A Liquefaction Analysis on the Seismic Behaviors of a Group-Pile Foundation

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1. Introduction

A lot of structures were destroyed during 1995 Hyogoken-Nanbu earthquake in Hyogo prefecture. It is found by field investigation that not only the pile heads but also the lower parts of piles of those structures cracked or failed. This phenomenon indicates that the bending moments to cause damage of the piles are not only from the inertial force of super-structures but also the kinematic action due to ground deformation. In particular, when the surrounding ground of the structures liquefied during an earthquake, piles will respond to the earthquake more complicatedly. In this case study, a five-floor building with pile foundations consisted of SC and two PHC piles (shown as figure 1) is studied by a 3-D numerical analysis. The pile foundations located in a reclaimed ground during the earthquake and were damaged[1]. This case study is focused on the development of excessive pore water pressure (E.P.W.P) and effective stress decreasing ratio (E.S.D.R, $(s'_{m0} - s'_{m})/s'_{m0}$) of the soil surrounding the infrastructure due to the earthquake in order to investigate the interaction between soils and structures, and also the influence of liquefaction on the response of structures.

2. Outline of the Study Case

The building was 350 m away from the closest coastal line. After the earthquake, the sand boiling phenomena, which indicates the occurring of liquefaction, were found in the region where the building is located. The building consists of a five-floor superstructure, footings and 500mm or 600mm diameter piles (shown as figure 1). In order to inspect the soundness of the piles, direct observations method for pile shaft surface, Bore-Hole Television System, and velocity logging method were used after the earthquake. The piles are consisted of SC part, two PHC-A parts at different depth in the ground. As to the degree of the damage of the piles (also see figure 1), cracks were found in the piles No. 1, No. 2 and No.3, and an intrusion of soil into cracked pile was found in the pile No.1. After the earthquake, 30-70 cm subsidence were observed at the footing F1 to F3 and this building was observed being tilted to the North- East side at the angles 1/80 to the north and 1/30 to the east.

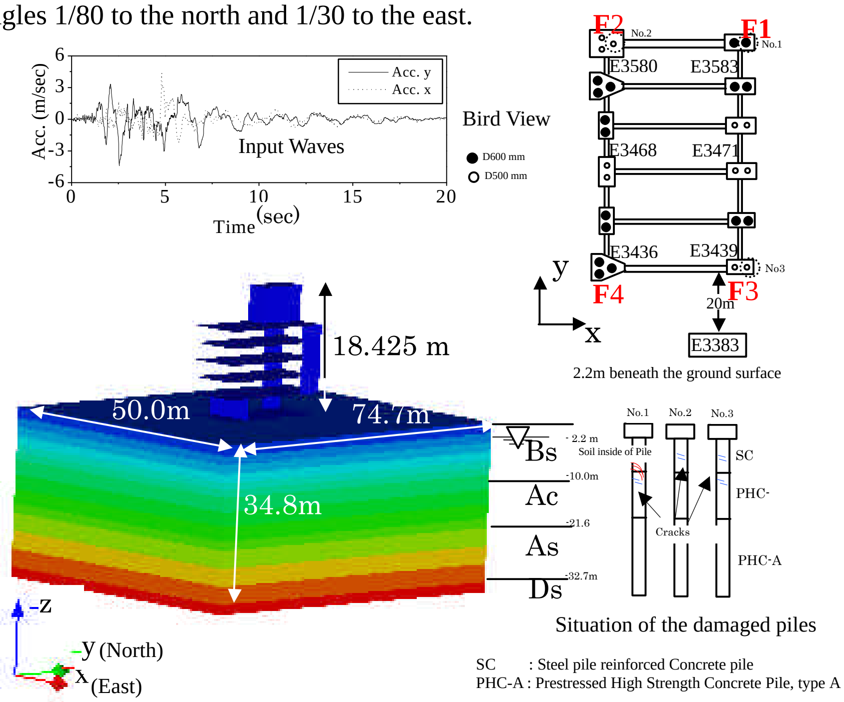

Figure 1 Layout of the study case

Table 1 Material parameters in the analysis

Name of Soil Profile	Bs	Ac
Density:	2.0	1.7
Initial Void Ratio:	0.42	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	1.230
Phase transformation ratio M_m	0.91	1.026
Hardening Parameter B_0	3500.0	55.0
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

3. Numerical Simulation

In this simulation, a cyclic elasto- plastic model[2] is used for sand and a cyclic elasto- viscoplastic model[3] is used for clay. Those models have been verified to reproduce the experimental results well under various stress conditions and have been applied to case studies. The material parameters used here are determined from the survey and are shown in Table 1. The governing equations for the coupling problems between soil skeleton and pore water pressure are obtained

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based on the two- phase mixture theory (Biot, 1962). Using a u-p (displacement of the solid phase- pore water pressure) formulation (Zienkiewicz, 1982), the liquefaction analysis is formulated. The full system is shown as figure 1, in which the piles are described by axial force dependent model[4)], the columns of upper structure are represented by elastic beam element, the slabs and walls are represented by the shell element, and the each weight of each floor has been distributed onto the each slab. The parameters of axial force dependent model of the piles are shown at table 2. The input waves N83E (X direction) and N383E (Y direction) are shown in figure 1. The side boundaries of the simulated system are assumed to be equi-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-b is used in this dynamic analysis with a time interval 0.002 sec.

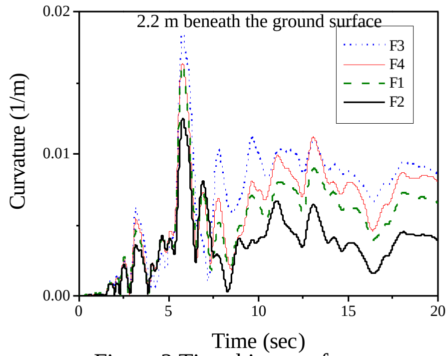

Figure 2 Time history of curvatures

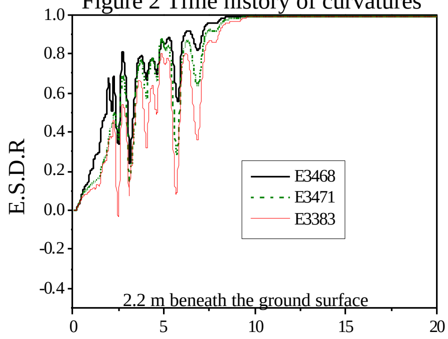

Figure 3 Time history of E.S.D.R

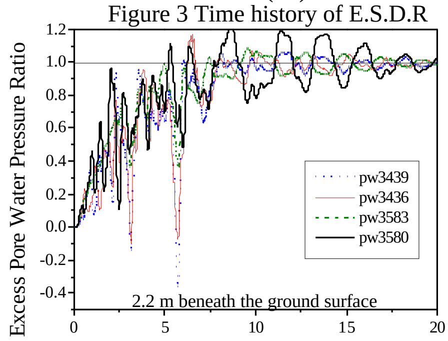

Figure 4 Time history of E.P.W.P

Table 2 Main parameters of piles of axial force dependent model

Concrete Young's Modulus Ec (KN/m^2) Steel Young's Modulus Es (KN/m^2)	24500000 205800000
Concrete compression strength σ_c (kPa), Concrete tension strength σ_t (kPa)	78400 4704
Degrading parameter of concrete Degrading parameter of steel	0.2069 0.4

4. Results and Discussions

Figure 2 shows that the calculated curvature (the root of addition of the x curvature square and the y curvature square) on the pile segments (-2.2m) of F1 to F4 respectively. In Figure 2, the curvature reached a large value at about 6 second. It also shows the curvature of F3 reaches the largest curvature but F2 reaches the smallest. The curvatures act at a longer period after liquefaction occurs (after about 7.5 sec, see figure 3). Figure 3 shows the E.S.D.R of soil elements (E3468 and E3471) inside the foundation and E3383, which locates 20m away from this infrastructure. The effective stress of the soils inside the structure (E3468 and E3471) decreases more rapidly than the far field soil elements. By comparing of E.S.D.R of E3468 and E3471, it can be shown that the soil and piles are not only pushing forth and back in the y direction but also x direction due to a 3-D simulation and the response of the soil elements (E3468 and E3471) behaves quite differently. Figure 4 shows the E.P.W.P of soil elements (E3583, E3580, E3439 and E3536) that the E.P.W.P of E3489 surrounding F3 is pushed back and forth hardly by F3 at about 6 second. On the other hand, E.P.W.P of E3580 responds extensively, as the others do not, after about 9 second. It is because the larger damaged pile segments have less influence of the loading transmission to the surrounding soil elements and the infrastructure, in this numerical simulation, has been transferred its weight to pile F2 due to this earthquake.

5. Conclusion

The soils within a foundation behave quite differently to the far-field soil. In this study, the effective stress of the soil elements inside the foundation decreases more rapidly than the far-field soil elements. The different mechanical behaviors of the far-field soils and the soils surrounding a pile foundation should be taken into consideration by a 3-D simulation. It also shows that a large deformation of pile segments occurs before the occurrence of liquefaction. Furthermore, the phenomena of uneven subsidence of the structure due to the earthquake should be investigated by more adequate boundary information.

References

1) Report on the Hanshin- Awaji Earthquake Disaster AIJ, JGS, JSCE, JSME, JSEE, Building Series Volume 4, Damaged to Building foundations (1998), pp.333-368 (in Japanese) 2) Oka, F., Yashima, A., Tateishi, A., Taguchi, Y. and Yamashita, S. (1999), Geotechnique, Vol.49, No.5, pp.661-680. 3) Oka. F. (1992), Proc. 4th Int. Sym. on Numerical Models in Geomechanics, Swansea, pp. 105-114. 4) Zhang, F., Yashima, A., Kimura, M. and R. Uzuoka, (2000), Vol.2, Proc. of the GeoEng2000, pp.423 (CD-ROM).