13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 3338

NUMERICAL STUDY OF STRUCTURE-SOIL-GROUP PILE FOUNDATIONS USING AN EFFECTIVE STRESS BASED LIQUEFACTION ANALYSIS METHOD

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SUMMARY

Seismic behavior of pile foundations is widely discussed by many researchers in order to do research of safer and more economic designs for the infrastructure during earthquakes. In usual, civil structures are constructed on a foundation with group piles installed rather than a single pile, moreover, the interaction among soils and piles depends on the liquefaction of surrounding ground of the infrastructure. The authors carried out a of liquefaction analysis using an effective stress based liquefaction analysis method on a real structure-pile foundation-ground, in which a five story building in Kobe area was shocked to damage in 1995 Hyogoken-Nambu Earthquake. In the present numerical simulations, we adopted a numerical code LIQCA3D which was developed by some of the authors. From the numerical simulations, it was found that the acceleration responses, velocity responses and displacement responses of the ground agree well with the observations, and the distance of the piles installed in the footing influences the built-up of the excess pore water pressure and the damage on both pile heads and low segments of the pile at the boundary takes place before the complete of liquefaction during these major earthquakes

INTRODUCTION

Many structures were damaged during the 1995 Hyogoken-Nambu Earthquake. It was found from the field investigations after the earthquake that not only the pile heads, but also the lower parts of the piles had cracked or failed. This phenomenon indicates that both the inertia force from the upper structure and the kinematic interaction between the piles and the ground play important roles in the mechanical behavior of piles. In particular, when the ground surrounding a structure liquefies due to seismic excitations, the behavior of the piles is more complicated. Damage related to liquefaction may involve cases in which the pile foundation is damaged due to the lateral flow of liquefied soils, and/or the piles fail at the boundary between two different soil layers, of which one liquefies while the other does not.

In the present study, we conducted numerical simulations of case study of the dynamic behavior of a single-pile foundation which was damage during 1995 Hyogoken-Nambu Earthquake employing a three dimensional liquefaction analysis method (LIQCA3D) to clarify the mechanism of the interactions among the soil-pile-structure.

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MODELS FOR SOILS AND PILES

The two-layer ground is typical at near shore of the major Japanese urban cities such as Kobe. In order to study the influence of soil characteristics, four different sandy materials are considered for upper sandy ground, that is, dense sand, medium dense sand, loose sand, and reclaimed soil. Table 1 shows the constitutive parameters in the soil constitutive models for different soils. On the other hand, an axial force dependent (AFD) model (Zhang et al., 2002) is used to describe the dynamic behavior of the pile which is 1.5 m in diameter. The parameters are shown in Table 2.

In the finite element analysis, a cyclic elasto-plastic model is used for sandy soils which has been developed by Oka et al. (1999). The model has been formulated based on: 1. infinitesimal strain theory, 2. elasto-plastic theory, 3. non-associated flow rule, 4. overconsolidated boundary surface, 5. non-linear kinematical hardening rule. The flow rule is a generalized one as:

$$d\varepsilon_{ij}^{p} = H_{ijkl}\frac{\partial f_p}{\partial \sigma'_{kl}} \tag{1}$$

where $d\varepsilon_{ij}^{p}$ is an plastic strain increment tensor, f_p is a plastic potential function and H_{ijkl} is a fourth order isotropic tensor of hardening modulus.

In the model, two yield surfaces are used: one is for the change of stress ratio and the other is for the change of mean effective stress. The yield surface for the change of stress ratio is as:

$$f_y = \{(\eta_{ij}^{*} - x_{ij}^{*})(\eta_{ij}^{*} - x_{ij}^{*})\}^{1/2} - k = 0 \tag{2}$$

$$\eta_{ij}^{*} = s_{ij} / \sigma'_m$$

where s_{ij} is the deviatoric stress tensor and σ'_m is the mean effective stress, and x_{ij} is a kinematic hardening parameter whose evolutional law is given by

$$dx_{ij} = B^*(A^* de_{ij}^{p} - x_{ij}^{*} d\gamma^{p*}) \tag{3}$$

$$d\gamma_p = (de_{ij}^{p} de_{ij}^{p})^{1/2}$$

where de_{ij}^{p} is the plastic deviatoric strain increment, A* and B* are material constants.

For clay layer of base ground, an elasto-viscoplastic model (Oka, 1992) was used.

NUMERICAL SIMULATION METHOD

The governing equations for the coupling problems between soil skeleton and pore water pressure are obtained based on the Biot type two-phase mixture theory. Using a u-p (displacement of the solid phase-pore water pressure) formulation, the liquefaction analysis is formulated. The side boundaries of the simulated system are assumed to be equal-displacement boundaries, the bottom of the system is fixed and boundaries except surface of the ground are impermeable. In this dynamic analysis, a stiffness-matrix-dependent type of Rayleigh damping is adopted and the direct integration method of Newmark-β is used in this dynamic analysis with a time interval 0.01 sec. Ground water table is at 1.5m beneath the ground surface. The mass of the superstructure is 80,000 Kg and the height of pier is 8m. Figure 1 shows the configuration of the single pile system and the seismic wave used in this study.

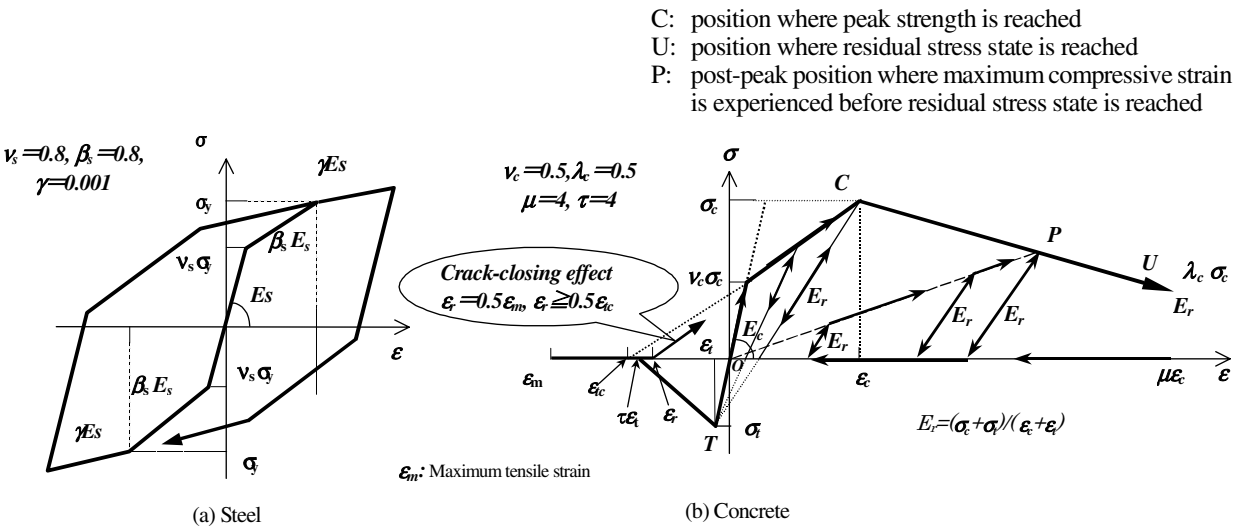


Fig. 1 Nonlinear properties of reinforcement and concrete

A CASE STUDY OF DAMAGE TO A PILE FOUNDATION DUE TO LIQUEFACTION BY THE 1995 HYOGOKEN-NAMBU EARTHQUAKE

In this section, the mechanical behavior of single-pile and group-pile foundations in a two-layer ground during seismic excitation was carefully studied. In reality, however, the upper structure is not an elastic beam element with a mass, but a more complicated structure. The piles should be modeled as more realistic structures. In this chapter, therefore, an actual case record will be discussed using the same numerical technique as that in the previous chapter.

The case study considered in this chapter involves a five-story building, located near East Kobe Harbor, which was damaged by the 1995 Hyogoken-Nambu Earthquake. The effective stress based numerical analysis is conducted using a full system that is composed of a five-story building, a group-pile foundation, and the ground. Uzuoka (2001b) studied the damage to the pile foundation employing a bilinear model to represent the M-F relation of the piles and Oka et al. (2002) showed that the influence from AFD model on the dynamic behavior of the structure. In this study, however, a detailed examination of the seismic behavior of the structure due to earthquake is presented.

Field observations at the site

The configurations of the piles are shown in Figure 2. The piles marked with a circular symbol, are the piles that were checked by a soundness investigation after the earthquake. Figure 3 shows the soil profile obtained through borehole tests. The first layer of the ground is reclaimed soil with a thickness of 11 m. The second layer is alluvial clay and the third is Pleistocene soil. The ground water table is about 2.2 m beneath the ground surface. Figure 4 shows the configuration of the mesh designed for the numerical simulation in this case. In the FE analysis, 4366 elements with 4803 nodes are used. The side boundaries of the simulated system are assumed to be equal-displacement boundaries. The bottom of the system is fixed and the boundaries, except for the surface of the ground, are impermeable.

The bottom of the system is fixed and the boundaries, except for the surface of the ground, are impermeable. The soil elements and the beam elements, whose numerical results will be discussed in detail later, are also shown in Figure 2.

The building located on the west side of Fukaehama, a reclaimed land that was completed in 1965. The closest distance to the coastal line of the reclaimed land is about 350 m. The five-story building, made of RC material, was constructed in 1988 and is supported by several group pile foundations. Each pile was installed by connecting several short bars made of an SC pile and two PHC-A piles at different depths in the ground in order to meet the requirements of seismic design. Since the piles are made from different materials, the mesh should be designed to reflect realistic conditions in the numerical analysis. Figure 5 shows the deformation of the ground on which the building stands after the earthquake. The subsidence of the pile heads was found to be about 70 cm. The building inclined to the north at an inclination of 1/80 and to the east at an inclination of 1/30.

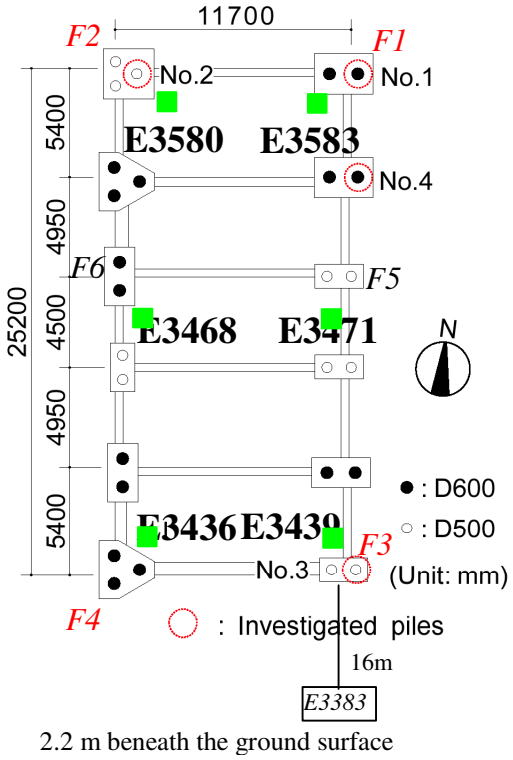


Fig. 2 Configurations of the pile foundation, soil elements, and beam elements

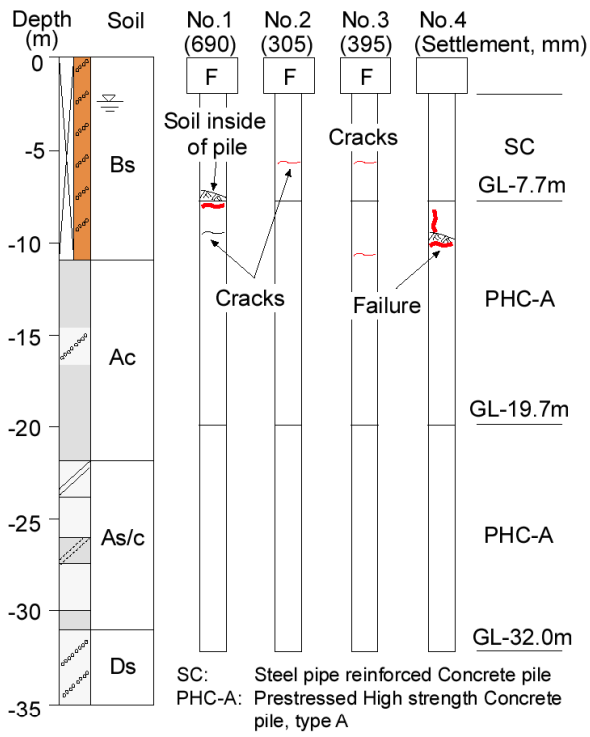


Fig. 3 Soil profile and damage of piles

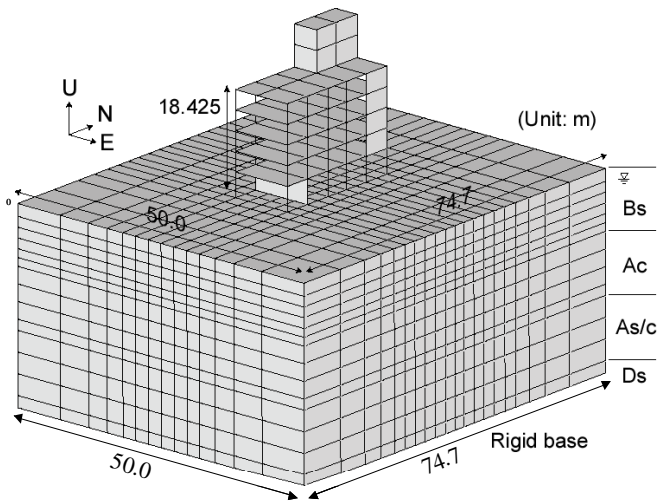


Fig. 4 Mesh used in the analysis

The superstructure itself, however, was not destroyed during the earthquake. In order to inspect the soundness of the piles, the direct observation method of the pile shaft surface, the borehole television method, and the velocity logging method were used after the earthquake. Cracks were found on piles No. 1, No. 2 and No.3 and an intrusion of the soils was found in pile No. 1. Sand boiling phenomena and subsidence of the ground were also found in the area. Therefore, liquefaction of the ground must have taken place during the earthquake. Furthermore, the damage occurred in the lower segments, which were located at the boundary between the sand layer and the clay layer, which may due to the influence of the large deformation of the reclaimed layer.

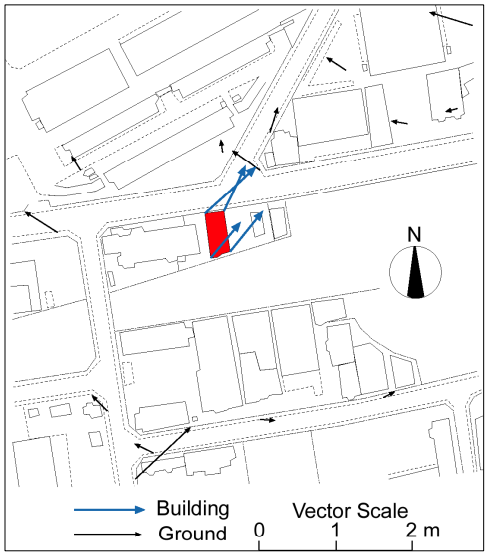


Fig. 5 Location of the building and deformation of the ground

Numerical procedure

The numerical analysis conducted based on the technique mentioned above. The piles are described using the AFD model, the superstructure is represented by an elastic beam element, the slabs and the walls are represented by shell elements, and the weight of each floor has been concentrated into each slab. In the present numerical simulation, the piles in one footing are simplified as a single pile whose area and inertia moment over the x-axis and the y-axis are the sums of the original ones. The parameters for the axial-force dependent model of the piles are listed in Table 1. The input waves, N83E in the x direction and N383E in the y direction, are shown in Figure 6. The side boundaries of the simulated system are assumed to be equal-displacement boundaries. The bottom of the system is fixed and the boundaries, except for the surface of the ground, are impermeable. In the analysis, a stiffness-matrix-dependent type of Rayleigh Damping is adopted and the direct integration method of Newmark-β is used with a time interval of 0.002 sec.

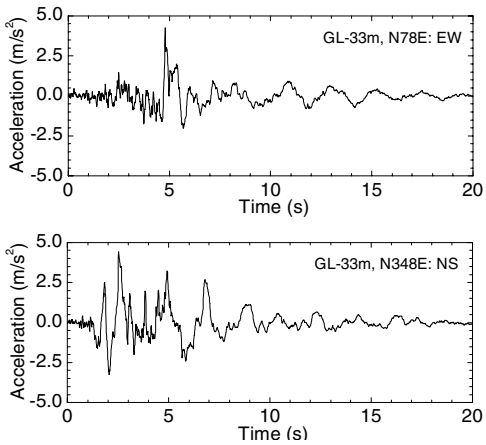


Fig. 6 Input waves

In order to simulate the mechanical behavior of the structure with a group-pile foundation, it is necessary to determine the parameters of the constitutive models for sandy soils and clayey soils. Numerical simulations of the sandy and the clayey soils are firstly conducted on one element. The stress path of reclaimed soils, which are above the groundwater table, will decrease but not reach zero. On the other hand, the stress path

response of reclaimed soils under the groundwater table (GL-2.2m~GL-10.7m) will lead to a zero effective stress state. Figure 7 shows a comparison between the liquefaction strength levels from the laboratory tests conducted on undisturbed samples from the field and the numerical simulations in which the parameters listed in Table 2 are used. In the present case, as expected, the Ac layer that is composed of a clayey soil did not liquefy.

Table 1 Pile Parameters

Pile types		SC		PHC-A	
Outer diameter	D (mm)	600	500	600	500
Inner diameter	D_i (mm)	510	420	510	420
Thickness	T (mm)	90	80	90	80
Thickness of steel pipe	t (mm)	6.0	6.0	0.0	0.0
Thickness of concrete	d_c (mm)	45	40	45	40
Diameter of reinforcement	ϕ (mm)	22.7	20.7	7.1	7.1
Number of reinforcements	N	28	28	19	14
Compression strength of concrete	σ_c (kPa)	7.84×10^4	7.84×10^4	7.84×10^4	7.84×10^4
Tensile strength of concrete	σ_t (kPa)	4.7×10^3	4.7×10^3	4.7×10^3	4.7×10^3
Yielding strength of steel	σ_s (kPa)	2.35×10^5	2.35×10^5	1.27×10^6	1.27×10^6
Failure bending moment	M_u (kNm)	803.5	578.3	681.1	465.9
Failure curvature	(1/m)	2.75×10^{-3}	3.77×10^{-3}	2.48×10^{-3}	3.48×10^{-3}

$E_c=4.50\times10^7$(kPa), $E_s=2.1\times10^8$(kPa)

RESULTS OF SIMULATION BY A 3-D DYNAMIC ANALYSIS

Comparison between the results of field observations and a numerical analysis

Figure 8 shows a comparison in which the acceleration responses of the ground surface from the observations and the computed accelerations in the EW direction agree well with each other in phase, but slightly disagree in amplitude. The computed results are smaller than the observed ones at 6 sec for the amplitude. Accelerations in the NS direction agree well with each other in both amplitude and phase.

Figure 9 shows a comparison of the velocity responses of the ground surface from the observations and the computation. The results both in NS and EW directions agree well with each other.

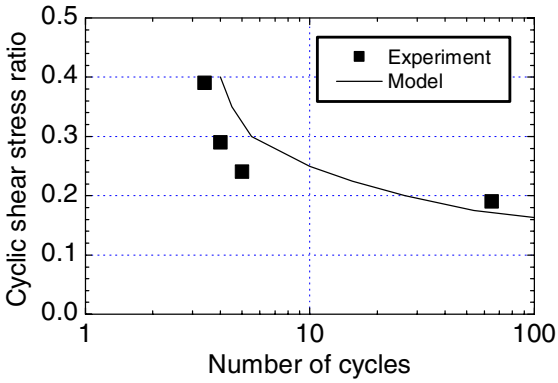


Fig.7 Liquefaction strength

Table 2 Parameters used in the analysis

NAME OF THE SOIL PROFILE	Bs	Ac
Density (t/m^3)	2.0	1.7
Initial void ratio	042	1.41
Coefficient of permeability (m/sec)	2.2E-5	3.8E-11
Compression index	0.0100	0.3310
Swelling index	0.0010	0.0425

Initial shear modulus ratio	1686.0	401.0
Failure stress ratio M^*_f	1.20	1.23
Phase transformation ratio M^*_m	0.91	1.03
Hardening parameter B_0, B_1, C_f for sand	3500.0, 70.0, 0	55.0, 0.0, 0.0
Parameter of anisotropy C_d	2000.0	
Dilatancy parameter D_0, n	1.0, 4.0	
Viscoplastic parameter m_0`, C_{01}, C_{02}(1/sec)		14.0, 5.54E-6, 7.76E-7

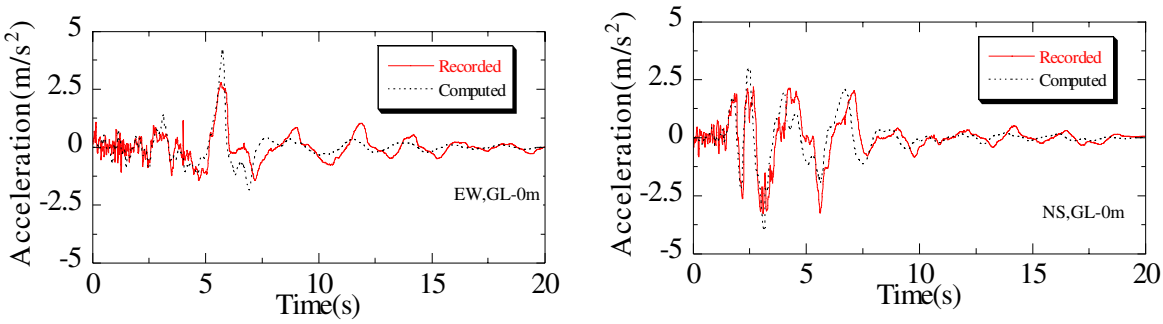


Fig. 8 Comparisons of the acceleration responses of the ground surface

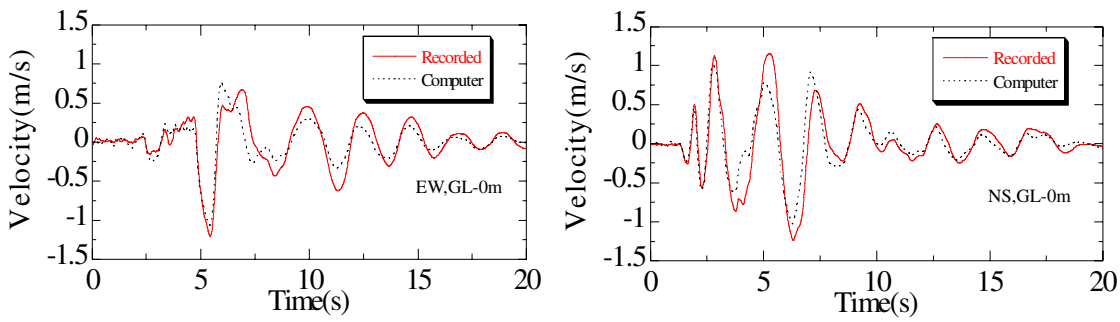


Fig. 9 Comparisons of the velocity responses of the ground surface

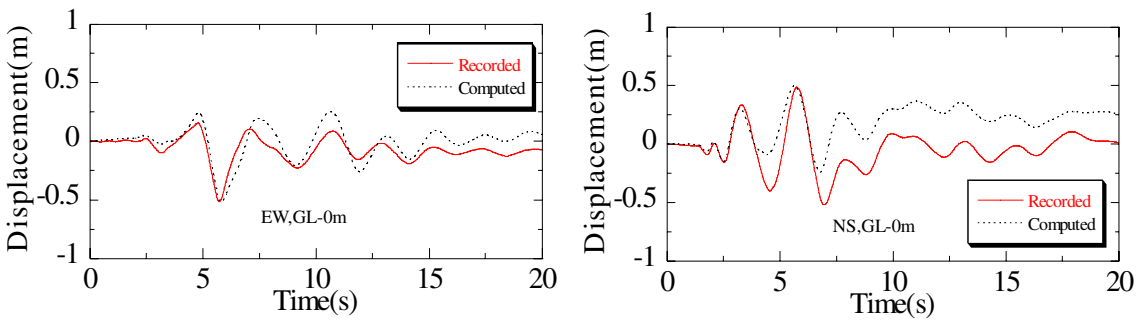
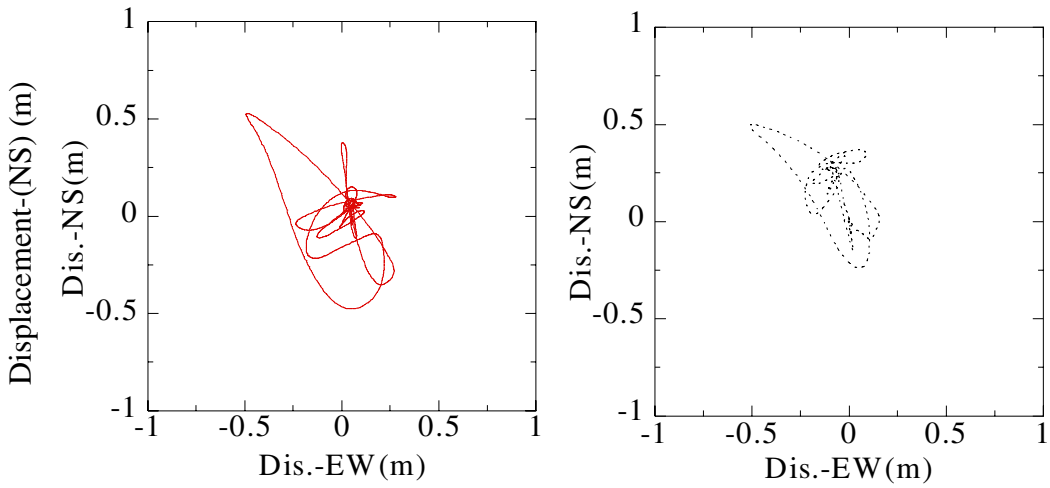


Fig. 10 Comparisons of the displacement responses of the ground surface



(Measured) (Computed)

Fig. 11 Comparisons of the orbits of displacement responses of the ground surface

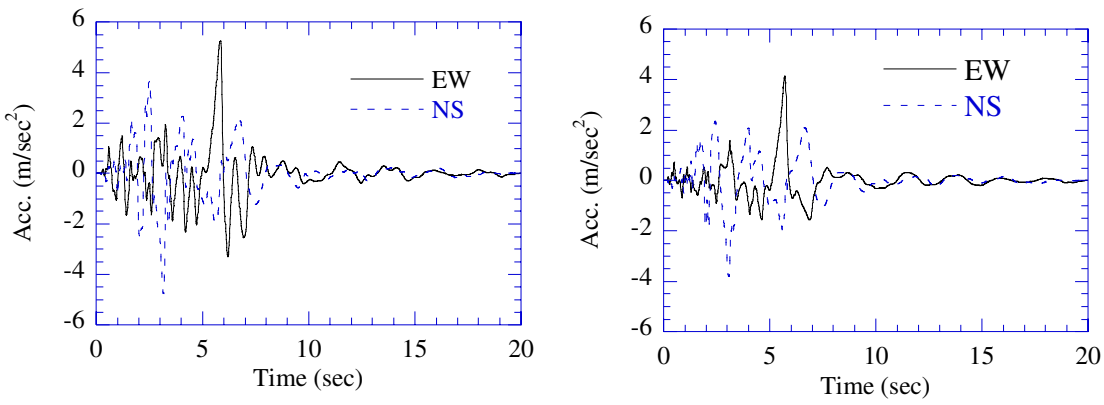
Figure 10 shows a comparison of the displacement responses of the ground surface from the observations and the computation. The general tendency is for the computed results to be larger than the observed ones in whole time duration in both NS and EW directions. The phases of the displacement responses, however, are in good agreement. It is also known that in the EW direction, the residual displacement obtained from the computation has an opposite tendency to that of the recorded data.

Figure11 shows a comparison of the orbit of the displacements of the computed and the recorded results. The computed and the observed results are in good agreement with respect to tendency.

Acceleration and Lateral displacements responses

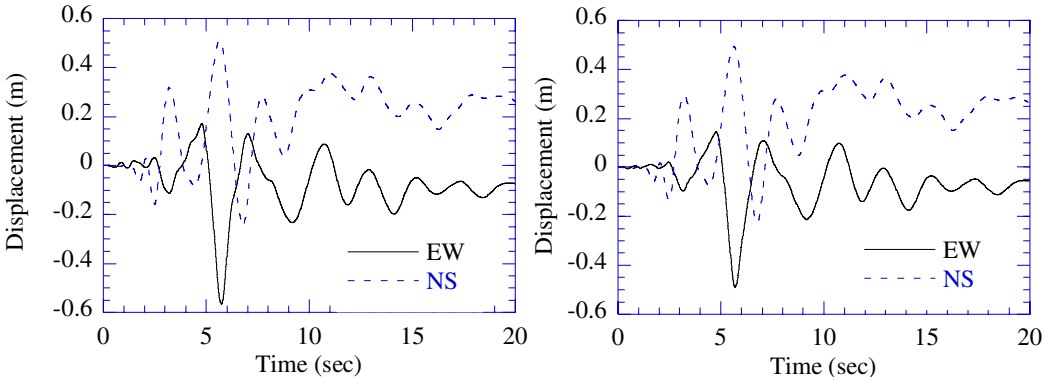
The acceleration responses of the superstructure and the ground surface at faraway field are shown in Figure 12. It was found that the acceleration responses of the building were larger than the acceleration responses of the ground. After liquefaction, the acceleration responses of both the structure and the ground surface decrease significantly.

The lateral displacements of the superstructure and the pile heads are shown in Figure 13. The displacement responses of the superstructure are similar to those of the ground surface. At the end of the calculation (t=20 sec), the building inclined and remained in a residual displacement towards the NW, which is different from the observed tendency in which the building inclined towards the NE. The reason is that the computation of the analysis only considers the behavior of the pile foundation and the ground during an extensive earthquake (t=20sec), in which the consolidation after the major waves, which would influence the inclination of the pile foundation in an actual site, is not computed.



(a) Acceleration responses of the structure (N1) (b) Acceleration responses of the ground surface (N2)

Fig. 12 Acceleration responses

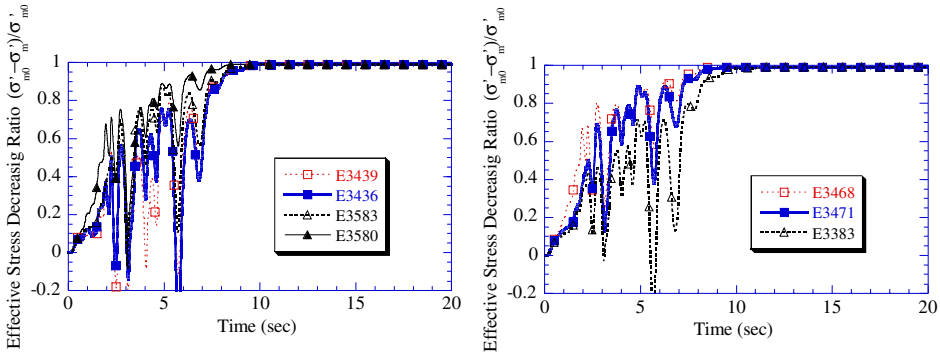


(a)Displacement responses of the structure (N1) (b) Displacement responses of the ground surface (N2)

Fig. 13 Displacement responses in the directions in the EW and NS

Effective stress decreasing ratio (ESDR)

Figures 14 shows the ESDR of the soil elements around the piles at the corners, at the neighboring areas of the pile foundation, and at faraway field shown in Figure 2, respectively. It can be seen that liquefaction took place at about 8 sec and that the excess pore water pressure ratio of the soil elements around footings F2 and F5 developed significantly, while the others did not. It shows that the interaction between these soil elements and the piles remained intensive after liquefaction. It was also found that the liquefaction of the soils within the foundation occurred faster than that of the soils at a faraway field. One possible reason for this is that the soils within the foundation were not firmly confined by the group piles, which were separated from each other at quite large distances.



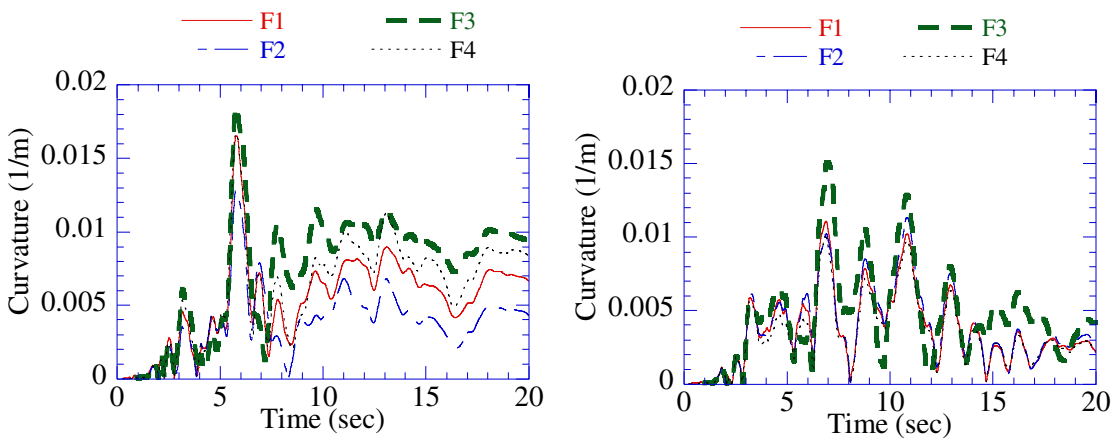
(a) Surrounding the piles at the corners (b) Within foundation and at faraway field

Fig. 14 Effective stress decreasing ratio of different soil elements

Curvature responses

Figure 15 shows a so-called resultant curvature, whose value is equal to the root of the summation of the square of the curvature in the x direction plus the square of the curvature in the y direction, at the pile head and at the bottom of the reclaimed layer for F1 to F4, respectively. The curvature reached a large value at about the 6 sec point, when a large acceleration in the x direction also took place at N1. The curvature responses show a longer period after liquefaction and a large residual curvature remaining on F3, which corresponds to the location of the cracks examined by the investigation at pile No. 3. In Figure 15(b), the curvature responses express a large value at about the 7 sec point. Compared to the upper part of the piles, the curvature of the piles at the bottom of the reclaimed layer after liquefaction vibrated for a relatively shorter period, indicating that the kinematic behavior of the interaction between the soil and the piles at the bottom of the Bs layer is different from that at the pile head. The residual deflections of the piles at the bottom of the reclaimed layer were less significant than those at the upper parts of the piles. However, a relatively larger residual deformation of the pile occurred at the bottom of the reclaimed layer at F3, which is shown in Figure 15(b).

According to Table 1, the failure curvatures of D=500 mm and D=600 mm of the SC pile and the PHC-A pile are 2.48×10^{-3} and 3.48×10^{-3} (1/m), respectively. In Figure 15(a), the curvature response at the pile heads exceeds both the failure curvatures of D=500 mm and D=600 mm, which shows that the pile segments would be damaged due to the earthquake. Also, it can be seen that the curvature responses of the segment at the bottom of the reclaimed (Bs) layer exceeds the failure curvatures of both D=500 mm and D=600 mm.



(a) At the pile heads

(b) At the bottom of the Bs layer

Fig. 15 Time histories of the curvature responses at different locations

CONCLUSION

In the present paper, a case study in which a building located on a reclaimed land in Fukaehama, near Kobe City, was damaged in the 1995 Hyogoken-Nambu Earthquake, was studied by the 3-D effective stress analysis with FEM. In order to simulate the mechanical behavior of the soils properly, the parameters used in the constitutive models for the soils were carefully investigated with laboratory tests and the liquefaction strength curve was examined. As a result, the computed acceleration responses, velocity responses, and displacement responses of the ground surface agreed well with the observations.

The obtained conclusions can be listed up as follows:

a) The occurrence of liquefaction shelters the upper structure from being excited by an earthquake.
b) The occurrence of liquefaction causes the large bending moment and shear stress developed in the pile segment at boundary between liquefied and non-liquefied soil layers which may damage the piles.
c) The distance of the piles installed in the footing influences the built-up of the excess pore water pressure.
d) The damage on both pile heads and low segments of the pile at the boundary takes place before the complete of liquefaction during these major earthquakes.
e) Even if the lateral spread of the ground dose not occur, the damage on the piles due to liquefaction still may take place.

ACKNOWLEDGEMENTS

The authors wish to thank Mr. Nishiraku of SHO-BOND Construction, Inc. and Mr. Koike of A & D Design Consultant, Inc. for providing the building data.

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