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LESSONS LEARNED FROM THE PERFORMANCE OF CAISSON TYPE QUAY WALLS AT 1995 GREAT HANSHIN EARTHQUAKE

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ABSTRACT

Many caisson type quay walls were damaged in Kobe Port during the 1995 Great Hanshin earthquake. Seaward displacements of the caisson walls were about 5 m maximum and 3 m average, settled about 1.5 m and tilted about 4 degrees toward the sea. The mode of deformation of the caisson walls involving the tilting toward the sea was quite different from that of sliding of a caisson, suggesting a problem of foundation soils beneath the caisson walls.

Effective stress analyses were performed to understand the mechanism of deformation in the caisson walls. The results suggested that the initial stress in the soils due to gravity was the main driving force to induce deformation while the excess pore water pressure increase in the foundation soil beneath the caissons and in the backfill soil reduced the resistance in the soils.

KEYWORDS

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

FAILURE MODE OF CAISSON WALLS

Seismic design of the caisson type quay walls in Japanese ports are accomplished by evaluating stability with respect to sliding, overturning, and bearing capacity of foundation (Ministry of Transport, 1991). In particular, the bearing capacity is evaluated with respect to an inclined load based on the circular slip analysis using the simplified Bishop's method. If one of the stability conditions is violated due to excessively large seismic load or reduction in resistance of the foundation subsoils, the caisson walls will displace in accordance with the failure mode corresponding to the violated stability condition. Thus, the current design procedure suggests that the failure mode will be either sliding, overturning, or circular slip failure of foundation subsoils.

During the 1995 Great Hanshin earthquake, the caisson walls at Kobe Port were shaken by a strong earthquake motion having the peak accelerations of 0.54g and 0.45g in the horizontal and vertical directions. In addition, there was extensive evidence of liquefaction of landfill soils. Most of the caisson walls in Kobe Port were designed by a pseudo static method using a seismic coefficient ranging from 0.10 to 0.18. These caisson walls 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. 1. About the same order of settlements were induced in the backfill soils behind the walls.

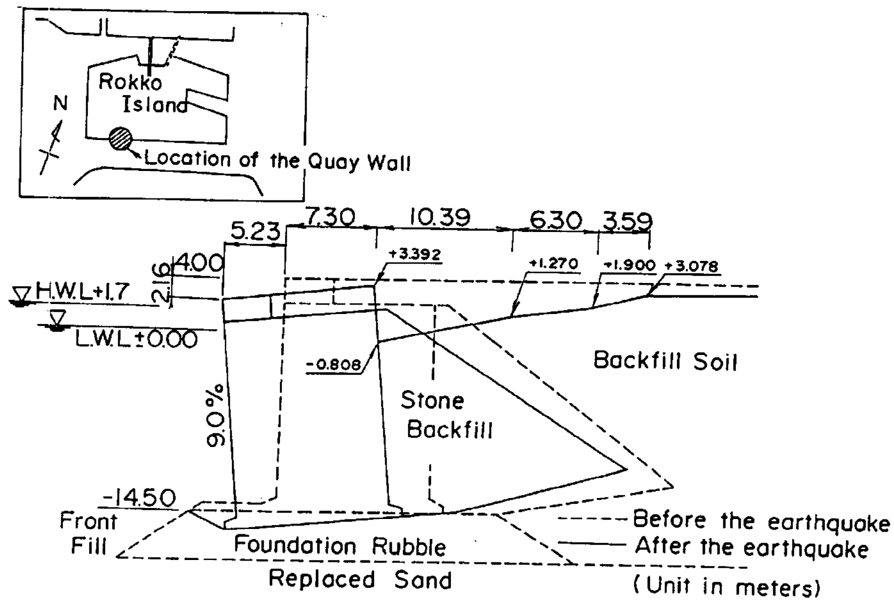


Fig. 1 Deformation of a caisson wall

Although the sliding mechanism could explain the large horizontal displacement of the caisson walls, this mechanism did not explain the large settlement and tilting of the caissons. Reduction in bearing capacity of foundation soils, therefore, was speculated as a main cause of the damage to the caisson walls at Kobe Port. In order to confirm this speculation, effective stress analysis was performed on the seismic performance of the caisson 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., 1992). The model has a capability to simulate the behavior of sand during rotation of principal stress axis, which plays an important role in the behavior of initially anisotropically consolidated sand under cyclic simple shear. 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. More details of the analysis are shown in the companion paper (Iai et al., 1996).

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. Major results of the computed initial stress states before the earthquake are shown in Fig. 2. The computed earth pressure behind the walls was about the same as the active earth pressure used in the design. The computed earth pressure coefficient at rest at the inland far behind the walls was about K_0 = 0.5 and also consistent with the commonly assumed value in design. The computed normal stress below the caisson indicates that the load from the caisson was inclined toward the sea, resulting in higher pressure at the toe of the caisson.

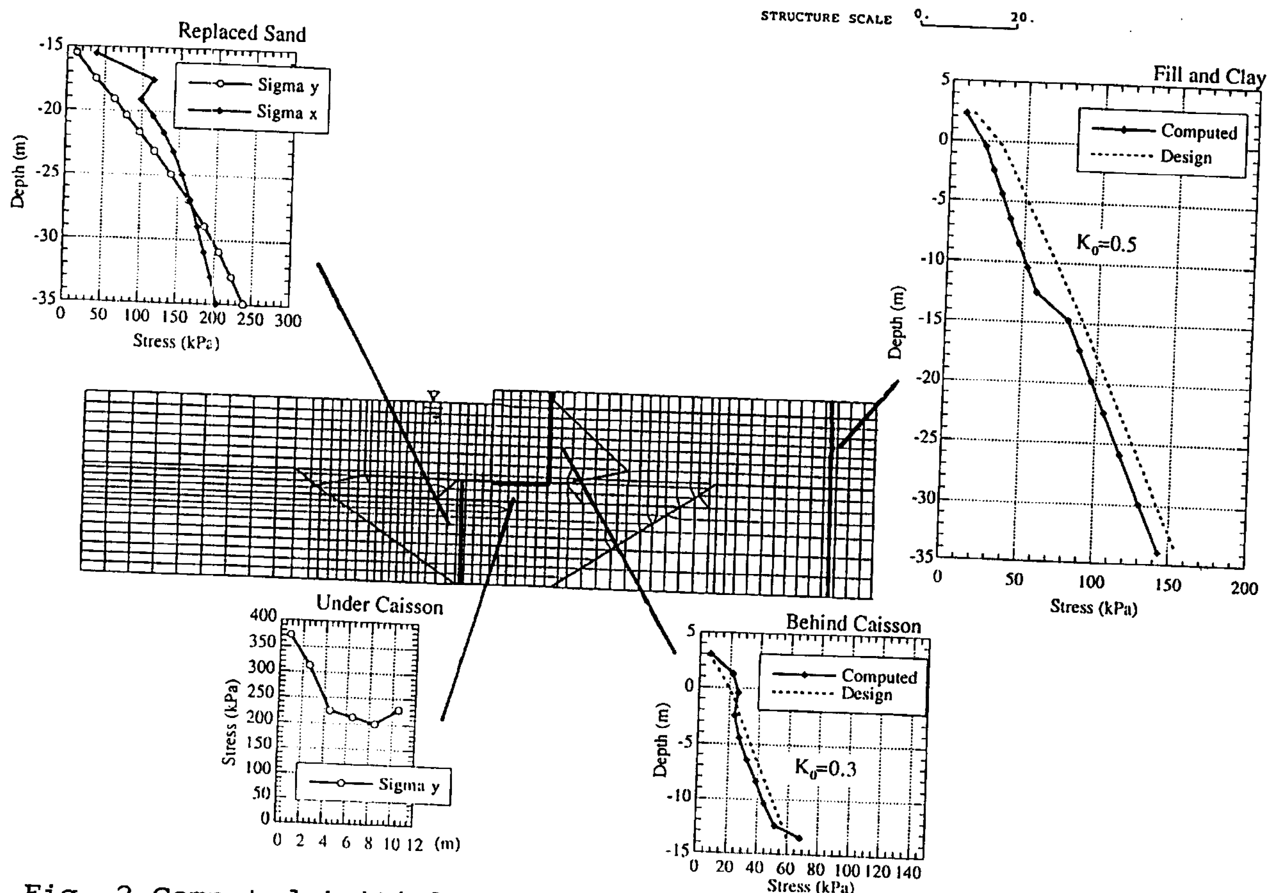


Fig. 2 Computed initial stresses in the subsoils due to gravity

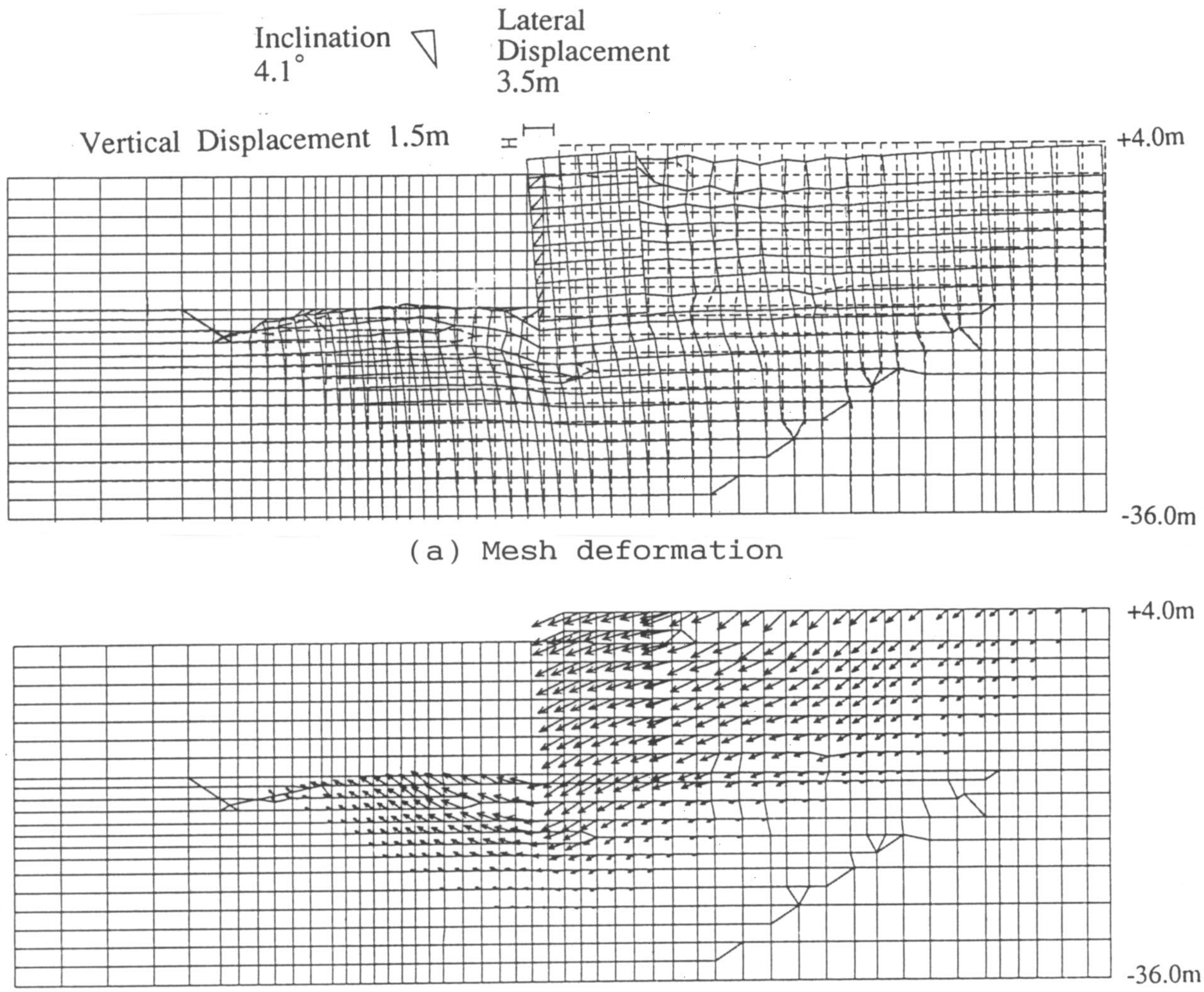
The earthquake response analysis resulted in the residual displacements of the caisson walls as shown in Fig. 3. The residual displacements of the walls were 3.5 m and 1.5 m in the horizontal vertical directions with tilting of 4 degrees toward sea. The order of magnitude of these results was consistent with the observed deformation of the caisson walls mentioned earlier and shown in Fig. 1.

The computed mode of deformation of the caisson walls shown in Fig. 3 was to tilt into and push out the foundation rubble beneath the caisson. This was also consistent with the observed mode of deformation, suggesting that the initial inclined load applied to the foundation soils shown in Fig. 2 triggered this mode of failure. The computed results in Fig. 3 also indicated that significant displacements were induced over wide cross sectional area in the foundation soils beneath the caisson. This mode of deformation of the quay wall is quite different from those caused by sliding but more consistent with the circular slip failure mode assumed for evaluating the bearing capacity of the foundation.

The effective stress analysis indicated that excess pore water pressures were increased both in the foundation soils and in the backfill soils. These pore water pressure build-up reduced the resistance of the foundation soils and increased the earth pressure on the walls, resulting in the large deformation of the caisson walls.

CONCLUSIONS

Observation of the failure mode of the caisson walls at the 1995 Great Hanshin earthquake and the effective stress analysis lead to the following conclusions.



(a) Mesh deformation

(b) Displacements vectors

Fig. 3 Computed deformation after the earthquake

(1) The results of the effective analysis were consistent with the observed deformation of the caisson walls, which moved seaward 5 m maximum, 3 m average, settled about 1 to 2 m, and tilted about 4 degrees toward the sea.
(2) The failure mode of the caisson walls were to tilt into and push out the rubble mound toward the sea. This is due to the initial inclined load applied to the foundation soils.
(3) The failure mode of the caisson walls also involved overall deformation of the foundation soils, which is consistent with a circular slip failure mode assumed for evaluating the bearing capacity of the foundation.
(4) Initial stress acting in the soils due to the gravity was the main driving force to cause the observed deformation of the caisson walls while the excess pore water pressure increase in the foundation soil beneath the caisson and in the backfill soil reduced the resistance of the soils below and behind the walls.

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