Seismic performance of caisson walls on loose saturated sand foundation
Performance au séisme d'un mur de quai sur une fondation de sable saturé

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ABSTRACT: An effective stress analysis is performed on the damage to the caisson type quay walls at Kobe Port, Japan, during the 1995 Great Hanshin earthquake. The analysis indicates that the gravity was the main driving force to cause the observed deformation of the caisson-foundation system while excess pore water pressure increase in the subsoils significantly increased the movement of the caisson.

RESUME: La stabilité due quai du port de Kobe lors du tremblement de terre de 1995 a été étudiée en contraintes effectives. Cette analyse montre que les déformations dans les fondations en caissons sont principalement causées par forces gravitationelles alors que les pressions interstitielles developpées dans les sols de fondations génèrent les déplacements du caisson.

1 INTRODUCTION

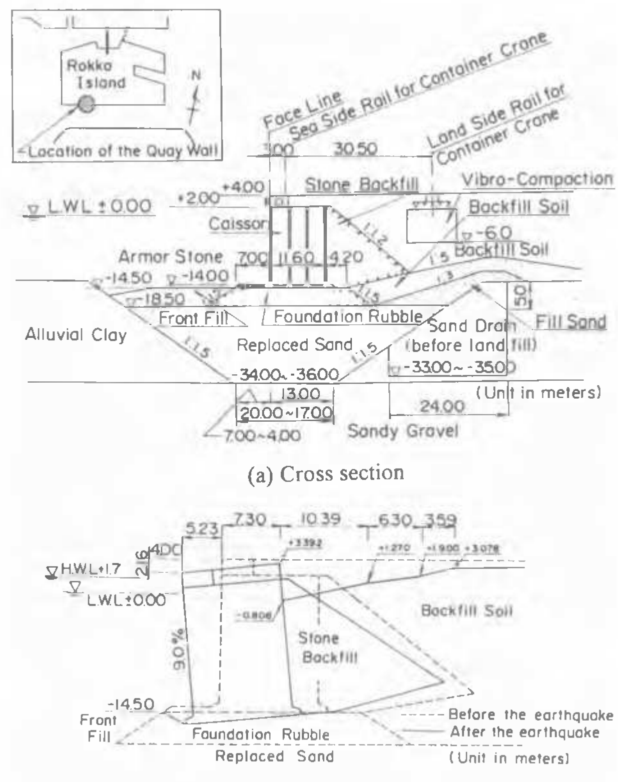
In the current practice of seismic design, caisson type quay walls are designed by evaluating the stability with respect to sliding, overturning and bearing capacity of foundation (Ports an Harbours Bureau, 1991). In particular, the bearing capacity of foundation is evaluated with respect to an inclined load, which is applied to the foundation subsoil due to earth pressure, dead weight and seismic inertia force of the caisson wall. The circular slip analysis is often performed for evaluating the bearing capacity against the inclined load. If one of the stability conditions is violated due to excessively large seismic load or reduction in resistance of the foundation subsoils because of excess pore water increase, the caisson walls will be likely to deform or move in the failure mode of the violated stability condition.

During the 1995 Great Hanshin earthquake, Japan, many caisson walls suffered serious damage at Kobe Port. These caisson walls were constructed on a loose saturated backfill sand, which was used for replacing the soft clayey deposit in Kobe Port in order to attain the required bearing capacity of foundation to support the caisson walls. A valuable case history was thus obtained for understanding the seismic performance of caisson walls founded on a loose saturated sand deposit. In this paper, analysis and discussions will be presented, based on this valuable case history, on the seismic performance of a shallow foundation with an inclined load.

2 DAMAGE MODE OF CAISSON WALLS

In the Great Hanshin earthquake, the caisson walls at Kobe Port were shaken with a strong earthquake motion having the peak accelerations of 0.54g and 0.45g in the horizontal and vertical directions. Most of the caisson walls were designed by a pseudo static method using seismic coefficients ranging from 0.10 to 0.18. These caisson walls displaced toward the sea about 5 m maximum, about 3 m average, settled about 1 to 2 m, and tilted about 4 degrees toward the sea. Figure 1 shows the cross section and deformation of the caisson wall at the south coast of the Rokko Island as an example. In fact, the deformation of this wall was among the largest in Kobe Port. There was little evidence of liquefaction at the backfill in the vicinity of the caisson walls but extensive evidence of liquefaction of landfill soils was observed at inland about 30 m or further from the walls (Inagaki et al, 1996).

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 the bearing capacity of foundation soils due to excess pore water increase, then, 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 performance of the caisson walls.



(a) Cross section

(b) Deformation

Figure 1. Cross section and deformation of a caisson wall (after Inagaki et al. 1996)

3 EFFECTIVE STRESS ANALYSIS OF CAISSON WALLS

The effective stress model used for this study was a multiple mechanism model defined in strain space (Iai et al., 1992). This model has a capability to simulate the behavior of sand subjected to rotation of principal stress axis, which plays an important role in the behavior of initially anisotropically consolidated sand under cyclic simple shear. With the effective stress and strain vectors written by

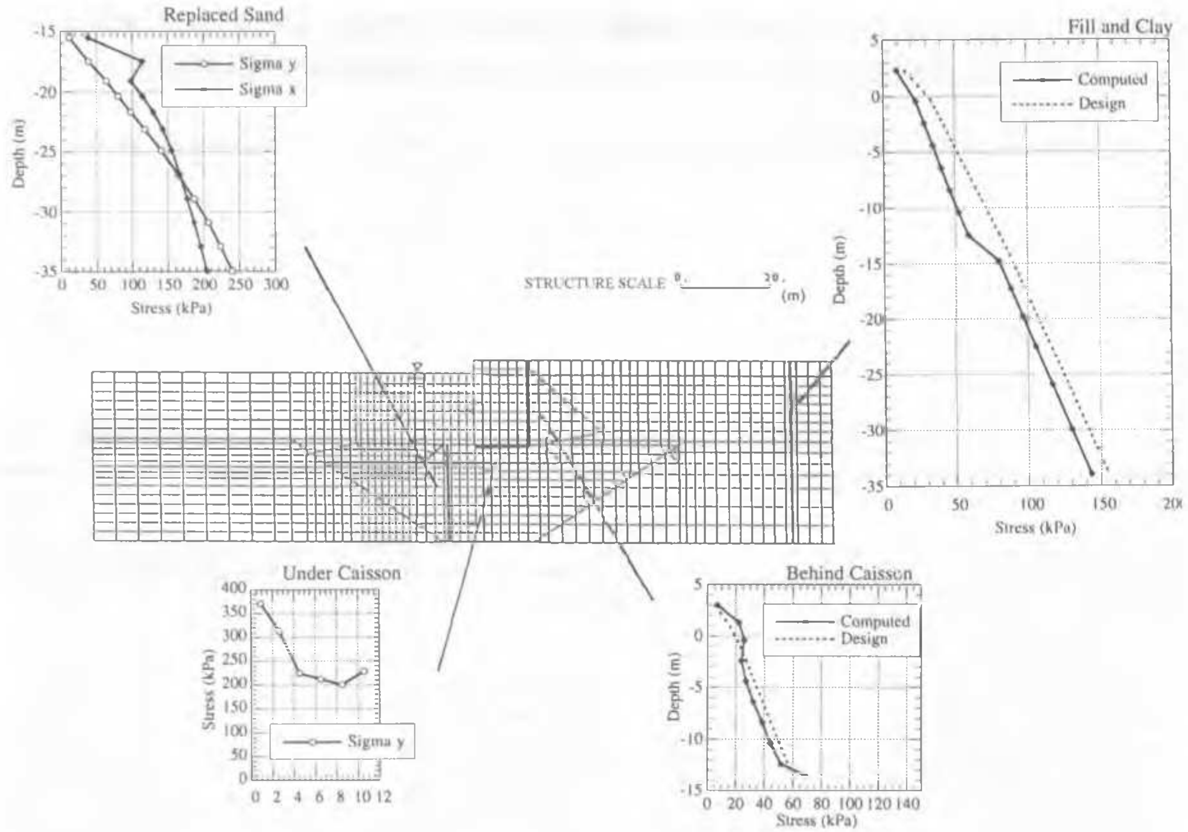


Figure 2. Computed earth pressures before the earthquake

$$\{\sigma'\}^T = \{\sigma'_x\ \sigma'_y\ \tau_{xy}\} \quad (1)$$

$$\{\varepsilon\}^T = \{\varepsilon_x\ \varepsilon_y\ \gamma_{xy}\} \quad (2)$$

the basic form of the constitutive relation is given by

$$\{d\sigma'\}^T = [\mathbf{D}](\{d\varepsilon\}-\{d\varepsilon_p\}) \quad (3)$$

in which

$$[\mathbf{D}] = K\{\mathbf{n}^{(0)}\}\{\mathbf{n}^{(0)}\}^T + \Sigma\, R^{(i)}{}_{L/U}\{\mathbf{n}^{(i)}\}\{\mathbf{n}^{(i)}\}^T \quad (4)$$

In this relation, the term $\{d\varepsilon_p\}$ in Eq. (3) represents the additional strain increment vector to take the dilatancy into account and is given from the volumetric strain increment due to the dilatancy $d\varepsilon_p$ as

$$\{d\varepsilon_p\}^T = \{d\varepsilon_p/2\ d\varepsilon_p/2\ 0\} \quad (5)$$

The second term in Eq. (4) represents the multiple shear mechanism. Each mechanism i = 1,..., I represents a virtual simple shear mechanism, with each simple shear plane oriented at an angle $\theta_i/2+\pi/4$ relative to the x axis. The tangential shear modulus $R^{(i)}{}_{L/U}$ represents the hyperbolic stress strain relationship with hysteresis characteristics. The direction vectors for the multiple shear mechanism in Eq. (4) are given by

$$\{\mathbf{n}^{(i)}\}^T = \{\cos\theta_i\ -\cos\theta_i\ \sin\theta_i\} \quad (\text{for } i = 1,\dots, I) \quad (6)$$

in which

$$\theta_i = (i-1)\Delta\theta \quad (\text{for } i = 1,\dots,I) \quad (7)$$

$$\Delta\theta = \pi/I \quad (8)$$

The loading and unloading for shear mechanism are separately defined for each mechanism by the sign of $\{\mathbf{n}^{(i)}\}^T\{d\varepsilon\}$.

The parameters of the effective stress model, total of ten, were determined by referring to the in-situ velocity measurements and the cyclic triaxial test results of the in-situ frozen samples having a diameter of 300 mm.

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. The results are shown in Fig. 2. As shown in this figure, the computed earth pressure behind the walls before the earthquake was about the same as the active earth pressure, having the earth pressure coefficient of about 0.3. At a further inland of landfill shown in Fig.2, the computed earth pressure corresponded to the earth pressure coefficient of about 0.5, being consistent with the general understanding of the earth pressure coefficient at rest for a loose sand deposit. The computed normal stress of foundation soil below the caisson indicated the load from the caisson was inclined toward the sea, resulting in higher pressure at the sea side toe of the caisson. This non-uniform distribution of the pressure constituted an overall inclined load applied to the foundation soil from the caisson wall.

With these earth pressures and other initial stresses given as an initial condition, an earthquake response analysis was performed using the input earthquake motion recorded at a depth of 32 m at the vertical seismic array station in the Port Island. The results of the response analysis indicated gradual increase in displacements of the wall as shown in Fig. 3, which resulted in the residual displacements at the top of the caisson being 3.5 m and 1.5 m in the horizontal and vertical directions.

The computed residual deformation of the quay wall is shown in Fig. 4 in mesh and vector representations. As shown in this figure, tilting of 4 degrees toward the sea was also computed together with the horizontal and vertical displacements mentioned earlier. These computed residual deformation of the wall were basically consistent with those observed and mentioned earlier.

4 MECHANISM OF DEFORMATION OF CAISSON WALLS

The computed mode of deformation of the caisson wall was to tilt into and push out the foundation soil beneath the caisson. This was also consistent with the observed deformation mode of the rubble foundation shown in Fig. 5, which was identified by a diving investigation. The inclined load due to gravity applied to the foundation through the caisson wall was considered to be the triggering mechanism to induce this mode of deformation.

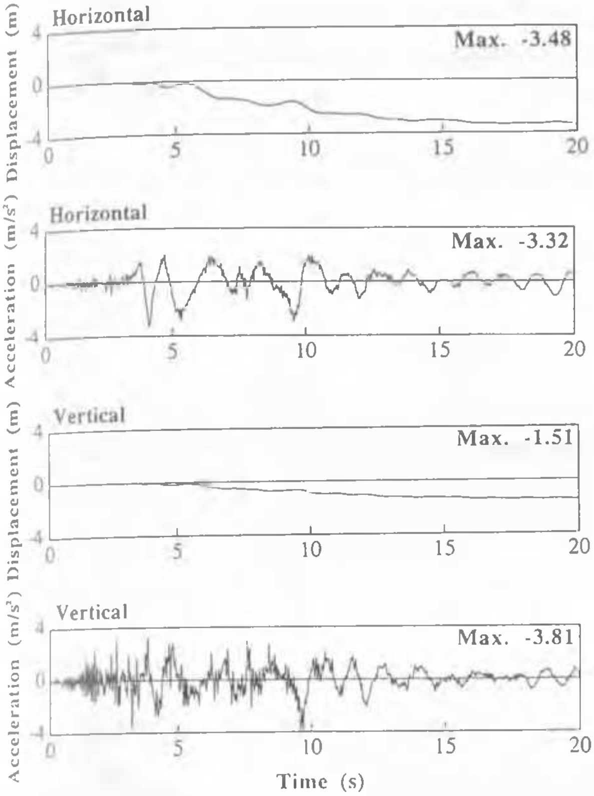


Figure 3. Computed time histories of displacements and accelerations at the sea side top corner of the caisson wall

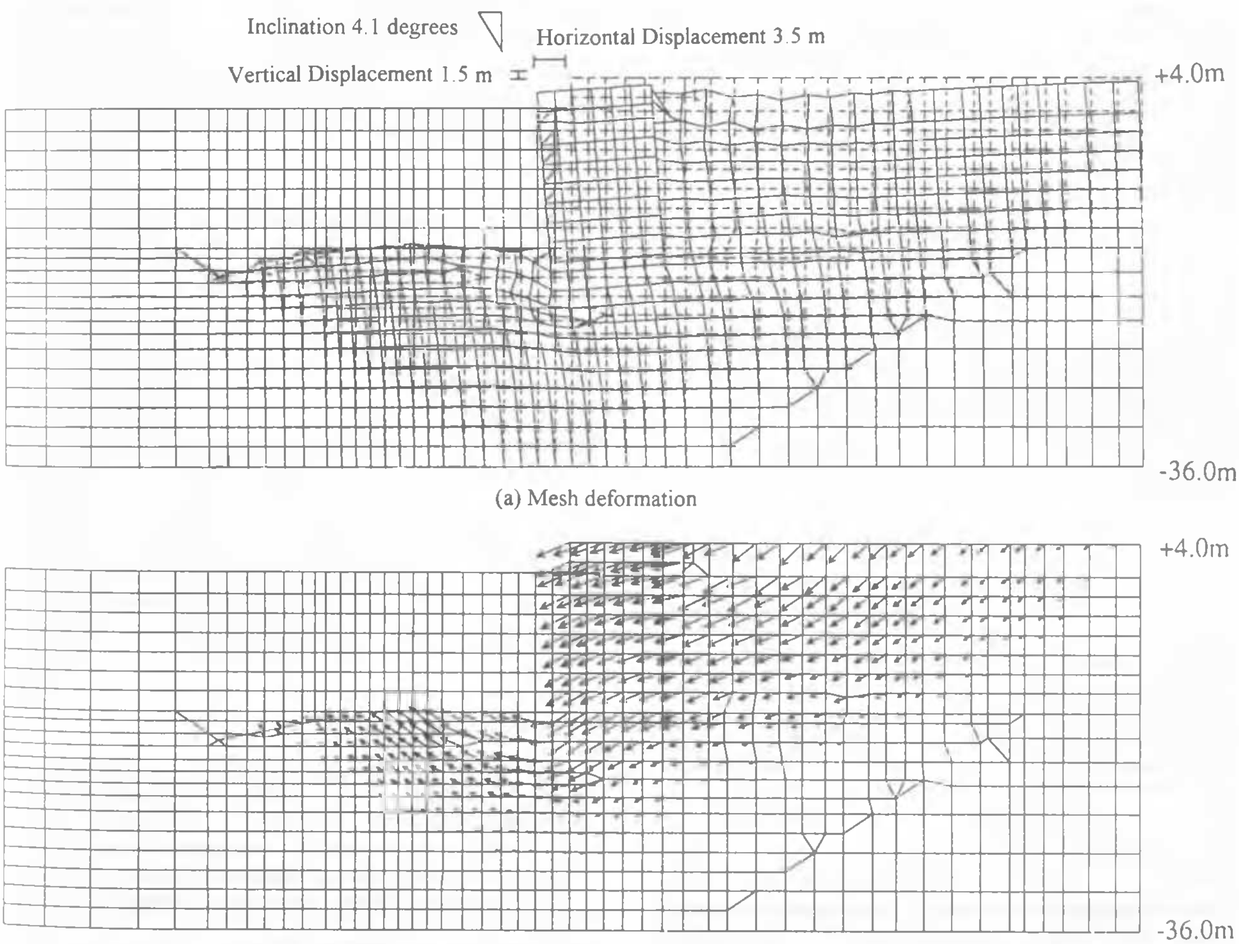
The computed results also indicated that significant displacements were induced over a wide cross-sectional area in the foundation soils beneath the caisson wall. This mode of deformation of the quay wall was quite different from the one based on the sliding mechanism of the caisson but more consistent with the circular slip failure mode assumed for evaluating the bearing capacity of the foundation.

The effective stress analysis also indicated that excess pore water pressures increased both in the foundation and backfill soils as shown in Fig. 6, in which the excess pore water pressure ratios were plotted and defined by $(1 - \sigma'_m/\sigma'_{m0})$ using current and initial confining pressures σ'_m and σ'_{m0}. These excess pore water pressure increases corresponded well to the gradual increase in the displacements shown in Fig. 3, resulting in the overall shear deformation of the foundation subsoils.

In particular, it should be noted that the displacements shown in Fig. 3 did not increased very much at about 4 to 5 seconds when the acceleration response of the caisson walls became the maximum. The displacements were gradually increased as the shaking continues in accordance with the increase in the excess pore water pressures shown in Fig. 6.

It is also understood from Fig. 6 that the excess pore water pressure ratio at the subsoils beneath the caisson never achieved the state of one hundred percent. This is considered due to the initial shear stress induced by the gravity load from the caisson wall. This may be more clearly shown by the stress path and stress-strain relationships of the soils beneath the caisson wall as shown in Fig. 7. The stress path, shown at the bottom, fluctuated in the vicinity of the failure line but never went through the origin because of the prevailing shear stress. In accordance with these fluctuations in the stress paths, strains were gradually induced in the soil as shown in the upper two rows of Fig. 7.

These results indicate that the deformation of caisson walls founded on a loose saturated sand deposit is induced by gradually



(a) Mesh deformation

(b) Displacement vectors

Figure 4. Computed residual displacements of caisson wall

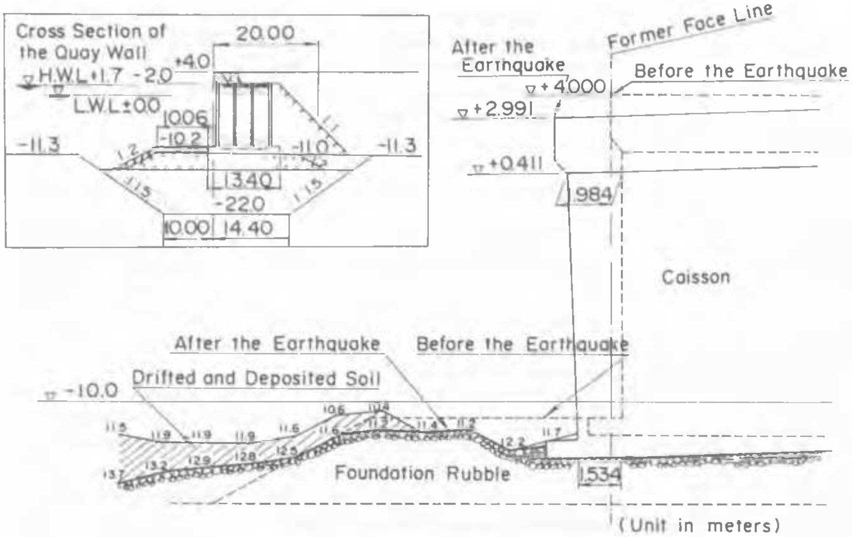
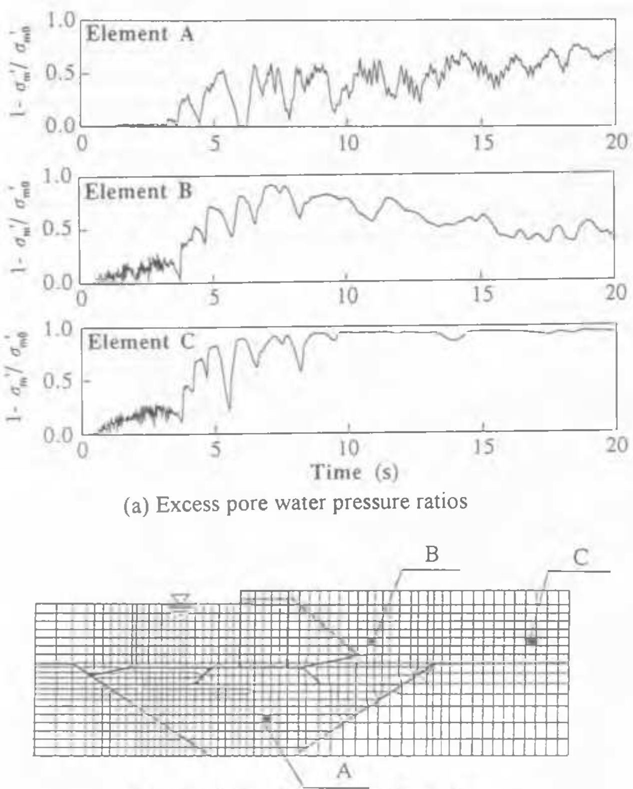


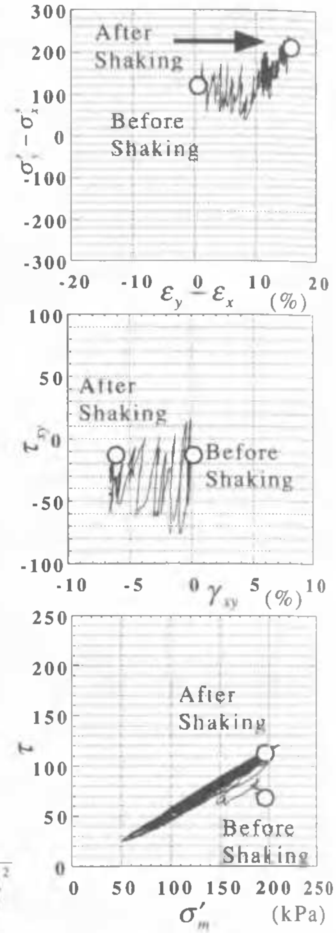
Figure 5. Deformation of rubble mound investigated by diving (after Inagaki et al. 1996)



(a) Excess pore water pressure ratios

(b) Locations of elements A,B, and C

Figure 6. Computed excess pore water pressure ratios



$$\tau = \sqrt{\tau_{xy}^2 + \{(\sigma_x' - \sigma_y')/2\}^2}$$

Replaced Sand under Caisson

Figure 7. Computed stress-strain relationships and stress path for the element A in Fig. 6

releasing the initial stress in the foundation soils due to gravity while the shear resistance of the foundation soil is gradually reduced due to excess pore water pressure increase in the foundation soil. These mechanism of deformation of caisson walls is quite different from the sliding block mechanism, often assumed in a simplified deformation analysis.

5 CONCLUSIONS

The effective stress analysis on a case history of damage to caisson type quay walls at Kobe Port during the 1995 Great Hanshin earthquake lead to the following conclusions on the seismic performance of caisson walls founded on a loose saturated sand deposit.

(1) The results of the effective stress analysis were consistent with the observed deformation of the caisson walls, which displaced 5 m maximum, 3 m average toward the sea, settled 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 foundation soils. This was considered because of the inclined load due to gravity applied to the foundation soils through the caisson wall.
(3) The failure of the caisson wall involved overall deformation of the foundation soil, which was similar the failure zone of a circular slip analysis often assumed for evaluating the bearing capacity of the foundation.
(4) Release of the initial stresses in the subsoils due to gravity were considered the main driving mechanism of the observed deformation of the caisson walls while the excess pore water pressure increase in the subsoils reduced the resistance of the soils below and behind the wall, resulting in large deformation of the wall founded on a loose saturated sand deposit.

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