Seismic performance of multi-story building with pile foundation in liquefiable ground including post-earthquake consolidation settlement

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ABSTRACT: A multi-story parking building with steel frame was designed and constructed according to the Building Standards of Japan Architecture. This study is to investigate the seismic performance of the building during and after a huge earthquake that will definitely hit the central part of Japan in near future. Special attention is paid to the differential settlement caused by the liquefaction and the long-term settlement after the earthquake. The analysis is carried out by 2D soil-water coupled dynamic/static finite element analysis using a program named as DBLEAVES (Ye, 2011). The input earthquake wave is a kind of 3 synchronization huge earthquake wave whose main shock lasts about 150s with a maximal acceleration of 182 gal. The ground behavior is described by Cyclic Mobility Model (Zhang et al., 2007, 2010, 2011). From the analysis, it is known that not only the liquefaction but also the long-term settlement after earthquake should be taken serious consideration. Meanwhile, even if the seismic judgment by the design code is OK, detailed calculation may reveal the risk of under estimation due to the shortcoming of such kind of overwhelming design code. In a word, sometime judgment by individual engineer will become crucial important.

INTRODUCTION

In evaluating the damage caused by earthquake, attention has been paid exclusively to ground liquefaction and displacement during or immediately after earthquakes. For this reason, only analyses of liquefaction in sandy ground during earthquakes have been performed in most dynamic analyses. On the other hand, to investigate the damage to a complex ground that contains sand, silt or clay layers, and long-term settlement over a period of several weeks or even years after the earthquake cannot be neglected, because of the long time for the dissipation of excess pore water pressure and the recovery of ground rigidity.

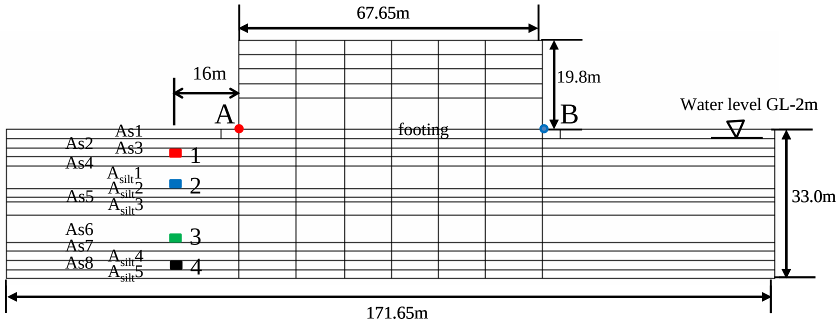
Different from the settlement induced by liquefaction in the pure sand ground, the issue of earthquake-induced settlement of foundation on natural ground is more complex. Many studies including tests and numerical simulation about the liquefaction and long-term settlement can be found in the references (Noda et al, 2009; Zhou et al, 2009; Mojtaba et al, 2012). Unfortunately, however, the mechanics of the post-liquefaction deformation of complex grounds has not been clarified sufficiently, and the counter measures against the damage due to the deformation still rely on experiments and empirical calculation.

In this study, the seismic performance of a 6-storey parking building in liquefiable ground is investigated by finite element method (FEM) considering the long-term consolidation settlement after the liquefaction. The calculations are carried out using a 2D and 3D soil-water coupling analysis program named as DBLEAVES (Ye, 2007; Ye, 2011). The program could not only analyze the static and dynamic behavior of natural complex grounds, but also could solve soil-structure interaction problems. The applicability of the proposed numerical method has been firmly verified by the investigation on group-pile foundations (Jin et al., 2010; Bao et al., 2012). A rotating hardening elastoplastic model named as Cyclic Mobility Model (CM Model) is adopted in this analysis code to properly describe the nonlinear behavior of soils during and after large earthquake motions. The constitutive model, which was developed at the bases of modified Cam-clay model, has a feature whereby the influence of the stress-induced anisotropy, the influence of density, the structure of the soil formed in the natural sedimentary process, different loading conditions and drained conditions can be properly described in a unified way.

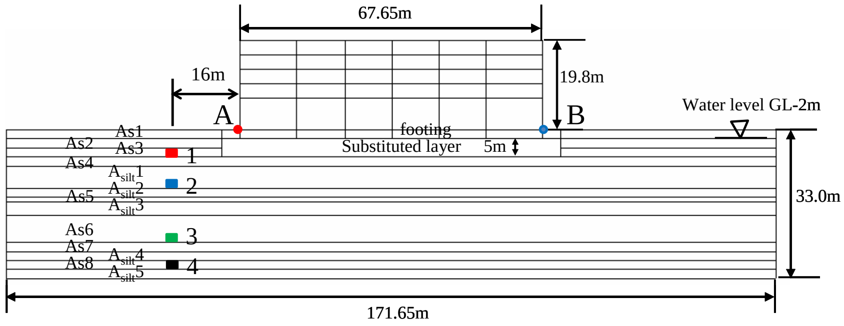
ANALYTICAL SECTION AND THE GROUND PROPERTY

Analysis range and soil layer division

According to the geometrical condition of the ground and the upper structure, the analysis of a full system, which consists of soil ground, upper structure and foundation, is carried out under plane strain conditions. The object section for the two dimension finite element analysis is selected as shown in FIG.1. The ground which composed of sand layers and silt layers is assumed to be uniform in horizontal direction. For the thickness of each layer, A_{s1}=2m、A_{s2}=2m、A_{s3}=2m、A_{s4}=2m、$A_{silt.1}$=5m、$A_{silt.2}$=2m、A_{s5}=1m、$A_{silt..3}$ =3m、A_{s6}=6m、A_{s7}=2m、$A_{silt..4}$=2m、A_{s8}=2m、$A_{silt..5}$=2m. The groundwater level locates at the depth of 2.0m below the ground (GL-2.0m). The ground condition used in the analysis is based on the result of the boring survey. In the analysis, point A in the left side and point B in the right side of the structure on the ground surface are selected to investigate the differential settlement. Element 1 in the upper sand layer (GL-4.0m), element 2 in the medium silt layer (GL-1.0m), element 3 in the bottom sand layer (GL-22.0m) and element 4 in the bottom silt layer (GL-29.0m) are also selected to investigate ground liquefaction.



(a) Case-1 long-pile type



(b) Case-2 improved ground（short-pile type）

FIG.1. Analysis range and ground profiles

Two types of foundation

Two cases with different kinds of foundation system are examined.

Case-1 : Long-pile type foundation

Piles are used for protecting structures by helping to keep the total and differential settlements small. As shown in FIG. 1, the piles are of length L31m, diameter D 1.2m and spaced 12.65m, 10.85m, 10.5m, 10.5m, 10.5m, 12.65m (from left to right in the analysis section). The footing is 75.65m long and 2m thick. A rigid connection is assumed between footing and piles (fixed-head piles).The piles are assumed to be elastic beams at the present study.

Case-2 : Short-pile type foundation

Numerous short piles with the length of 5m were constructed in the real situation according to the changed design plan. The short-pile type foundation is simplified as an improved ground with the zone of 75.65m in length and 5m in depth to avoid the enormous computation. In other words, the foundation with many short piles is considered to be an improved ground with an elastic material. As for the rigidity of the improved ground, it is evaluated in the way that the rigidity of short piles and the rigidity of soils are averaged with weights based on each volume ratio.

Constitutive model for soils and ground parameter

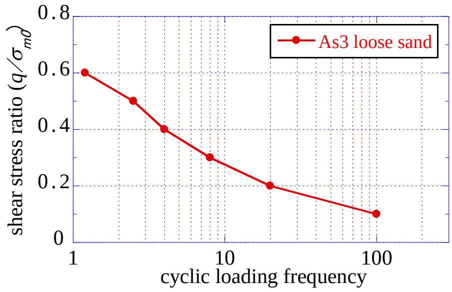
In the dynamic analysis with FEM for the ground, the sand and silt are modeled with the CM Model, a rotating hardening elastoplastic model with the main feature to describe the static and dynamic behavior of sand and silt, considering the effect of the stress-induced anisotropy, the density and the structure in a unified way. In the model, eight parameters are employed, among which five parameters, M, N, λ, κ and ν, are the same as those in the Cam-clay model. The other three parameters, a: the parameter controlling the collapse rate of the structure, m: the parameter controlling the loosing rate of the overconsolidation ratio or the change in density of the soil, and b_r: the parameter controlling the developing rate of the stress-induced anisotropy, have clear physical meanings and can be easily determined by undrained triaxial cyclic loading tests and drained triaxial compression tests. A detailed description of this model can be found in the references (Zhang et al., 2007, 2010, 2011). The eight ground parameters of each soil layer used in calculation are shown in Table 1. The initial values of the state variables employed in the constitutive model are given in Table 2. The liquefaction strength curve of loose sand layer A_{s2} (the layer A_{s2} is a typical loose sand layer that may liquefy easily) is shown in FIG.2.

Table 1. Material parameters of each soil layer

Layer	λ	κ	ν	R_f	e_0	a	b_r	m
Very loose sand A_{s1} A_{s4}	0.050	0.010	0.30	4.60	0.80	2.2	1.5	0.10
Loose sand A_{s2} A_{s3} A_{s5} A_{s6}	0.030	0.0060	0.30	4.60	0.78	2.2	1.5	0.10
Medium dense sand A_{s7} A_{s8}	0.024	0.0048	0.30	4.60	0.75	2.2	1.5	0.10
Loose silt $A_{silt.1\sim3}$	0.207	0.041	0.35	3.50	1.1	0.10	0.10	3.8
Bottom silt $A_{silt.4\sim5}$	0.207	0.035	0.35	3.50	1.1	0.10	0.10	3.8
Improved material	E=10^{5}kPa ; ν=0.25							

Table 2. Physical and state variables of each soil layer

Layer	OCR	D_r (%)	R^*_0	ζ	k (m/sec)	γ (kN/m^3)
Very loose sand A_{s1} A_{s4}	4.0	53	0.80	0	1.0E-4	17.6
Loose sand A_{s2} A_{s3} A_{s5} A_{s6}	5.0	58	0.80	0	1.0E-4	17.6
Medium dense sand A_{s7} A_{s8}	6.0	78	0.80	0	1.0E-4	17.6
Loose silt $A_{silt.1\sim3}$	2.5		0.60	0	1.0E-6	16.7
Bottom silt $A_{silt.4\sim5}$	2.5		0.60	0	1.0E-6	17.6
Improved material						



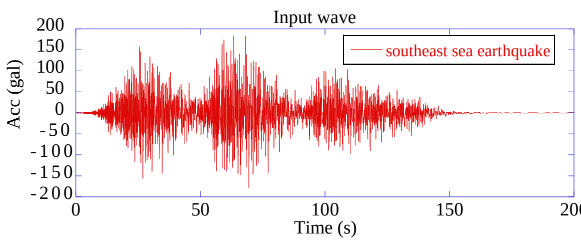


FIG.2. Liquefaction strength curve **FIG.3. Time history of acceleration of 3**

of the alluvial sand layerA_{s2} **synchronization earthquake wave**

EARTHQUAKE WAVE AND FINITE ELEMENT SIMULATION

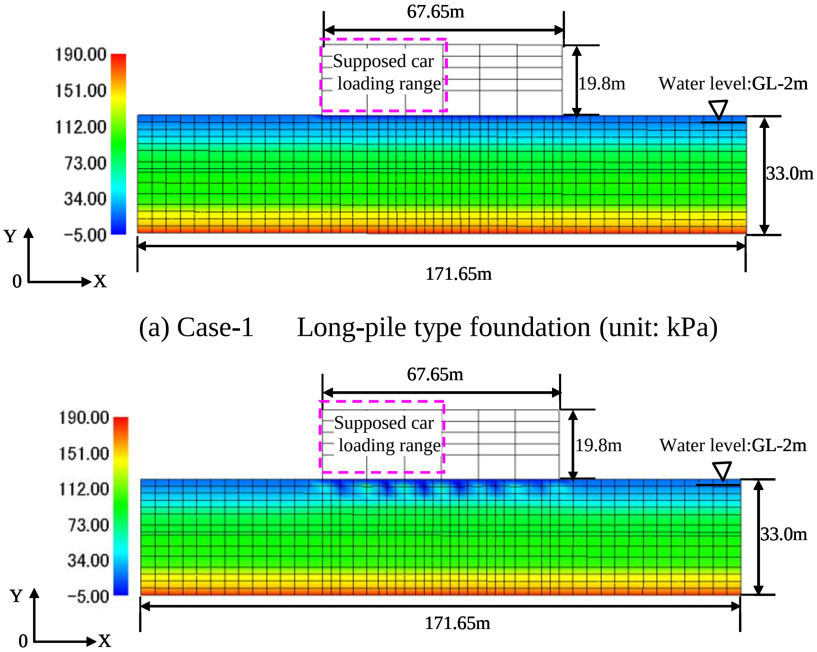
The input earthquake wave is an artificial 3-synchronized earthquake wave considering the faults laying in the Eastern sea, the Southeast sea and the South sea of Japan. FIG. 3 shows the time history of acceleration of the earthquake wave. The whole wave lasts for 200s, and the main shock lasts about 150s with a maximal acceleration of 182 gal. The seismic intensity of the artificial wave is less than 5 degree with the seismic standard of Japan.

FIG. 4 shows the finite element mesh of the ground and the upper structure. In the case of dynamic analysis, an equal displacement boundary condition, sometime also called as periodic boundary condition, is used for two side boundaries to deal with the energy loss problem. On the other hand, the bottom is assumed to be fixed in horizontal and vertical directions. The drained boundary is set the same as hydraulic boundary at the ground level of -2m. Newmark-β method is used and the integration time interval is 0.002s. Rayleigh type of initial-rigidity-proportional attenuation is used and the damping values of the soils, the structure and the piles are assumed to be 2% and 10% for the first and second modes respectively in the dynamic analysis of the full system.

Before the dynamic analysis, a static analysis considering the structure-ground as a whole system is carried out to get the initial effective stress of the ground. The distribution of initial mean effective stress caused by the self-weight of upper structure, the parking-car load and ground is also shown in FIG. 4. The living load caused by the parking cars from the second floor to the sixth floor is non-uniform, so the worst condition that the parking concentrates on the whole left half sides of all the floors is assumed in the analysis. The static analysis of 3.5 years of consolidation in liquefied ground is also conducted after the dynamic analysis of the earthquake motion.

RESULTS AND DISCUSSION

FIG. 5 shows the distribution of excess pore water pressure ratio (EPWPR) in the ground at the end of earthquake motion, and the time histories of EPWPR at selected elements, in the free field 16m away from the foundation. From the results it is understood that, the ground beside the foundation liquefied in the depths of GL-4.0~-6.0m and GL-19.0~-25.0m (EPWPR≒1.0) in both two cases. The ground inside the foundation, however, liquefied severely in the depth of GL-2.0~-6.0m in the case of Long-pile type foundation, and did not occur in the case of Short-pile type foundation. In other words, the improved ground has a better capacity to resist liquefaction. FIG. 6 shows the excess pore water pressure (EPWP) distribution in the ground at the end of earthquake motion and the mean effective stresses at the selected elements during the earthquake motion. Due to the low permeability of the medium silt layer (GL-8.0~-19.0 m), the EPWP mostly develops in the bottom sand layers (GL-19.0~-25.0 m). Case-1 and Case-2 present similar EPWP distribution variations at the end of earthquake motion, except the zone beneath the foundation. The EPWP is generated to the maximum value of approximately 165kPa at the end of earthquake motion in both cases. It takes, however, about 3.5 years, quite a long time to a complete dissipation of EPWP in both cases.

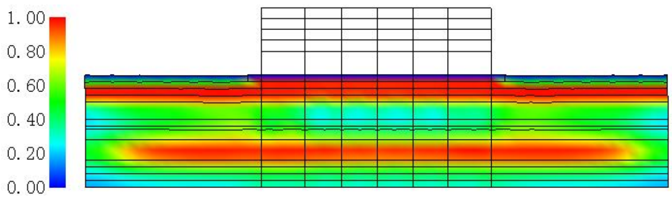


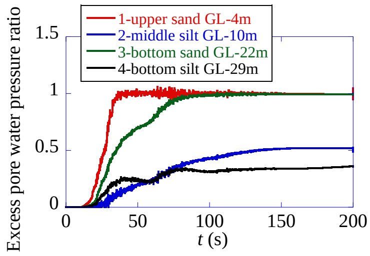
(a) Case-1 Long-pile type foundation (unit: kPa)

(b) Case-2 Short-pile type foundation (unit: kPa)

FIG.4. Mean effective stress distribution of the ground due to self-weight

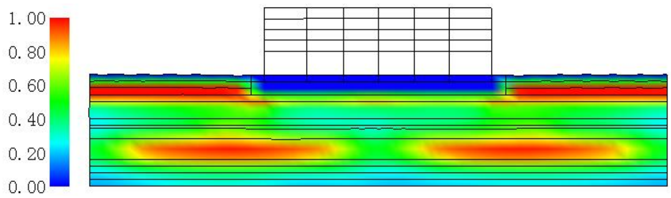
FIG. 7 shows the distribution of displacement vector at the end of earthquake motion. Obviously, for the upper structure and the foundation, larger horizontal displacement occurred in the case of Short-pile type foundation than in the case of Long-pile type foundation. But, for the ground in the free field at the two sides away from the foundation, horizontal displacement occurred severely in both two cases. The displacement is mainly in horizontal direction during earthquake motion. FIG. 8 shows the distribution of displacement vector 3.5 years after earthquake. Along with the post-liquefaction consolidation of the ground, part of the horizontal displacement occurred during the earthquake motion may recover somehow, but the vertical displacement increased to a large level due to the long-term consolidation of the ground. The amount of displacement of the upper structure and the foundation is larger in the case of Short-pile type foundation than in the case of Long-pile type foundation, while the ground displacement of free field at the two sides away from the foundation is also the same in both cases.

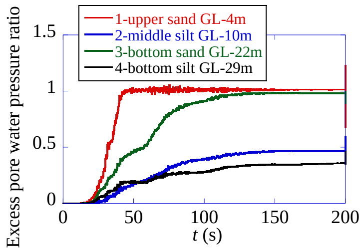




(i) Distribution of EPWPR (ii) Time histories of EPWPR

(a) Case-1 Long-pile type foundation

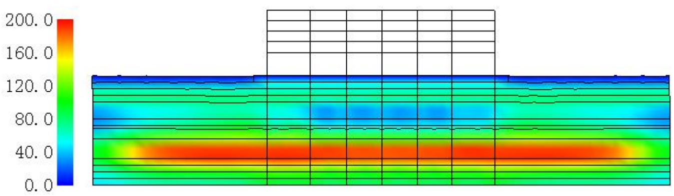


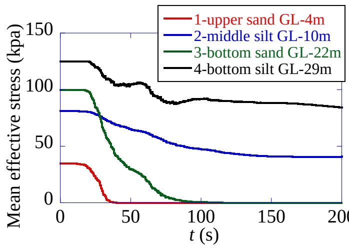


(i) Distribution of EPWPR (ii) Time histories of EPWPR

(b) Case-2 Short-pile type foundation

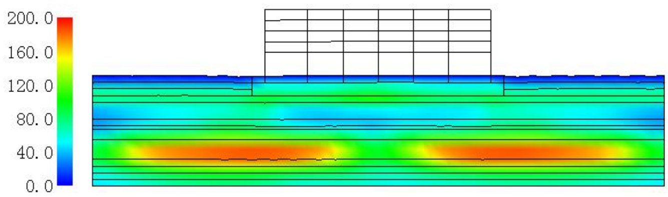
FIG.5. Distribution of EPWPR immediately after earthquake and time histories of EPWPR of the selected elements during earthquake

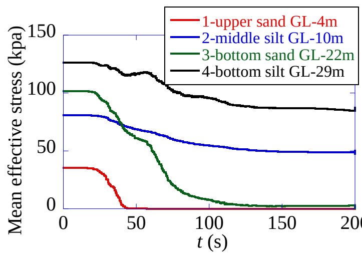




(i) Distribution of EPWP (ii) Time histories of EPWP

(a) Case-1 Long-pile type foundation

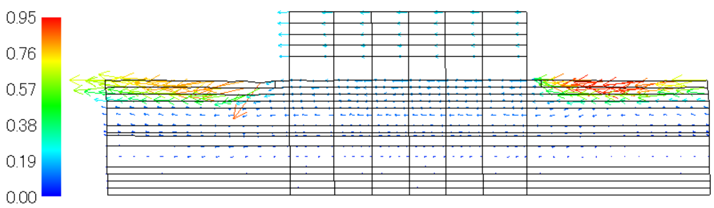




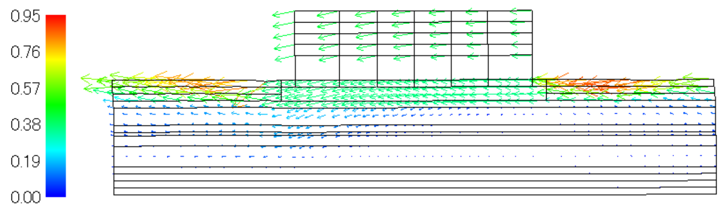
(i) Distribution of EPWP (ii) Time histories of EPWP

(b) Case-2 Short-pile type foundation

FIG.6. Distribution of EPWP immediately after earthquake and time histories of mean effective stresses of the selected elements during earthquake

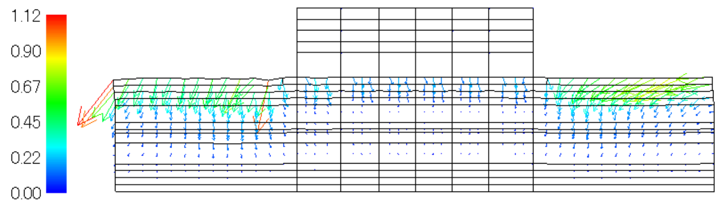


(a) Case-1 Long-pile type foundation (unit: m)

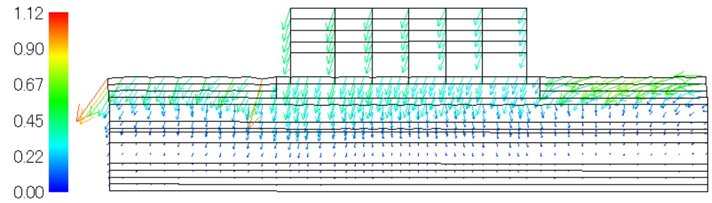


(b) Case-2 Short-pile type foundation (unit: m)

FIG.7. Displacement vector immediately after earthquake



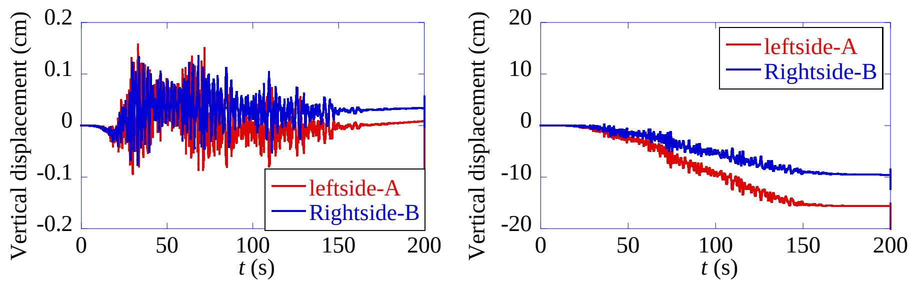
(a) Case-1 Long-pile type foundation (unit: m)



(b) Case-2 Short-pile type foundation (unit: m)

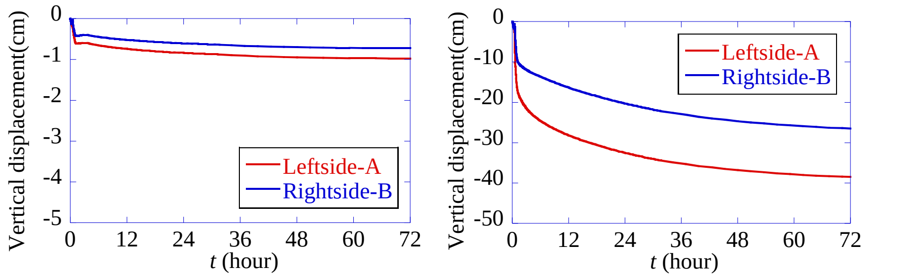
FIG.8. Displacement vector 3.5 years after earthquake

The instantaneous settlements during earthquake of point A and B at the two sides of the structure are shown in FIG. 9. Large differential settlements occurred under the structure foundation. The differential settlement of the Short-pile type foundation is obviously larger than that of the Long-pile type foundation. The time histories of long-term settlements within 72 hours after the earthquake motion are shown in FIG. 10. It is clear that the total settlements of foundation are composed of instantaneous settlement and long-term post-earthquake settlement, and most of the differential settlement occurs immediately after the earthquake while the post-earthquake settlement is relatively uniform despite its large amplitude.



(a) Case-1 Long-pile type foundation (b) Case-2 Short-pile type foundation

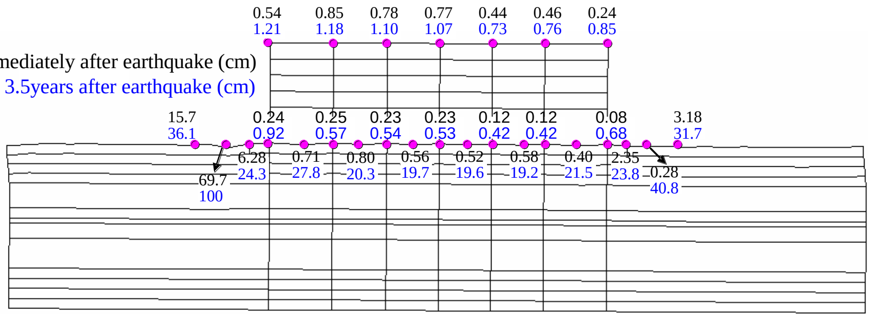
FIG.9. Settlements on the surface of two sides of the structure foundation during earthquake



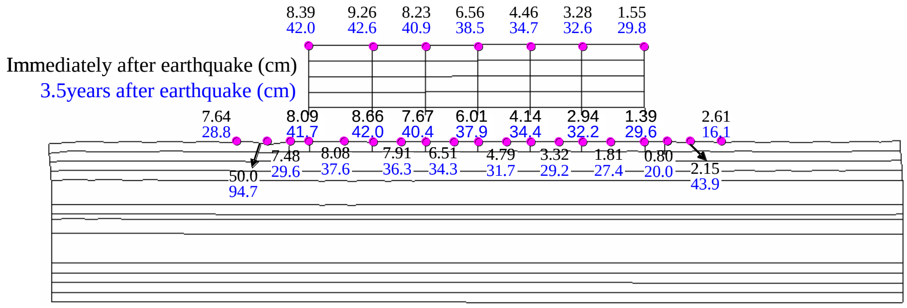
(a) Case-1 Long-pile type foundation (b) Case-2 Short-pile type foundation

FIG.10. Settlements on the two sides of the structure 72 hours after earthquake

FIG.11 shows the comparison of instantaneous settlement and long-term settlement of the building between the two cases. In the case of Long-pile type foundation, the differential settlement of the left and right side ends is 0.16cm immediately after earthquake and 0.24cm 3.5 years after earthquake. However, in the case of Short-pile type foundation the differential settlement is 6.70cm immediately after earthquake and 12.10cm 3.5 years after earthquake. It means that about 60% percent of the differential settlement occurred immediately after earthquake. In the case of Short-pile type foundation, the inclination degree of building is 1.8‰ based on the amount of differential settlement when assuming it to be uniform. This detailed calculation reveals the risk of under estimation of the differential settlement in Case-2 that cannot be evaluated accurately by the design code.



(a) Case-1 Long-pile type foundation



(b) Case-2 Short-pile type foundation

FIG.11. Comparison of instantaneous and long-term settlements of the building

CONCLUSIONS

In this study, the liquefaction and the post-liquefaction deformation of a layered sandy/silt ground on which a pile-supported building is laid, was investigated using soil-water coupling elastoplastic dynamic/static finite element analysis program (DBLEAVES). Two cases of analyses, which are the Long-pile type foundation and Short-pile type foundation, were carried out. The main results are outlined below:

Liquefaction occurred mainly in the sand layers including loose sand and medium dense sand in the ground. The ground below the foundation liquefied more severely in the case of Long-pile type foundation than in the case of Short-pile type foundation, while it showed the same results in the free field ground at the two sides away from the foundation in both cases.

The differential settlement in the building caused by liquefaction and the long-term settlement due to consolidation of the soils after the earthquake can be confirmed at a maxim value of 12.10cm in the case of Short-pile type foundation, which did not occur in the case of Long-pile type foundation. In another word, the Long-pile type foundation has a better capacity of resisting differential settlement while the Short-pile type foundation (improved ground) has a better capacity of resisting ground liquefaction. No matter in what cases, most part of the differential settlement occurs immediately after the earthquake while the post-earthquake settlement is relatively uniform despite its large amplitude.

The seismic stability of the structure foundation was well evaluated by this numerical analysis, although the damage of building in the case of Short-pile type cannot be confirmed by the liquefaction ground. Attention should be paid not only to the liquefaction behavior of the ground during the earthquake, but also to the long-term settlement after the earthquake motion. The parking building with steel frame was designed and constructed according to the Building Standards of Japan Architecture, however, in some critical condition, even if the seismic judgment by the design code is OK, detailed calculation may reveal the risk of under estimation of differential settlement that may give rise to serious problem. In another words, sometime judgment by individual engineer will become crucial important.

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