5. Simulation of Damage Process

Numerical analyses were carried out to understand the damage process of port facility and buildings during the earthquake. In section 5.1, the damage process of the caisson-type quaywall in Taichung Harbor was analyzed by using the 2-dimensional dynamic effective stress method. We discussed the extent of liquefaction behind the quaywall and the permanent movement of the quaywall. In section 5.2, a high-rise RC apartment building is used to investigate the damage process and mechanism when subjected to the near fault acceleration records in different level of PGA. Three-dimensional frame model considering the nonlinear behavior of individual structural members is used in the analysis to calculate the building's responses to the ground motions.

5.1 Quaywall in Taichung Harbor

During the 1999 Chi-Chi Earthquake, the Taichung Harbor was partially damaged. The caisson quaywalls moved seaward and the unloading facilities were damaged at the Piers No.1 to No.4A located beside the North Docking Channel. These damages were considered to be due to liquefaction of reclaimed sand, because many sand boils were observed behind the quaywalls. Liquefaction analysis of the soil-structure system was performed to understand the damage process during the earthquake.

5.1.1 Damaged Quaywall in Taichung Harbor

In Taichung Harbor, remarkable damages were observed at the North terminal only. Damage description about Taichung Harbor has already been made in section 2.5, therefore a brief description about the damage is made here. Structural conditions of quaywalls and soil conditions of backfill are especially described here.

Figure 5.1.1 shows a typical cross section of Pier No.1 to No.3 of Taichung Harbor (Chen and Hwang, 2000). Each pier consists of ten caissons. Each concrete caisson has four cells, with a height of 19.6 m and a width of 17.6 m. These quaywalls at Pier No.1 to No.4 were designed by the "Design Manual of Harbor Structures in Japan 1967" (Japan Society of Civil Engineers, 1999). The design seismic coefficient is 0.15 for all caissons. The quaywalls were constructed as follows; 1) excavation of the foundation, 2) construction of layers of cobbles and boulders, 3) installation of the RC caissons and filling the backfill cobbles and boulders. The ground behind the

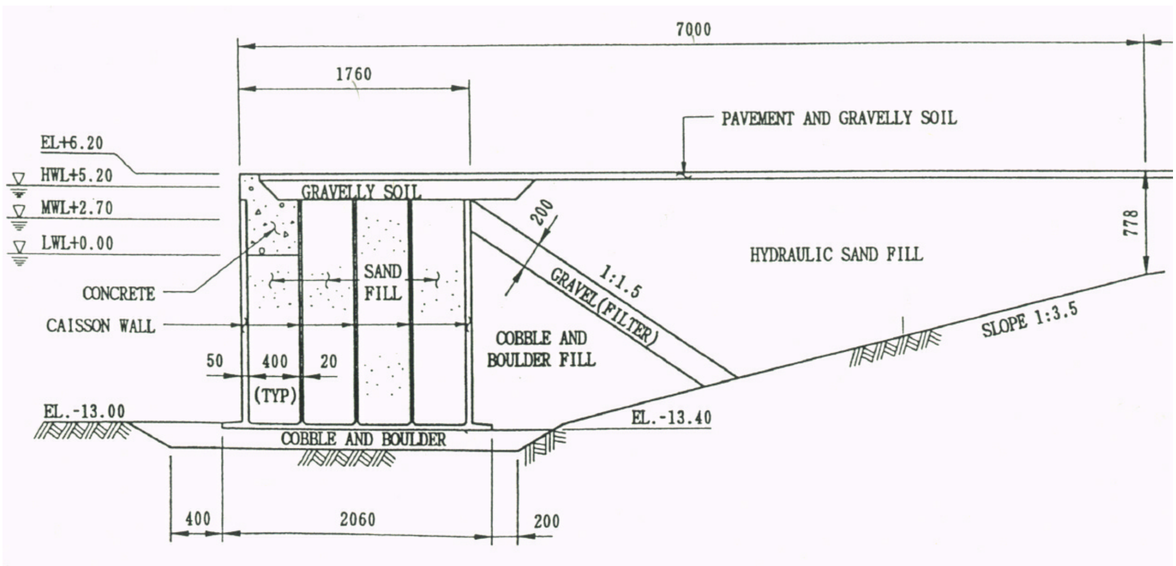


Figure 5.1.1. Typical cross section of Pier No.1 to No.3 of Taichung Harbor (Chen and Hwang, 2000).

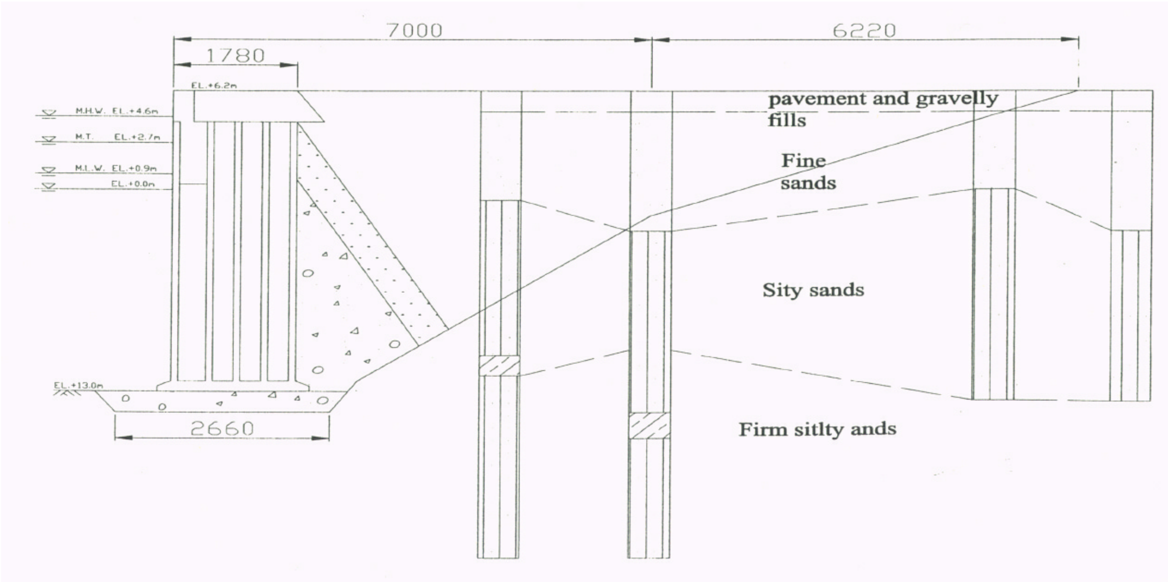


Figure 5.1.2. Soil profiles along the cross section of Pier No.3 (Chen and Hwang, 2000).

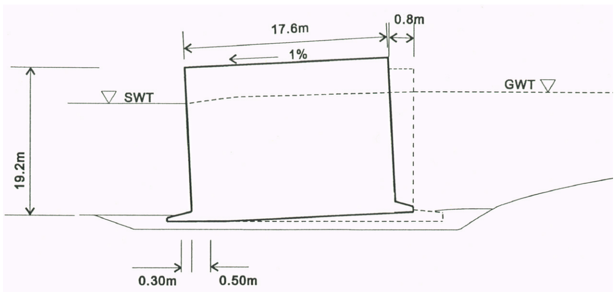


Figure 5.1.3. Damaged configuration of the caisson in Pier No.2
(Center of Harbor and Marine Technology, 1999).

quaywalls was hydraulically filled with sands dredged from the sea, then a gravel of thickness 30 cm and asphalt pavement was layered.

Figure 5.1.2 shows a schematic section of soil profiles near Pier No.3 in the North terminal, which is based on the results of geological boring logs performed after the earthquake (Chen and Hwang, 2000). The original seabed consists mainly of sands of loose to medium dense, interbedded with several thin layers of clayey silt or silty clay. SPT-N values of the seabed range over about 20. The averaged SPT-N value from GL-20m to GL-40m was about 25 based on the in-situ tests at Pier No.1 (Japan Society of Civil Engineers, 1999). Above the original seabed, sands dredged from the navigation channel and nearby areas are filled hydraulically to the present ground level. The hydraulic sand fills consist of mainly fine sands. The sand fills are quite loose, and their SPT-N values range from 5 to 14 (Chen and Hwang, 2000). Therefore, the hydraulic sand fills under the water table could be liquefied during the earthquake.

Ground failures and structural damage observed in the investigated area are concluded as shown in Figure 2.5.2. Many sand boils were observed at several sites mainly around the ground depressions and the vertical gaps occurred between the caissons and backfill soils. The backfill soils subsided due to seaward movements and inclinations of the caissons (Photo. 2.5.1), and length and width of the subsided area were about 800 m and 40 m, respectively. Most of the caissons were damaged slightly or not. Maximum values of inclinations and horizontal displacements of the caissons were about 1-3 degree and 1.7 m, respectively as shown in Figures 2.5.3. Maximum settlement of backfill soils was about 1.4 m as shown in Figures 2.5.4. The deformed configuration of the caisson at Pier No.2 was measured by the Center of Harbor and Marine Technology as shown in Figure 5.1.3 (Center of Harbor and Marine Technology, 1999). The top surface of the caisson experienced a horizontal displacement of 80cm and a tilt angle of 1 degree. It was estimated that the foundation of the caisson will have a horizontal displacement of 50cm.

5.1.2 Numerical Method

Governing Equations

In this study, the governing equations of for the coupling problems between soil skeleton and pore water were obtained with the two phase mixture theory (Biot, 1962). Using a u-p (displacement of the solid phase - pore water pressure) formulation (Zienkiewicz and Bettes, 1982), a simple and practical numerical method for the two-dimensional liquefaction analysis was formulated. The finite element method (FEM) has been usually used for the spatial discretization of the governing equations. In this study, however, the finite element method (FEM) was used for the spatial discretization of the equilibrium equation, while the finite difference method (FDM) was used for the spatial discretization of the pore water pressure in the continuity equation (Akai and Tamura, 1978). The accuracy of the proposed numerical method was verified by Oka et al. (1994) through a comparison of numerical results and analytical solutions for transient response of saturated porous solids. As details of this method were given in Oka et al. (1994), only a brief description of the method is given below. The governing equations are formulated by the following assumptions; 1) the infinitesimal strain, 2) the smooth distribution of porosity in the soil, 3) the small relative acceleration of the fluid phase to that of the solid phase compared with the acceleration of the solid phase, 4) incompressible grain particles in the soil. The equilibrium equation for the mixture is derived as follows:

$$\rho \ddot{u}_i^s = \sigma_{ij,j} + \rho b_i \tag{5.1.1}$$

where ρ is the overall density, $\ddot{u}_i$ is the acceleration of the solid, σ_{ij} is the total stress tensor and b_i is the body force. The continuity equation is derived as follows:

$$\rho^f \ddot{\varepsilon}_{ii}^s - p_{,ii} - \frac{\gamma_w}{k}\dot{\varepsilon}_{ii}^s + \frac{n\gamma_w}{kK^f}\dot{p} = 0 \tag{5.1.2}$$

where ρ^f is the density of the fluid, p is the pore water pressure, γ_w is the unit weight of the fluid, k is the coefficient of permeability, ε_{ii}^s is the volumetric strain of the solid, n is porosity and K^f is the bulk modulus of the fluid.

Constitutive Models

The constitutive equation used for sand is a cyclic elasto-plastic model (Oka et al., 1992, Tateishi et al., 1995). The constitutive equation is formulated by the following assumptions; 1) the infinitesimal strain, 2) the elasto-plastic theory, 3) the non-associated flow rule, 4) the concept of the overconsolidated boundary surface, 5) the non-linear kinematic hardening rule. The performance of the constitutive model was verified by Tateishi et al. (1995). The model succeeded in reproducing the experimental results well under various stress conditions, such as isotropic and anisotropic consolidated conditions, with and without the initial shear stress conditions, principal stress axis rotation, etc.

5.1.3 Numerical Conditions

The quaywall of Pier No.2 was selected for this study, because its backfill area is an open space without any structures. Figure 5.1.4 shows the 2-dimensional FE model made from the typical cross section as shown in

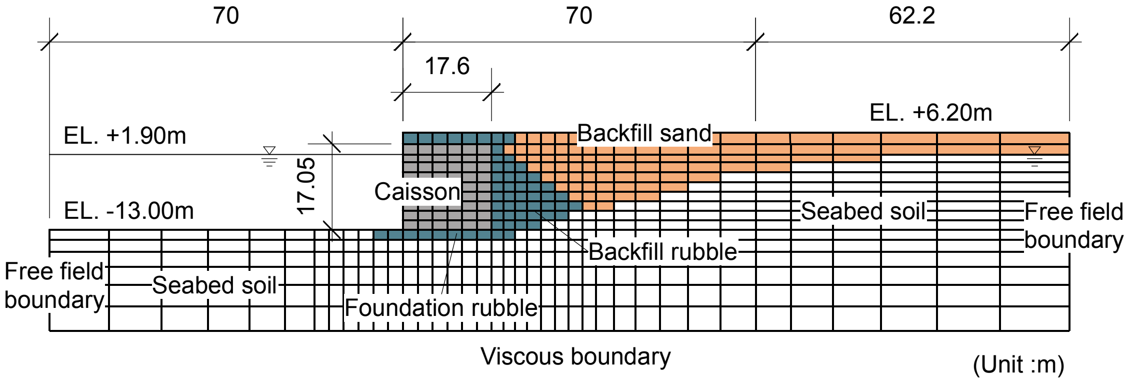


Figure 5.1.4. FE model for Pier No.2 of Taichung Harbor (Refer to color figure 7).

Figure 5.1.1. The depth of the model was determined to be EL –33m, because SPT-N value was over 50 at the depth according to the in-situ tests at Pier No.1 (Japan Society of Civil Engineers, 1999). The width of the model was set to be enough wide, considering the liquefiable sand fill area as shown in Figure 5.1.2.

We applied the cyclic elasto-plastic model for sand to all soil materials, which were backfill sand, original seabed, backfill rubble and foundation rubble. The model parameters are divided to two categories, basic parameters and fitting parameters. The basic parameters, density, internal friction angle, initial shear modulus were normally determined by the in-situ tests and laboratory tests using undisturbed. These basic parameters were derived from the static stability analysis of the caisson (Chen and Hwang, 2000). The initial shear modules were estimated from the mean SPT-value, which were 10 in the backfill sand and 25 in the original seabed. The fitting parameters were determined by adjusting technique. For the backfill sand, the parameter values were selected by trial and error in order to describe the liquefaction strength curve. The liquefaction strength curves were estimated from the mean SPT-N value by using the method of Japanese Highway Bridge Specification (Japan Road Association, 1996). The cyclic shear stress ratio of backfill sand, required to cause 5% axial strain in 20 cycles, was about 0.22. For other soils, which were original seabed, backfill rubble and foundation rubble, dilatancy effect was neglected and nonlinear characteristic of shear deformation was only taken into account.

The concrete caisson quaywall was modeled by linear elastic elements. The slip between the caisson and rubble was modeled by joint elements whose parameters were derived from the static stability analysis of the caisson (Chen and Hwang, 2000). The friction factors were 0.50 on the foundation rubble and 0.27 on the backfill rubble. The initial stress state was computed by the static elasto-perfectly-plastic analysis, in which the Drucker-Prager type failure surface model was employed.

In Taiwan, an intense strong motion array called TSMIP has been installed island-wide. Near Taichung Harbor, there are several seismograph stations in near distance. Among them, the closest one is the station located at the Elementary School of Chigshui (Station Code: TCU059, on ground), where is about 4.7 km southeast of the Harbor. During the main shock of the earthquake, the accelerations of ground motions at the site were successfully recorded and are available tentatively (Lee et al., 1999). Figure 5.1.5 (a) and (b) show the acceleration time series of the two orthogonal horizontal motions (EW and NS components) recorded at TCU059. Peak ground accelerations (PGA) of the both motions are about 160 gal. The input motion for the analysis as shown in Figure 5.1.5 (c) was corrected to N6.5E of the direction of analytical model section. The corrected motion was input from the viscous bottom boundary of the FE model. The time integration step of 0.001 was adopted, and the simulation was performed for 72 seconds. The Rayleigh damping proportional to initial stiffness was used to reproduce the damping in high frequency domain.

5.1.4 Numerical Results

Figure 5.1.6 (a) shows the distribution of excess pore water pressure ratio after the earthquake. Complete liquefaction occurred in the backfill sandy layer, which agreed with the observation of many sand boils at the backfill sand. Few excess pore water pressure occurred in the seabed and rubbles, because dilatancy effect was neglected in these layers. Figure 5.1.6 (b) shows the deformed configuration after the earthquake and the amount of displacement at the top of the caisson. The scale of deformation is the same as that of the FEM mesh and the arrows in the figure shows the direction of the magnitude of displacement. The simulation reproduced a horizontal displacement of about 0.80 m at the top of the caisson, which agreed with the measured displacement of about 0.80 m in Pier No.2 as shown in Figure 5.1.3. On the other hand, the vertical displacement at the top of the caisson was about 0.41 m, which was larger than the measured displacement of about 0.18 m in Pier No.2 as shown in Figure 5.1.3. The predicted mode of deformation of the caisson agreed with the measured one as shown in Figure 5.1.3. The deformation of the caisson was due to the increased lateral pressure of the backfill liquefied soils and deformation of foundation rubble and seabed soil. According to the measured configuration of the caisson as shown in Figure 5.1.3, the caisson seems to move seaward with slip at the bottom. However, Little slip between the foundation rubble and the caisson bottom occurred in the predicted deformed configuration,

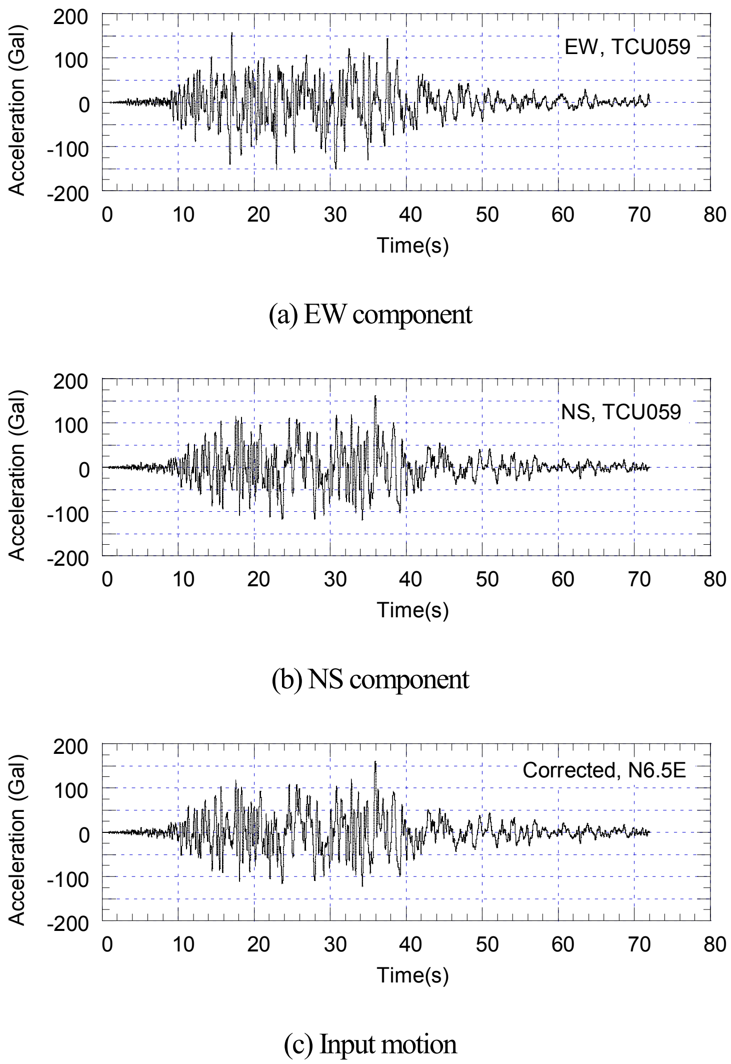
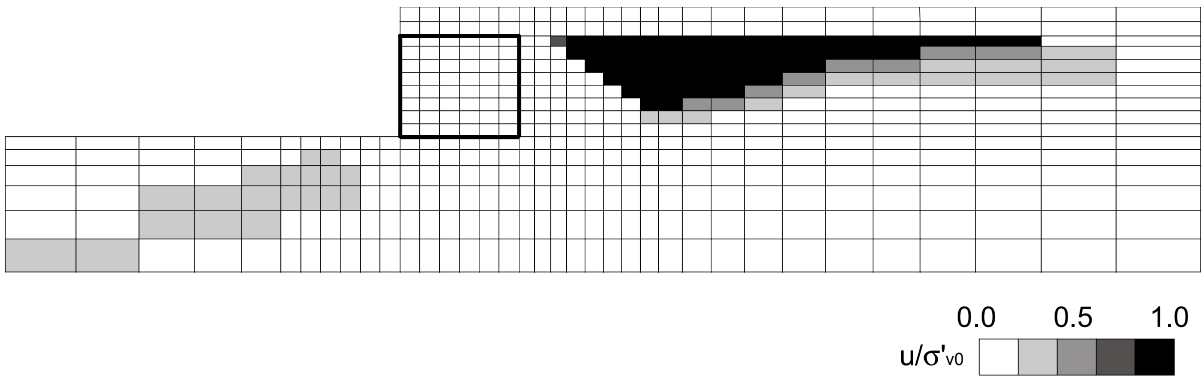
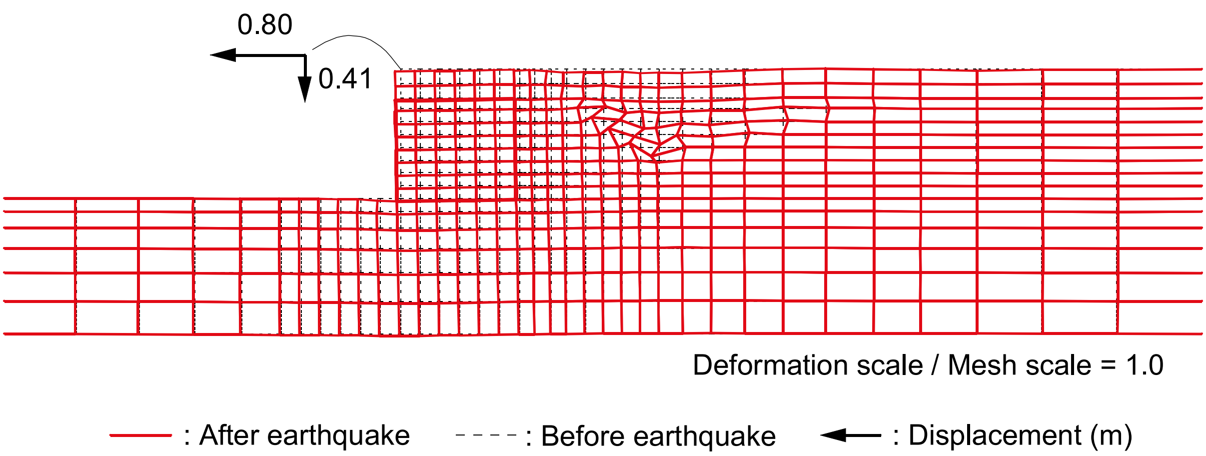


Figure 5.1.5. Acceleration records at TCU059 (Lee *et al*., 1999) and input motion.



(a) Distribution of excess pore water pressure ratio



(b) Deformed configuration after earthquake

Figure 5.1.6. Numerical results (Refer to color figure 8).

although the backfill rubble slightly slipped along the caisson wall. We had better need further investigation about model parameters for the slip joint elements and foundation seabed soils.

5.1.5 Summary

Liquefaction analysis of the soil-structure system was performed to understand the damage process of the caisson quaywall in Taichung Harbor during the earthquake. We discussed the extent of liquefaction behind the quaywall and the permanent movement of the quaywall. The numerical simulation reproduced liquefaction of backfill sand and seaward deformation of the caisson quaywall. For more accurate prediction, we had better need further investigation about soil parameters especially about the foundation seabed soils.

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