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Mehmet Uz
University of Wollongong, meuz@uow.edu.au

Muhammad N. S Hadi
University of Wollongong, mhadi@uow.edu.au

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Investigating the Effect of Pounding for Inelastic Base Isolated Adjacent Buildings under Earthquake Excitations

M.E. Uz & M.N.S. Hadi
School of Civil, Mining and Environmental Engineering, University of Wollongong, Wollongong, NSW, Australia

ABSTRACT: In this study, an investigation is carried out to analyse the earthquake induced pounding between two insufficiently separated base isolated buildings considering the inelastic behaviour of the structures' response. The buildings are modelled as a four-story and a three-story system to simulate the structural behaviour. The resulting systems of second order constant coefficient equations are reformulated as a system of first order ordinary differential equations and solved using the ordinary differential equation solver of MATLAB. Numerical study revealed that pounding behaviour of the buildings has considerable influence on the behaviour of the lighter building causing substantial amplification of the response and leading to considerable permanent deformations due to yield. The parametric investigation has led to the conclusion that the peak displacement of the lighter and more flexible building is very sensitive to the structural parameters by varying gap size, story mass, the friction coefficient for sliding, and structural stiffness, whereas the effect of the heavier base isolated building was negligible.

1 INTRODUCTION

Residential buildings in metropolitan cities which are located in seismically active regions are often built close to each other due to the economics of the land use or architectural reasons. Existing spacing between buildings may become not enough to avoid pounding if either historic restoration or seismic rehabilitation for existing fixed base buildings is done with the use of base isolation systems. Thus there is a need to study the effect of base isolation on pounding of buildings as well as of pounding on these base isolated buildings. The probability distribution of required separation distance of adjacent buildings to avoid seismic pounding were examined by Lin & Weng (2001), Stavroulakis & Abdalla (1991) and Maison & Kasai (1992). Zhang & Xu (2000) studied the response of two adjacent shear buildings connected to each other at each floor level by visco-elastic dampers represented by Voigt's model. Although the study on earthquake-induced structural pounding has been recently much advanced, the above review indicates that very few studies are reported on the behaviour of base-isolated buildings during impact. Pounding between closely-spaced buildings having different dynamic properties was studied by Chau & Wei (2001), Uz & Hadi (2009), Hadi & Uz (2009), Jankowski et al. (1998) and Jankowski (2008). The aim of the present study is to conduct a detailed investigation on pounding-involved response of inelastic two base isolated buildings of unequal heights with using non linear visco-elastic model of collisions.

2 EQUATIONS OF MOTION

The adjacent buildings have been modelled as four and three storey buildings. In order to investigate the behaviour of colliding base isolated buildings, a three dimensional model with the help of each storey's mass lumped on the floor level has been conducted in this study. An elastic-plastic approximation of the storey drift-shear force relation has been fulfilled for the longitudinal (x) and transverse (y) directions, whereas the two buildings are assumed to be in the linear elastic range for the vertical direction (z). The dynamic equation of motion for the two base isolated buildings can be expressed in Equation 1, including the pounding involved responses of base isolated buildings modelled with inelastic systems at each floor level as:

$$\begin{bmatrix} M_1 & 0 & 0 \\ 0 & M_1 & 0 \\ 0 & 0 & M_1 \end{bmatrix}\begin{bmatrix} \ddot{X}(t) \\ \ddot{Y}(t) \\ \ddot{Z}(t) \end{bmatrix} + \begin{bmatrix} C_x & 0 & 0 \\ 0 & C_y & 0 \\ 0 & 0 & C_z \end{bmatrix}\begin{bmatrix} \dot{X}(t) \\ \dot{Y}(t) \\ \dot{Z}(t) \end{bmatrix} + \begin{bmatrix} F_x^s(t) \\ F_y^s(t) \\ F_z^s(t) \end{bmatrix} + \begin{bmatrix} F_x^p(t) \\ F_y^p(t) \\ F_z^p(t) \end{bmatrix} \qquad (1)$$

$$= \begin{bmatrix} F_x(t) \\ F_y(t) \\ F_z(t) \end{bmatrix} - \begin{bmatrix} M_2 & 0 & 0 \\ 0 & M_2 & 0 \\ 0 & 0 & M_2 \end{bmatrix}\begin{bmatrix} \ddot{X}(t) \\ \ddot{Y}(t) \\ \ddot{Z}(t) \end{bmatrix} - \begin{bmatrix} M_3 & 0 & 0 \\ 0 & M_3 & 0 \\ 0 & 0 & M_3 \end{bmatrix}\begin{bmatrix} \ddot{X}_g(t) \\ \ddot{Y}_g(t) \\ \ddot{Z}_g(t) \end{bmatrix}$$

where M_1, M_2 and M_3 are mass matrices of both buildings, whereas damping coefficient matrices of both buildings in the longitudinal (x), transverse (y) and vertical (z) directions are shown as C_x, C_y and C_z respectively. The subscript i= 1, 2, 3, 4, 5, 6, 7 denotes the quantities pertaining to the storeys of Building A and Building B, m_i (i=1,.., 7) can be shown as mass of a single storey of both buildings in mass matrices. Moreover, m_{Bi} (i = 1, 2) denotes the mass of the base of both buildings, respectively. In the study, the pounding force in the longitudinal direction in Equation 2, F_{xij}^{P} (t) (i=1, 2, 3, 4; j=5, 6, 7), has been arranged with the help of nonlinear visco-elastic model according to the formula (Jankowski et al. 1998, Jankowski 2006, Jankowski 2008):

$$\begin{aligned} &F_{xij}^{p}(t)=0 && \delta_{ij}(t)\le 0 \\ &F_{xij}^{p}(t)=\overline{\beta}(\delta_{ij}(t))^{3/2}+\overline{c}_{ij}(t)\dot{\delta}_{ij}(t) && \delta_{ij}(t)>0\ \dot{\delta}_{ij}(t)>0 \\ &F_{xij}^{p}(t)=\overline{\beta}(\delta_{ij}(t))^{3/2} && \delta_{ij}(t)>0\ \dot{\delta}_{ij}(t)\le 0 \end{aligned} \qquad (2)$$

where δ_{ij} (t) and $\dot{\delta}_{ij}(t)$ are the total relative displacement and the total relative velocity between both buildings with respect to the foundation respectively. D is the initial gap between buildings exposed to different ground motion excitations. According to the example of results obtained by Jankowski (2006), $\overline{\beta}=2.75\times10^{9}$ N/m$^{3/2}$ and $\xi=0.35$ (e=0.65) have been applied for the impact stiffness parameter and the damping ratio related to a coefficient of restitution accounts, respectively. The initial gap, D, between the buildings has been taken as 0.02 m. In this study, the time interval Δt is selected as 0.002 sec (Hadi & Uz 2009). The value of the friction coefficient of the sliding bearing is 0.10. The value of the friction coefficient can be calculated by Equation 3.

$$\mu u_i = f_{max} - \Delta f \times e^{-a|\dot{U}|} \qquad (3)$$

where f_{max}, Δf, a, and $\dot{U}$ are the coefficient of friction at large sliding velocity, the differences between f_{max} and the coefficient of friction at low sliding velocity, the constant value, and the sliding velocity, respectively.

3 RESPONSE ANALYSES

3.1 *Numerical Examples*

The dynamic equations derived the most general in Equation 1 for the validation of the numerical models can be conducted to analyse substantially different dynamic properties of adjacent building systems. The numerical results presented in this study are obtained using the MATLAB software. The following basic values describing the structural characteristics in Table 1 have been used in this study. Table 2 shows the properties of Buildings in the longitudinal, transverse and vertical direction, respectively.

Table 1. The structural characteristics of buildings

Building A (Reference Building)							
Storey no	m (kg) (10^3)	k (N/m)			c (kg/sec)		
		x (10^6)	y (10^6)	z (10^{10})	x (10^4)	y (10^4)	z (10^6)
1	25	3.46	3.46	1.246	6.60	6.60	3.96
2	25	3.46	3.46	1.246	6.60	6.60	3.96
3	25	3.46	3.46	1.246	6.60	6.60	3.96
4	25	3.46	3.46	1.246	6.60	6.60	3.96
Building B (Heavier and Stiffer)							
Storey no	m (kg) (10^6)	k (N/m)			c (kg/sec)		
		x (10^9)	y (10^8)	z (10^{11})	x (10^7)	y (10^6)	z (10^8)
1	1.0	2.21	5.53	2.215	1.05	5.28	1.05
2	1.0	2.21	5.53	2.215	1.05	5.28	1.05
3	1.0	2.21	5.53	2.215	1.05	5.28	1.05

Table 2. Properties of Buildings in the longitudinal, transverse and vertical direction

Properties	Building A			Building B		
	x	y	z	x	y	z
First mode time period (sec)	1.54	1.54	0.026	0.3	0.6	0.03
Second mode time period (sec)	0.53	0.53	0.01	0.1	0.2	0.01
First frequency (mod/sec)	4.08	4.08	245	21	10	209
Second frequency (mod/sec)	11.7	11.7	706	59	29	586

The Elcentro (18.05.1940) and the Duzce (12.11.1999) earthquake records have been conducted in this study as the input with the N-S, E-W, and U-D components of the ground motion in the longitudinal, transverse, and vertical directions, respectively (see Table 3).

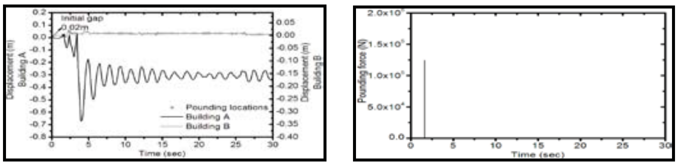
Table 3. Earthquake records used in this study

Earthquake	MW	Station	PGA (g) (N-S, E-W, U-D)	Duration (sec)
Elcentro, 1940	7.0	117 El Centro	0.31,0.22,0.2	39.9
Duzce, 1999	7.1	375 Lamont	0.97,0.51,0.2	41.5

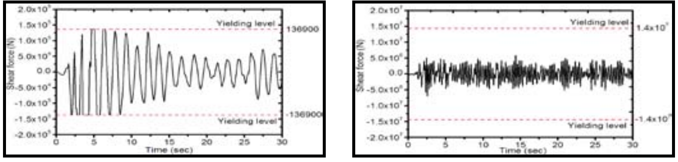
The equation of motion has been derived and solved using step by step solution by the fourth-order Runge-Kutta method with impact and without impact.

3.2 *Results of Response Analysis*

The results of the analysis in the longitudinal and the transverse directions including the displacement, pounding force, and shear force time histories are shown in Figures 1-3 for the all story levels of the buildings, respectively. Additionally, a comparison between pounding-involved and independent vibration (providing large separation distance to avoid contacts) displacement responses of the third story levels of the buildings is shown in Figure 4. It can be seen in Figures 1-2a that after the first contact, Building A which is lighter and more flexible building than Building B recoiled so significantly that it entered into the yield level at the all story levels (see Figs 1-3c).



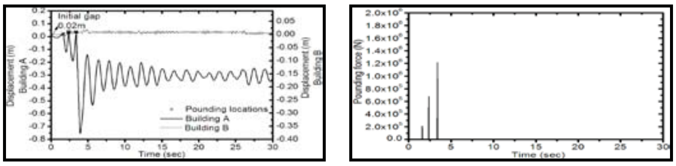
a) Displacement history b) Pounding force history



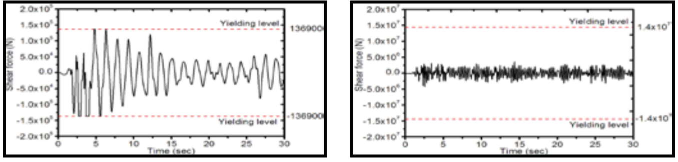
c) Shear force history in Building A d) Shear force history in Building B

Figure 1. Time histories in the longitudinal direction for the second story levels of buildings

Due to the fact that Building B kept small displacements, shear forces of Building B stayed in the elastic range. In Figures 1-3d, shear forces in Building B are mainly in the effect of intensive ground motion. Figures 1-2b indicate that both buildings come into contact three times during the earthquake, although the three collisions took place only at the third story level. As there is no contact in the first story levels of the buildings, the pounding force is zero. Hence, the first story level is not shown here.



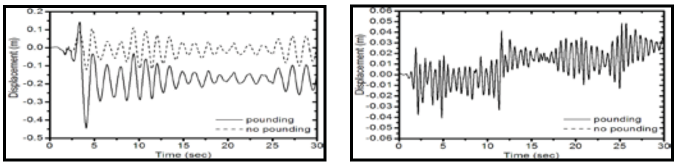
a) Displacement history b) Pounding force history



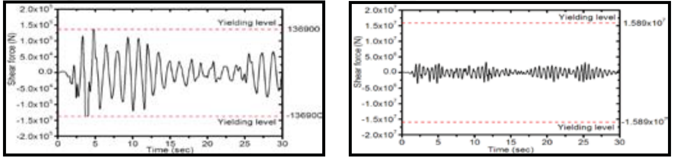
c) Story shear force history in Building A d) Story shear force history in Building B

Figure 2. Time histories in the longitudinal direction for the third story levels of buildings

The results shown in Figures 1-3b indicate that the most critical one for pounding problem is the highest contact point of buildings close to each other (at the third story level) in view of the fact that contacts causing the maximum pounding force took place three times during the earthquake at this point (see Fig. 2b).



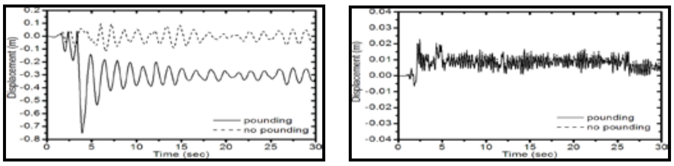
a) Displacement history b) Displacement history



c) Shear force history in Building A d) Shear force history in Building B

Figure 3. Time histories in the transverse direction for the third story levels of buildings

As can be seen from Figure 1b, the effect of contacts at the lower story can be neglected by simplifying the numerical model defined in Eq. 2, considering collisions only at the upper story.



a) Building A b) Building B

Figure 4. Pounding-involved and independent vibration displacement time histories of the third story levels of buildings in the longitudinal direction

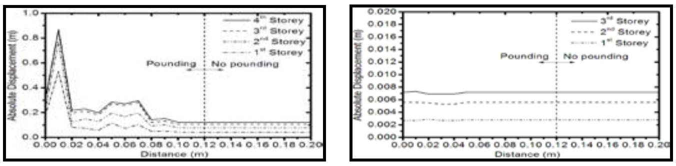
Entering into the yield range at all floors finally resulted in a substantial permanent deformation of the structure as can be seen in Figure 4a. On the other hand, Building B (the heavier and the stiffer one) does not change any considerable level in the response of the earthquake induced pounding between the structures (see Fig. 4b).

4 PARAMETRIC STUDY

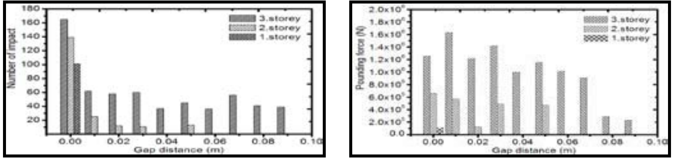
In this study, a parametric study has also been conducted in order to determine the influence of different structural parameters on pounding response of buildings. For various values of gap distance between buildings, story mass, structural stiffness, and friction coefficient of base isolation the numerical analysis has been carried out. When the effect of one parameter has been investigated, the values of others have been kept unchanged. For the parametric analysis, the Duzce 1999 earthquake is used in this study.

4.1 *Effect of Gap Size between Buildings*

The gap distance is one of the important parameters, which describes the influence on the pounding response of neighbouring buildings. In Figures 5, 9, the peak absolute displacements of colliding buildings with the different values of this parameter are shown in the longitudinal and transverse, respectively.



a) Building A b) Building B



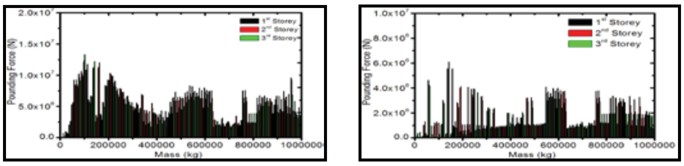
c) Total number of impacts Gap distance d) Maximum pounding force Gap distance

Figure 5. Variation of peak displacement, the number of impacts and pounding force in the longitudinal direction in terms of the width of the gap between buildings

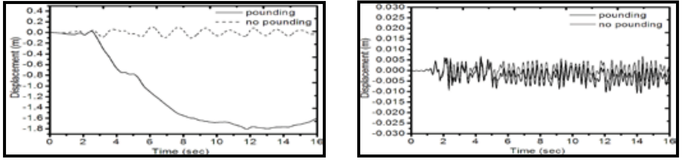
On the other hand, the peak displacements of the response in the vertical direction are similar to transverse direction in almost all the ranges of the gap distance, mass, stiffness, and the friction coefficient. Hence, they are not shown in this study. It can be seen in Figures 5a, c, 9a that the three dimensional response of Building A is very responsive to the gap size value. In the case of the longitudinal and transverse directions, an increase in the gap distance is associated with a reduction in the absolute displacement, although the peak displacement increases significantly in the lowest gap size values. As the gap size increases up to around 0.01 m, the absolute displacement also reaches the peak values. As can be observed from Figures 5b, 9b, there are no differences in the lowest gap size values. According to the results of the parametric studies in this study, a gap size of 0.12 m is required in order to prevent the pounding between the analysed buildings under the Duzce 1999 ground motion. Here, it should be underlined that the minimum required distance between neighbouring buildings depends on both the dynamic characteristics of colliding buildings and the intensity of ground motion.

4.2 *Effect of story mass*

The story mass is a vital structural parameter of the colliding buildings, which has an effect directly on the pounding response of buildings during impact. The pounding response and the independent vibration displacement of the third story of Building A in the longitudinal direction is shown in Figures 6c, d with the story mass $m_i=1.4\times10^5$ kg corresponding to the peak pounding force in Figure 6a. The results of the parametric study illustrate that the response of Building A is affected significantly by changing the considered parameter.



a) Longitudinal direction b) Transverse direction



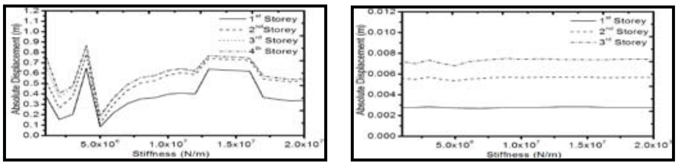
c) Building A d) Building B

Figure 6. a-b) The peak pounding force and story mass in the longitudinal and transverse directions, c-d) Pounding-involved and independent vibration displacement time histories of the third story levels of buildings in the longitudinal direction for $m_i=1.4\times10^5$ kg (i=1, 2, 3, 4)

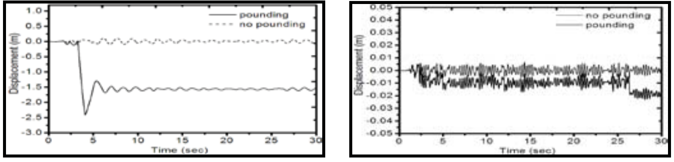
As can be observed in Figure 6a, it reaches the high value of pounding forces for the story mass up to about $m_i=2.0\times10^5$ kg. Then, it falls down and follows a steadily increasing slope. The pounding result in a significant change in the structural behaviour including entering into the yield level is clearly shown with providing the comparison between the pounding response and the independent vibration displacement of Building A in the longitudinal direction in Figure 6c. The pounding responses and the independent vibration displacements of Building B are considerably different during only a short period after one of the collisions in Figure 6d.

4.3 *Effect of structural stiffness*

One of the important dynamic properties of the buildings is the structural stiffness. Structural stiffness values are conducted in this study. Results of the parametric study are shown in Figure 7 in the longitudinal.



a) Building A b) Building B



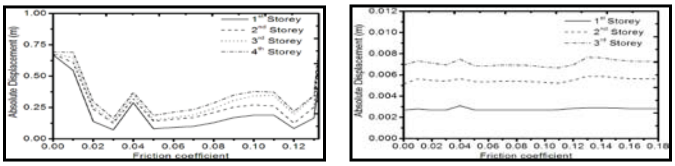
c) Building A d) Building B

Figure 7. a-b) Peak Displacements with respect to story stiffness, k_{xi} (i=1, 2, 3, 4), c-d) Pounding-involved and independent vibration displacement time histories of the third story levels of buildings in the longitudinal direction for $k_{xi}=3.4\times10^6$ N/m

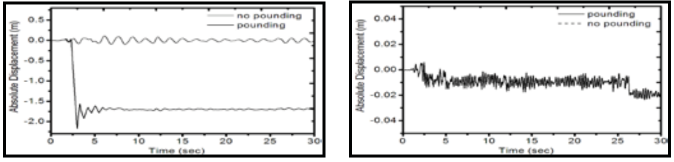
Moreover, the independent vibration displacement and pounding response of the third story of the buildings are also illustrated in the longitudinal direction in Figures 7c, d for the structural stiffness k_i= 3.4×10^6 N/m corresponding to the peak displacement in Figure 7a. It can be seen from Figure 7a, the plots of the peak displacements differ greatly for Building A. In case of the longitudinal direction, the peaks have high values in the vicinity of k_{xi}=3.4×10^6 N/m and k_{xi}=1.5×10^7 N/m. In a comparison between pounding response and the independent vibration displacement of the third story levels of the buildings, Figures 7c, d indicate that pounding has a vital influence only on the behaviour of both buildings in the longitudinal direction.

4.4 *Effect of friction coefficient*

The results of the parametric studies carried out for the different values of the sliding coefficient of friction are illustrated in the three directions in Figure 8. Furthermore, a friction coefficient, mu_a=0.01, corresponding to the peak displacement in Figure 8a in a plot of the compression between the pounding-involved response and the independent vibration displacement is used in order to understand the effect of pounding on the behaviour of the buildings. It can be seen from Figures 8c, 9a that the pounding – involved results of Building A have two ranges of a considered increase in the longitudinal and transverse directions till the parameter considered up to vicinity of mu_a=0.13. The first one is around mu_a=0.01, while the second one can be observed in the vicinity of mu_a=0.13 in both directions.



a) Building A b) Building B



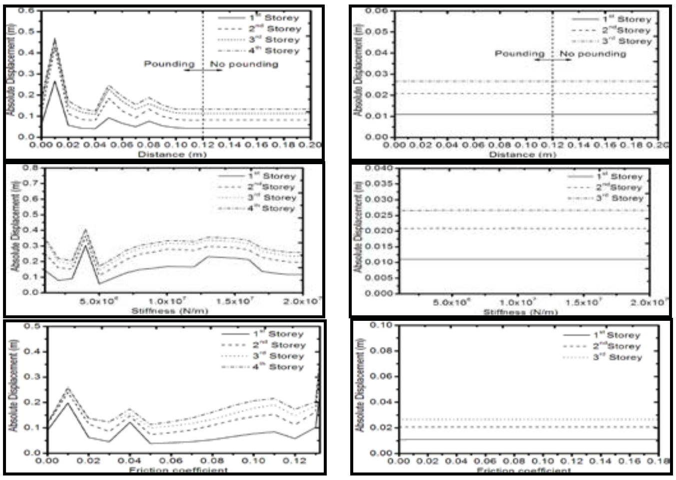
c) Building A d) Building B

Figure 8. a-b) Peak Displacements with respect to friction coefficient, mu_a, c-d) Pounding-involved and independent vibration displacement time histories of the third story levels of buildings in the longitudinal direction for mu_a=0.01

Moreover, Building B is unaffected by changing the friction coefficient ranges especially in the high friction values. It can be seen in Figure 8c that Building A enters into the yield level, even though Building B is nearly identical for the considered friction coefficient value as shown in Figure 8d.

4.5 *Effect of parametric values in the transverse direction*

In Figure 9, the peak absolute displacements of colliding buildings with the different values of these related parameters are shown in the transverse direction.



a) Building A b) Building B

Figure 9. Variation of peak displacement in the transverse direction in terms of the width of the gap, stiffness, and friction coefficient between buildings, respectively

In the transverse direction, Figure 9a indicates that the peak displacement also increases substantially between the two ranges of structural stiffness, although the response for the other values of story stiffness is quite similar.

5 CONCLUSIONS

In this study, non-linear analysis has been carried out for the earthquake-induced pounding of unequal height buildings having significantly different dynamic properties. For non-linear analysis, inelastic multi degree of freedom lumped mass systems have been modelled for the structures and the nonlinear visco-elastic model for impact force during collisions have been incorporated on the three dimensional pounding between two adjacent four and three story buildings. The results of the parametric investigation carried out with changing the values of structural parameters have also been presented.

According to the results of the response analysis in this study demonstrate that pounding of the structures during ground motion excitation has a significant influence on the behaviour of the lighter building in the longitudinal direction. This pounding may lead to substantial amplification of the response, which may finally cause a considerable permanent deformation of the structure because of the yield level. In contrast, the results of the response analysis show that the behaviour of the heavier building in the longitudinal, transverse, and vertical directions is practically unchanged by pounding of structures.

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