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SEISMIC RESPONSE OF NONLINEAR SOIL-PILE SYSTEM

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ABSTRCT

Seismic response of nonlinear soil-pile system is studied by means of the dynamic experiments on a full-size pile group. The soil-pile system is simulated by the boundary zone model with non-reflective interface, and the impedance of group (stiffness and damping) are back-calculated from the measured data. The seismic response of the pile group is computed with the input of Tangshan earthquake records. Comparisons between the linear soil-pile system and the nonlinear system reveal the influence of nonlinearity on seismic response of piles, and it is concluded that the response may be increased substantially with the nonlinearity of soil-pile system in a seismic event.

KEYWORDS

soil-pile interaction, dynamics, seismic response, pile group, nonlinear vibration, full-scale testing.

INTRODUCTION

Many high rise buildings and heavy structures were supported on pile foundations in some seismically active area. The behavior of such structures is greatly affected by nonlinear soil-pile foundation interaction during strong earthquakes. Seismic response of pile foundations has been a subject of broad interest, but the difficulty is how to evaluate the soil-pile interaction in a nonlinear situation. This subject includes two important aspects: analytical model of soil-pile system and experimental studies.

In this study, a boundary zone model with non-reflective interface was developed, to account for the nonlinear response of soil-pile system as an approximate approach. With the boundary zone model, the nonlinear response of soil-pile system under strong excitation can be reproduced.

The dynamic experiments were conducted on a full-size pile group, under different levels of lateral harmonic excitation which produced linear vibration and nonlinear vibration, respectively. The validity of the boundary zone model is evaluated through comparison of theoretical results with the measured data. The parameters of the full-size pile group, stiffness and damping, are back-calculated from the measurements on the top of pile group. Seismic response of the full-size pile group is computed in terms of Tangshan earthquake records. Comparisons between the linear soil-pile system and the nonlinear system reveal the influence of nonlinearity on seismic response of piles.

BOUNDARY ZONE MODEL

A number of approaches are available to account for dynamic soil-pile interaction but they are usually based on the assumptions that the soil behavior is governed by the law of linear elasticity or viscosoil and the pile is rarely perfect and slippage or even separation often occurs in the contact area. Furthermore, the soil region immediately adjacent to the pile can undergo a large degree of straining, which would cause the soil-pile system to behave in a nonlinear manner. Both theoretical and experimental studies have shown that the dynamic response of the piles is very sensitive to the properties of the soil in the vicinity of the pile shaft (Han and Novak, 1988; Han, 1989; Han and Vaziri, 1992; El-Marsafawi et al.,1992).

To account for the nonlinearities resulting from loss of contact in an approximate way, Novak and Sheta (1980) proposed including a cylindrical annulus of softer soil (an inner weakened zone or so-called boundary zone) around the pile in their plane strain analysis. One of the simplifications involved in the original boundary zone concept was that the mass of the inner zone was neglected to avoid the wave reflections from the interface between the inner boundary zone and the outer zone. To overcome this problem, Veletsos and Dotson (1986, 1988) proposed a scheme that can account for the mass of the boundary zone. Some of the effects of the boundary zone mass were investigated by Novak and Han (1990) who found that a homogeneous boundary zone with a non-zero mass yields undulation impedances due to wave reflections from the fictitious interface between the two media, the near field and the far field. The ideal boundary zone should have properties smoothly approaching those of the outer zone to alleviate wave reflections from the interface.

Consequently, the impedance functions for a composite soil layer are formulated based on a new model of the boundary zone with a non-reflective interface, as shown in Fig. 1, by Han and Sabin (1995). A parabolic variation of the medium properties is assumed, and the properties of the soil medium for each region are defined as

$$G^*(r) = \begin{cases} G_i^* & r = r_o \\ G_o^* f(r) & r_o < r < R \\ G_o^* & r \geq R \end{cases} \tag{1}$$

and

$$\left.\begin{aligned} G_i^* &= G_i(1 + i2\beta_i) \\ G_o^* &= G_o(1 + i2\beta_o) \end{aligned}\right\} \tag{2}$$

in which G_i and G_o = shear moduli of the inner and outer zones; r_o = radius of the cylindrical cavity in the medium; R = radius of boundary zone; r = radial distance to an arbitrary point; β_i and β_o = damping ratio for the two zones; $i = \sqrt{-1}$; and $f(r)$ is the parabolic function assumed.

The stiffness and damping factors obtained from this analysis were compared with those obtained for the Novak-Sheta and Veletsos-Dotson idealizations. The Veletsos-Dotson solution produces pronounced oscillations (undulations) caused by wave reflection from the interface. Since the interface between the two zones is only fictitious in reality, the undulating impedances may not be expected for many applications. Although the Novak-Sheta solution avoids this difficulty, it does so by taking the mass of the boundary zone to be zero. The present analysis assumes that boundary zone has a non-zero mass and a continuous variation in the boundary zone. The present analysis does not oscillate for a wide range of frequencies.

With the impedances of the soil layer described as above, the element stiffness matrix of soil-pile system can be formed in the same way as the general finite element method. Then the overall stiffness matrix of a single pile can be assembled for different modes of vibration. The dynamic stiffness and damping of a single pile can be described in terms of complex stiffness (impedance functions):

$$k = k_1 + ik_2 \tag{3}$$

in which k_1 is the real part representing the true stiffness of pile; k_2 is the imaginary part and $k_2 = \omega c$, where c is the constant of equivalent viscous damping.

The pile-soil-pile interaction for the group is considered using dynamic interaction factor approach. The dynamic interaction factors were presented in a chart form by Kaynia and Kausel (1982). The complex stiffness of the group with a rigid cap can be written in the general form for all of the vibration modes of interest

$$K = k \sum_{i=1}^{n} \sum_{j=1}^{n} \varepsilon_{ij} \tag{4}$$

where k is the static stiffness of a single pile; n is the number of piles in the group; ε_{ij} are the elements of $[\alpha]^{-1}$, and $[\alpha]$ is the interaction matrix.

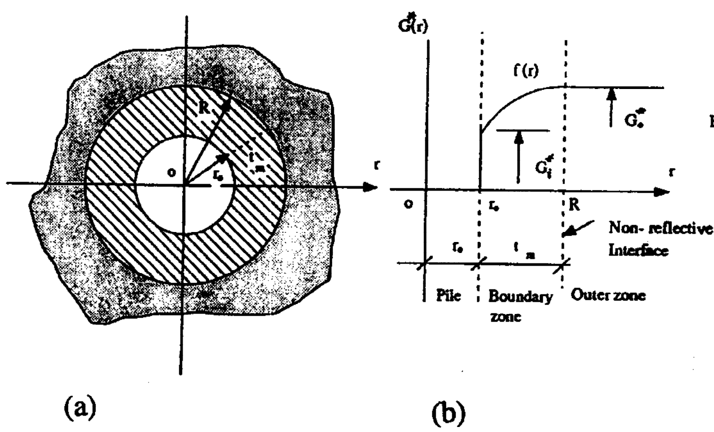

Fig. 1 Model of Boundary Zone with Non-reflective interface : (a) Composition of Zones; and (b) Variation of Shear Modulus with Radial Distance

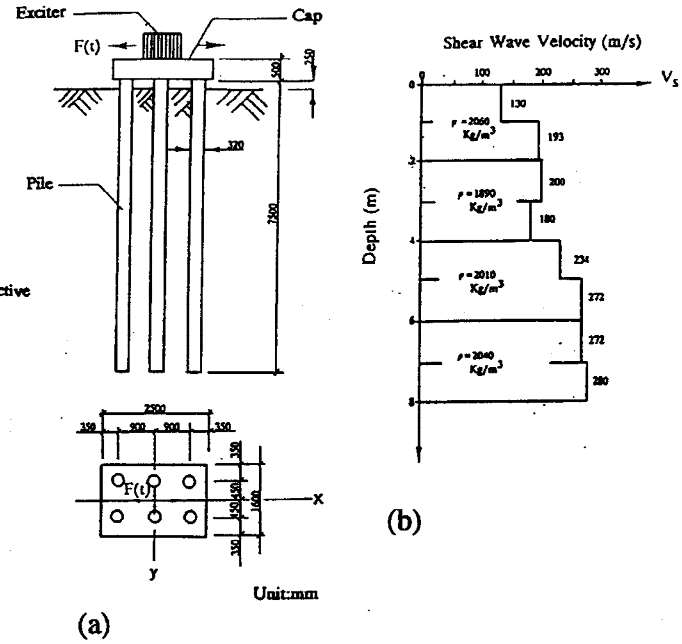

Fig. 2 (a) Layout of Pile Group and (b) Shear Wave Velocity and Mass Density of Soil

EXPERIMENTAL STUDIES ON FULL-SIZE PILE GROUP

Field tests on a full-scale pile group, comprising six piles, were carried out at a site within the grounds of the Institute of Engineering Mechanics, Harbin, China. The details for the dynamic experiments had been described by Han and Novak (1992). The subsurface investigation indicated that the test site was underlain by a relatively homogeneous layer of silty clay with occasional lenses of sandy clay mixture down to a depth of 30 meters. The ground-water table was established at 20 m below the ground surface.

The pile group under study was comprised of six cast-in-place reinforced concrete piles; each pile was 7.5 m long and 0.32 m in diameter as shown in Fig. 2(a). The concrete cap was 2.5 m long (X - direction), 1.6 m wide (Y - direction), and 0.5 m thick, weighing 49 kN and having a clearance of 0.25 m above the ground surface. The pile slender ratio (L/d) and spacing ratio (s/d) are 23.4 and 2.81, respectively, where L is the pile length, s is the pile spacing, and d is the pile diameter. The shear wave velocity and mass density of soil for the site profile are shown in Fig. 2(b).

An exciter with two counter-rotating eccentric masses was bolted to the pile cap to produce the harmonic excitation. Different excitation intensities were used in the experiments, and the magnitude of the exciting force was changed by adjusting the angle of the eccentric mass. Two exciters were used in the experiments respectively. The smaller one was used to produce linear vibration of the pile group and the larger one was used for nonlinear vibration.

Two horizontal displacement pick-ups (to measure the horizontal vibration) and two vertical displacement pick-ups (to measure the rocking vibration) were mounted on the pile cap. The steady-state dynamic response of the pile group under horizontal excitation was measured under different frequencies and for different excitation intensities.

Linear Vibration of Pile Group

A smaller exciter, weighing 1.18 kN, was fixed on the pile cap by foundation bolts to produce the harmonic excitation. The active component of the horizontal excitation was situated 0.2 m above the cap surface, and the center of gravity of the cap-exciter system was 0.25 m below the cap surface.

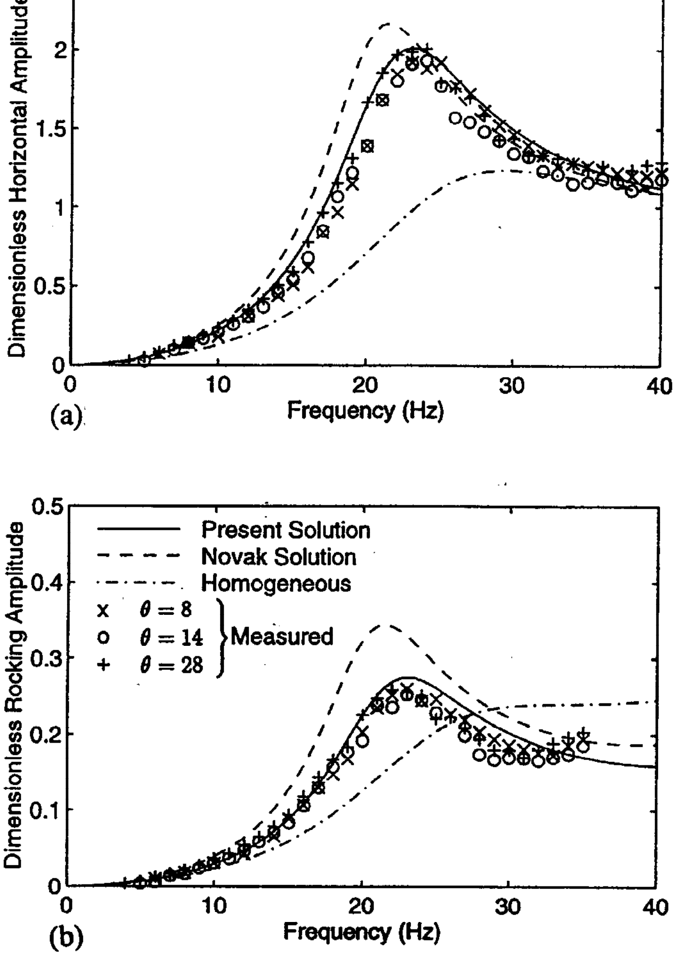

Fig. 3 Linear Vibration of Pile Group under Harmonic Excitation (a) Horizontal Response; and (b) Rocking Response

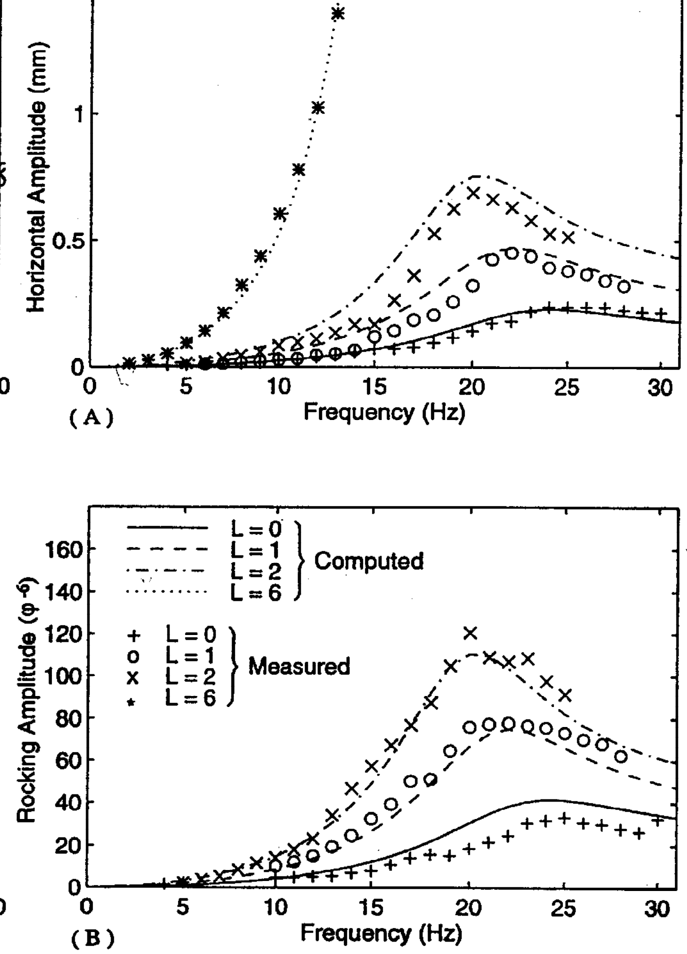

Fig. 4 Nonlinear Vibration of Pile Group under Harmonic Excitation (a) Horizontal Response; and (b) Rocking Response

Several levels of excitation intensities were used in the experiments, as indicated, $\theta = 8, 14, 28$ corresponding to excitation intensity of 96, 171 and 259 kg.mm, respectively. The maximum horizontal amplitude of the pile group was measured to be 0.104 mm at top of the pile cap; this is considered to be a small amplitude vibration or linear vibration.

Under horizontal excitation, the cap produced coupled horizontal and rocking vibration. For linear vibration, the amplitude response of the group can be normalized to give dimensionless amplitudes. A comparison of the experimental response curves with the theoretical predictions is shown in Fig. 3 for horizontal and rocking vibration of the pile cap. The theoretical predictions were done in different ways. Using the present solution, the boundary zone is included for each pile in the group. The relevant parameters of the boundary zone are as follows: $G_i/G_o = 0.1, t_m/r_o = 0.7, \beta_i = 0.07$, and $\beta_o = 0.035$. Poisson's ratio of soil is assumed, $\nu = 0.3$. The calculated curves are shown by a solid line in Fig. 3.

With the same parameters of boundary zone, Novak's solution is used to generate the response curves as shown by the dashed lines in Fig. 3. In the third way, the soil is considered to be radially homogeneous, to ignore the boundary zone effects (i.e., the boundary zone was omitted). The dash dot lines in Fig. 3 represent the theoretical results without boundary zone. It can be seen that the present solution (solid lines) agree with the measured results quite well, particularly for the horizontal vibration; the Novak's solution differs a little from the present solution due to different assumption on boundary zone. Without the boundary zone, the calculated response curves have a higher resonant frequency and lower peak amplitude. It is indicated that the stiffness and damping of soil-pile system are overestimated in the radially homogeneous model.

A larger exciter, weighing 4.9 kN, was fixed on the pile cap by foundation bolts to produce the harmonic excitation. The active component of the horizontal excitation was situated 0.2 m above the cap surface, and the center of gravity of the cap-exciter system was 0.24 m below the cap surface. As indicated, $L = 0, 1, 2, 6$ correspond to the excitation intensities of 472, 887, 1360 and 3870 kg.mm, respectively, in the experiments. The maximum horizontal amplitude of the pile group was measured to be 1.4 mm (or $4.4 \times 10^{-3}d$) at top of the pile cap, with a corresponding maximum acceleration of $1.13g$; this represents a rather intense harmonic vibration, resulting in a nonlinear vibration of the pile group as described in the following.

The theoretical predictions and measurements of horizontal and rocking displacements on the cap of the group in different excitation intensities, excited in Y - direction, are shown in Fig. 4. From the response curves shown in Fig. 4, it can be observed that the resonant frequencies reduce with increasing excitation intensity and the peak amplitudes are not proportional to excitation intensity at all frequencies. These are typical features of nonlinear vibration.

To depict the nonlinear response theoretically, the boundary zone concept, which accounts for yielding of soil around the pile, was incorporated into the linear-elastic-based mathematical model. This model provides for the gradual expansion of the yield zone as the excitation level increase. In matching the measured data, allowance had to be made for the modification of boundary zone parameters and the pile separation. For instance, the thickness of boundary zone changed from $0.5r_o$ to $0.9r_o$ as the excitation increased from $L = 0$ to $L = 1$. As the excitation intensity increased further, the separation between the pile and soil might occur. Such as, the separation length was 100 mm corresponding to excitation $L = 2$, and separation was 430 mm to excitation $L = 6$. It should be mentioned that the variation of boundary zone parameters and the separation lengths could not be physically measured yet in the field at present; the values given here were inferred by using a trial-and-error technique of matching the theoretical and measured response curves.

Some of the salient features of the nonlinear vibration of the pile group are listed in Table 1 as the excitation was on Y-direction. The stiffnesses and damping ratios shown in this table correspond to the resonant frequency for the first mode of vibration of the pile group. It can be seen that the stiffnesses of the pile group are reduced and these reductions are quite pronounced. For instance, the horizontal stiffness of the pile group reduces by almost half, when the excitation intensity increases from $L = 0$ to $L = 6$. This reduction can principally be attributed to an increase in the thickness of the boundary zone and the soil-pile separation effects.

Table 1. Behavior of the pile group in nonlinear vibration

Excitation intensity L	Resonant frequency (Hz)	Horizontal		Rocking	
		Stiffness (MN/m)	Damping ratio, D_x	Stiffness ($10^2 MN.m$)	Damping ratio, D_ψ
0	24	114	0.34	5.40	0.039
1	22	97.7	0.29	4.97	0.039
2	20	87.7	0.25	4.67	0.039
6	15.8	54.0	0.20	4.31	0.018

It can be noted that the theoretical predictions agree with the measured results quite well for both horizontal and rocking vibrations. It can be concluded from these results that the employed mathematical model, incorporating a boundary zone, is capable of capturing the nonlinear vibration of a pile group. The results show that the resonant frequency of the pile group reduces and the resonant amplitude increases as the excitation intensity increases. For instance, when the excitation level is increased from $L = 0$ to $L = 6$, the resonant frequency of the pile group reduces from 24 Hz to 15.8 Hz.

SEISMIC RESPONSE OF PILE GROUP

The theoretical analysis of piles under seismic loading can be done in two ways: inertial interaction analysis and kinematic interaction. The inertial interaction analysis is based on the following assump-

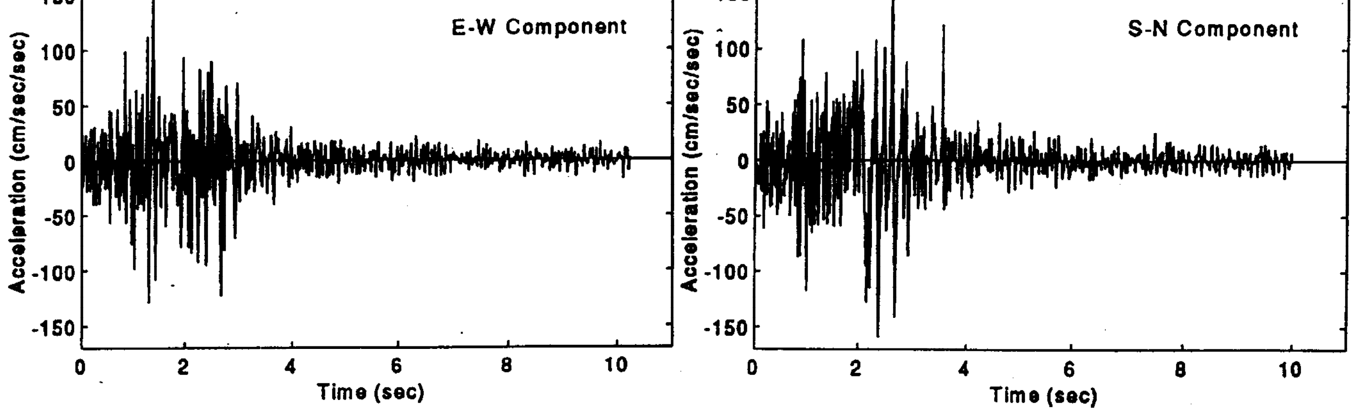

Fig. 5 Time History of Ground Acceleration from Tangshan Earthquake (1976); (a) in East-West Direction (EW); (b) in South-North Direction (SN)

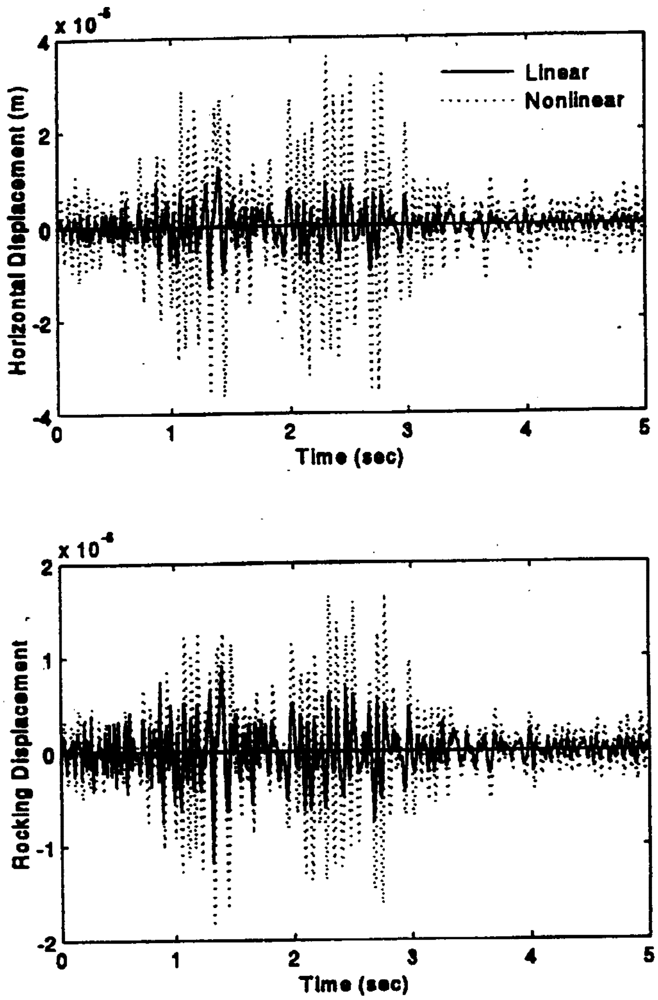

Fig. 6 Seismic Response of Pile Group in X - Direction (EW); (a) Horizontal Displacement; and (b) Rocking Displacement

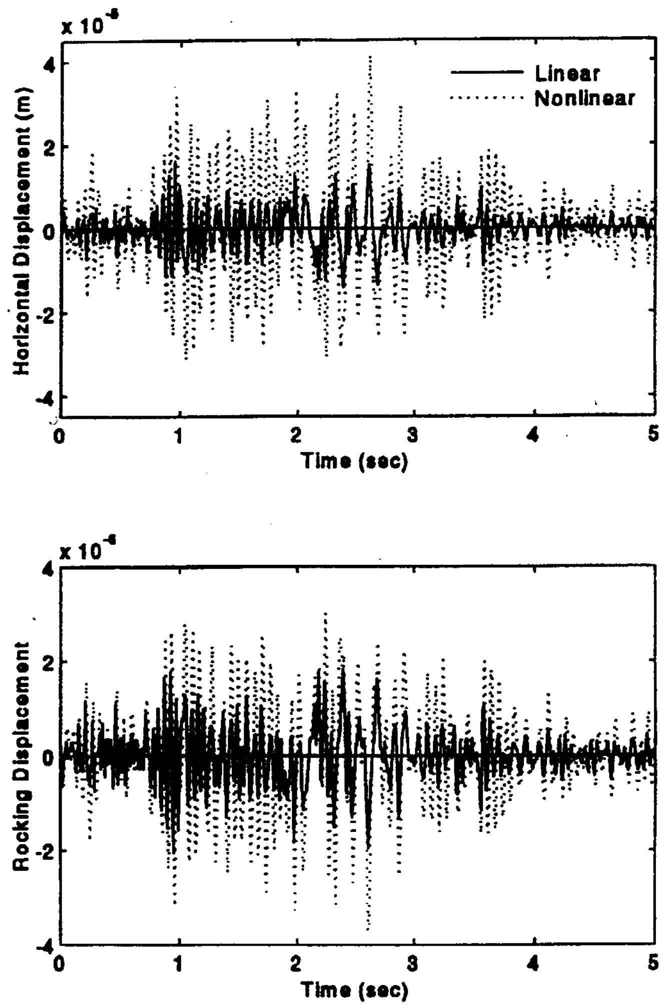

Fig. 7 Seismic Response of Pile Group in Y - Direction (SN); (a) Horizontal Displacement; and (b) Rocking Displacement

the piles and their cap, soil-pile interaction analysis is conducted separately to yield the pile foundation impedances; and, the seismic response is obtained using standard analysis, even response spectra (see Novak and El-Hifnawy, 1984). The assumption of the input ground motion not being affected by the presence of the piles is based on the idea that the dominant seismic wave length are much larger than the pile diameter, and given the bending flexibility of slender piles, the piles will follow the horizontal motion of the ground.

The kinematic interaction involves consideration of the wave scattering effect, i.e., the absolute values of the horizontal displacements of the embedded pile head and the ground surface motion in the absence of the piles are allowed to be different (Gazetas, 1984). Analysis of this type was conducted by Wass and Hartman (1984), Wolf and Von Arx (1982), Hadjian et al (1990) and Kobori et al (1991). One limitation of the accuracy of most analysis using kinematic interaction is that soil linearity is assumed. It is well known that for strong earthquakes linear site response analysis can yield unrealistic displacements and stresses.

To account for the nonlinearity of a soil-pile system, the method of inertial interaction analysis has been employed in this study. The time history of the horizontal ground acceleration employed is shown in Fig. 5, from the record of Tangshan earthquake, in 1976.

The impedances (stiffness and damping) of the group had been determined in linear vibration and nonlinear vibration, respectively, as described previously. The seismic response of the cap can be computed using the input of the Tangshan earthquake records. The maximum horizontal ground accelerations are 0.15 g. in EW direction and 0.16 g in SN direction, respectively. The Wilson-θ method of step-by-step integration with respect to time was used to calculate the seismic response (Clough and Penzien, 1975), and the time step is taken as 0.01 second. The nonlinear vibration impedances correspond to the excitation intensity of $L = 6$. The seismic response of the cap is shown in Fig. 6 for X - direction (EW) and Fig. 7 for Y - direction (SN), in which the solid lines represent linear response and dash lines represent the nonlinear response. It can be seen that the peak values of the nonlinear response are about 2-3 times that of the linear for both horizontal and rocking displacements, even though the nonlinearity is not very strong in the case undertaken. This indicates that the nonlinearity of soil-pile system may increase the response of pile foundation substantially in a seismic event.

The recent investigations of earthquake damages in Japan shown that some bridges supported with pile foundations collapsed during strong earthquake, although these bridges were built based on aseismic design code. One more important factor is that the behavior of pile foundations in nonlinear situation is very different with that of those in linear condition. Consequently, the effects of nonlinearity of the soil-pile system should be evaluated reasonably to increase the safety of structures built on pile foundations. In this case, the peak values of the nonlinear response are about 2-3 times that of the linear response.

CONCLUSION

The behavior of nonlinear soil-pile system was investigated based on the dynamic experiments with a full-size pile group in the field, and the Tangshan earthquake records were used as input to compute the seismic response of the pile group. This procedure is much closed to that one in real seismic environment.

Some of the conclusions that can be made from the study performed are as follows:

1. The pile group exhibits typical nonlinear features under strong lateral excitation, although the maximum horizontal amplitude is only 1.4 mm in this case. The stiffness and damping of the soil-pile system reduced as the excitation intensity increases. This in turn results in a reduction of the resonant frequency and an increase in the resonant amplitude.

2. The boundary zone model with non-reflective interface was verified to predict effectively the nonlinear response of pile foundations. With the excitation intensity increased, the thickness of boundary zone may be increased, the shear modulus of soil in the boundary zone reduced and the damping ratio in the zone increased. This approach is approximate, but it is a simple and realistic method to evaluate the soil- pile interaction, even included the effect of nonlinearity of soil-pile system .

the vicinity of the pile shaft may be weakened, even slipped or separated, which results in the stiffness of soil-pile system reduced and the peak amplitudes increased. In this case, the peak amplitudes increased about 2-3 times due to the nonlinearity of soil-pile system.

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