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SEISMIC RESPONSE EVALUATION FOR ISOLATED AND NON-ISOLATED BUILDINGS CONSIDERING POUNDING

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ABSTRACT

Previous research on pounding between seismically isolated buildings during earthquakes has been focused on impacts at the bases of structures. However, the effect of simultaneous interactions at the bases and at the superstructures has not been studied. In this research, the seismic responses of adjacent buildings considering impacts between bases and/or superstructures are investigated. The study is carried out in two parts for two cases of adjacent buildings: (i) both structures have fixed bases, (ii) both structures have base isolation systems. The nonlinear viscoelastic model of impact force during impact is used to capture the pounding forces at the bases and at the superstructures. The responses of the buildings and the bearings with and without impacts under the El Centro ground motion record are presented. It has been observed that the pounding-involved responses of the buildings mainly depend on the type of structural base systems of both structures. The acceleration and displacement responses at a base of the isolated building increase due to impacts. The acceleration at the level of the base isolation increases approximately with the same trend, whereas the displacement responses of the base isolated buildings decrease due to impact in the same cases. The results of the study indicate that pounding in base-isolated buildings may lead to considerable structural damage and therefore is should be avoided at the design stage of structures.

Introduction

The pounding-involved response of structures subjected to earthquake loading, which can result in severe structural damage, has recently been a subject of intensive study. A number of different methods has been considered in order to minimize the probability of collisions between adjacent buildings or bridge segments (see, for example, Xu et al. 1999; Jankowski et al. 2000; Bhaskararao and Jangid 2006; Anagnostopoulos and Karamaneas 2008). On the other hand, the use of seismic isolation (see, for example, Kelly 1986; Buckle and Mayes 1990; Komodromos 2000; Faravelli 2001) results in larger structural displacements increasing the probability of collisions.

Structural pounding in isolated buildings may occur either at the foundation (base) or at the storey level if the structures with different dynamic properties are insufficiently separated.

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A number of analyses on pounding between seismically isolated buildings during earthquakes has been focused on impacts at the bases of structures. Malhotra (1997) performed a study investigating the effect of seismic impacts between the base of an isolated building and the surrounding retaining wall. An investigation on the earthquake-induced dynamic response of the base-isolated multi-storey building considering pounding against the retaining wall was conducted by Matsagar and Jangid (2003). Komodromos et al. (2007) studied pounding between the seismically isolated building and the surrounding moat wall in order to investigate the influence of various design parameters and conditions on the peak storey accelerations and displacements.

The above papers indicate that the conducted analyses concerned only collisions at the base level of isolated buildings. Moreover, analysed structures were usually modelled as elastic systems with linear modelling of isolation devices. The objective of the present paper is to investigate the seismic response of isolated and non-isolated buildings considering pounding at the bases as well as at the storey levels. In the analysis, the superstructures of buildings have been modelled as inelastic systems with nonlinear model of isolation devices and using nonlinear viscoelastic model of pounding force during impact.

Numerical Model

The study has been carried out in two parts for two cases of adjacent buildings: (i) both structures have fixed bases, (ii) both structures have base isolation systems. In the study, the discrete elastoplastic lumped mass models of interacting three-storey shear buildings with isolated and non-isolated base systems have been used (see Fig. 1 and Fig. 2).

Equations of Motion

Two Non-isolated Buildings

Let m_i^l, c_i^l, u_i^l, R_i^l and m_i^r, c_i^r, u_i^r, R_i^r (i=1,2,3) be the masses, damping coefficients, displacements and resisting forces for the left and the right building, respectively. In the case of two structures with fixed bases (see Fig. 1), the nonlinear dynamic equation of motion can be written as

$$\begin{pmatrix} \mathbf{M}^l & \mathbf{0} \\ \mathbf{0} & \mathbf{M}^r \end{pmatrix}\begin{pmatrix} \ddot{\mathbf{U}}^l \\ \ddot{\mathbf{U}}^r \end{pmatrix}+\begin{pmatrix} \mathbf{C}^l & \mathbf{0} \\ \mathbf{0} & \mathbf{C}^r \end{pmatrix}\begin{pmatrix} \dot{\mathbf{U}}^l \\ \dot{\mathbf{U}}^r \end{pmatrix}+\begin{pmatrix} \mathbf{R}^l \\ \mathbf{R}^r \end{pmatrix}+\begin{pmatrix} \mathbf{F}_{ps} \\ -\mathbf{F}_{ps} \end{pmatrix}=-\begin{pmatrix} \mathbf{M}^l & \mathbf{0} \\ \mathbf{0} & \mathbf{M}^r \end{pmatrix}\begin{pmatrix} \mathbf{I} \\ \mathbf{I} \end{pmatrix}\ddot{u}_g, \quad (1a)$$

$$\mathbf{M}^l=\begin{pmatrix} m_1^l & 0 & 0 \\ 0 & m_2^l & 0 \\ 0 & 0 & m_3^l \end{pmatrix}, \qquad \mathbf{M}^r=\begin{pmatrix} m_1^r & 0 & 0 \\ 0 & m_2^r & 0 \\ 0 & 0 & m_3^r \end{pmatrix}, \quad (1b)$$

$$\mathbf{C}^l=\begin{pmatrix} c_1^l+c_2^l & -c_2^l & 0 \\ -c_2^l & c_2^l+c_3^l & -c_3^l \\ 0 & -c_3^l & c_3^l \end{pmatrix}, \qquad \mathbf{C}^r=\begin{pmatrix} c_1^r+c_2^r & -c_2^r & 0 \\ -c_2^r & c_2^r+c_3^r & -c_3^r \\ 0 & -c_3^r & c_3^r \end{pmatrix}, \quad (1c)$$

$$\mathbf{R}^l = \begin{pmatrix} R_1^l - R_2^l \\ R_2^l - R_3^l \\ R_3^l \end{pmatrix}, \qquad \mathbf{R}^r = \begin{pmatrix} R_1^r - R_2^r \\ R_2^r - R_3^r \\ R_3^r \end{pmatrix}, \qquad \mathbf{F}_{ps} = \begin{pmatrix} F_{11} \\ F_{22} \\ F_{33} \end{pmatrix}, \tag{1d}$$

$$\mathbf{U}^l = \begin{pmatrix} u_1^l \\ u_2^l \\ u_3^l \end{pmatrix}, \qquad \dot{\mathbf{U}}^l = \begin{pmatrix} \dot{u}_1^l \\ \dot{u}_2^l \\ \dot{u}_3^l \end{pmatrix}, \qquad \ddot{\mathbf{U}}^l = \begin{pmatrix} \ddot{u}_1^l \\ \ddot{u}_2^l \\ \ddot{u}_3^l \end{pmatrix}, \tag{1e}$$

$$\mathbf{U}^r = \begin{pmatrix} u_1^r \\ u_2^r \\ u_3^r \end{pmatrix}, \qquad \dot{\mathbf{U}}^r = \begin{pmatrix} \dot{u}_1^r \\ \dot{u}_2^r \\ \dot{u}_3^r \end{pmatrix}, \qquad \ddot{\mathbf{U}}^l = \begin{pmatrix} \ddot{u}_1^r \\ \ddot{u}_2^r \\ \ddot{u}_3^r \end{pmatrix} \tag{1f}$$

where $\mathbf{M}^l$, $\mathbf{C}^l$ and $\mathbf{M}^r$, $\mathbf{C}^r$, are the mass and damping matrices of the left and the right building, respectively; $\mathbf{R}^l$ and $\mathbf{R}^r$ are vectors consisting of the system resisting forces; $\mathbf{U}^l$, $\dot{\mathbf{U}}^l$, $\ddot{\mathbf{U}}^l$ and $\mathbf{U}^r$, $\dot{\mathbf{U}}^r$, $\ddot{\mathbf{U}}^r$ denote the displacement, velocity and acceleration vectors of the left and the right structure, respectively; $\mathbf{F}_{ps}$ is a vector containing the forces due to impact; **I** is a vector with all its elements equal to unity and $\ddot{u}_g$ is the earthquake acceleration. During the elastic stage, resisting forces, R_i^l, R_i^r, take the form: $R_i^l = k_i^l(u_i^l - u_{i-1}^l)$, $R_i^r = k_i^r(u_i^r - u_{i-1}^r)$, while during the plastic stage: $R_i^l = \pm f_{yi}^l$, $R_i^r = \pm f_{yi}^r$, where k_i^l, k_i^r and f_{yi}^l, f_{yi}^r are the storey stiffness coefficients and yield forces for the left and the right building, respectively.

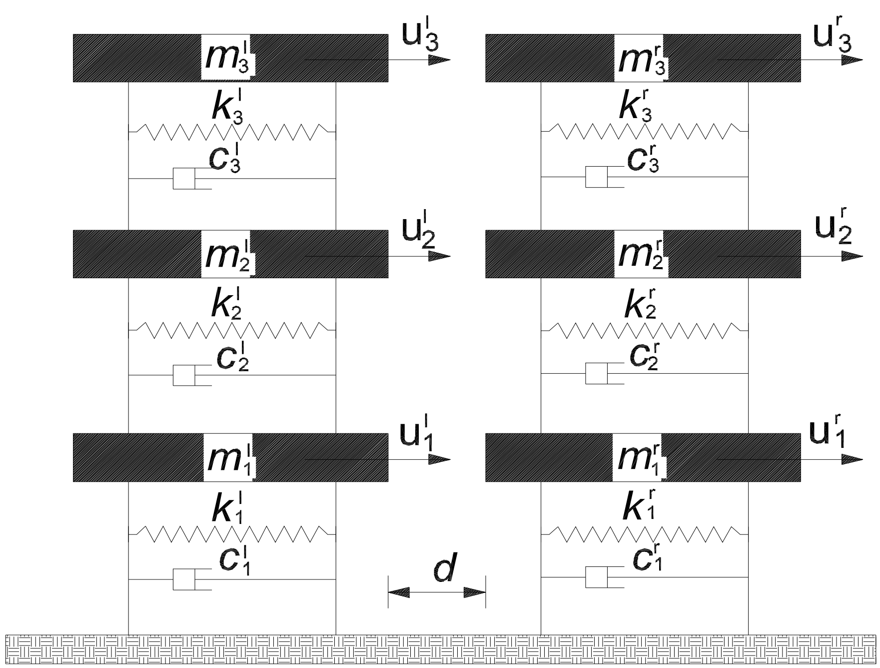

Figure 1. Model of colliding three-storey buildings with fixed bases.

Two Isolated Buildings

In the case of two base-isolated buildings (see Fig. 2), the nonlinear dynamic equation of motion can be written as

$$\begin{pmatrix} \mathbf{M}_b^l & \mathbf{0} \\ \mathbf{0} & \mathbf{M}_b^r \end{pmatrix} \begin{pmatrix} \ddot{\mathbf{U}}_b^l \\ \ddot{\mathbf{U}}_b^r \end{pmatrix} + \begin{pmatrix} \mathbf{C}_b^l & \mathbf{0} \\ \mathbf{0} & \mathbf{C}_b^r \end{pmatrix} \begin{pmatrix} \dot{\mathbf{U}}_b^l \\ \dot{\mathbf{U}}_b^r \end{pmatrix} + \begin{pmatrix} \mathbf{R}_b^l \\ \mathbf{R}_b^r \end{pmatrix} + \begin{pmatrix} \mathbf{F}_{bps}^l \\ -\mathbf{F}_{bps}^r \end{pmatrix} = - \begin{pmatrix} \mathbf{M}_b^l & \mathbf{0} \\ \mathbf{0} & \mathbf{M}_b^r \end{pmatrix} \begin{pmatrix} \mathbf{I} \\ \mathbf{I} \end{pmatrix} \ddot{u}_g, \tag{2a}$$

$$\mathbf{M}_b^l = \begin{pmatrix} m_b^l & 0 & 0 & 0 \\ 0 & m_1^l & 0 & 0 \\ 0 & 0 & m_2^l & 0 \\ 0 & 0 & 0 & m_3^l \end{pmatrix}, \qquad \mathbf{M}_b^r = \begin{pmatrix} m_b^r & 0 & 0 & 0 \\ 0 & m_1^r & 0 & 0 \\ 0 & 0 & m_2^r & 0 \\ 0 & 0 & 0 & m_3^r \end{pmatrix}, \tag{2b}$$

$$\mathbf{C}_b^l = \begin{pmatrix} c_b^l + c_1^l & -c_1^l & 0 & 0 \\ -c_1^l & c_1^l + c_2^l & -c_2^l & 0 \\ 0 & -c_2^l & c_2^l + c_3^l & -c_3^l \\ 0 & 0 & -c_3^l & c_3^l \end{pmatrix}, \qquad \mathbf{C}_b^r = \begin{pmatrix} c_b^r + c_1^r & -c_1^r & 0 & 0 \\ -c_1^r & c_1^r + c_2^r & -c_2^r & 0 \\ 0 & -c_2^r & c_2^r + c_3^r & -c_3^r \\ 0 & 0 & -c_3^r & c_3^r \end{pmatrix}, \tag{2c}$$

$$\mathbf{R}_b^l = \begin{pmatrix} R_b^l - R_1^l \\ R_1^l - R_2^l \\ R_2^l - R_3^l \\ R_3^l \end{pmatrix}, \qquad \mathbf{R}_b^r = \begin{pmatrix} R_b^r - R_1^r \\ R_1^r - R_2^r \\ R_2^r - R_3^r \\ R_3^r \end{pmatrix}, \qquad \mathbf{F}_{bps}^l = \begin{pmatrix} F_{bb} - F_{pb}^l \\ F_{11} \\ F_{22} \\ F_{33} \end{pmatrix}, \qquad \mathbf{F}_{bps}^r = \begin{pmatrix} F_{bb} - F_{pb}^r \\ F_{11} \\ F_{22} \\ F_{33} \end{pmatrix} \tag{2d}$$

$$\mathbf{U}_b^l = \begin{pmatrix} u_b^l \\ u_1^l \\ u_2^l \\ u_3^l \end{pmatrix}, \qquad \dot{\mathbf{U}}_b^l = \begin{pmatrix} \dot{u}_b^l \\ \dot{u}_1^l \\ \dot{u}_2^l \\ \dot{u}_3^l \end{pmatrix}, \qquad \ddot{\mathbf{U}}_b^l = \begin{pmatrix} \ddot{u}_b^l \\ \ddot{u}_1^l \\ \ddot{u}_2^l \\ \ddot{u}_3^l \end{pmatrix}, \tag{2e}$$

$$\mathbf{U}_b^r = \begin{pmatrix} u_b^r \\ u_1^r \\ u_2^r \\ u_3^r \end{pmatrix}, \qquad \dot{\mathbf{U}}_b^r = \begin{pmatrix} \dot{u}_b^r \\ \dot{u}_1^r \\ \dot{u}_2^r \\ \dot{u}_3^r \end{pmatrix}, \qquad \ddot{\mathbf{U}}_b^r = \begin{pmatrix} \ddot{u}_b^r \\ \ddot{u}_1^r \\ \ddot{u}_2^r \\ \ddot{u}_3^r \end{pmatrix}, \tag{2f}$$

where m_b^l, u_b^l, $\dot{u}_b^l$, $\ddot{u}_b^l$, m_b^r, u_b^r, $\dot{u}_b^r$, $\ddot{u}_b^r$ is the mass, displacement, velocity and acceleration of the base of the left and the right building, respectively, $R_b^l = k_b^l u_b^l$, $R_b^r = k_b^r u_b^r$ are the resisting forces of the isolation systems, k_b, c_b denote stiffness and damping coefficients, F_{bb} is the pounding force between the structural bases and F_{pb}^{l}, F_{pb}^{r} are the pounding forces due to collisions of the retaining wall and the base of the left and the right building, respectively.

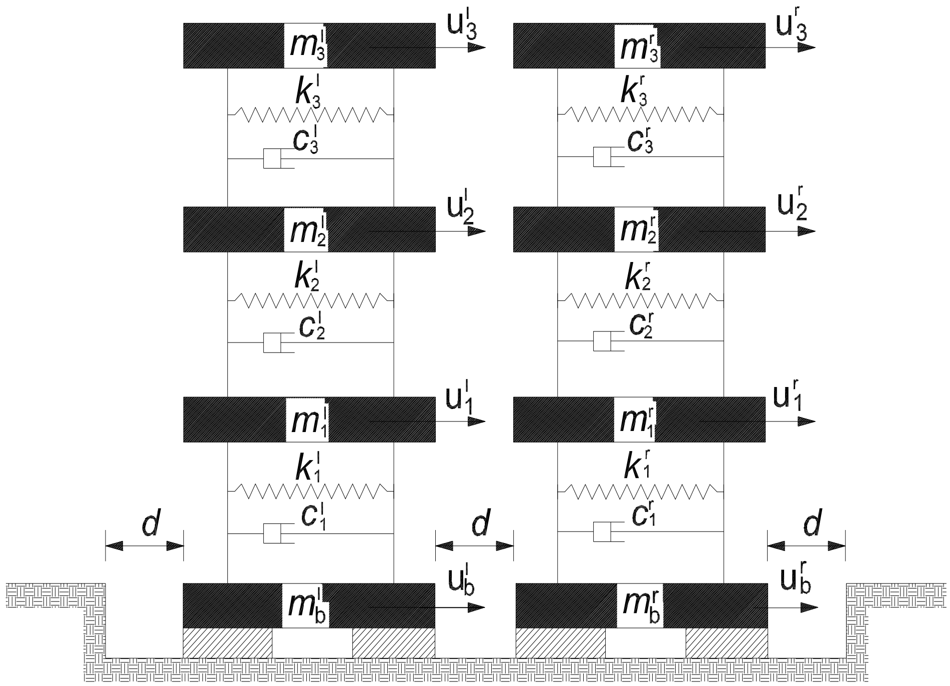

Figure 2. Model of colliding three-storey base-isolated buildings.

Model of Isolation Devices

Seismic isolation of structures provides the mitigation of seismic damage of structures being a reliable and cost-effective method. Among the developed various types of isolation systems, the use of High Damping Rubber Bearings (HDRBs) is one of the most attractive solutions. These bearings have also been used in this study as the isolation device. In order to simulate the behaviour of HDRBs, a nonlinear strain rate dependent model has been applied (see Jankowski 2003). The model describes the behaviour of the bearing by a nonlinear elastic spring-dashpot element, for which stiffness and damping coefficients, k_b, c_b, are obtained at a given time based on the actual values of displacement, u_b, and velocity, $\dot{u}_b$, using formulas (Jankowski 2003)

$$k_b = a_1 + a_2(u_b)^2 + a_3(u_b)^4 + \frac{a_4}{\cosh^2(a_5\dot{u}_b)} + \frac{a_6}{\cosh(a_7\dot{u}_b)\cosh(a_8 u_b)} \tag{3a}$$

$$c_b = \frac{a_9 + a_{10}(u_b)^2}{\sqrt{a_{11}^2 + (\dot{u}_b)^2}} \tag{3b}$$

where a_1 - a_{11} are parameters of the model which are obtained by fitting the experimental data using the method of the least squares.

Model of Structural Pounding

Pounding between adjacent structures is a highly complex phenomenon. Therefore, in order to accurately simulate impact, an appropriate impact force model must be adopted. The nonlinear viscoelastic model (Jankowski 2005) has been employed in the analysis of earthquake-induced pounding between adjacent buildings presented in this paper. According to the nonlinear viscoelastic model, the value of pounding force between the *i*th (*i*=1,2,3) storeys of two adjacent buildings is calculated as (Jankowski 2005)

$$\begin{aligned} F_{ii} &= 0 && \text{for } \delta_{ii} \le 0 && \text{(no contact)} \\ F_{ii} &= \overline{\beta}\delta_{ii}^{\frac{3}{2}} + \overline{c}_{ii}\dot{\delta}_{ii} && \text{for } \delta_{ii} > 0 \text{ and } \dot{\delta}_{ii}(t) > 0 && \text{(contact-approach period)} \\ F_{ii} &= \overline{\beta}\delta_{ii}^{\frac{3}{2}} && \text{for } \delta_{ii} > 0 \text{ and } \dot{\delta}_{ii}(t) \le 0 && \text{(contact-restitution period)} \end{aligned} \quad (4)$$

where $\delta_{ii} = (u_i^l - u_i^r - d)$ is the relative displacement (*d* denotes the initial separation gap), $\overline{\beta}$ is the impact stiffness parameter and $\overline{c}_{ii}$ and is the impact element's damping

$$\overline{c}_{ii} = 2\overline{\xi}\sqrt{\overline{\beta}\sqrt{\delta_{ii}\frac{m_i^l m_i^r}{m_i^l + m_i^r}}}, \qquad \overline{\xi} = \frac{9\sqrt{5}}{2}\frac{1-e^2}{e(e(9\pi - 16) + 16)} \quad (5)$$

Here $\overline{\xi}$ is an impact damping ratio related to a coefficient of restitution, *e* (Jankowski 2006).

Response Analysis

In the study, the seismic response of adjacent three-storey buildings with isolated and non-isolated bases considering pounding effect has been investigated. The feasibility of the study has been demonstrated by a series of numerical simulations. A parametric study has been carried out for a number of cases, including different parameters of the storeys mass, stiffness, damping coefficients and yielding forces. A number of different cases for pounding between isolated and non-isolated building systems has been analyzed for different separation distances between the adjacent buildings. The NS component of the El Centro earthquake of 1940 has been considered in this study.

System Parameters

The dynamic parameters of structures considered in the study are shown in Table 1 (see Jankowski 2008). These parameters make one of the buildings to be more flexible and lighter when compared to the other one, which is stiffer and heavier. In the analysis, the left building has been considered to be a flexible one, whereas the right structure to be a stiff one. The fundamental natural periods of the non-isolated structures have been calculated as equal to 1.2 s and 0.30 s for the flexible and stiff building, respectively.

The base-isolated left building has been equipped with 4 circular HDRBs with the parameters of the bearing's model described in example 3 of the paper by Jankowski (2003). On the other hand, the base-isolated right building has been equipped with 4 square HDRBs with the parameters of the bearing's model described in example 1 of the paper by Jankowski (2003). The following parameters of the nonlinear viscoelastic model of pounding force have also been applied in the analysis: $\overline{\beta}=2.75\cdot 10^9$ N/m$^{3/2}$, $\xi=0.35$ (e=0.65) (see Jankowski 2008).

Table 1 Properties of buildings used in the study

Storey parameter	Flexible building	Stiff building
Mass	$25 \cdot 10^3$ kg	10^6 kg
Damping	5%	5%
Stiffness	$3.46 \cdot 10^6$ N/m	$2.215 \cdot 10^9$ N/m
Yielding force	$1.369 \cdot 10^5$ N	$1.589 \cdot 10^7$ N

Solution procedure

The Newmark method with a constant step size of 0.001s (optimized through the convergence study) has been used to solve the equations of motion. In order to attain high degree of numerical stability, the constant average acceleration approach with $\gamma = 0.5$ and $\beta = 0.25$ has been applied.

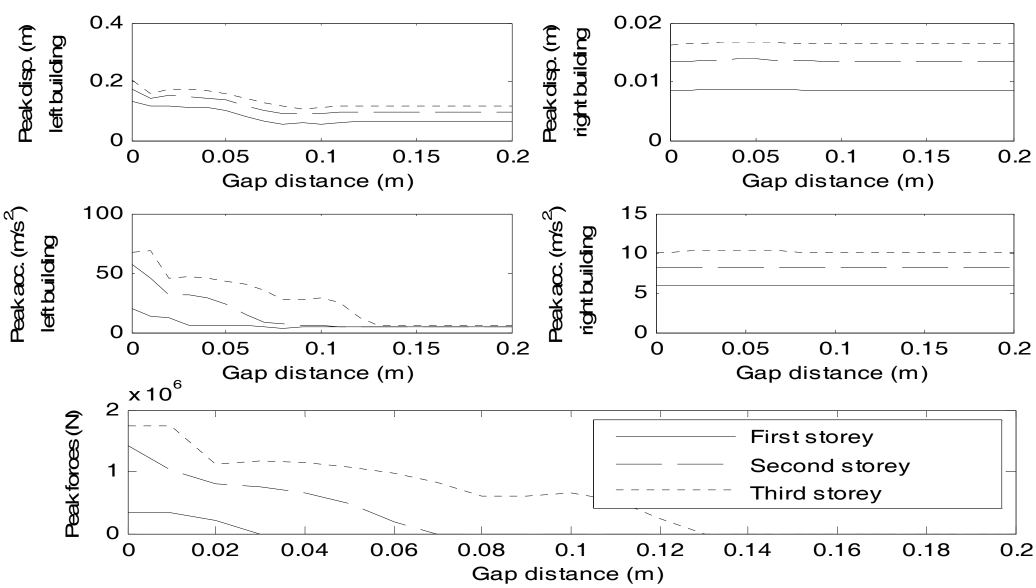

Figure 3. Peak displacements, accelerations and pounding forces with respect to the gap distance between adjacent non-isolated buildings under the El Centro earthquake.

Pounding Between Two Non-isolated Buildings

First, the pounding-involved seismic response of two buildings with non-isolated bases (see Fig. 1) has been investigated. The examples of the results of the analysis in the form of the peak displacements, accelerations and pounding forces with respect to the gap size between structures are presented in Fig. 3. It can be seen from the figure that the peak displacements and accelerations for the right building (stiffer and heavier one) show almost constant values for all gap distances. On the other hand, the results obtained for the left building (more flexible and lighter one) indicate that collisions lead to significant changes in its response. It can be seen from the figure that, the peak displacements and accelerations of the storeys of the left building decrease with the increase in the separation distance. Similar relation concerns also the peak pounding force. It can also be seen from Fig. 3 that the gap size of 0.13 m is required in order to prevent pounding between non-isolated buildings. For such a gap size, the peak structural displacements are equal to 0.11 m and 0.017 m for the left and the right building, respectively.

Pounding Between Two Isolated Buildings

Then, the pounding-involved seismic response of two buildings with isolated bases (see Fig. 2) has been considered. The peak displacements and accelerations of the storeys (with relation to the base), peak displacements and accelerations of the bases as well as peak pounding forces are shown in Fig. 4. It can be seen from the figure that both isolated buildings are considerably influenced by structural pounding. The comparison between Fig. 3 and Fig. 4 indicates that the influence of the base isolation on the structural response is significant. In particular, the values of the peak accelerations and pounding forces are substantially higher comparing to the case of non-isolated buildings. For both buildings, the peak displacements and accelerations for the storeys increase up to a certain value of the gap distance and with further increase in the gap distance a decrease trend can be observed. The impact forces first increase with the increase in the gap distance and then they show a decrease trend after passing a certain gap size value. All the storeys provide almost similar peak displacements. On the other hand, the peak pounding forces show significant differences between higher and lower storey levels. Regarding the left and the right isolated bases, it can be seen from Fig. 4 that the base peak displacements increase with the increase in the gap distances. The curves of the base peak accelerations and peak pounding forces at the bases show the initial increase with further decrease trend as the gap size increases. It can also be seen from Fig. 4 that the gap size of 0.08 m is required in order to prevent pounding between isolated buildings. For such a gap size, the total peak structural displacements (displacements of the storeys plus displacements of the bases) are equal to 0.15 m and 0.08 m for the left and the right building, respectively. Smaller (comparing to the case of non-isolated buildings) minimum gap size preventing pounding is the result of introducing isolation system which led to more in-phase vibrations of buildings.

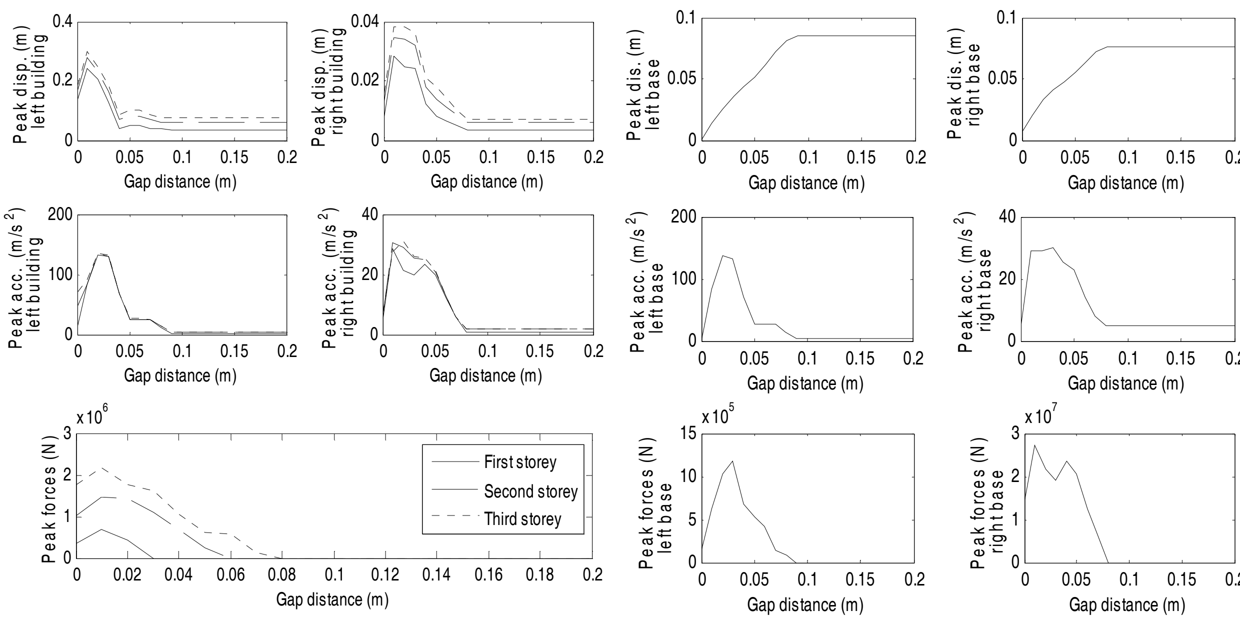

Figure 4. Peak displacements, accelerations and pounding forces with respect to the gap distance between adjacent isolated buildings under the El Centro earthquake.

Final Remarks

The results of the study focused on the seismic response evaluation for isolated and non-isolated buildings considering pounding have been presented in this paper. In the analysis, inelastic three-storey structures with isolated and non-isolated bases and different dynamic parameters of the superstructures have been considered.

The results of the study demonstrate that the variation of the peak storeys accelerations and displacements, considering the effect of pounding, substantially depends on the type of the base. In the case of the non-isolated buildings, the variation of the storeys peak accelerations and displacements is relatively low. On the other hand, for the base-isolated buildings, a significant difference can be expected in the obtained peak storeys accelerations and displacements.

The pounding-involved response of the isolated building with flexible superstructure contain contributions from both isolated base as well as from the superstructure due to higher vibration modes. It has been observed that the flexibility of the superstructure tends to increase the peak storeys accelerations; however, this contribution is relatively low in comparison with the isolated base mode. Thus, the peak storeys accelerations of the base-isolated building with flexible superstructure become higher than the values obtained for the base-isolated building with stiff superstructure.

The results of the study clearly indicate that pounding in the base-isolated buildings may lead to considerable structural damage and therefore it should be avoided at the design stage of structures.

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