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Influence of soil-structure interaction on pounding response of adjacent buildings due to near-source earthquakes

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A numerical approach to analyse pounding responses of adjacent buildings on subsoil to earthquakes is presented. The nonlinear calculation of the soil-structure system is performed subsequently in the Laplace and the time domain. The adjacent buildings and the subsoil are described by finite elements and boundary elements, respectively. In the numerical investigation the effect of Kobe, Northridge and Chi-Chi near-source earthquakes is considered. The result reveals that both the subsoil and long-period pulses in the ground motions can increase the pounding potential of buildings. In addition, poundings can amplify the induced floor vibrations. In contrast, soil-structure interaction has reduction effect on the induced vibrations. In order to estimate the distance required to prevent pounding the influence of the soil-structure interaction is significant.

Key Words: Structural poundings, nonlinear soil-structure interaction, near-source earthquakes

1. Introduction

Pounding problem of adjacent buildings occurs when the lateral space between the buildings is not sufficient to allow the buildings to vibrate freely. Severe damage can also happen due to pushing of the end building of a series of very closely built buildings (Michoacan earthquake in 1985 in Mexico and Chi-Chi Taiwan earthquake 1999). Even the buildings are far apart poundings can take place due to the collapse of the opposite buildings (Chi-Chi earthquake) or the collapse of a nearby structure next to an express way (Kobe earthquake[1] in 1995). Poundings can also take place when one or several pedestrian bridges link the neighbouring multi-storey residential buildings (Kobe earthquakes[1]). Poundings between bridge girders are also often observed in the recent earthquakes[2]. Collisions of buildings against their surrounding can occur as well when base-isolated buildings experience stronger ground excitations than it is expected[3].

After the Mexico earthquake in 1985 pounding problems attracted many researchers. Two main objectives of their investigations are to determine the necessary distance between structures to avoid pounding and to find a proper measure to reduce the effect of severe poundings. Anagnostopoulos[4] and Athanassiadou et al.[5] investigated the pounding behaviour of a series of buildings by using single-of-degree-freedom (SDOF) systems. An extension of the investigations to multi-degree-of-freedom (MDOF) systems is performed by Anagnostopoulos and Spiliopoulos[6]. Filiatrault and Wagner[7] as well as Zhu et al.[8] performed experimental investigations to verify their numerical model. Kasai[9] indicated the significant of a consideration of phase between building vibrations in the determination of the required distance. Penzien[10] and Hao et al.[11] suggested procedures for a determination of the distance between buildings. Recent investigations by Zanardo et al.[12] indicate that it is important to consider the spatially varying ground motions, since they can significantly amplify the pounding effect. Ruangrassamee and Kawashima[13] as well as DesRoches and Fenves[14] investigated pounding behaviour of bridge girders by using SDOF systems. Jankowski[15] et al. studied the performance of hard rubber bumpers between bridge girders to reduce the poundings. In order to reduce the pounding potential Luco and Barros[16] proposed uniformly distributed viscous dampers between the buildings. To reduce the pounding effect an application of collision walls is suggested in the European code[17].

Numerous papers on the pounding issues have been published in the past 15 years. However, investigation on pounding response including soil-structure interaction (SSI) is very few. Recently, Rahman et al.[18] used frequency independent soil stiffness in their investigations on the effect of soil on structural poundings. Investigation on pounding responses of adjacent buildings due to strong near-source earthquakes is not known.

In this work the response of two adjacent buildings to the Kobe, Northridge and Chi-Chi near-source earthquakes is considered. A pedestrian bridge links the two buildings. Since buildings with an assumed fixed base can behave differently than buildings with subsoil, the influence of the subsoil on the building responses is taken into account.

2. Numerical approach

2.1 Adjacent buildings with bridge link and subsoil

In the numerical analysis the two adjacent buildings and the bridge link are described by finite element method, and the subsoil by boundary element method. By coupling the two subsystems the dynamic stiffness of the whole system adjacent buildings with bridge link and subsoil can be determined.

The dynamic stiffness of the building member can be determined by solving the equation of motion in the Laplace domain analytically. The effect of the continuous distribution of the mass of the structural members on the building response can be therefore taken into account. Compared to the lumped-mass and consistent-mass formulation in the time domain the considered continuous-mass model can produce more precise results. By adding the stiffness of the each member the dynamic stiffness $\left[\widetilde{K}_b\right]$ of each building can be defined. $\{\widetilde{\ }\}$ indicates a vector or matrix in the Laplace domain. Details of the determination of the building stiffness are given by the author[19].

We obtain the dynamic stiffness of the subsoil by transforming the wave equation

$$(c_p^2 - c_s^2)\, u_{j,ji} + c_s^2\, u_{i,jj} + p_i = \ddot{u}_i \tag{1}$$

into the Laplace domain

$$(c_p^2 - c_s^2)\, \widetilde{u}_{j,ji} + c_s^2\, \widetilde{u}_{i,jj} + s^2 \widetilde{u}_i = -\widetilde{p}_i \tag{2}$$

where c_p and c_s are the compressive and shear wave velocity, respectively. p_i is the component of the body force per unit mass. i and j = 1, 2, 3. By using the full-space fundamental solution of the displacement $\widetilde{U}_{ij}$ and traction $\widetilde{T}_{ij}$ and by assuming a distribution of the displacement u_i and traction t_i along the boundaries we obtain a number of algebraic equations

$$\left[\widetilde{T}\right]\{\widetilde{u}\} = \left[\widetilde{U}\right]\{\widetilde{t}\} \tag{3}$$

An assumption that the surface of the subsoil is traction free leads to the relationship at the contact area between the foundation of the buildings and the subsoil. A consideration of the area of the elements leads to the dynamic stiffness of the subsoil.

$$\left[\widetilde{K}_s\right]\{\widetilde{u}_s\} = \{\widetilde{P}_s\} \tag{4}$$

The coupling of the building and the subsoil is achieved by equating the displacements and by equilibrating the forces at the interface between building foundations and subsoil. We then obtain the governing equation of the system *building I with subsoil* and of the system *building II with subsoil*

$$\left[\widetilde{K}^I\right]\{\widetilde{u}^I\} = \{\widetilde{P}^I\} \tag{5}$$

and

$$\left[\widetilde{K}^{II}\right]\{\widetilde{u}^{II}\} = \{\widetilde{P}^{II}\}, \tag{6}$$

respectively.

It is assumed that all location of both of the building foundations will experience the same ground excitation. After transforming the ground excitation from the time domain into the Laplace domain

$$\{\widetilde{P}(s)\} = \int_0^\infty \{P(t)\} e^{-st}\, dt\,, \tag{7}$$

where $s = \delta + i\omega$ is the Laplace parameter and $i = \sqrt{-1}$, we can obtain the linear response $\{\widetilde{u}(s)\}$ of the building I and II by using Eq. (5) and Eq. (6), respectively. A transformation of the results from the Laplace to the time domain gives us the time history response of the buildings

$$\{u(t)\} = \frac{1}{2\pi i} \int_{\delta - i\omega}^{\delta + i\omega} \{\widetilde{u}(s)\} e^{st}\, ds \tag{8}$$

Discussion about accurate numerical Laplace transform is given by Hillmer[20].

2.2 Nonlinear analysis

Nonlinear behaviour occurs when the two buildings come into contact or when the buildings separate again. In the nonlinear analysis of the soil-structure systems this nonlinear structural behaviour is described by piecewise linear behaviours. In the course of each of these linear behaviours, the buildings are either in contact or separated, the response of the buildings will be calculated in the Laplace domain. The change from one linear behaviour to the following one will be defined in the time domain.

(1) Single-degree-of-freedom (SDOF) system

In order to introduce this numerical approach the nonlinear calculation of a SDOF system subjected to a triangle load is considered. The material data of the system is given in Fig. 1(a). It is assumed that the spring can only behave elastic or perfectly plastic (Fig. 1(h)). The governing equation of the system is

$$\widetilde{k} \quad \widetilde{x} = \widetilde{p}\,, \tag{9}$$

and the dynamics stiffness of the system is

$$\widetilde{k} = s^2 + 2\beta\omega_e s + \omega_e^2\,, \tag{10}$$

where s is the Laplace parameter, ω_e is the natural frequency of the system, and the viscous damping $\beta = c/(2m\omega_e)$.

For the given load $\widetilde{p}$ we obtain the linear response $\widetilde{x}_{lin}$ of the system in the Laplace domain. After transforming the result to the time domain, we can determine the time when the spring leaves the elastic range (Fig. 1(b)). When the displacement response passes through the limit of the elastic range x_e, at that instant, the spring starts to yield. In the considered case this happens at the time t_e of 0.49s. Beyond this instant the system has no stiffness any longer, since the spring stiffness has zero value (range (2) in Fig. 1(h)). Now, only the inertial force of the mass and the damping force determine the vibration of the system.

In order to incorporate the change of the nonlinear spring behaviour we correct the linear response $x_{lin}(t)$ by adding a corrective result beginning from the time t_e. We obtain the corrective result from a subsequent calculation using the actual dynamic stiffness of the system and a corrective load.

In general this load can be defined as

$$\widetilde{p} = -\Delta\widetilde{x}_j(\widetilde{k}_{actual} - \widetilde{k}_{before}) \tag{11}$$

where $\Delta\widetilde{x}_j$ is the Laplace transform of

$$\Delta x_j(t) = x_{before}(t) - x_{limit}$$

j is the number of the change in the system

$t > t_c$, and t_c is the time when the change occurs.

After transforming the differential value $\Delta x_1(t)$ we define the unbalanced load $\widetilde{p}_1$ from Eq. (11). Since it is difficult to have an insight in Laplace domain the unbalanced load $p_1(t)$ is presented in the time domain in Fig. 1(c). The resulting corrective result $x_1(t)$ in Fig.1(d) is obtained from Eq. (9). By adding this result to the linear result we correct the system response. Fig. 1(e) shows the time history of the corrected response $x_{i1}(t)$. An examination of the result indicates that the system has its maximum response x_m of 0.092m at the time t_m of 1.21s. The maximum takes place when the velocity response of the system is zero. Beyond this instant the system is in the releasing phase (3) (see Fig. 1(h)), and the spring has its stiffness k_F again.

To incorporate this new condition we define the differential value $\Delta x_2(t) = x_{i1}(t) - x_m$ in the time domain. After transforming this value into the Laplace domain we obtain the corresponding corrective load $\widetilde{p}_2$ from Eq. (11). Fig. 1(f) shows the load $p_2(t)$. We obtain the resulting corrective result from Eq. (9). The transformation of the result $x_2(t)$ into the time domain is displayed in Fig. 1(g). By adding this result to the last corrected result $x_{i1}(t)$ we have the actual vibration response $x_{i2}(t)$. An examination of this response shows that there is no more change in the system behaviour. The response is therefore the final nonlinear response $x(t)$ of the system to subjected load. The heavy dark line in Fig. 1(h) indicates the path that was passed by the system. The result $x(t)$ coincides well with the result obtained by using a step-by-step numerical integration procedure[21] directly in the time domain with a time step of Δt of 0.012s.

(2) Pounding and separation cases

In pounding cases we define the differential values $\{\Delta u\}$ at the contact-degree-of-freedom (CDOF) from the instant t_i when poundings take place. In order to determine the unbalanced load we condense one of the buildings into the CDOF. With the dynamic stiffness $[\widetilde{K}_c]$ of the condensed building-soil system the unbalanced load in the Laplace domain is

$$\{\widetilde{P}_u\} = [\widetilde{K}_c]\{\Delta\widetilde{u}\} \tag{12}$$

Using the governing equation of the uncondensed system we define the corrective result $\{\widetilde{u}_n\}$ due to the unbalanced load. Since both systems are now in contact the dynamic stiffness $[\widetilde{K}_c]$ has to be added to the equation. After transforming the corrective result into the time domain we correct the previous linear responses beginning at the time t equal t_i.

If the buildings are separated at time t_j, for example, the unbalanced load is equal to the contact forces $\{\widetilde{P}_c\}$. From the time t_j both of the systems experience at the previous contact locations the unbalanced loads. The corrective results in the Laplace domain can be obtained from Eq. (5) and Eq. (6). We then correct the response of the buildings from the time t_j.

We examine again the actual responses in the time domain whether further poundings occur. The analysis is complete when there is no more pounding or separation.

3. Response of buildings with poundings and SSI

Fig. 2 displays the considered adjacent buildings with a pedestrian bridge at the third floor. The bridge is moveable at the right end. The distance between the bridge end and the second building is sd. The material data of the buildings and bridge link are given in Table 1. The same number in the parentheses indicates that the members consist of the same material. The material damping is described by a complex modulus of elasticity, which is governed by the Kelvin parameters E1 and En[22], so that the buildings have almost frequency independent material damping. For the chosen value E1 = 0.1 and En = 10^{28} the equivalent damping ratio is about 1.4%. The fundamental frequency of the first and second building with an assumed fixed base and without the bridge link is f^{I}_{f} of 1.065Hz and f^{II}_{f} of 1.248Hz, respectively.

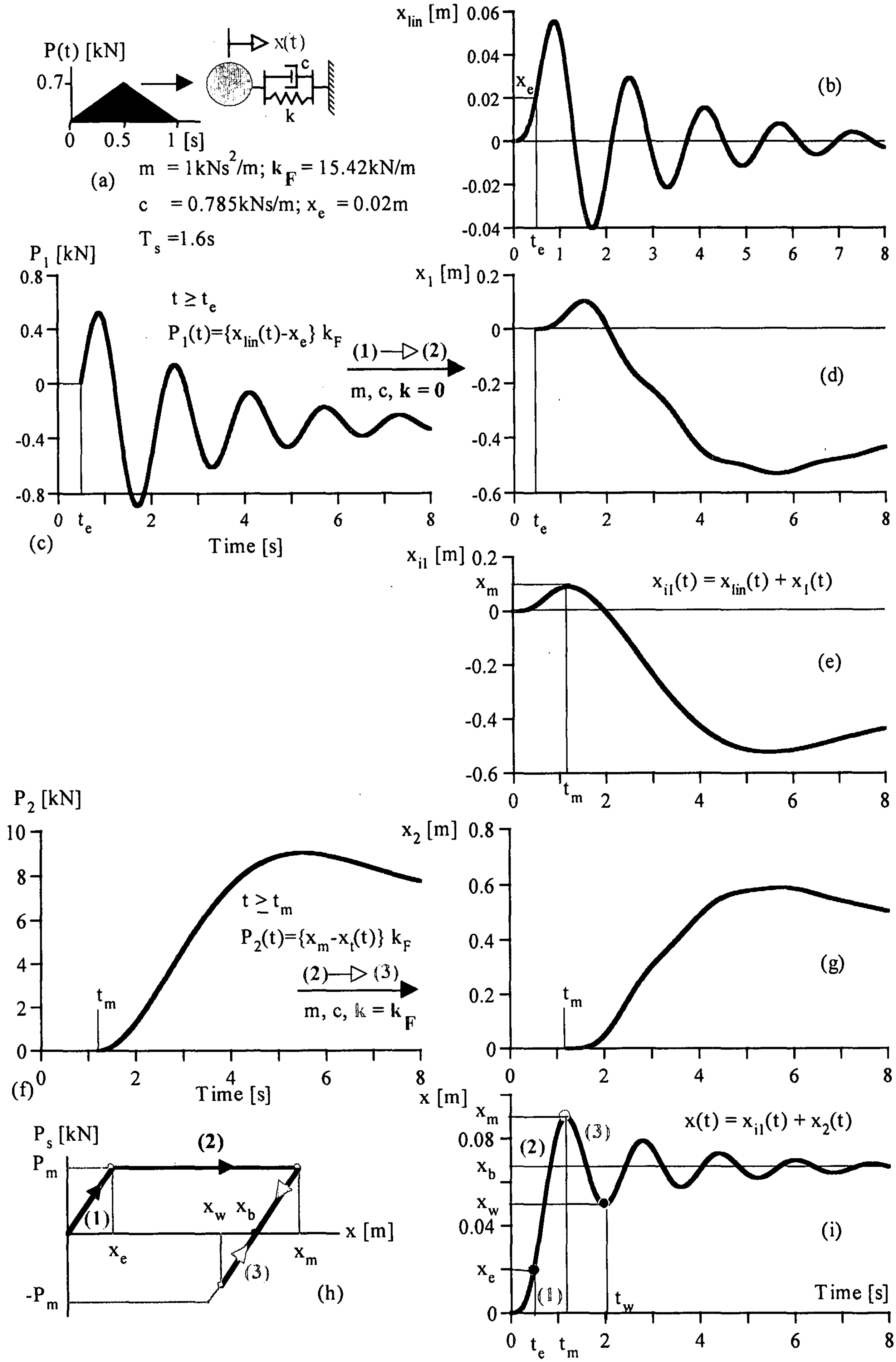


Fig. 1(a)-(i) Nonlinear analysis of the response of a SDOF system in the Laplace and time domain

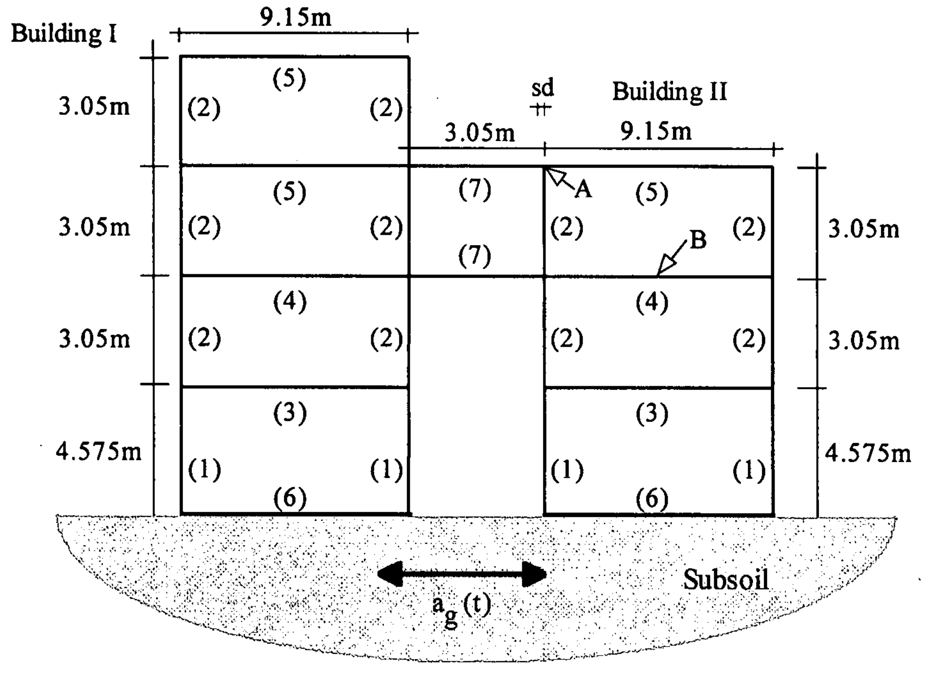


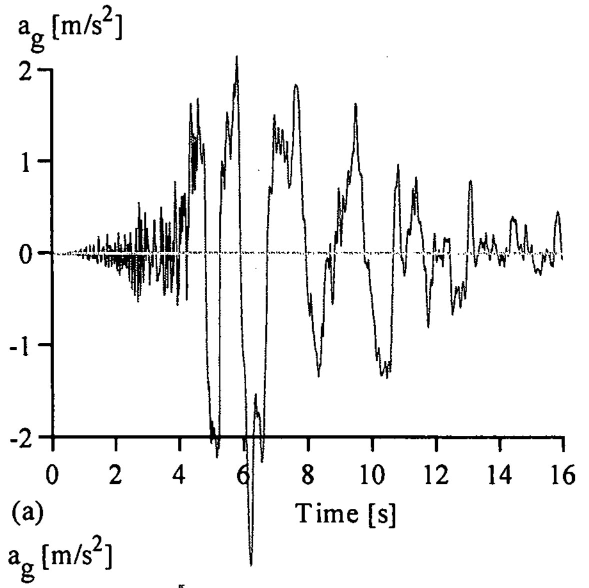
Fig. 2 Adjacent buildings with bridge link and subsoil

Table 1 Material property of the buildings and the bridge link

No of the structural member	Mass [kg/m]	EA [10^6 kN]	EI [10^3 kNm^2]
(1)	67	1.72	21
(2)	33	0.837	9.8
(3)	2447	3.19	200
(4)	2358	3.19	200
(5)	1209	3.19	110
(6)	4500	54	200
(7)	314	0.837	9.8

It is assumed that the subsoil is a half-space with no material damping. The soil has the shear wave velocity of 100m/s and the density of 2000kg/m^3. The Poisson ratio is 0.33. The ground excitation is the ground accelerations in the north-south direction at the Kobe Port Island. The long-period pulses in the time history in Fig. 3(a) due to forward rupture directivity effects[23] can be clearly seen in the dominance of the excitation in the frequency range around 0.5Hz and 1Hz (Fig. 3(b)). The vertical lines in the figure show the location of the fundamental frequency of the buildings. These locations indicate that the first building will experience stronger ground excitation than the second building.

Pounding potential is not necessarily determined by the maximum response of the adjacent buildings. Poundings often take place because of a different phase between the vibrations of the adjacent buildings. This means the subsequent large building responses are more important in the determination of the required distance between the buildings.



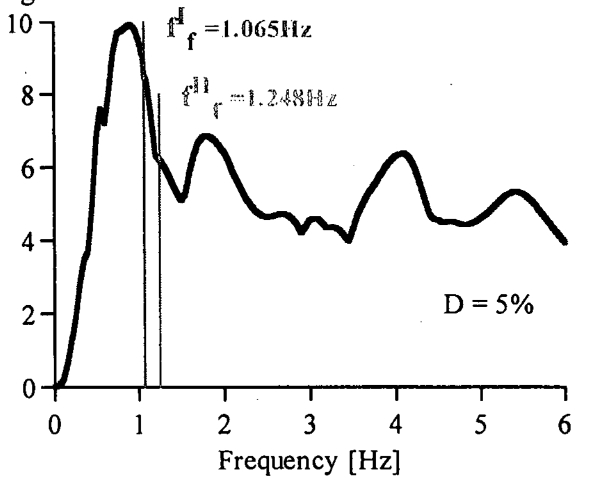
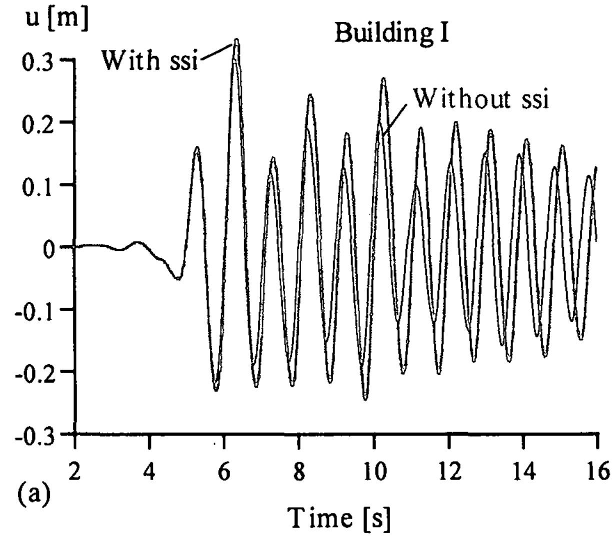


Fig. 3(a) and (b) Kobe earthquake at the Kobe Port Island. (a) North-south component and (b) its response spectrum



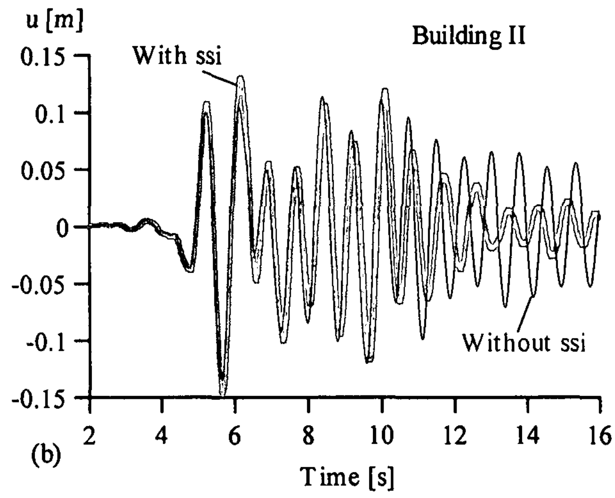


Fig. 4(a) and (b) Influence of SSI on the displacement at the top of the buildings, (a) building I and (b) building II

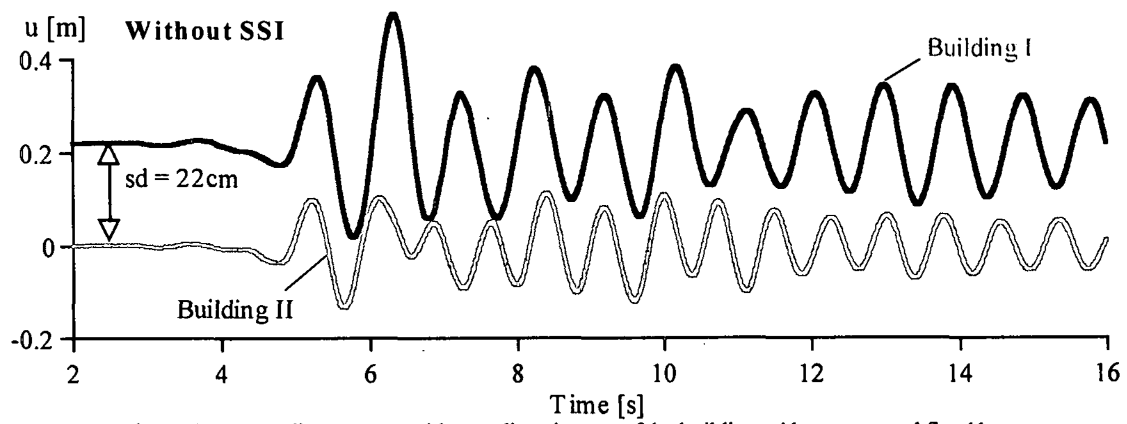


Fig. 5 Necessary distance to avoid poundings in case of the building with an assumed fixed base

Fig. 4(a) and (b) show the horizontal displacement at the top of the first and second building, respectively. Poundings are neglected. The response of the first building is stronger than the one of the second building, because the fundamental frequency of the first building is closer to the dominant frequency of the ground excitation. The SSI effect can be seen in the lagging of the response with a longer natural period. The lengthening of the natural period of the buildings due to the existing subsoil can cause an additional increase of the effect of the ground excitation, since in the considered case a slight shift of the natural frequency towards lower frequency means stronger excitation, as we can in Fig. 3(b). Although only a slight increase of the maximum response can be observed. However, a recognizable increase of the subsequent large responses can be seen in the response of the first building.

The result shows that an assumption of a fixed base can clearly underestimate the pounding potential of the buildings, since the soil can change not only the phase of the vibration of the adjacent buildings, it can also increase the subsequent large responses, especially, when the near-source ground motions with a pronounced dominant frequency range are of concern.

Fig. 5 shows the horizontal displacement of the two buildings at the height of the roof of the second building (location A in Fig. 2). In order to see the effect of the soil in the later comparison the soil-structure interaction is neglected first. This result clearly shows that not the maximum response of the buildings is significant, but the phase and consequently the subsequent large responses are determined in the occurrence of poundings. Even though the maximum response of the first building is 27.7cm, only a distance sd of 22cm is necessary to avoid poundings.

Fig. 6(a) and (b) display the horizontal displacement response without and with a consideration of SSI at the location A (see Fig. 2), respectively. The lowering of the fundamental frequency of the buildings due to the subsoil brings a stronger excitation of the buildings. This factor and the flexibility of the soil cause therefore larger amplitude of the response. The distance sd of 22cm is no longer sufficient, as we can see in Fig. 6(b). In case of building with subsoil the required distance to prevent pounding is 26.5cm, an increase of 20.4%.

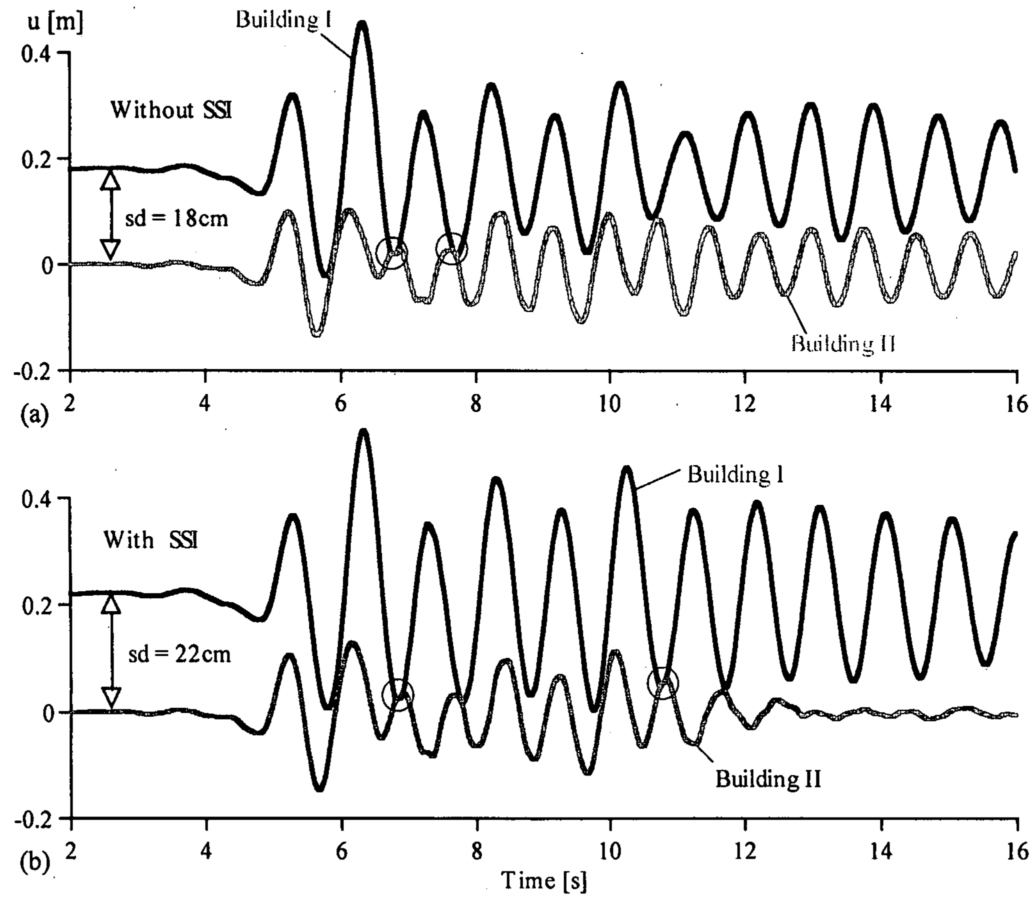


Fig. 6(a) and (b) Pounding responses, (a) without and (b) with a consideration of SSI

An assumption of a separation distance sd of 22cm will cause pounding at 6.85s and 10.8s, indicated by the circles in the figure. In case of buildings with fixed base poundings occur at 6.78s and 7.7s, if a separation distance sd is 18cm (Fig. 6(a)).

Fig. 7 shows the average axial force in the upper girder of the bridge link. It is not the contact force, which will become zero when the buildings separate again. The axial force is caused by the propagating waves in the girder induced by the collisions. The pounding between the bridge girders and the second building causes a sudden jump in the axial force. Corresponding to the time of the pounding occurrences the jump in case of no SSI takes place slightly earlier. The jump in case of with SSI is much stronger. The second jump in the axial force without and with a consideration of SSI occurs at 7.7s and 10.8s, respectively (see also Fig. 6(a) and (b)). The vertical lines in Fig. 7 indicate the time of these occurrences. The information of the axial forces can be useful for a development of a proper reduction measure at the possible contact locations.

Fig. 8(a) and (b) shows the development of the bending moment at the support of the lowest left column of the second and first building, respectively. In Fig. 8(a) only the pounding effect is considered. No increase of the maximum bending moment due to the poundings can be observed. In Fig. 8(b) the soil-structure interaction effect is considered. SSI does not increase the maximum bending moment, however, an increase of the subsequent large bending moment can be observed.

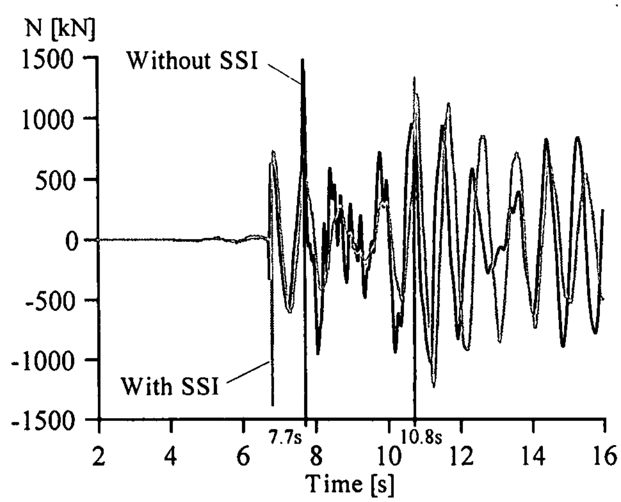


Fig. 7 Influence of SSI on the development of the axial force in the upper girder of the bridge

Table 2 shows that a neglect of the influence of soil-structure interaction can underestimate the necessary distance between the buildings to avoid poundings. In the considered cases up to more than 25% larger distance is required. The considered near-source earthquakes are displayed in Fig. 3(a) and Fig. 9.

Fig. 10 shows the induced horizontal accelerations at the middle of the middle girder of the second building (location B in Fig.2). The determination of these induced vibrations is significant for the design of secondary structures in the buildings.

Secondary structures are structures attached to the floors, roof or walls, which do not bear load, like mechanical and electrical equipments. In the common case a secondary structure will be described by a SDOF system with an assumed fixed base. The maximum response of the SDOF system of certain natural frequency and damping to the induced vibrations is then displayed in a response spectrum.

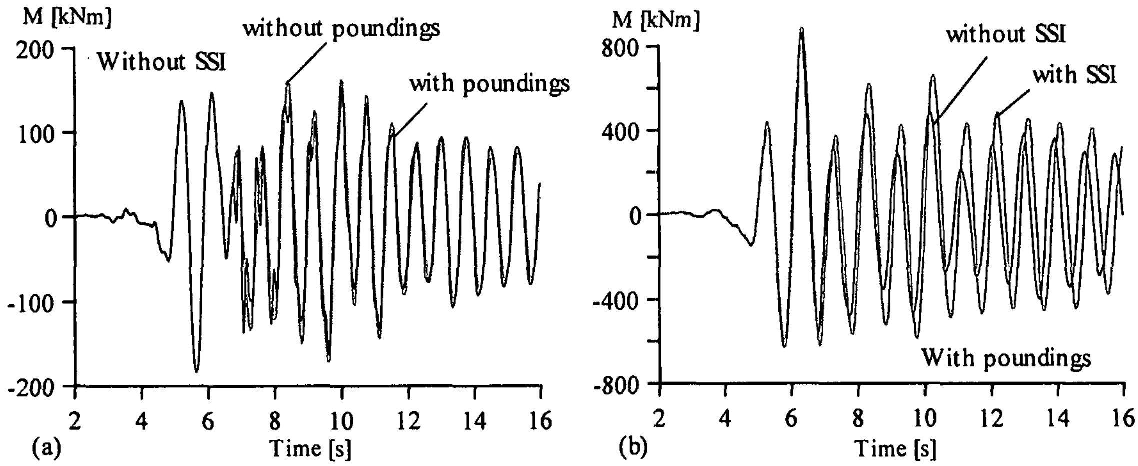


Fig. 8(a) and (b) Influence of SSI and pounding on the bending moment in (a) building I and (b) building II

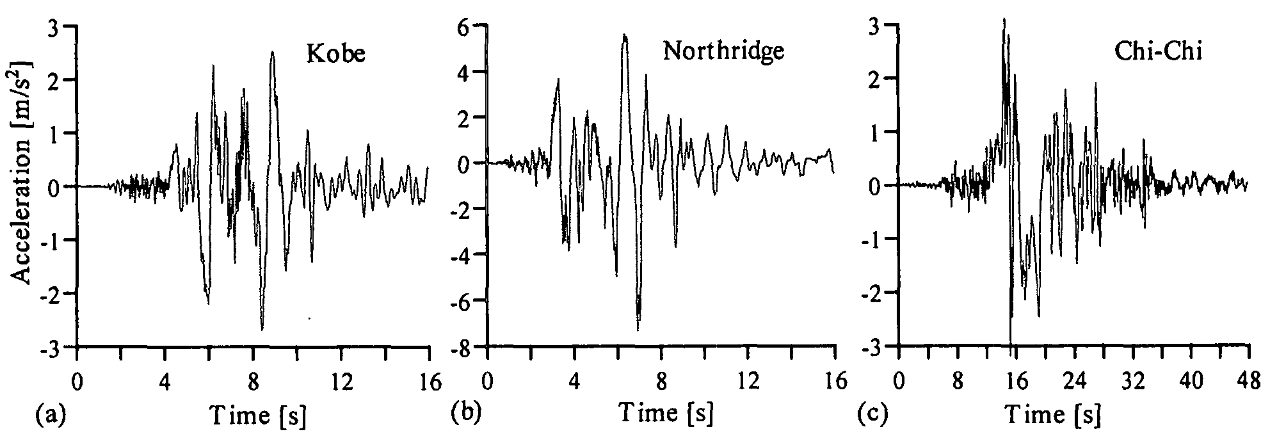


Fig. 9(a)-(c) Acceleration time histories of considered near-source earthquakes

Table 2 Influence of SSI on the minimum required distance between the two buildings

Earthquake	Station	Direction	Epicentral Distance [km]	Peak acceleration [m/s^2]	Required distance		
					Without SSI [cm]	With SSI [cm]	$\frac{\text{With SSI}}{\text{Without SSI}}$ [%]
Hanshin-Awaji (Japan, 1995)	KPI	N-S	20.0	3.41	22.0	26.5	20.4
	KBU	N-S	25.3	2.70	22.0	25.0	13.6
Northridge (USA, 1994)	SCG	S38E	10.8	7.39	75.0	94.0	25.3
Chi-Chi (Taiwan, 1999)	TCU068	N-S	46.3	3.23	20.5	22.5	9.8

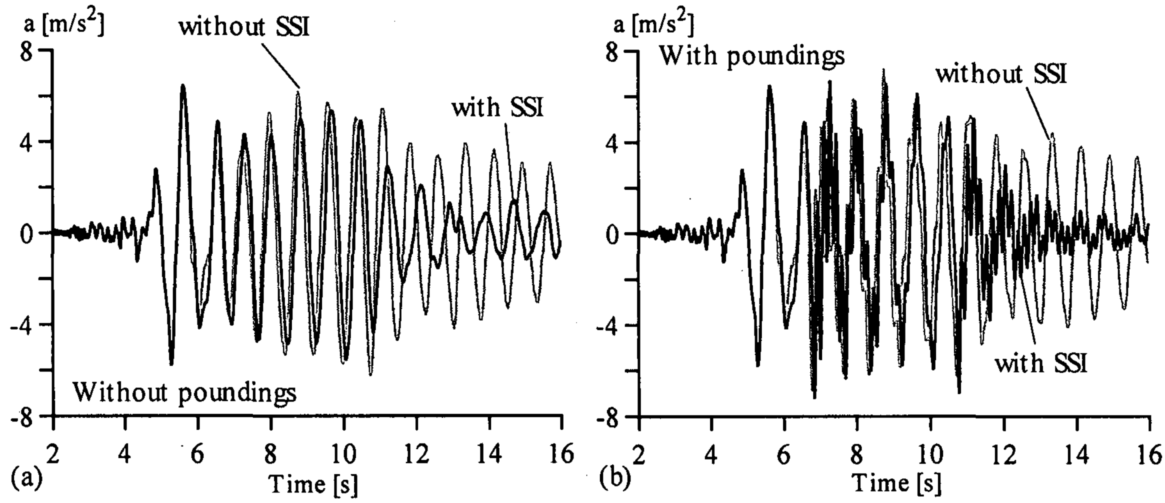


Fig. 10(a) and (b) Influence of poundings and SSI on induced vibrations

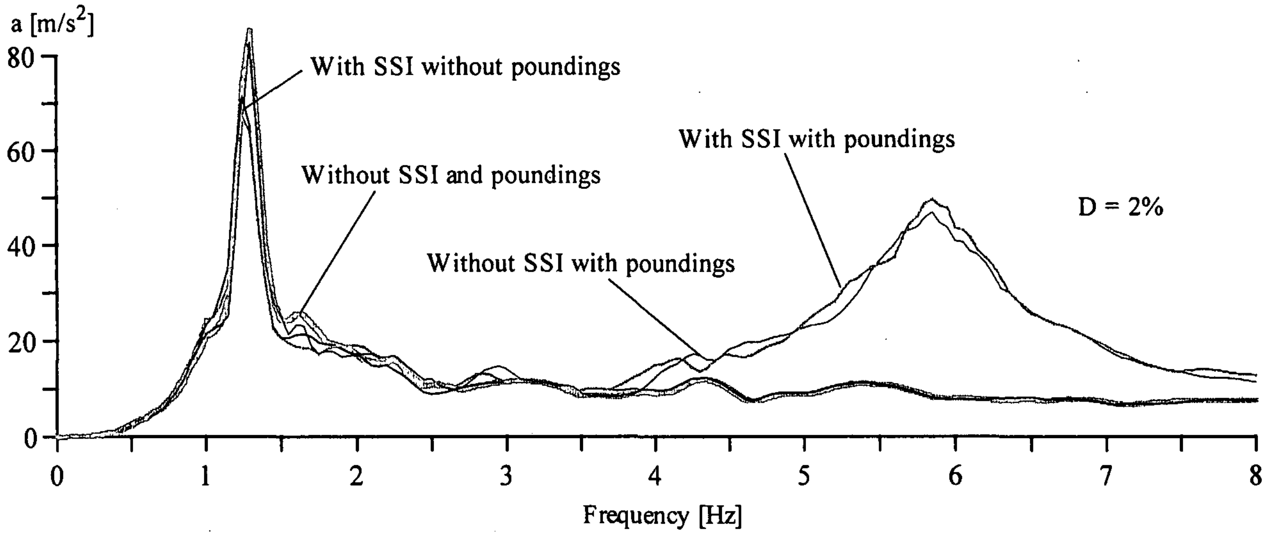


Fig. 11 Effect of poundings and SSI on the response of secondary structures

The soil-structure interaction causes a decrease of the subsequent induced vibrations (Fig. 10(a)). A simultaneous effect of the soil-structure interaction and poundings in Fig. 10(b) shows that subsequent induced vibrations decrease, and high frequency vibrations are induced into the buildings.

Fig. 11 shows the response acceleration spectrum for a secondary structure, which is attached at the middle of the middle girder of the second building (location B in Fig.2). It is assumed that the interaction between the secondary structure and the girder can be neglected. The results without poundings indicate that only the first mode of the building is excited by the ground motions. Secondary structures with a natural frequency close to the fundamental frequency of the second building will therefore the strongest excitation.

The poundings excited the third mode of the building, since in this mode the building has a largest lateral vibration at the middle girder. Poundings at these locations due to the bridge link cause therefore an amplification of induced vibration in the third natural frequency range around 5.7Hz. Secondary structures with a natural frequency close to 5.7Hz will be therefore strongly excited. In the considered case the soil-structure interaction causes only a slight decrease of the secondary structure response.

4. Conclusions

In the numerical analysis the adjacent buildings and the subsoil are described by finite elements and boundary elements, respectively. The poundings and the separations of the buildings are determined in the time domain. The response of the buildings is calculated in the Laplace domain.

The investigation reveals:

Strong long-period pulses in near-source ground motions cause stronger excitations at the low frequency range. As a consequence of this, high buildings are likely stronger affected than low buildings. Strong near-source ground excitations may therefore increase the pounding potential of adjacent buildings.

Since subsoil tends to lengthen the fundamental vibration period of the buildings, depending on the natural frequencies of the buildings, buildings will experience stronger ground excitations than the buildings with an assumed fixed base, when the shifted fundamental frequency meets the dominant frequencies of the ground motions, as we can see from the discussion of Fig. 4 and Fig. 6 in connection with Fig. 3(b).

Even if the subsoil causes only a slight reduction of the fundamental frequency of the buildings, a neglect of the soil-structure interaction effect can underestimate the building pounding potential, since the flexibility of the soil causes larger amplitude of the building responses.

Poundings might not increase the forces in the structural members, they may, however, excite the higher modes of the buildings. Consequently, the induced vibrations can have additional higher frequency content. These induced vibrations can cause larger excitation of secondary structures, since secondary structures have in general high natural frequencies.

Soil-structure interaction can cause a reduction of the induced vibrations.

In order to be able to determine the controlling parameters of pounding responses of neighbouring buildings to near-source earthquakes further investigations are necessary.

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