Soil-water coupling analysis on seismic behavior of an alternately layered sand-silt ground with different foundation system

Xiaohua Bao i), Guanlin Ye ii), Bin Ye iii) and Feng Zhang iv)

i) Assistant Professor, Department of Civil Engineering, Shenzhen University, Nanshan District, Shenzhen 518060, China. ii) Associated Professor, Department of Civil Engineering, Shanghai Jiao Tong University, Minhang District, Shanghai 200240, China. iii) Associated Professor, Department of Geotechnical Engineering, Tongji University, Yangpu District, Shanghai 200092, China. iv) Professor, Department of Civil Engineering, Nagoya Institute of Technology, Showa-ku, Gokiso-cho, Nagoya 466-8555, Japan.

ABSTRACT

In evaluating the damage caused by earthquakes, attention has been paid exclusively to ground liquefaction of sandy grounds during earthquakes. However, studies of post-seismic damage, especially damage to clayey/silty grounds have also been reported. In this paper, the behavior of an actual alternately layered sand-silt ground and foundation-superstructure system during and after an earthquake is investigated. The calculations are carried out using a 2D/3D soil-water coupling analysis program named as DBLEAVES that can not only describe the static and dynamic behavior of natural complex grounds, but also can solve soil-structure interaction problems. Two cases with different foundations (long-pile type foundation and dense short-pile type foundation) are analyzed. A rotating hardening elastoplastic model named as Cyclic Mobility model (CM Model) is adopted in the analyses to properly describe the nonlinear behavior of soils during and after large earthquake motions. The results show that the long-pile type foundation is more suitable to resist uneven settlement while the short-pile type foundation has a better resistance to ground liquefaction. No matter what kind of case may be, not only the liquefaction but also the long-term settlement after the earthquake should be taken into consideration seriously.

Keywords: numerical analysis, liquefaction, differential settlement, constitutive model

1 INTRODUCTION

In evaluating the damage caused by earthquakes, 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. Different from the settlement induced by liquefaction in the pure sand ground, the issue of earthquake-induced settlement of foundation on natural ground, which may contain cohesive strata, is more complex because of the low permeability of soils (Matsuda & Ohara, 1991). Noda et al. (2009) investigated the behavior of an actual alternately layered sand-clay ground and embankment-coupled system before, during and after an earthquake using finite element analysis. Zhou et al. (2009) conducted dynamic centrifuge model tests to study the earthquake-induced differential settlement of foundation on cohesive ground. Other studies including tests and numerical simulations about the liquefaction and long-term settlements can be found in the references (Tokimatsu et al, 2012 and Mirjalili 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 calculations.

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 2D/3D soil-water coupled analysis program DBLEAVES (Ye, 2011). The applicability and accuracy of the program have been firmly verified by the investigation on group-pile foundations in real scale (Jin et al., 2010) and shaking table tests (Bao et al., 2012). A rotational kinematic hardening elasto-plastic model named as Cyclic Mobility Model (CM Model) is adopted in this analysis code to properly describe the nonlinear behavior of cohesiveless soils under both dynamic and static loadings, especially the cyclic mobility of sand during liquefaction. With the CM Model and effective stress based FEM code, the mechanical behavior of soil, the change of excess pore water pressure (EPWP) and the consolidation etc. can be well understood.
2 GROUND PROPERTIES AND EARTHQUAKE WAVE

2.1 Ground properties

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 dimensional finite element analysis is selected as shown in Figure 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, As1=2.0 m, As2=2.0 m, As3=2.0 m, As4=2.0 m, Asilt.1=5.0 m, Asilt.2=2.0 m, As5=1.0 m, Asilt.3 =3.0 m, As6=6.0 m, As7=2.0 m, A silt..4=2.0 m, As8=2.0 m, A silt.5=2.0 m. The groundwater level is located at the depth of 2.0 m below the ground (GL-2.0 m). 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. Four elements from different ground layers as shown in Table 1 are also selected to investigate response acceleration and liquefaction of ground.

Two cases with different kinds of foundation system are examined. Those are Case-1 with Long-pile type foundation and Case-2 with Short-pile type foundation. In the dynamic analysis with FEM for the ground, the sand and silt are modeled with the CM Model. 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 2. The initial values of the state variables employed in the constitutive model are given in Table 3. The liquefaction strength curve of sand layer As3, which is a typical loose sand layer that may liquefy easily, is shown in Figure 2.

2.2 Earthquake wave and boundary condition

The input earthquake wave is an artificial 3-synchronized earthquake wave considering the faults lying in the Eastern Sea, the Southeast Sea and the South Sea of Japan. The wave is an E+F seismic excitation, in which ‘E’ represents the emitting excitation at the base and the ‘F’ represents the reflecting excitation at the upper free surface. Figure 3 shows the time history of acceleration of the earthquake wave. The whole wave lasts for 200 s, and the main shock lasts for 150 s with a maximal acceleration of 182 gal.

For the boundary condition, the base nodes of the FE mesh were assumed to be fixed in both x and y direction. The side boundary nodes at the same elevation were all “tied” together to experience the same accelerations. The drained boundary is set the same as hydraulic boundary at the ground level of -2 m. After earthquake motion, the analysis automatically turns to static analysis to simulate post-liquefaction consolidation

2.3 Static analysis

A static analysis considering the ground-structure as a whole system is carried out to get the initial effective stress of the ground before the dynamic analysis. The distribution of initial mean effective stress caused by the self-weight of upper structure and ground is also shown in Figure 4 (the mesh size is 4 m in length and 2 m in height). The live load caused by 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 considering consolidation in 3.5 years is also conducted after the dynamic analysis of the earthquake motion.

Table 2. Material properties of each soil layer.

<table>
<thead>
<tr>
<th>Layer</th>
<th>OCR</th>
<th>Dc (%)</th>
<th>R’o</th>
<th>ζ (m/sec)</th>
<th>k (kN/m^2)</th>
<th>γ (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very loose sand A_{12} A_{14}</td>
<td>4.0</td>
<td>53</td>
<td>0.80</td>
<td>0</td>
<td>1.0E-4</td>
<td>17.6</td>
</tr>
<tr>
<td>Loose sand A_{12} A_{14} A_{58}</td>
<td>5.0</td>
<td>58</td>
<td>0.80</td>
<td>0</td>
<td>1.0E-4</td>
<td>17.6</td>
</tr>
<tr>
<td>Medium dense sand A_{12} A_{58}</td>
<td>6.0</td>
<td>78</td>
<td>0.80</td>
<td>0</td>
<td>1.0E-4</td>
<td>17.6</td>
</tr>
</tbody>
</table>

Table 3. Physical and state variables of each soil layer.
3 RESULTS AND DISCUSSION

3.1 Excess pore water pressure and mean effective stress

Figure 5 shows the distribution of excess pore water pressure ratio (EPWPR) in the ground at the end of the earthquake motion. EPWPR is defined as the ratio of excess pore water pressure to the initial vertical effective stress. EPWPR > 0.95 means liquefaction. From the results it is understood that, the ground beside the foundation liquefied at the depths of GL-4.0~6.0 m and GL-19.0~25.0 m (EPWPR > 1.0) in both two cases. The ground inside the foundation, however, liquefied severely in the depth of GL-2.0~6.0 m in the case of the long-pile type foundation, and did not occur in the case of the short-pile type foundation. In other words, the improved ground has a better capacity to resist liquefaction. Figure 6 shows the excess pore water pressure (EPWP) distribution in the ground at the end of the earthquake motion. The excess pore water pressure is generated with a maximum value of approximately 165 kPa at the end of earthquake motion in both two cases. 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 the earthquake motion, except the zone beneath the foundation at the bottom sand layers. It takes, however, quite a long time (approximate 3.5 years) to complete the dissipation of EPWP in both cases according to the calculation.

Figure 7 shows time histories of EPWPR and mean effective stress of the selected elements in the free field 16.0 m away from the foundation during earthquake motion. The EPWPR increased to 1 while the mean effective stress decreased to 0 in the upper and bottom sand layers. It shows that the sand layers liquefied severely, and the silt layers did not liquefy. The EPWPR showed the same tendency in the free field outside the foundation in Case-1 and Case-2.

3.2 Response acceleration

Figure 8 shows the time histories of horizontal accelerations at the four locations of the selected elements. The accelerations occurred with respect to the input earthquake wave in both two cases. High frequency component of the motion are damped out by the soils. Along the vertical direction from bottom to top, the peak value of horizontal acceleration decreases slightly in the ground in both two cases.

3.3 Displacement vector

Figure 9 shows the distribution of displacement vector at the end of the earthquake motion. Obviously, for the upper structure and the foundation, horizontal displacement occurred in the case of the short-pile type foundation is larger than in the case of the 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. Figure 10 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 in the case of the short-pile type foundation is larger than in the case of the long-pile type foundation, while the ground displacements of free field at the two sides away from the foundation are almost the same in both cases.

3.4 Settlements

The instantaneous settlements of points A and B at the two sides of the structure during earthquake motion are shown in Figure 11. It is known that differential settlements occurred and the differential settlement of the short-pile type foundation is 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 Figure 12. It is clear that the total settlements of foundation are composed of instantaneous settlement and long-term post-liquefaction settlement, and most of the differential settlement occurs immediately after the earthquake while the post-liquefaction settlement is relatively uniform despite its large amplitude.

The phenomenon of the differential settlement can be explained through Figure 13, which shows the calculated vertical accelerations of point A and B at the two sides of the structure. From the recorded acceleration of the two points between 60s and 64s during earthquake motion, it is clear that the acceleration behaves in almost the same value but have opposite sign. It means the subsoil below the structure might bear the vertical dynamic compressive stress on one side, while bear the vertical dynamic extensional stress at the same time on the other side. Usually, in the case of large deformation of the soils, the difference in the vertical acceleration will cause different deformation of subsoil on the two sides of the structure. Compare Case-1 with Case-2, the amplitude of the response vertical acceleration of the two sides in Case-2 is larger than that in Case-1, so the amount of settlement in Case-2 is obviously larger than in Case-1.

The instantaneous settlements of points A and B at the two sides of the structure at the end of earthquake motion are shown in Table 4. The long-term settlements of points A and B at the two sides of the structure 3.5 years after earthquake motion are shown in Table 5. In the case of the long-pile type foundation, the differential settlement of the left and right side ends is 0.16 cm immediately after earthquake and 0.24 cm 3.5 years after earthquake. However, in the case of the
short-pile type foundation the differential settlement is 6.70 cm immediately after earthquake and 12.10 cm 3.5 years after earthquake. It means that about 60% percent of the differential settlement occurred immediately after earthquake. Generally, the reasons for the earthquake induced differential settlement of the building on natural ground are regarded as the asymmetry of the buildings, the non-uniform distribution of the ground, the asymmetry and irregularity of the seismic loading. In the present condition, considering the symmetric building and uniform ground layers, the differential settlement might be influenced by the asymmetry of the static car loading and irregularity of the earthquake wave. In the case of the short-pile type foundation, the inclination degree of building is 1.8‰ based on the wave. In the present condition, considering the symmetric buildings, the non-earthquake. Generally, the reasons for the earthquake of the differential settlement occurred immediately after earthquake wave. 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.

Fig. 5. Distribution of EPWPR at the end of earthquake.

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

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

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

Fig. 3. Time history of acceleration of 3 synchronization earthquake wave.
Fig. 6. Distribution of EPWP at the end of earthquake.

Fig. 7. Time histories of EPWPR and mean effective stress of the selected elements during earthquake.

Fig. 8. Time histories of horizontal acceleration during earthquake (16m away from the left side of the foundation).

Fig. 9. Displacement vector at the end of earthquake.

Fig. 10. Displacement vector 3.5 years after earthquake.

Fig. 11. Settlements on the surface of two sides of the structure foundation during earthquake.

Fig. 12. Settlements on the two sides of the structure 72 hours after earthquake.
not only to the liquefaction behavior of the ground during the earthquake motion, but also to the long-term settlement after the earthquake. 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 evaluation per the design code is OK, detailed calculation may reveal the risk of under estimation of differential settlement that may give rise to serious problems.

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