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ANALYSIS OF DAMAGE TO QUAY WALLS DURING 1995 GREAT HANSHIN EARTHQUAKE, JAPAN

SUSUMU IAI, KOJI ICHII AND TOSHIKAZU MORITA

Port and Harbour Research Institute, Ministry of Transport
Nagase 3-1-1, Yokosuka, 239 Japan

ABSTRACT

Many caisson type quay walls were damaged in Kobe Port during the 1995 Great Hanshin earthquake. The caisson walls moved seaward about 5 m maximum and 3 m average, settled about 1 to 2 m, and tilted about 4 degrees toward the sea. Based on the results of the geotechnical investigations including in-situ soil freezing sampling and cyclic triaxial tests, effective stress analysis was performed to identify the mechanism of deformation in the caisson wall. The results of the analysis were consistent with the observed deformation of the caisson wall, which tilted into and pushed out the rubble mound underneath the caisson toward the sea. The results of the analysis suggested that the excess pore water pressure increase in the foundation soil and the backfill increased the deformation of the caisson walls about twice as large as that purely caused by the seismic inertia force. Despite the large vertical component of earthquake motion (about 0.5g), the analysis suggested that the vertical input motion had minor effect on the performance of the caisson walls.

KEYWORDS

Case history; caisson type quay walls; effective stress analysis; earthquake damage; earthquake resistant; effective stress analysis; liquefaction; sandy soil; soil-structure interaction

INTRODUCTION

Caisson type quay walls maintain their stability by the friction at the bottom of the caisson in order to resist seismic inertia force and earth pressures acting on the wall. About 90 percent of quay walls at Kobe Port were caisson type. A typical cross section is shown in Fig. 1. Since the natural soil deposit in Kobe Port was a soft clayey layer about 10 to 20 m thick, the foundation soil beneath the caisson wall was replaced with sandy soil to achieve enough bearing capacity. The same sandy soil was used for land fill behind the wall. During the 1995 Great Hanshin earthquake, these caisson walls were shaken by a strong earthquake motion having the peak accelerations of 0.54g and 0.45g in the horizontal and vertical directions and the peak velocities of 122 and 34 cm/s, respectively. In addition, there was extensive evidence of liquefaction of landfill soils. Most of the caisson walls in Kobe Port displaced toward sea about 5 m maximum, about 3 m average, settled about 1 to 2 m, and tilted about 4 degrees toward the sea as shown in Fig. 2. About the same order of settlements were induced in the backfill soils behind the walls.

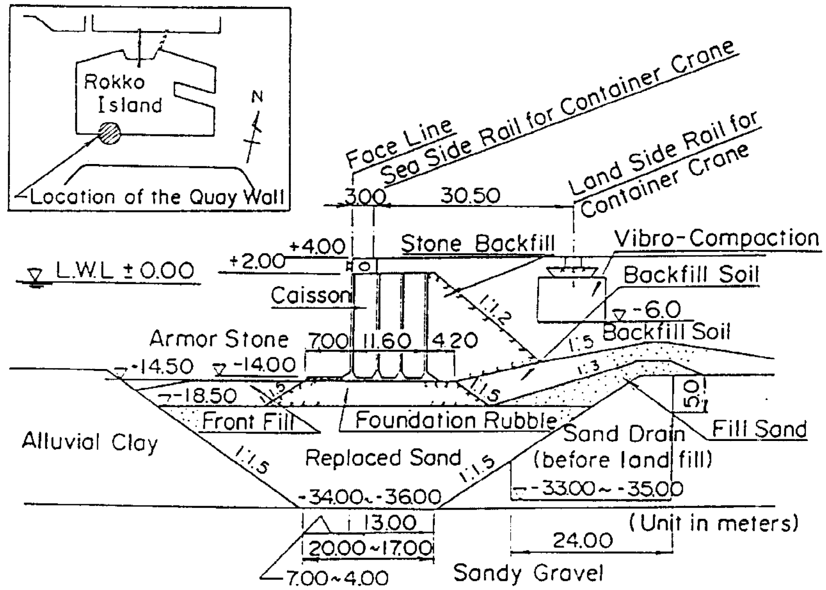

Fig. 1 Cross section of a caisson type quay wall at Kobe Port

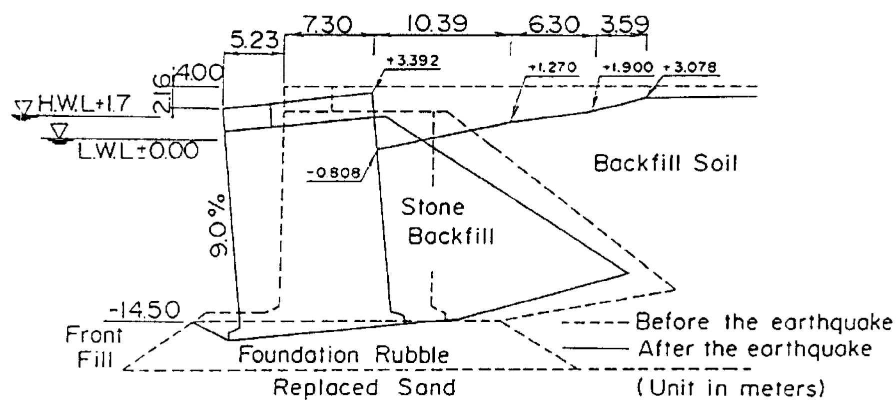

Fig. 2 Deformation of a quay wall at Kobe Port

Based on the results of the geotechnical investigations including in-situ freezing sampling of sandy soils and in-situ velocity measurements, effective stress analysis was performed in this study to identify the mechanism of the deformation of the caisson type quay walls.

EFFECTIVE STRESS ANALYSIS

The effective stress model used for the present study was a multiple mechanism model defined in strain space (Iai *et al.*, 1992a). The model has a capability to simulate the behaviour of sand during rotation of principal stress axis, which plays an important role in the behaviour of initially anisotropically consolidated sand under cyclic loading (Iai *et al.*, 1992b; Iai *et al.*, 1994). The model parameters were determined by referring to the in-situ velocity measurements and the cyclic triaxial test results of the in-situ frozen samples. In particular, liquefaction resistance evaluated by the laboratory tests and back-fitted curve by the effective stress model are shown by solid marks and lines in Fig. 3.

The finite element mesh shown in Fig. 4 was used for the analysis under plane strain and undrained conditions. Joint elements were used to simulate the friction type slide along the concrete surfaces at back and bottom of the caisson. At the side end boundaries of the cross sectional region used for the finite element analysis, free field response motions were given through the transmitting boundaries to approximate in-coming and out-going waves to and from the finite element region. At the bottom boundary, the earthquake motion recorded at a depth of 32 m at Port Island, shown in Fig. 5, was used as the bedrock motion. Before the earthquake response analysis, a static analysis was performed to simulate the stress conditions before the earthquake to take the effect of gravity into account.

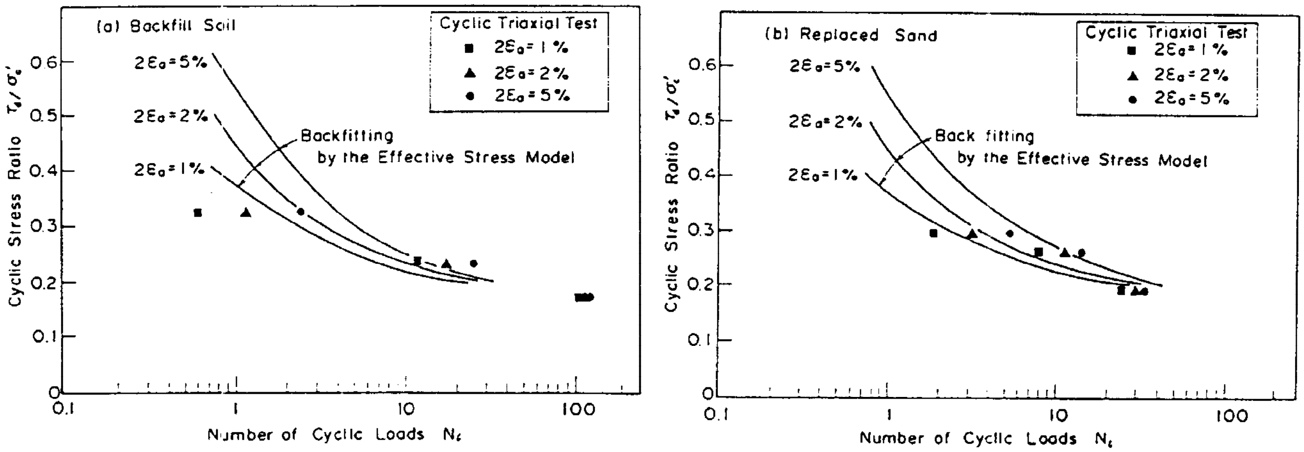

Fig. 3 Cyclic triaxial test results of in-situ freezing sample and back-fitted curve by effective stress model

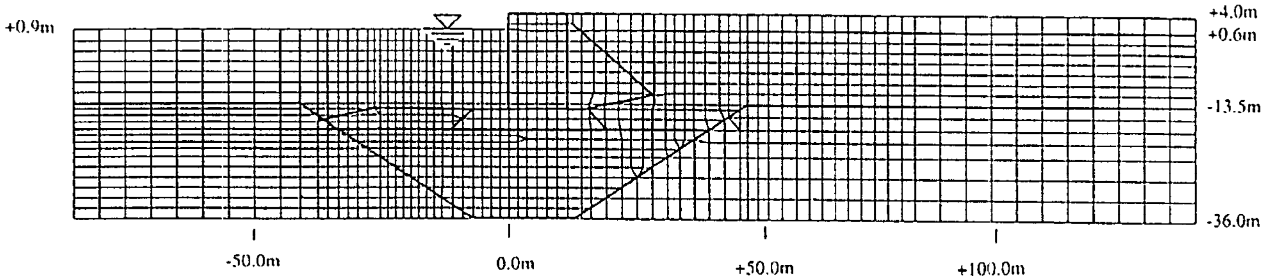

Fig. 4 Finite element mesh for the analysis

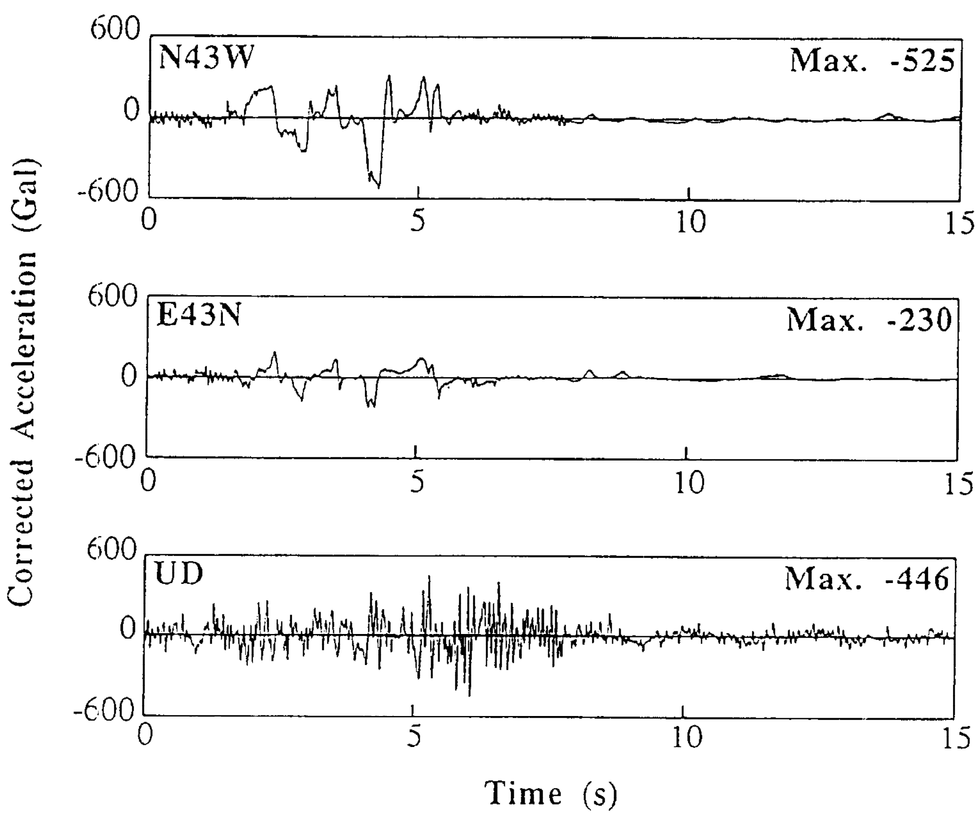

Fig. 5 Earthquake motion at 1995 Great Hanshin earthquake recorded at GL -32 m at Port Island

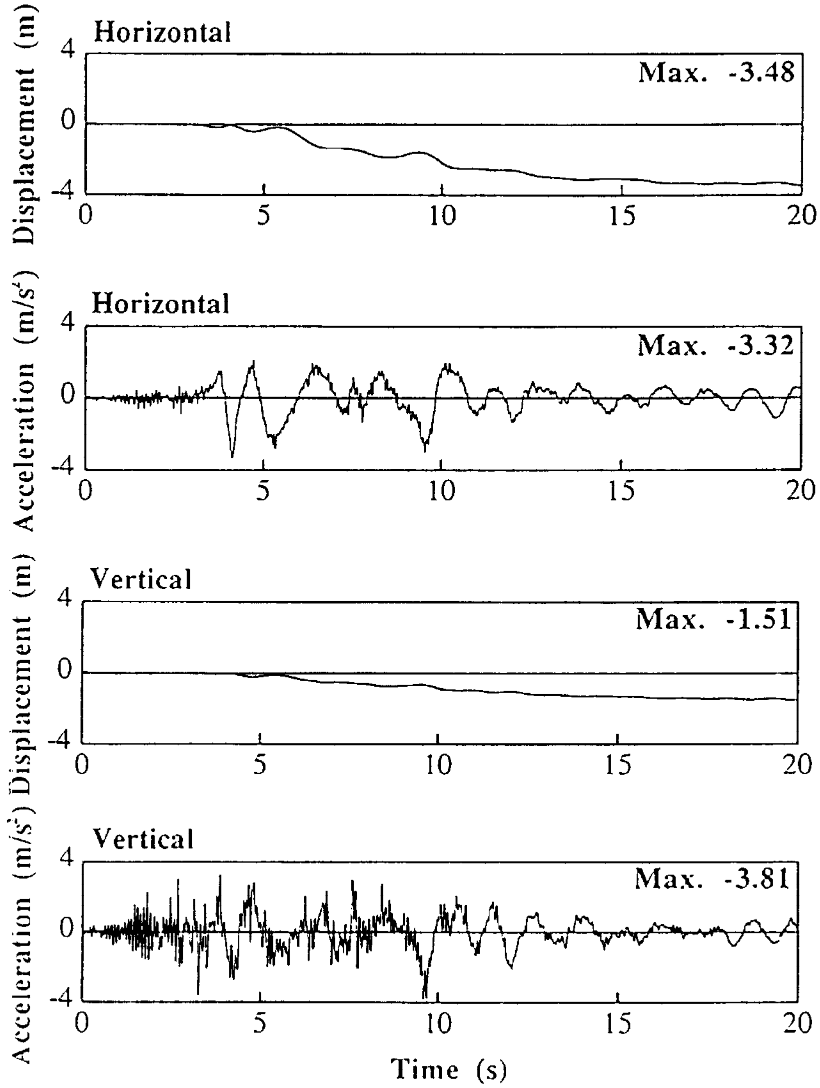

Fig. 6 Computed accelerations and displacements at the upper seaside corner of the caisson

RESULTS OF THE ANALYSIS

The results of the effective stress analysis showed that the displacements of the caisson wall were gradually increased for about 10 seconds to cease at the residual displacements as shown in Fig. 6. The residual displacements were 3.5 m and 1.5 m in the horizontal and vertical directions with tilting of 4 degrees toward sea as shown in Fig. 7. The order of magnitude of these results is consistent with the observed deformation of the caisson walls mentioned earlier and shown in Fig. 2.

The mode of deformation of the caisson walls as seen in Fig. 7 was to tilt into and push out the foundation rubble beneath the caisson. This is also consistent with the actual mode of the deformation identified by the investigation in the sea after the earthquake shown in Fig. 8. The computed results in Fig. 7 also indicated that significant displacements were induced over wide cross sectional area in the sand replacement beneath the caisson. This mode of deformation of the quay wall is quite different from those caused by sliding along the bottom of the caisson often assumed for a simplified sliding block analysis.

To evaluate the degree of liquefaction in the soils beneath and behind the caisson walls, the excess pore water pressure ratios were also computed and specified in terms of $(1 - \sigma'_m/\sigma'_{mo})$, where σ'_m, σ'_{mo} = current and initial mean effective stresses. Excess pore water pressures were gradually increased for about 10 seconds as shown in Fig. 9. As shown in Fig. 9(a), the excess pore water pressure ratio beneath the caisson was less than 0.8. This is considered due to large initial deviator stresses applied due to the dead load of the caisson and earth pressures from behind the caisson.

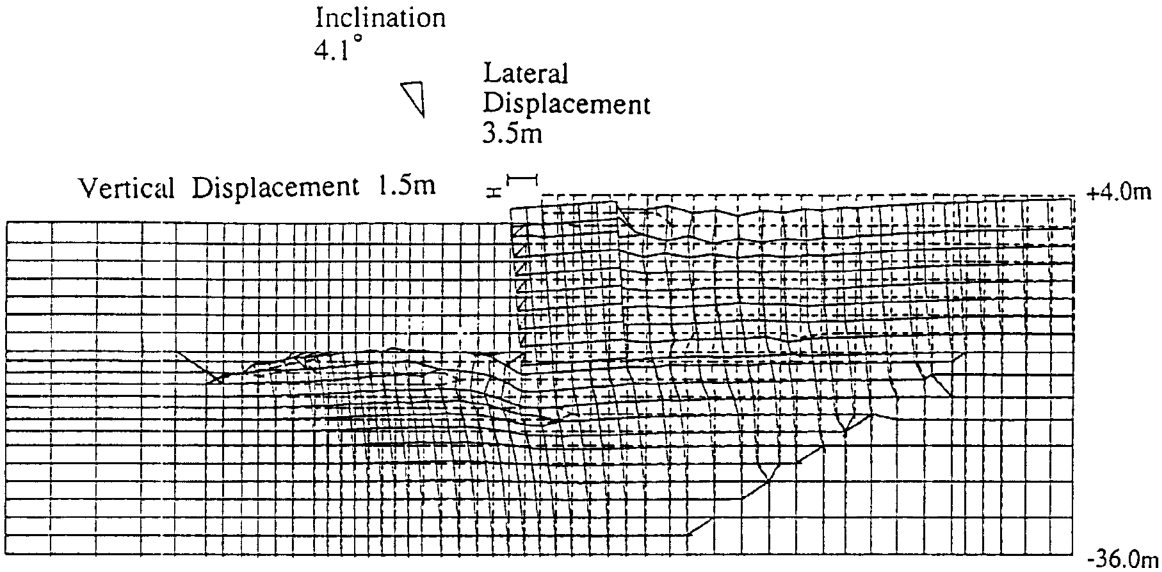

Fig. 7 Computed residual displacements of the quay wall

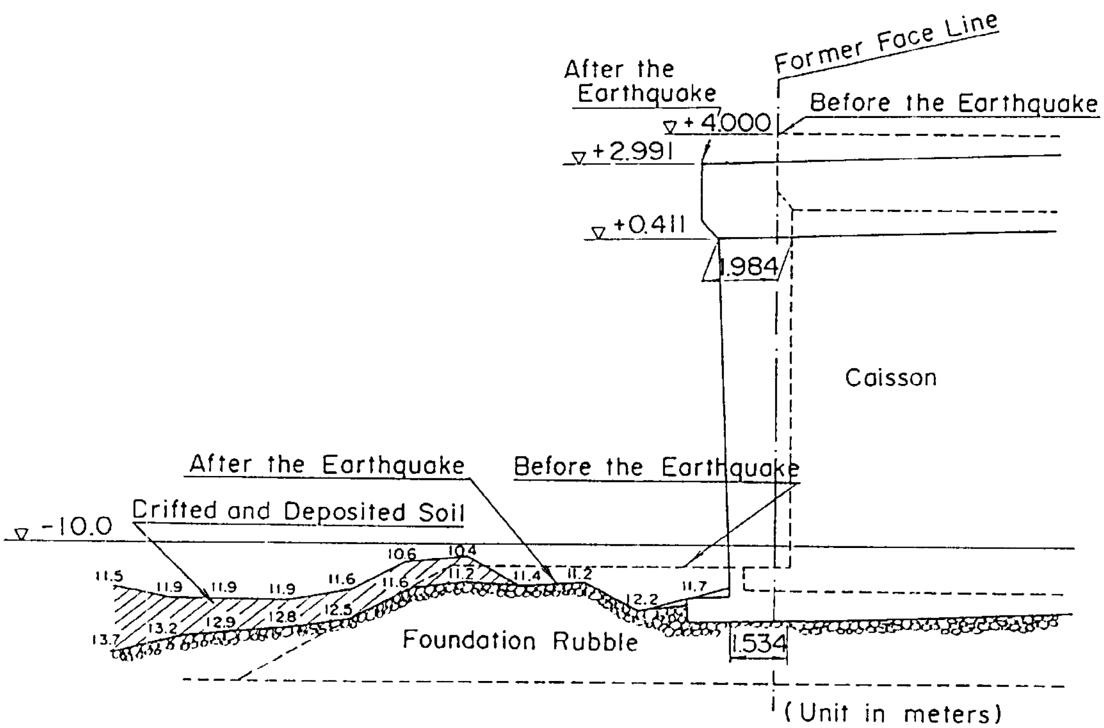

Fig. 8 Deformation of rubble mound beneath the caisson

It is also noted in Fig. 9(b) that the portion of the soil immediately behind the caisson wall had the excess pore water pressure ratios increase close to 0.9 but then decrease to less than about 0.5 during shaking. This may be due to the seaward displacement of the wall because of large inertia force and excess pore water pressure increase in the foundation soil; the wall dragged out the backfill soil, leading to reduction in the excess pore water pressures behind the caisson wall. This is also consistent with the general observation in Kobe Port that there were lack of sand boils in the immediate vicinity of the caisson walls. As shown Fig. 9(c), the excess pore water pressures further inland reached about 100 % excess pore water pressure ratio. This is also consistent with the evidence of liquefaction in inland portion of reclaimed land.

In summary, the sand replacement foundation and the part of the backfill soil in the vicinity of the caisson wall did not achieve the excess pore water pressure ratio beyond 0.8 but their excess pore water pressures have certainly increased during the earthquake. The deformation induced throughout the foundation soils indicates that the excess pore water pressure increase significantly affected the deformation of the quay walls.

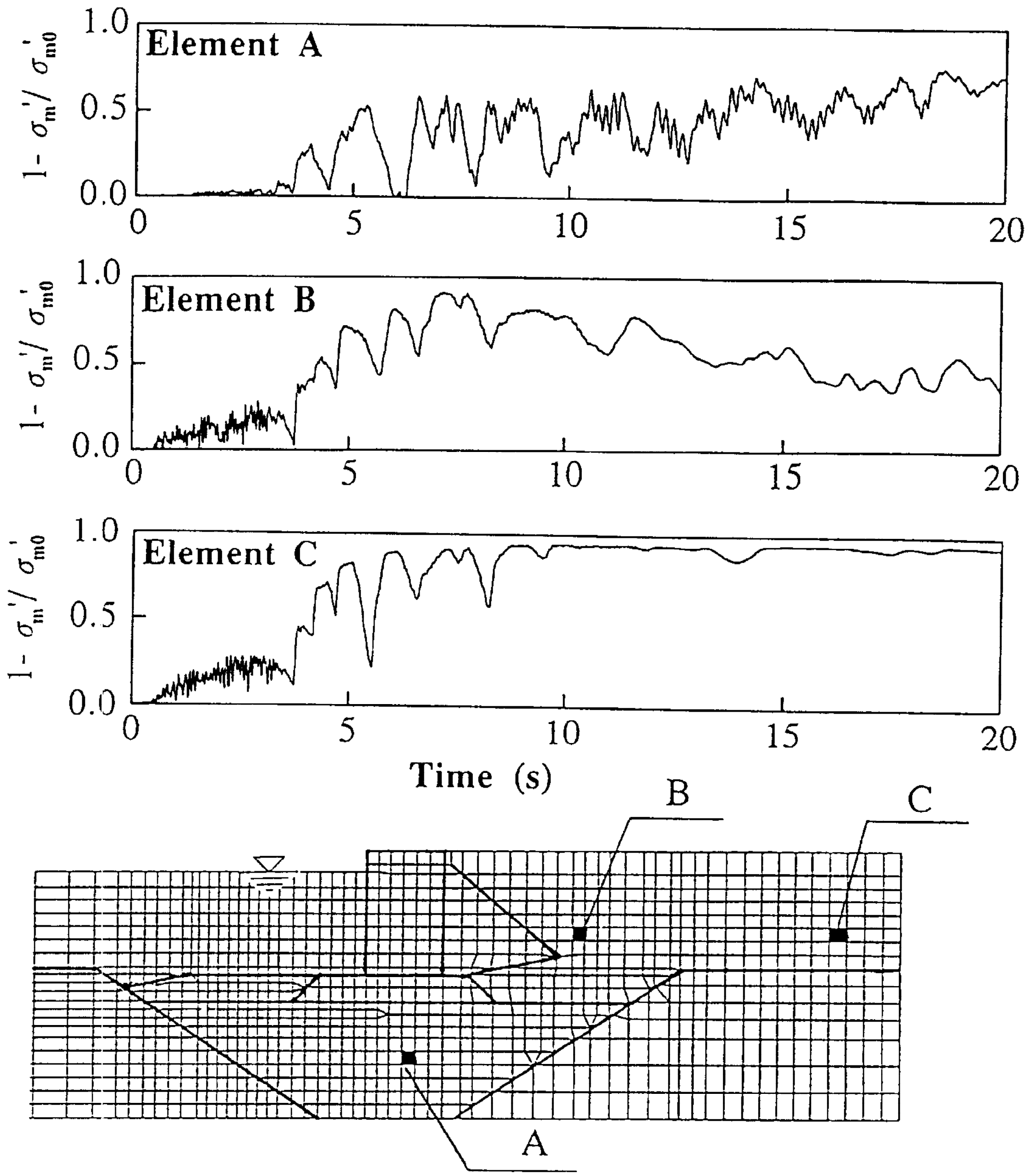

Fig. 9 Computed excess pore water pressures

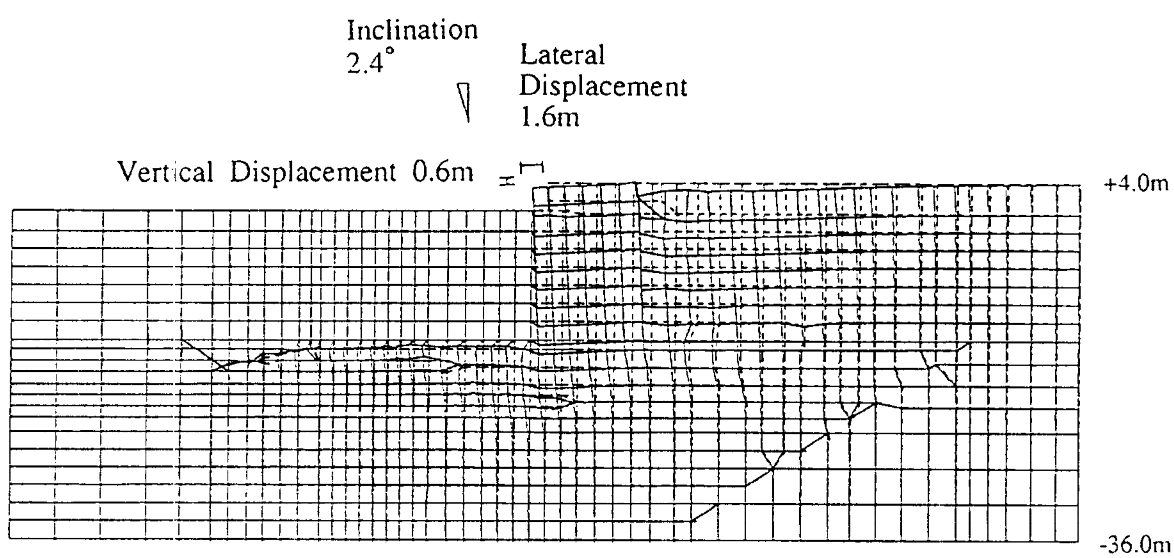

Fig. 10 Computed residual displacements of the quay wall using non-liquefiable soil model

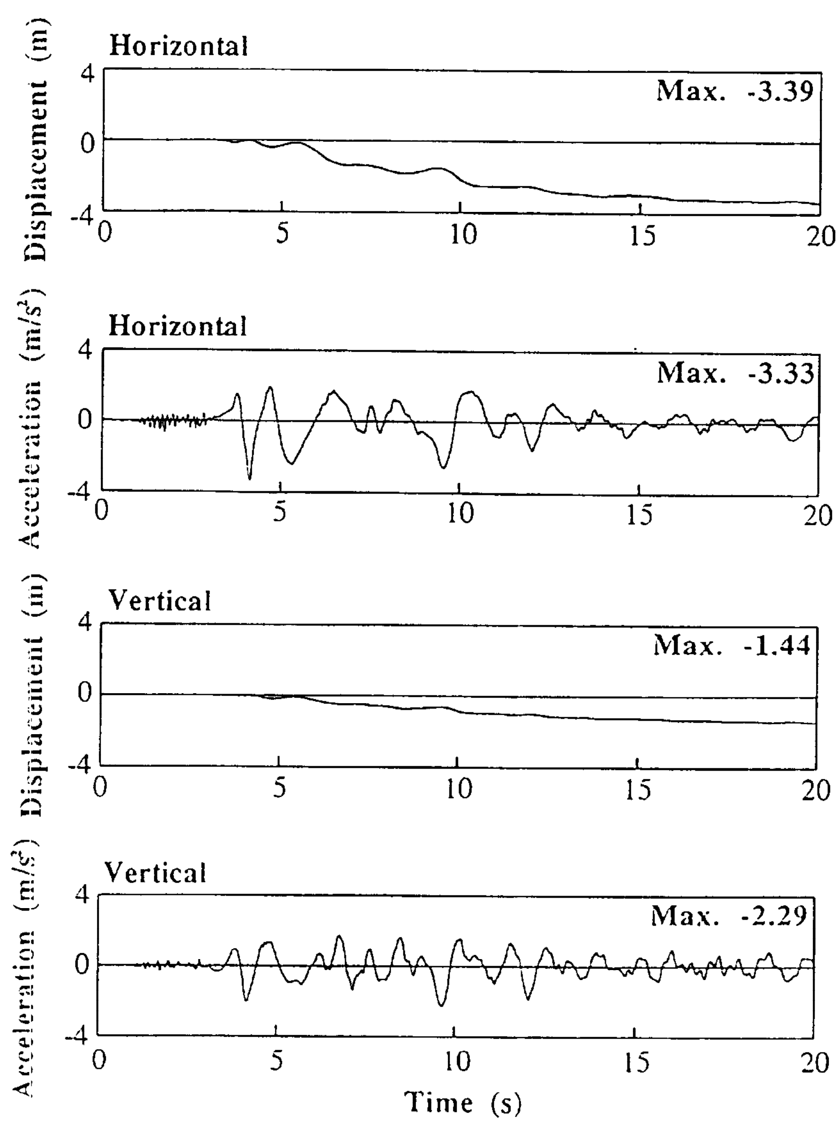

Fig. 11 Computed time histories of the caisson response without vertical input motion

INFLUENCE OF EXCESS PORE WATER PRESSURE INCREASE

In order to quantify the effect of the excess pore water pressure increase in the foundation and backfill soils, the following analysis was performed by introducing an artificial soil model, to be called non-liquefiable soil, which has the same properties as those used in the aforementioned analysis but completely lacks the characteristics of dilatancy.

In order to distinguish the cases for analyses, the case which dealt with the actual quay wall during the earthquake shown in the previous chapter is designated as Case-1 and the case using non-liquefiable soil is designated as Case-2.

The results of Case-2 show that the caisson wall will move about 1 to 2 m in horizontal direction and incline about 2 to 3 degrees purely due to the seismic inertia force as shown in Fig. 10. The comparison of the results of Case-2 in Fig. 10 with those of Case-1 in Fig. 7 indicates that the effect of excess pore water pressure increase in the soils behind and beneath the caisson wall is to increase the displacements and tilting of the caisson wall about twice as those induced purely by inertia force.

The results shown in Fig. 10 also indicate that, when there is no excess pore water pressure increase, the deformation previously seen at depth of the foundation soil in Fig. 7 in Case-1 is quite localized around the foundation rubble in Case-2.

INFLUENCE OF VERTICAL SEISMIC MOTION

An important feature of the ground motions of the 1995 Great Hanshin earthquake in the epicentral region including Kobe Port was noted in the high vertical accelerations of about 0.5g at the ground surface. In order to evaluate the influence of the vertical seismic motion on the performance of the caisson type quay walls, another analysis was performed using only the horizontal input earthquake motion in comparison to Case-1 which was computed with the full vertical input motion.

The computed time histories of the caisson without the vertical component of input motion are shown in Fig. 11. They are very similar to those shown in Fig. 6, which were computed with the full vertical input motion. In particular, the vertical responses computed with and without the vertical input motion are similar to each other because they are mainly induced by rocking motion excited by the horizontal input motion. Similarity is also noted in other responses, though not shown, such as excess pore water pressures. It was concluded that, despite the strong vertical input earthquake motion, its effect was minor on the performance of the caisson wall.

CONCLUSIONS

The 1995 Great Hanshin earthquake provided a rare opportunity to evaluate earthquake resistance of caisson type quay walls. The effective stress analyses performed in this study lead to the following conclusions.

(1) The results of the analysis were consistent with the observed deformation of the caisson type quay wall which moved seaward 5 m maximum, 3 m average, settled about 1 to 2 m, and tilted about 4 degrees. In particular, the caisson tilted into and pushed out the rubble mound toward the sea.

(2) The analysis indicates that, though complete liquefaction occurred in the backfill soils far away behind the wall, the excess pore water pressures in the foundation soils beneath the caisson wall and backfill soil immediately behind the caisson wall did not reach 80% of the initial confining pressures.

(3) Excess pore water pressure increase in the foundation soil underneath the caisson and in the backfill soil increased the deformation of the caisson walls about twice as large as that purely caused by the seismic inertia force.

(4) When there is excess pore water pressure increase in the foundation soil, the caisson wall deformation is significantly affected by the deformation throughout the foundation soil. These modes of deformation are quite different from the sliding mechanism often assumed for the conventional simplified analysis.

(5) Though the vertical ground motion was very large, the analysis suggested that vertical input earthquake motion had minor effect on the performance of the caisson walls because the vertical response motion of the caisson was excited mainly by rocking.

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