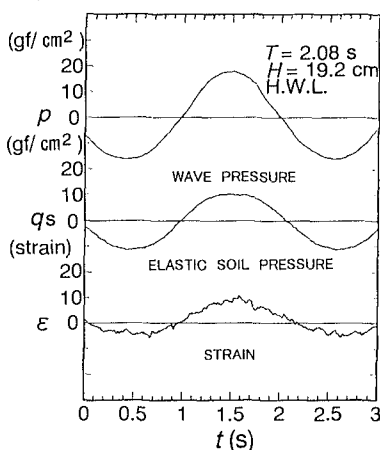


Fig. 5 Caisson wall strain due to non-breaking waves.

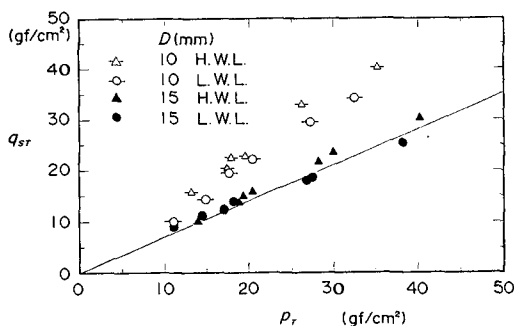


Fig. 6 Soil pressure vs. wave pressure.

Dynamic Response of Caisson Wall Against Impulsive Breaking Waves

Figure 7 shows the soil pressure q , strain ε , and acceleration α after a wave with impulsive pressure p impacts the front wall. The resultant pressure acting on the wall ($p-q$) is also shown. Acceleration data indicates that the wall vibrates strongly, and that the soil pressure due to the "added mass effect" is proportional to acceleration. We term the internal soil pressure due to the added mass effect as "inertia" soil pressure, q_{sp} . The dynamic response of the front wall and associated pressures can be described using two phases (Fig. 7), namely, phases I and II represented in Fig. 8. In phase I, p shows a positive peak coinciding with positive peaks in α and q ; and, due to q_{sp} , ($p-q$) and accordingly ε show no peak. In Phase II, ε and ($p-q$) contrastively show nearly coincident peaks while α shows a negative peak. Note that even though q is reduced below its peak value, ε and ($p-q$) nevertheless increase due to the negative inertia soil pressure. However, due to the coincidently acting positive elastic soil pressure, ($p-q$) decreases. In other words, in this case the dynamic response of the caisson wall reduces the peak in ε by about 50%.

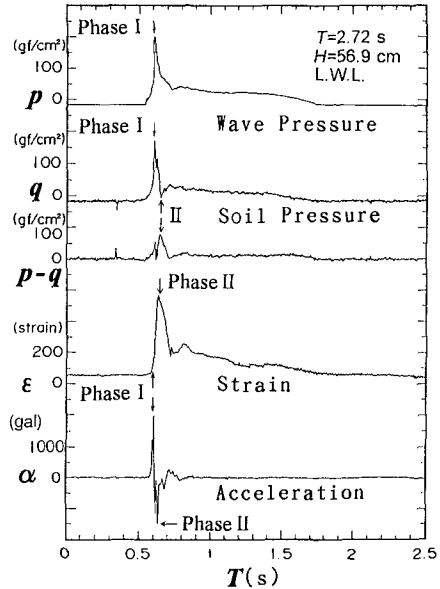


Fig. 7 Dynamic response of a caisson wall against impulsive breaking waves.

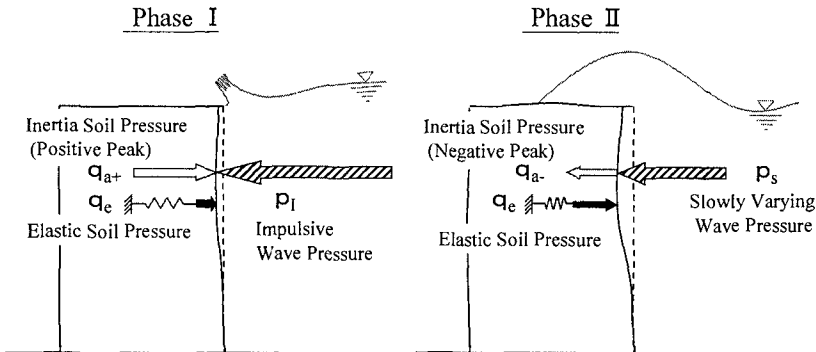


Fig. 8 Diagrammatic representation of the dynamic response.

Figure 9 shows the peaks of impulsive wave pressure and inertia soil pressure in Phase I, and Fig. 10 shows the peaks of wave pressure and wall strain. The inertia soil pressure is 70 – 90% of the impact wave pressure, which indicates that the strain at Phase I is greatly reduced due to soil pressure, making the strain at Phase II more

significant. By comparing the peak strains for the sand-filled and water-filled caissons due to the soil pressure the peak strain is reduced 40 ~ 80%.

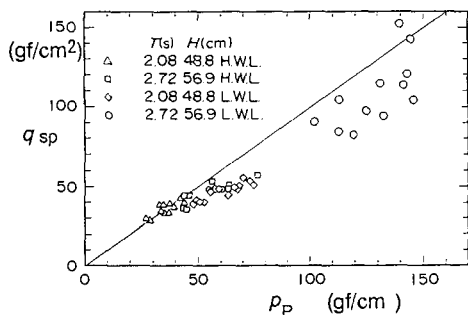


Fig. 9 Impulsive pressure vs. inertia soil pressure.

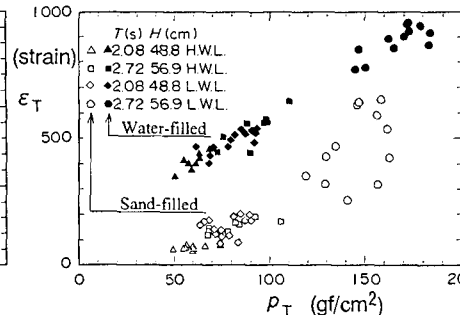


Fig. 10 Impulsive pressure vs. wall strain.

4. FEM SIMULATION

FEM Model

Figure 11 shows the mesh system used to carry out time-dependent, three-dimensional FEM numerical calculations simulating the dynamic response of a caisson wall.

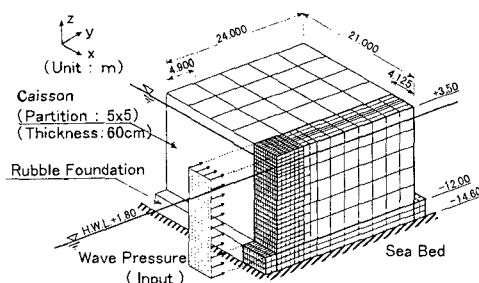


Fig. 11 Mesh system used for FEM simulation.

Experiments vs. Calculations

Figure 12 compares measured and calculated profiles of p , α , q , and ϵ where the apparent good agreement between them indicates that employed FEM simulation is suitable for approximating the caisson's dynamic response.

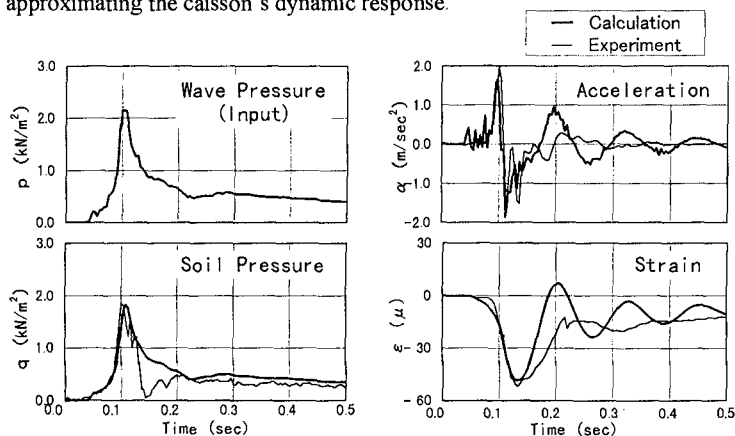


Fig. 12 Measured vs. calculated results.

Dynamic Response of a Prototype Caisson Wall

Figure 13 shows simulated profiles for a prototype caisson (height, 15.5 m; chamber width, 4.125 m; wall thickness, 60 cm) located at Mutsu-Ogawara Port, where results are indicated for an input wave pressure profile with an impact duration τ of 0.01 or 0.06 s. At $\tau = 0.01$ s, the inertia effect is the same as that of the model caisson, *i.e.*, it delays the peak in wall displacement and reduces its value by about 30%. At $\tau = 0.06$ s, however, the wall displacement peak almost coincides with the peak in wave pressure, with its peak value being the same as the static value. In other words, the effect of the inertia soil pressure is limited only when the impact duration is short.

It should be noted, however, that wall deformation δ_{st} includes the effect of elastic soil pressure, which is about one third of input wave pressure; and therefore the resultant wall deformation is in turn reduced by one third.

Figure 14 shows the effect of τ on the inertia effect for a wall with filling sand and water, *i.e.*, the caisson's internal soil pressure, displacement, and deflection

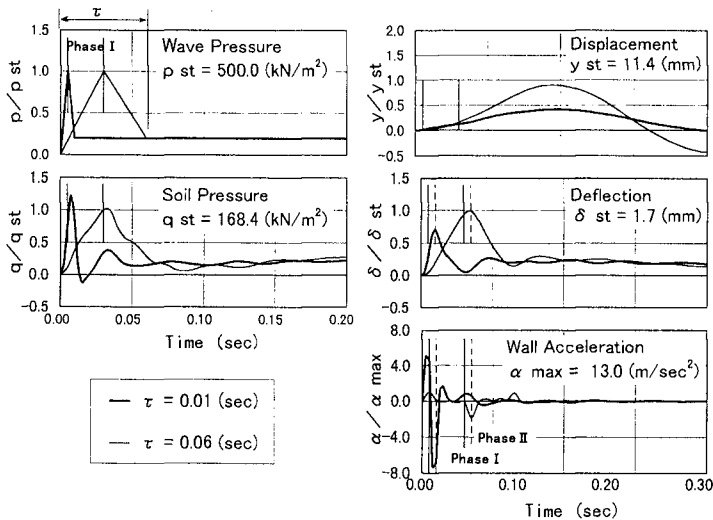


Fig. 13 Dynamic response of a prototype caisson wall.

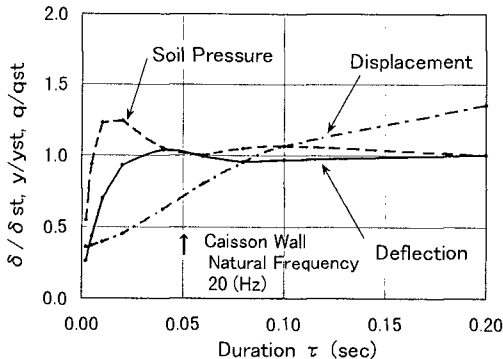


Fig. 14 Duration and inertia effect.

which are all non-dimensionalized by static values. As can be seen, the reduction in deflection, namely strain, due to the inertia effect appears when the duration of impact is much less than the wall's natural period (20 Hz).

The natural frequency of the wall in air is 40 Hz, whereas in water it is 20 Hz for a sand-filled caisson; being slightly higher than standard frequencies because the considered wall thickness of 60 cm is slightly larger than the standard value. These results indicate that the reduction in strain due to the dynamic response, especially that due to the inertia effect, appears when τ is short, *i.e.*, around 40 ms.

Rubble Mound Stiffness and Dynamic Response

The above simulation considered a relatively stiff rubble mound foundation that is lower and wider than standard ones. Therefore, to determine if rubble mound stiffness affects the dynamic response, the value of stiffness was reduced to 1/10th the original value. Figure 15 compares the simulations, where it should be noted that no marked differences appear during the duration of impact, which results clearly indicate that the dynamic response effect of the caisson wall is not affected by the rubble mound foundation.

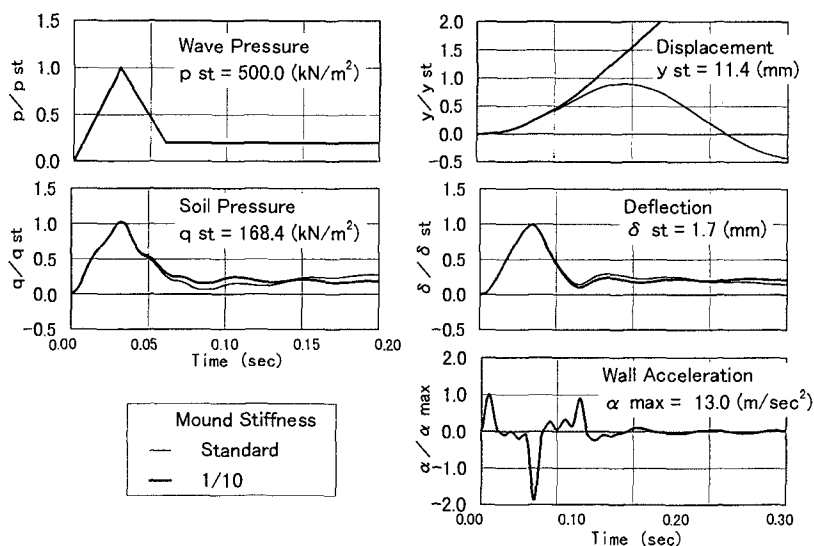


Fig. 15 Effect of stiffness of rubble mound on dynamic response.

Bending Moment

Figure 16 shows the horizontal and vertical bending moments of the caisson wall with $\tau = 0.06$ s, where the sand filling markedly reduces both bending moments by $\approx 30\%$. Also shown are corresponding profiles calculated using the conventional design method in which a reinforced concrete plate is fixed along three edges and no filling sand is considered. Note that the horizontal moment is overestimated and the vertical moment underestimated, especially around S.W.L. where huge impulsive wave pressure are present.

Figure 17 shows the effect of wall thickness and partition distance on wall compressive stress due to bending moment, where these results clarify that increasing wall thickness is much more effective than decreasing partition distance.

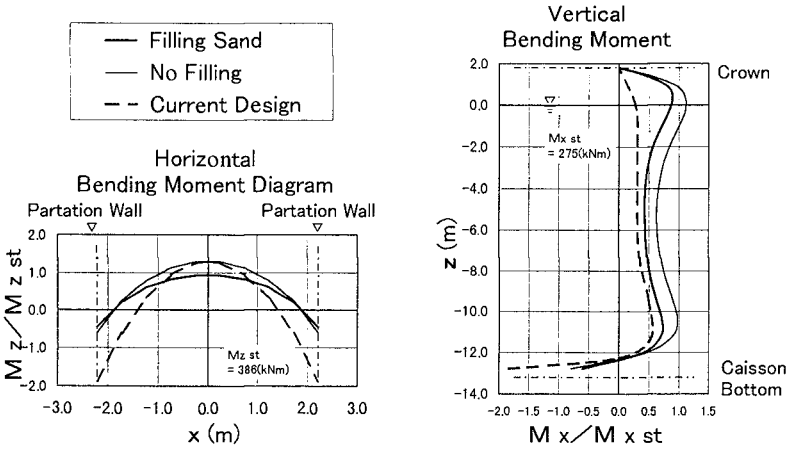


Fig. 16 Effect of filling sand on bending moment.

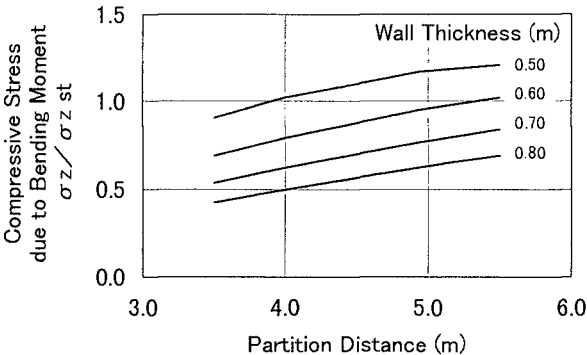


Fig. 17 Effect of wall thickness and partition distance on wall compressive stress.

Mutsu-Ogawara Case

Table 3 summarizes the results of applying our 3D, time-dependent FEM model to simulate the actual effects of a strong, 1991 winter storm on the No. 7 (damaged) and adjoining No. 8 (undamaged) caissons of the Mutsu-Ogawara breakwater. From previous work (Takahashi *et al.*, 1994), if we apply the modified Goda formulae (Goda, 1974) assuming an impulsive pressure coefficient of α_i , then the predicted value of the impulsive wave pressure impacting the breakwater would be about $2.8 w_0 H$, *i.e.*, since maximum incident wave height during the storm was 14.88 m, the breakwater was hit by an impulsive pressure of 500 kN/m².

As indicated, the 45-cm-thick wall of the No. 7 caisson was subjected to a bending moment of 315 kNm, which is much higher than the allowable value of 240 kNm. On the other hand, the 70-cm-thick wall of the No. 8 caisson was subjected to a resultant bending moment of 421 kNm, which is much lower than the allowable bending moment of 540 kNm. Such correspondence with field survey observations points towards the suitability of using our model to establish a practical design method against impulsive pressures.

Table 3 FEM simulation results applied to the caisson walls at Mutsu-Ogawara Port.

Caisson No.	No.7	No.8
Thickness of Caisson Wall (m)	0.45	0.70
Allowable Bending Moment (kNm)	240	540
Calculated Bending Moment (kNm)	315	421
Judgement	Failure	No Failure

* Input

Maximum wave height : Hmax = 14.88 (m)

Impulsive wave pressure coefficient : $\alpha_i = 1.10$

Maximum wave pressure : $p = 2.8w_0H$

5. CONCLUSIONS

Main results are summarized as follows:

- 1) Although failures of caisson walls rarely occur, they are possible if the impulsive pressure is quite large and the thickness of the caisson wall is insufficient.
- 2) Model experiments successfully demonstrated the dynamic response of the caisson wall against impulsive breaking wave pressures.
- 3) Based on measured and calculated results showing good agreement, the employed 3D, time-dependent FEM model is considered to effectively simulate the dynamic response of a wall of a sand/water-filled caisson.
- 4) Internal soil pressures, namely, elastic and inertia soil pressures, play important roles in reducing the strain in a caisson wall, *i.e.*, the strain in the caisson wall due to the elastic soil pressure is dependent on wall stiffness and can reduce the bending moment by as much as 30%.
- 5) Wall strain due to the inertia effect appears when the impact duration time of

impulsive pressure is shorter than the natural period of the wall, *i.e.*, around 40 ms.

- 6) The wall's dynamic response is not affected by the stiffness of the rubble mound foundation.
- 7) Our simulation model is expected to provide a good foundation for continued analysis of caisson wall failures such that a practical design method can be effectively established against impulsive pressures. The ultimate design method will be based on applying the Goda pressure formulae (Goda, 1974) in conjunction with the impulsive pressure coefficient α_i (Takahashi *et al.*, 1994).

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