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ABSTRACT

An exact analytical method for the interaction analysis of a fully coupled soil-pile-structure system under seismic excitation is investigated in this paper. Only horizontal shaking induced by harmonic SH waves was considered. The soil mass, pile and building were all considered as elastic with hysteretic type damping. Geometrically, the soil is modeled as an elastic isotropic homogeneous continuum, and both pile and structures are simplified as beam models. The structure and piles are coupled through a rigid foundation at the ground level. Buildings of various heights in Hong Kong designed to withstand wind load were analyzed using the present model. Only the shaking at the ground level is considered in this study. It was discovered that the maximum shaking of the piled-structures at ground level is generally larger than that of a free field ground shaking except near the first natural frequency of the coupled soil-pile-structure system. This first resonant frequency depends strongly on the natural frequency of the structure.

INTRODUCTION

Piles have been used for hundreds of years (e.g. Poulos and Davis, 1980), but the theoretical analysis for piles only started about a century ago, while more intensive studies on the dynamic interaction was not available until the early 1970s (e.g. Takemiya and Yamada, 1981). It is not uncommon that damages were induced in piles during earthquakes (e.g. Matsui and Oda, 1996). Thus, the interaction of pile-soil-structure and the mechanism of these pile damages under the seismic excitation need to be examined comprehensively. Nevertheless, it is one of the most difficult problems in geotechanical engineering because of its complexity. In fact, the rigorous results of the soil-pile-structure interaction have yet to be incorporated into seismic design code (e.g. Kumar and Prakash, 1997). Much remains to be learned about the fundamental aspects of seismic soil-pile-structure response so that a more reliable seismic design procedure for pile-supported structures can be established with higher degree of confidence.

For a more comprehensive review on soil-pile-soil interaction, we refer to Novak (1991). In particular, numerical analyses for soil-pile-structure interaction include the finite element analysis (e.g. Cai, et al., 2000; Finn et al., 1997) and boundary element method (e.g. Guin and Banerjee, 1998). But, a time history analysis on a realistic model of soil-pile-structure system using either finite element method or boundary element method is extremely time-consuming. Therefore, other semi-analytical approach has also been adopted, such as the spring-dashpot method (e.g. Mylonakis et al., 1997; Makris et al., 1996), substructure model (e.g. Ahn and Gould, 1989), and matrix model (e.g. Kumar and Praskash, 1997). The soil-pile-structure interaction has also been investigated by experimental approaches, such as the shaking table test and the centrifuge test (e.g. Finn and Gohl, 1992; Meymand, 1998; Boulanger et al. 1999). Many studies have also investigated the effect of pile group on the seismic response of structures using various approaches. For example, continuum model based on elastic wave theory has been used by Tajimi (1969) and Tazoh et al. (1987). Soil-pile interaction in pile groups has also been modeled by Winkler model (e.g. Nogami, 1985; Makris and Gazetas, 1992). The method of interaction factor has been very popular (e.g. Makris and Gazetas, 1992; El Nagger and Novak, 1995). Variational method was proposed by Shen et al. (1999), and in such an approach no discretization of the pile is needed.

In most of the previous studies, the interaction between soil, pile and structure has in fact been uncoupled. The coupling effect of soil-pile-structure interaction has yet been examined comprehensively. It is always advisable to examine the essential phenomenon approximately before a time-consuming and rigorous numerical analysis is considered. Thus, a simple analytic method that can capture qualitatively the essence of the soil-pile-structure interaction is needed.

Koo et al. (2000) proposes a simple two-dimensional plane strain model to examine the qualitative characteristics of the coupled soil-pile-structure system with infinite end-bearing piles, and this study is an accompany of the Koo's work. It considers a building resting on a finite pile group. More specifically, in this study the seismic wave will be modeled by SH waves. A typical soil-pile-structure system shown in Fig. 1(a) will be modeled by a system having an equivalent 3×1 piles-system as shown in Fig. 1(b), and both piles and buildings will be modeled by a simple beam model. By allowing all the soil displacement, the traction between the pile and the soil, and the shear force transferring from the soil-pile system to the structures as unknowns simultaneously, a coupled system for the soil-pile-structure interaction is formulated. The numerical results of the coupled system are compared with those of the free field soil response, it can be seen that the ground level shaking is affected and modified by the presence of the building due to the full coupling between soil, pile and structure.

FREE FIELD RESPONSE OF SOIL

The free field vibration v^f of an infinite soil layer with complex shear modulus $G^*=G(1+2i\zeta)$, hysteretic damping ratio ζ, mass density ρ and thickness H subject to a harmonic shaking at the bedrock level of

$$\bar{v}_r = v_r H e^{\omega t i} \quad (1)$$

can be expressed as (Koo et al., 2000)

$$v^f = v_r H \sum_{n=1}^{\infty} \frac{4(-1)^{n+1}}{(2n-1)\pi} \left[\frac{(\omega/\omega_{sn})^2}{1-(\omega/\omega_{sn})^2 + 2\zeta\, i} \right] \cos(\alpha_n \frac{z}{H}), \quad (2)$$

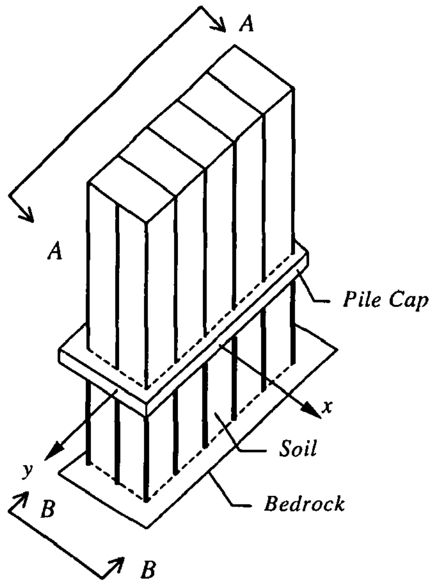
where $f_n(z) = \cos(\alpha_n \frac{z}{H})$, $\alpha_n = (2n-1)\pi/2$ and $\omega_{sn} = \alpha_n (G/H^2\rho)^{\frac{1}{2}}$ are the natural circular frequencies of the soil and G is the elastic shear modulus of the soil.

RESPONSE OF PILE-SOIL-SYSTEM

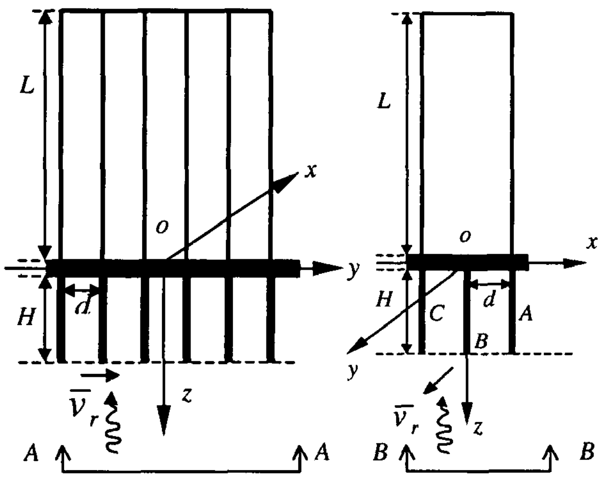
In order to examine the interaction of the soil layer and the piles, a three-pile system shown in Fig. 1(b) is considered. As shown in Fig. 2(a), we divide the soil region $y>0$ into four sub-regions (I, II, III and IV). In view of symmetry, the responses of soil in sub-region I and II are the same as those of soil in sub-region IV and III, respectively. Therefore, only soil response in sub-regions I and II will be considered below.

Response of Soil in Sub-region I

Once seismic shaking of the soil-pile-structure system is triggered, force is transmitted between the soil and the pile. In particular, the shear force acting on the pile A (at $x=d, y=0$) along the y-direction induced by soil in sub-region I is denoted



(*a*) A soil-pile-structure system



(*b*) The equivalent system

Figure 1. A sketch for a soil-pile-structure system containing a 3x1 array of piles

by $\bar{t}_y^{\,\mathrm{I}}$ (see Fig.2(b)). This shear force can be expressed in terms of the following Fourier cosine expansion:

$$\bar{t}_y^{\,\mathrm{I}} = t_y^{\mathrm{I}} e^{\omega t i} = G^* H \sum_{n=1}^{\infty} t_n^{\mathrm{I}} f_n(z) e^{\omega t i}, \quad (3)$$

where t_n^{I} is a non-dimensional constant to be determined.

Denote $\bar{v}^{\mathrm{I}}(x,y,z,t)$ as the displacement of the soil in sub-region I along the y-direction. Considering the decay conditions, and the top and bottom boundary conditions of the soil. the following soil response is assumed

$$\bar{v}^{\mathrm{I}} = v^{\mathrm{I}}(x,y,z)e^{i\omega t} = H\sum_{n=1}^{\infty} C_n^{\mathrm{I}} f_n(z) e^{-k_n(x-d+y)} e^{i\omega t}, \tag{4}$$

where C_n^{I} is unknown constant to be determined. The constant k_n can be found approximately by the equilibrium equation of the soil, which calls for

$$k_n = \sqrt{\frac{1-2\nu}{3-4\nu}} \frac{\alpha_n}{H}, \tag{5}$$

where ν is the Poisson's ratio of the soil.

To establish the relationship between C_n^{I} in (4) and t_n^{I} in (3), the principle of virtual work is employed (for details, see Chau, et al., 2000), and it gives

$$t_i^{\mathrm{I}} = D_i^{\mathrm{I}} C_i^{\mathrm{I}}, \quad i = 1,2,3,\cdots \tag{6}$$

where

$$D_i^{\mathrm{I}} = \frac{1}{2(1-2\nu)}\left(3-4\nu-2\nu^2 - \frac{3-4\nu}{2(1+2\zeta i)}\frac{\omega^2}{\omega_{si}^2}\right), \quad i = 1,2,3,\cdots \tag{7}$$

Response of Soil in Sub-region II

Soil in sub-region II is subjected to shear forces at the locations of piles A and B. Similar to the solution form given in (3), these shear forces can be expressed as

$$\begin{aligned}\bar{t}_y^{\mathrm{II}} &= t_y^{\mathrm{II}} e^{\omega i} = G^* H \sum_{n=1}^{\infty} t_n^{\mathrm{II}} f_n(z) e^{\omega t i}, \\ \bar{t}_y^{\mathrm{III}} &= t_y^{\mathrm{III}} e^{\omega i} = G^* H \sum_{n=1}^{\infty} t_n^{\mathrm{III}} f_n(z) e^{\omega t i},\end{aligned} \tag{8}$$

where t_n^{II} and t_n^{III} are non-dimensional unknown constants to be determined, acting along y-direction and at $x = d, y = 0$ and $x = 0, y = 0$, respectively.

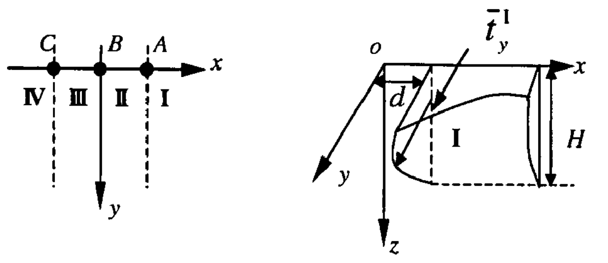
The general solution of the soil response in sub-region II ($\bar{v}^{\mathrm{II}}$) can be taken as

$$\begin{aligned}\bar{v}^{\mathrm{II}} &= v^{\mathrm{II}}(x,y,z)e^{\omega t i} \\ &= H\sum_{n=1}^{\infty}\left(\frac{1}{2}C_{0n}^{\mathrm{II}} + \sum_{m=1}^{\infty} C_{mn}^{\mathrm{II}} g_m(x)\right) f_n(z) e^{-k_n y} e^{\omega t i},\end{aligned} \tag{9}$$

where $g_m(x) = \cos(\beta_m x/d)$, $\beta_m = m\pi$, and C_{mn}^{II} are unknown constants to be determined from boundary conditions.

Again, similar to the procedure for soil I, virtual work principle is applied to yield the relation between the magnitude of shaking and the magnitude of shear forces. In particular, we have

$$t_j^{\mathrm{III}} + (-1)^i t_j^{\mathrm{II}} = D_{ij}^{\mathrm{II}} C_{ij}^{\mathrm{II}}, \quad i = 0,1,2,\cdots, \; j = 1,2,3,\cdots \tag{10}$$



(a) The sub-region of the soil (b) Response of the soil I

Figure 2. Schematic diagrams for analyzing: (a) a plane view of the three-pile-system (A, B and C) in the soil layer, which is divided into 4 sub-regions I, II, III and IV; (b) the free field response $\bar{v}^s$ of a semi-infinite layer of soil of thickness H.

where

$$D_{ij}^{\mathrm{II}} = \frac{d}{H}\alpha_j\sqrt{\frac{3-4\nu}{1-2\nu}}\left\{\frac{(1+\nu)(1-2\nu)}{2(3-4\nu)} + \frac{1}{4}\left[1+\left(\frac{H}{d}\frac{\beta_i}{\alpha_j}\right)^2\right] - \frac{1}{4(1+2\zeta i)}\frac{\omega^2}{\omega_{sj}^2}\right\}, \quad i = 0,1,2,\cdots, \; j = 1,2,3,\cdots \tag{11}$$

The continuity of displacement of the soil at $x = d$ requires

$$\bar{v}^{\mathrm{II}}(d,y,z) = \bar{v}^{\mathrm{I}}(d,y,z), \tag{12}$$

Substitution of (4) and (9) into (12) leads to the following expression relating C_i^{I} and C_{ij}^{II}

$$C_i^{\mathrm{I}} = \sum_{j=1}^{\infty}(-1)^j C_{ji}^{\mathrm{II}} + \frac{1}{2}C_{0i}^{\mathrm{II}}, \quad i = 1,2,3,\cdots. \tag{13}$$

Vibration of Pile A

Because of symmetry, the seismic motions of piles A and C are identical, so only the motion of pile A will be considered here. By superposition, the force acting the profile of pile A is

$$\bar{t}_y^A = t_y^A e^{\omega t i} = -2(\bar{t}_y^{\mathrm{II}} + \bar{t}_y^{\mathrm{III}}) = -2G^* H \sum_{n=1}^{\infty}(t_n^{\mathrm{I}} + t_n^{\mathrm{II}}) f_n(z) e^{\omega t i} \tag{14}$$

Let the deflection of the pile A relative to the bedrock be $\bar{v}^A(z,t)$ and assume a harmonic response for pile A to be $\bar{v}^A(z,t) = Hv^A(z)e^{\omega t i}$, it is easy to show that (Chau, et al., 2000)

$$v^A = v_h^A(z) + v_{pt}^A(z), \tag{15}$$

where $v_{pt}^A(z)$ and $v_h^A(z)$ are expressed as follows

$$v_{pt}^A(z) = -2\sum_{n=1}^{\infty} T_n(t_n^{\mathrm{I}} + t_n^{\mathrm{II}}) f_n(z) - v_r, \quad v_h^A(z) = \left[f_p\left(\frac{z}{H}\right)\right]\left\{\tilde{A}_p^A\right\}. \tag{16}$$

where $\lfloor f_p(z) \rfloor$ and T_n have been given in Eq. (25) and (26) of Koo et al. (2000) and will not be repeated here. If we further denote $\overline{P}_A = (E_p^* I_p / H^2) P_A e^{\omega t i}$, in which P_A is the non-dimensional shear force at pile A, then, from the boundary conditions, we find

$$\{\overline{A}_p^A\} = \{A_P^A\} P_A + \{A_r^A\} v_r, \quad (17)$$

$$\{A_P^A\} = \frac{1}{2\lambda_p^3} \left\{ -\frac{\sinh \lambda_p}{\cosh \lambda_p} \quad 1 \quad \frac{\sin \lambda_p}{\cos \lambda_p} \quad -1 \right\}^T, \qquad \{A_r^A\} = \frac{1}{2} \left\{ \frac{1}{\cosh \lambda_p} \quad 0 \quad \frac{1}{\cos \lambda_p} \quad 0 \right\}^T. \quad (18)$$

Finally, substitution of (16) and (17) into (15) yields

$$v^A(z) = \left[f_p(\frac{z}{H}) \right] \{A_P^A\} P_A + \left[f_p(\frac{z}{H}) \right] \{A_r^A\} v_r - 2\sum_{n=1}^{\infty} T_n (t_n^{\mathrm{I}} + t_n^{\mathrm{II}}) f_n(z) - v_r \quad (19)$$

The conditions of the requirement of the displacement compatibility, $\overline{v}^A(z) = \overline{v}^f(z) + \overline{v}^{\mathrm{I}}(z)$, at $x = d, y = 0$, calls for

$$C_n^{\mathrm{I}} + 2T_n (t_n^{\mathrm{I}} + t_n^{\mathrm{II}}) - C_{Pn}^A P_A = (C_m^A - a_n^f) v_r, \quad n = 1,2,3,\cdots \quad (20)$$

where C_{Pn}^A and C_m^A are the same as those of C_{Pn} and C_m given in (34) of Koo et al. (2000) and a_n^f is given in (2).

Vibration of Pile B

Similarly, the shear force acting on pile B can be expressed as

$$\overline{t}_y^B = t_y^B e^{\omega t i} = -4\overline{t}_y^{\mathrm{III}} = -4G^* H \sum_{n=1}^{\infty} t_n^{\mathrm{III}} f_n(z) e^{\omega t i}. \quad (21)$$

Let the deflection of the pile B relative to the bedrock be $\overline{v}^B(z,t) = H v^B(z) e^{\omega t i}$ and denote $\overline{P}_B = (E_p^* I_p / H^2) P_B e^{\omega t i}$. Then, the deflection of the pile B can be expressed as

$$v^B(z) = \left[f_p(\frac{z}{H}) \right] \{A_P^B\} P_B + \left[f_p(\frac{z}{H}) \right] \{A_r^B\} v_r - 4\sum_{n=1}^{\infty} T_n t_n^{\mathrm{III}} f_n(z) - v_r, \quad (22)$$

where $\{A_P^B\}$ and $\{A_r^B\}$ are the same as $\{A_P^A\}$ and $\{A_r^A\}$ given in (18), respectively.

The displacement compatibility at pile B $\overline{v}^B(z) = \overline{v}^f(z) + \overline{v}^{\mathrm{II}}(z)$ gives

$$\frac{1}{2} C_{0n}^{\mathrm{II}} + \sum_{m=1}^{\infty} C_{mn}^{\mathrm{II}} + 4T_n t_n^{\mathrm{III}} - C_{Pn}^B P_B = (C_m^B - a_n^f) v_r, \quad n = 1,2,3,\cdots \quad (23)$$

where C_{Pn}^B and C_m^B are the same as those of C_{Pn} and C_m given in (34) of Koo et al. (2000).

RESPONSE OF STRUCTURE AND PILE CAP

Since the structure is modeled by a beam element, the results given above for the pile response can be directly adopted to the case of building. It is straightforward to obtain that the response of the structure is

$$v^b(z) = \left[f_b(\frac{z}{L}) \right] \{A_b\} v_0 \quad (24)$$

where $\{A_b\}$ and $\lfloor f_b(z) \rfloor$ have been given in (41) and (38) of Koo et al. (2000), respectively. The shear force of the building at the bottom $z = 0$ is

$$\overline{Q}_0 = Q_0 e^{\omega t i} = E_b^* I_b \left. \frac{\partial^3 \overline{v}^b}{\partial z^3} \right|_{z=0} = \frac{E_b^* I_b H}{L^3} \lfloor f_b'''(0) \rfloor \{A_b\} v_0 e^{\omega t i} \quad (25)$$

The shaking of the rigid pile cap is denoted as: $\overline{v}_0 = H v_0 e^{\omega t i}$, and the pile cap is subjected to shear forces $\overline{P}_B$, $2\overline{P}_A$ and $\overline{Q}_0$. Then, force equilibrium at the rigid pile cap can be expressed as

$$2P_A + P_B + K_f v_0 = 0, \quad (26)$$

where K_f has been given in (45) of Koo et al. (2000).
On the other hand, the displacement compatibility at the pile cap requires

$$\overline{v}_0 = \overline{v}_r + \overline{v}^A(0,t) = \overline{v}_r + \overline{v}^B(0,t). \quad (27)$$

Substitution of (19) and (22) into (27) yields

$$\lfloor f_p(0) \rfloor \{A_P^A\} P_A - \lfloor f_p(0) \rfloor \{A_P^B\} P_B - 2\sum_{n=1}^{\infty} T_n (t_n^{\mathrm{I}} + t_n^{\mathrm{II}}) + 4\sum_{n=1}^{\infty} T_n t_n^{\mathrm{III}} = 0, \quad (28)$$

and

$$v_0 = \lfloor f_p(0) \rfloor \{A_P^B\} P_B + \lfloor f_p(0) \rfloor \{A_r^B\} v_r - 4\sum_{n=1}^{\infty} T_n t_n^{\mathrm{III}}. \quad (29)$$

Finally, substitution of (29) into (26) gives

$$2P_A + (1 + K_f \lfloor f_p(0) \rfloor \{A_P^B\}) P_B - 4K_f \sum_{n=1}^{\infty} T_n t_n^{\mathrm{III}} = -K_f \lfloor f_p(0) \rfloor \{A_r^B\} v_r. \quad (30)$$

By now, the formulation for the coupled soil-pile-structure system is completed. In particular, the unknown constants are $t_n^{\mathrm{I}}, t_n^{\mathrm{II}}, t_n^{\mathrm{III}}, C_n^{\mathrm{I}}, C_{mn}^{\mathrm{II}}, P_A$ and P_B, and their governing equations are given by (6), (10), (13), (20), (23), (28) and (30). If the number of terms used for indices *n* and *m* are *N* and *M* respectively, the numbers of unknowns and governing equations of are both equal to (*M*+4)*N*+2. Once the solutions for $t_n^{\mathrm{I}}, t_n^{\mathrm{II}}, t_n^{\mathrm{III}}, C_n^{\mathrm{I}}, C_{mn}^{\mathrm{II}}, P_A$ and P_B are obtained, all responses of the building, soil and piles can be calculated.

NUMERICAL RESULTS AND DISCUSSIONS

As remarked earlier, the details of the stress distribution in a building depends on its exact structural form. The main objective here is to investigate the role of full coupling between soil, pile and structure, and not the detailed structural responses of a building with a particular structural form. To illustrate effect of coupling, we define the following amplification factor K_0 at the pile cap level:

$$K_0 = \begin{cases} \left|\dfrac{v_0}{v_r}\right| & \text{for the coupled system} \\ \left|\dfrac{v_s}{v_r}\right| = \left|1+\sum_{n=1}^{\infty} a_n^f\right| & \text{for the free field} \end{cases} \quad (31)$$

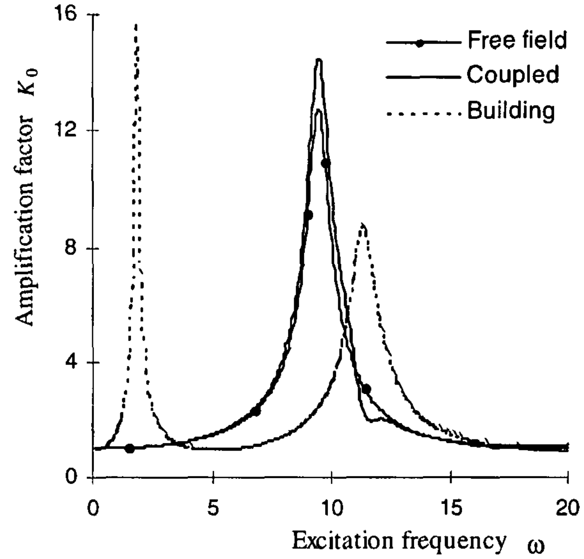


Fig.3 Amplification factor vs. Excitation Frequency for soil thickness H=40 (m) and 30 story building

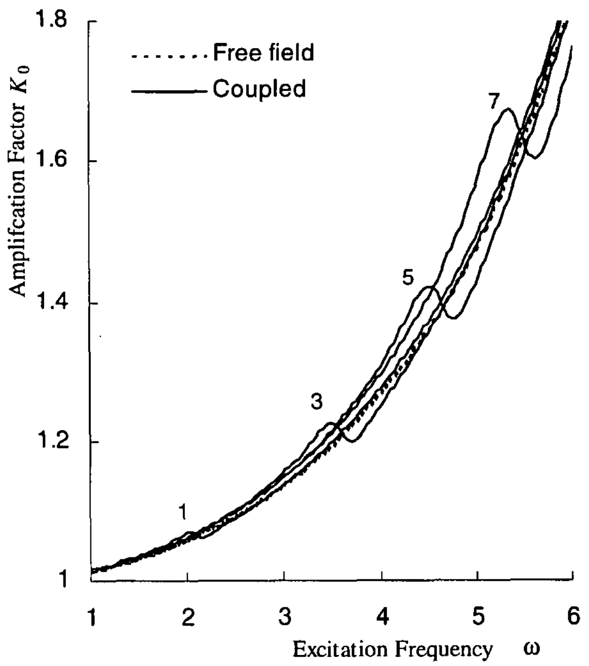


Fig.4 Amplification factor vs. Excitation Frequency for 20 story building, soil thickness H=40(m) with different building stiffness E_bI_b=k $(E_bI_b)_0$

where the pile cap and the free field soil responses relative to the bedrock shaking are used to signify the amplification of the coupled system and the free field response respectively

The calibration of the model parameters are similar to that by Koo et al. (2000), except that a pile group system of 3x6 piles is considered as an example. The dimension of the pile cap is axb, where $a=2d$, $b=5d$, and d=3.77m. The pile diameter is assumed as 1.2m. All parameters used are the same as those used in Koo et al. (2000) except for the following:

$$M_f = 1.1370\times10^4 L \text{ (kg)}, \text{ and } m_b = 5.6852\times10^4 \text{ (kg m}^{-1}\text{)}$$

In addition, the stiffness of the building is given by

$$E_bI_b = 4.7358\times10^3(L^2 + 197.46L - 362.08)L^2 \ (N\,m^2) \quad (32)$$

For a 30-story building built on a soil of thickness H=40m, the variation of K_0 with the excitation frequency ω is shown in the Fig. 3, together with the response of a building (dotted line). The predominant frequencies of the soil, the pile and the building are $\omega_s = 9.449\text{s}^{-1}, \omega_p = 1.336\text{s}^{-1}$, and $\omega_b = 1.801\text{s}^{-1}$ respectively. Similar to the observation by Koo et al. (2000), except near the excitation frequency slightly larger than that of the building, installation of pile foundation actually leads to increase in the magnitude of shaking at the ground level comparing to the free field soil shaking.

To examine the effect of the building stiffness, Fig. 4 plots K_0 versus ω for various values of relative stiffness ratio $k=(E_bI_b)/(E_bI_b)_0$ for the building, where $(E_bI_b)_0$ is the stiffness estimated using (32). Although the natural frequency of the building increases with $(E_bI_b)^{1/2}$ and K_0 shifts to the right, the magnitude of K_0 increases very rapidly with the building stiffness such that the system response at ground level remains larger than the free field response at the predominate frequency. Therefore, simply increasing the building stiffness may not necessarily improve the earthquake resistance of the building, in the sense that a larger ground level shaking is actually induced. This conclusion is the same with that of the case of infinite pile array obtained in Koo et al. (2000). This is very important in the sense that some structural engineers seems to believe that a stiffer building (as required by wind-resistant design) will also behave more favorably under earthquake shaking. Fig. 4 clear shows that it may not be the case if the soil-pile-structure interaction is incorporated. More studies are clearly needed to further clarify this paradox, especially for the case of finite pile group system.

CONCLUSIONS

In this study, the analysis given in Koo et al. (2000) is generalized to the case of finite pile group. In particular, it is shown that the amplification factor at the ground level decreases with pile stiffness. More importantly, in contrast to conclusion by Koo et al. (2000) for the case of infinite pile array, increase in the building stiffness does not necessarily improve the seismic performance of the building. Actually, on the contrary, a stronger coupled ground level shaking may take place because of the increase in the building stiffness. Much effort is still needed to investigate the effects of soil nonlinearity, soil-pile gapping, slippage and separation, and the cracking of concrete pile on the soil-pile-structure interactions.

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