COUPLED ANALYSIS OF NONLINEAR SOIL EFFECTS DURING LARGE EARTHQUAKES

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In this paper, the response of a liquefiable site during the 1995 Kobe earthquake is studied using vertical array records, with particular attention on the effects of nonlinear soil behavior and liquefaction on the characteristics of ground motions. The characteristics of ground motions during shaking events are analyzed first. A fully coupled, inelastic finite element analysis is thereafter performed to simulate the earthquake response of the array site. The stress-strain histories of soils and excess pore water pressures at different depths are calculated, and their relations to the characteristics of ground motions are addressed.

Key Words: *site response, ground motion, nonlinear effects, coupled analysis*

1. INTRODUCTION

The vertical array records obtained at Port Island during the 1995 Kobe earthquake are of particular interest in earthquake geotechnical engineering, due to that the site consisted of a thick reclaimed surface layer which was fully liquefied during shaking events. The records have been used by many researchers to study various issues of interest. Particularly, the characteristics of vertical amplification, on which the available discussion is limited, have been discussed by Yang et al. (1999a)[1]. In this paper, the nonlinear response of the reclaimed site is analyzed using a fully coupled finite element method. The attention of the present study is focused on the correlation of soil behavior and the buildup of pore water pressure to the characteristics of ground motions during the shaking history.

2. NONLINEAR AMPLIFICATION

The acceleration records in both E-W and N-S directions clearly indicate that nonlinear amplification occurred during the shaking events, as shown in Fig. 1. It is seen that a significant reduction of the amplitudes of horizontal motions occurred when seismic waves travelling through the surface reclaimed layer. A spectral analysis has shown a typical feature that the predominant periods of surface motions in both components were elongated during shaking history[2], as given in Fig. 2, in which the Fourier spectra in three time phases are included for N-S direction. Obviously, in the second phase which corresponds to strongest shaking, long period waves were dominant.

The spectral ratios between the motions recorded at the surface and three stations along the depth (denoted as SP0/SP83, SP0/SP32, SP0/SP16) are calculated, which can be regarded to well represent the transfer functions for different locations. Fig. 3 shows the spectral ratios in three time phases for N-S component. The spectral ratios for E-W component are basically similar to those for N-S component for all three time phases[2], therefore, the discussion is focused here on N-S component. It can be seen that, in Phase 1, the spectral ratios SP0/SP83 and SP0/SP32 are similar, with peak frequencies of around 4.5, 7, and 11 Hz, and 4.2, 6, 10.5 Hz, respectively. The dominant seismic waves were those with high frequencies in this phase. The spectral ratio SP0/SP16, however, exhibited a little different feature: the peak frequencies were around 3.2 and 5 Hz. This indicates that, even though the shaking during this stage was not strong, seismic motions with relatively longer period became dominant when the waves travelling through the surface reclaimed layer. Further, by comparing the spectral ratios in Phase 1 and 2, the influence of nonlinear soil behavior, which was attributed to a strong shaking, was clearly observed: the frequency content was obviously shifted to low frequency end, in other words, the predominant periods were lengthened when the strong motions occurred. Especially, it is noticed that, in phase 2 the amplitudes for the waves with frequency higher than 5 Hz were dramatically reduced, with amplification ratios below 1, while in phase 1 these waves were amplified notably. The difference for the two phases indicates that the increase in the predominant period was caused primarily by a strong decrease in the amplitude of low-period waves, rather than by amplification of long-period motions.

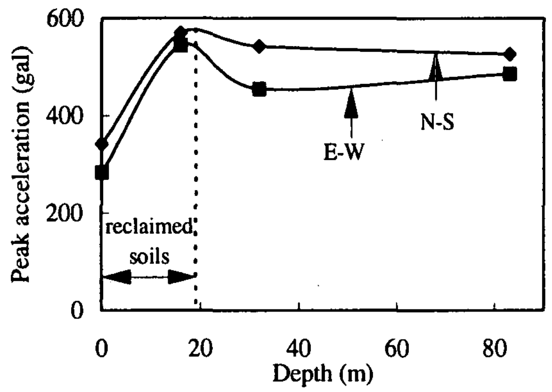

Fig. 1 Distribution of horizontal peak accelerations along depth

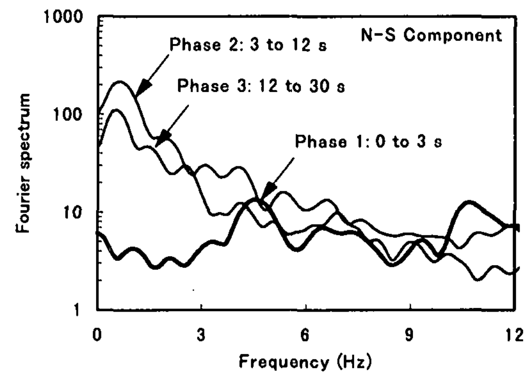

Fig. 2 Fourier spectra for surface motions in three phases

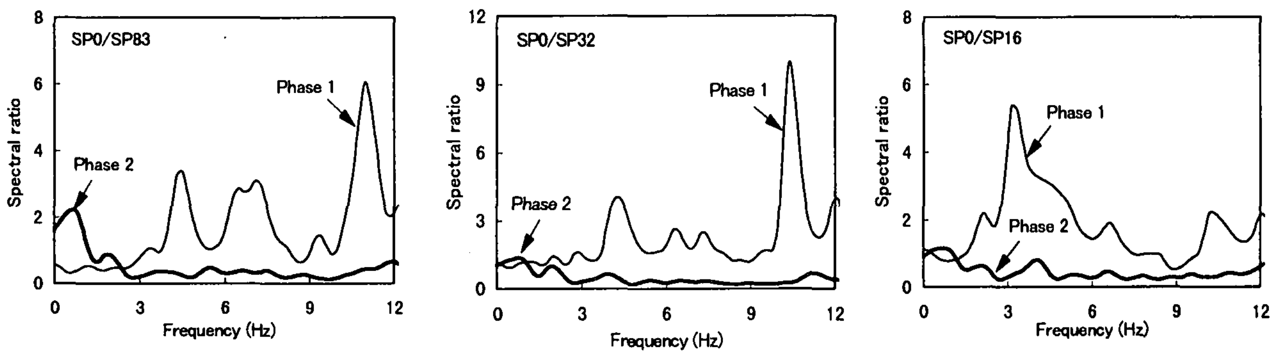

Fig. 3 Spectral ratios of ground motions in N-S component in Phase 1and 2

As have been shown above, the surface reclaimed layer produced a significant influence in ground motions. This layer consisted of decomposed granite which has a high liquefaction susceptibility. Based on the array records, it is possible to back-calculate the variation of soil stiffness during earthquake. A simple yet reliable approach has been applied to evaluate the shear modulus of the surface reclaimed layer before and after the strongest shaking[2], the identified shear wave velocity and shear modulus for N-S and E-W components are shown in Table 1. These values correspond respectively to the average soil stiffness before and after the strongest earthquake shaking. Clearly, the reduction of shear modulus during the shaking event is remarkable.

3. COUPLED NUMERICAL ANALYSIS

The preceding analyses have indicated that the nonlinear amplification was mainly attributed to the behavior of the reclaimed soils, which is expected to relate to the effects of pore water pressure buildup in soils. In this section, a coupled nonlinear finite element analysis is performed of the site response to the recorded motions. The inelastic soil model incorporated in numerical procedure is a hypo-plasticity bounding surface model[3,4] which was developed within the framework of bounding surface theory. The state-of-the-art model is capable of realistically simulating the soil behavior under a wide range of loading conditions. Some essential effects are captured, such as the compression and dilation induced effective stress change, the lateral stress change due to shaking and the significant reduction of stiffness upon liquefaction, etc. A typical undrained response of reclaimed soils simulated using the model is shown in Fig. 4, together with the cyclic strength data obtained in laboratory.

Table 1 Identified shear wave velocity and shear modulus before and after strongest shaking

Component	V_{s1} time phase 1	V_{s2} time phase 2	G_1 time phase 1	G_2 time phase 2	Reduction of G $(G_1-G_2)/G_1$ (%)
N-S	202 m/s	51 m/s	77.5 MPa	4.9 MPa	94%
E-W	154 m/s	57 m/s	45.1 MPa	6.2 MPa	86%

PS logging : Vs=198 m/s G_0=74.5 MPa (average values for the surface layer, G.L. 0 m to G.L. -16 m)

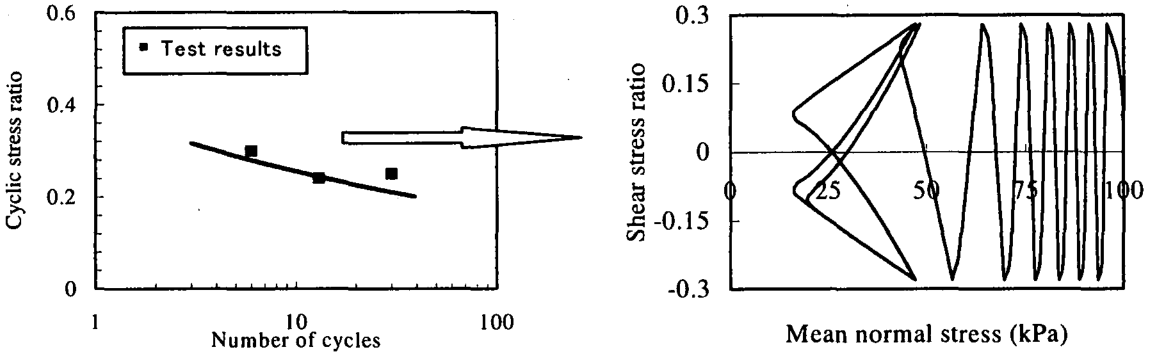

Fig. 4 Undrained cyclic response of soil model

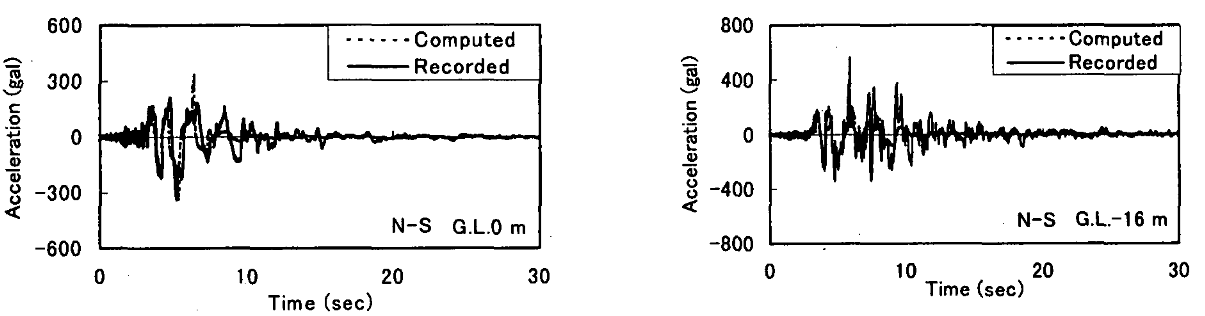

Fig. 5 Calculated and recorded acceleration time histories at surface and the depth of 16 m

Due to the limited space, in the following only the calculated results for N-S direction are presented. Fig. 5 shows the calculated and recorded acceleration time histories at the surface and the depth of 16 m. It can be seen that the calculated acceleration histories are very similar to the field records. The simulated stress-strain histories at depths of 7.7 m and 51.3 m during the shaking are shown in Fig. 6. Here, the shear stress is normalized to the initial effective vertical stress. Obviously, the soils at the shallow depth (7.7 m) showed a dramatic reduction of soil stiffness. Especially, it is noted that the shear modulus of soils was almost reduced to zero while the shear strain remained at a large level at the final stage of strong shaking, this indicates that the soils fully liquefied. The soils at deeper layer (51.3 m), on the other hand, responded generally in a linear manner, with no appreciable reduction of stiffness and with a low level of strain. Fig. 7 depicts the excess pore water ratios at the depths of 7.7 m and 51.3 m during the shaking. It is seen that an abrupt rise in excess pore water pressure took place during the phase of strongest excitation (Phase 2, 5 s to 8 s). For the soils at the depth of 7.7 m, the excess pore pressure reached the value of initial effective vertical stress around at 8 s, which resulted in a full soil liquefaction.

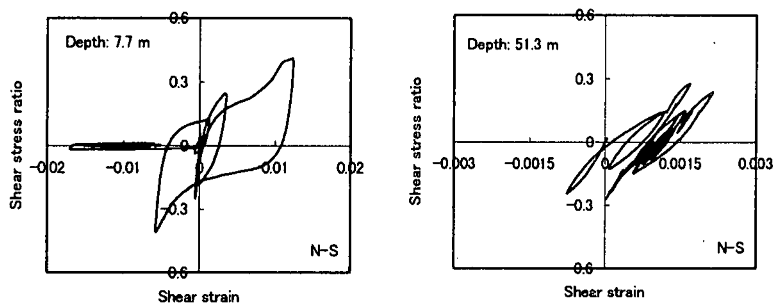

Fig. 6 Stress-strain histories of soils at different depths

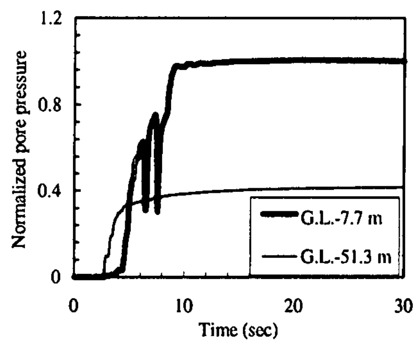

Fig. 7 Pore water pressure response

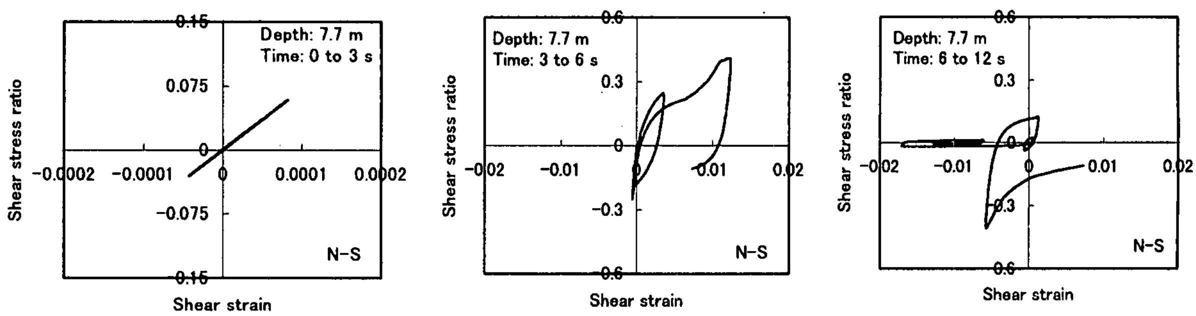

Fig. 8 Stress-strain behavior of reclaimed soils at different time stages

To clearly show the relation of ground motions and the soil behavior during shaking history, in Fig. 8 the stress-strain histories of soils at the depth of 7.7 m at different time stages are given. It is observed that, at the first time stage which corresponds to weak shaking, the soil responded linearly with the strain level at the order of 0.01%, which is the typical threshold strain for sand, meanwhile the generated excess pore water pressure at this stage was very small, as shown in Fig. 7. Corresponding to this stage, the spectra of the surface motions was dominated by high frequency components (with peak frequencies of 4.5 Hz and 10.8 Hz). During the second stage which corresponds to strong shaking, the soil exhibited an obvious nonlinearity with the peak strain of the order of 1%. The generated excess pore pressure reached the value of 75% of initial effective vertical stress at 6 s. During the period from 6 s to 12 s, the abrupt loss of soil stiffness upon liquefaction was clearly observed. The amplitude of shear strain approximately reached the value of 2%. Correspondingly, the long-period waves were dominant in surface motions (with peak frequency of 0.6 Hz).

4. CONCLUSIONS

The present study indicates that: there exist large differences in the characteristics of ground motions before and after the strongest earthquake shaking; the surface liquefiable soils played a key role in the variation of the characteristics of ground motions; the increase in the predominant period of surface motions was caused primarily by a strong decrease in the amplitude of low-period waves, rather than by amplification of long-period motions; the characteristics of ground motions at different depths are closely related to soil behavior and pore water pressure buildup during shaking.

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