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Seismic pounding effects on adjacent buildings in series with different alignment configurations

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Abstract. Numerous urban seismic vulnerability studies have recognized pounding as one of the main risks due to the restricted separation distance between neighboring structures. The pounding effects on the adjacent buildings could extend from slight non-structural to serious structural damage that could even head to a total collapse of buildings. Therefore, an assessment of the seismic pounding hazard to the adjacent buildings is superficial in future building code calibrations. Thus, this study targets are to draw useful recommendations and set up guidelines for potential pounding damage evaluation for code calibration through a numerical simulation approach for the evaluation of the pounding risks on adjacent buildings. A numerical simulation is formulated to estimate the seismic pounding effects on the seismic response demands of adjacent buildings for different design parameters that include: number of stories, separation distances; alignment configurations, and then compared with nominal model without pounding. Based on the obtained results, it has been concluded that the severity of the pounding effects depends on the dynamic characteristics of the adjacent buildings and the input excitation characteristics, and whether the building is exposed to one or two-sided impacts. Seismic pounding among adjacent buildings produces greater acceleration and shear force response demands at different story levels compared to the no pounding case response demands.

Keywords: adjacent buildings in series; seismic pounding; time history analysis; separation gap; response demands; alignment configurations

1. Introduction

Several urban seismic vulnerability inspections after several major earthquakes have recognized pounding as one of the main hazards to buildings and bridges (Bertero 1987, Kasai and Maison 1991, Abdel Raheem 2006, 2009, Kawashima *et al*. 2011, Cole *et al*. 2012, Abdel Raheem and Hayashikawa 2013, Inel *et al*. 2013). Although numerous recent codes require a minimum seismic separation gap, it is as yet insufficient as codes essentially lag behind the recent research (ICBO 1997, IS 2002, ECS 2004, ICC 2009, ASCE 2010). Pounding damage was inspected during the 1944 Elcentro earthquake, the 1985 Mexico earthquake, the 1988 Sequenay earthquake, the 1992 Cairo earthquake, the 1994 Northridge earthquake, and the 1995 Kobe earthquake. In the Mexico City catastrophic earthquake, around 40% of the damaged structures faced certain level of pounding and structural collapse for 15% of them are observed (Rosenblueth 1986, Anagnostopoulos 1996) In the 1989 Loma Prieta earthquake, more than 200 pounding incidents through over 500 buildings were revealed at sites over 90 km from the picentre (Kasai and Maison 1997), thus endorsing the potential disastrous damages in the future earthquakes. Pounding among adjacent buildings in series could have more awful destruction as nearby structures have out of phase vibration characteristics and insufficient separation gap or mitigation measure of energy dissipation system to accommodate the relative deformations of adjacent buildings. Examination of structural pounding damage during recent earthquakes (Kawashima and Unjoh 1996, Naserkhaki *et al*. 2013, Abdel Raheem 2013, 2014) has identified buildings of several configuration categories that are susceptible to pounding damage: equal story height pounding; non-equal story height (mid-column) pounding; heavier adjacent buildings pounding; eccentric pounding and buildings in series.

The buildings in many highly-congested municipal cities constitute a foremost concern for seismic pounding

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damage. The pounding among adjacent structures in series during earthquakes causes a repeated hammer that is exerted on each other, hence could lead to damages that ranges from slight non-structural local damage to serious structural global damage that could prompt buildings total failure. The damage due to end building pounding of in-row adjacent buildings is a standout amongst the most widely recognized vulnerabilities as urban areas are brimming with rows of slightly separated or in contact buildings (Jeng and Tzeng 2000). So, the seismic pounding of adjacent buildings has been thoroughly investigated by using several structural and impact models (Anagnostopoulos and Spiliopoulos 1992, Davis 1992, Jankowski 2006, Mahmoud and Jankowski 2011, Abdel Raheem 2014). Anagnostopoulos (1988) investigated the pounding among adjacent buildings in series using idealized single degree of freedom (SDOF) systems and linear viscoelastic impact model, it was concluded that the seismic response depends mainly on the vibration characteristics of the adjacent structures; relative in-row position of the structure whether exterior or interior structure, input excitation frequency content and the separation gap size. The exterior buildings are much more severely penalized than the interior buildings, the response of interior building was observed to be increased or decreased relying upon whether it has a smaller or higher fundamental period than the adjacent structures; stiffer structures usually display an amplified response, while the flexible structures encountering a response reduction. The stiffer structure within the row got less magnification than their external location. Athanassiadou *et al*. (1994) did comparable reproductions on the ground motion phase shift effect; it is observed that the stiffer structure, irrespective of its row relative position, undergone the most response magnification. Anagnostopoulos and Spiliopoulos (1992) concluded based on numerical simulation of three buildings that occasionally pounding generated higher response amplification for external building position than for internal building. In contrast, damage assessment analysis in Christchurch 2011 earthquake displayed various situations where the interior structures of the row were seriously damaged, while the exterior structures of the same row endured (Cole *et al*. 2011). A shake table examination on pounding interaction among buildings in series (Khatiwada and Chouw 2013) has recognized that an external building a row of buildings is extremely vulnerable to pounding damage, while interior buildings could be safer. Therefore, it is required to evaluate the seismic pounding effect on buildings response demands to promote an improved damage control and more competent utilization of land. Despite the extensive research carried out on the seismic collision of buildings during the last two decades, which has been mainly reported earlier, the findings of many works have been refuted by other pertinent studies. According to Cole *et al*. (2010), this discrepancy has to do with the high level of complexity inherent in the problem.

This study focuses on the seismic pounding effects on the seismic response demands among adjacent buildings in series with equal story heights, where the pounding predominantly affects the global response demands. A numerical modelling based on a finite element method is developing for the simulation of the pounding among adjacent buildings in series. The seismic response of adjacent buildings in series is investigated for different design parameters that include: number of stories, separation distances; alignment configurations, and then compared with that of the model without pounding. The global performance is examined through the maximum responses for the story displacement, acceleration and story shear seismic demands. Moreover, the responses for selected input excitation are presented to discuss the effect of the input excitation characteristics.

2. Numerical simulation for seismic response analysis

2.1 Nonlinear dynamic analysis procedures

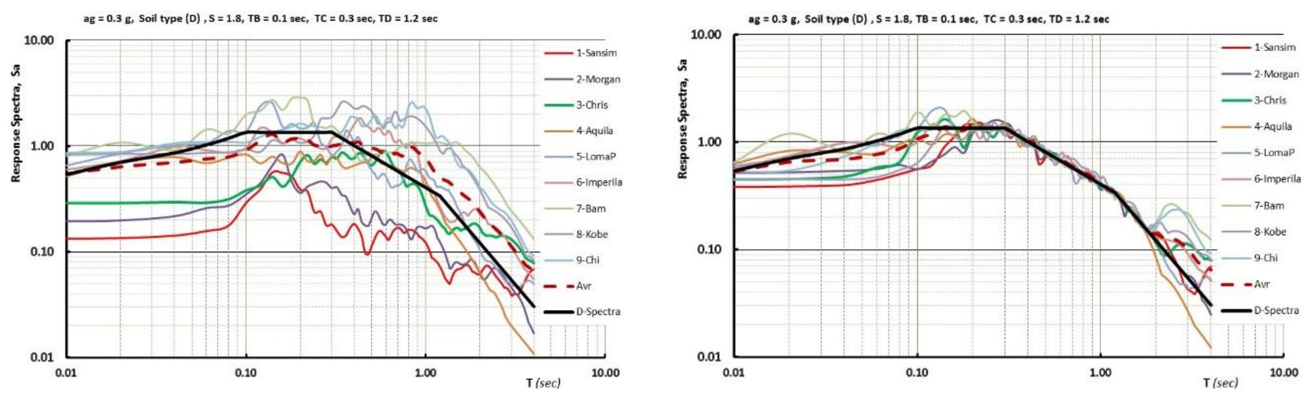
The finite element software package ETABS (CSI 2016) is used for nonlinear dynamic analysis of the three-dimensional structural model of adjacent buildings, where the geometric and material nonlinearities are considered during structural FE modeling and analysis. The equilibrium equations for nonlinear static and nonlinear time history analysis take into account the deformed configuration of the structure. The material nonlinearity could be captured with the inelastic behavior in the form of a nonlinear force-deformation relation, which affords insight into ductility and limit-state behavior. Yielding and post-yielding behavior are modeled using plastic hinge, hinge properties can be calculated automatically based on element material and section properties according to FEMA-356 (ASCE 2000) or ACSE 41-13 criteria. The fibre P-M2-M3 hinge simulates the axial behavior of a number of axial fibres distributed across the frame element cross section. Each fibre has a location, a tributary area, and a stress-strain curve. The axial stresses are integrated over the section to calculate the values of P, M2 and M3.

2.2 Input motions and scaling procedure

For the nonlinear dynamic analysis of the seismic pounding among adjacent buildings in series, a set of nine-ground motion time histories is chosen for grasping the input excitation effect. The input excitation in the form of acceleration time histories is required to be well-matched with the design response spectra at the target site. A time domain scaling method is used to scale the selected real ground motion records (PEER 2013) to match the proposed elastic design spectrum (ECP 2008) using SeismoMatch software (Abrahamson 2006). The real and matched ground motion spectra are plotted against design response spectrum as shown in Fig. 1. For the response-history analysis, the key parameters as indicator of the damage potential of the earthquake excitation are calculated for real and matched ground motion records and presented in Table 1. The ground motions are scaled in accordance with the wldkprovisions of seismic codes (NEHRP 2003) such that for each period between $0.2T_1$ and $1.5T_1$, the average of the 5% damped response spectra for the suit of ground motion

Table 1 key parameters of real and matched nine-ground motion records

Earthquake / Station	Mw	Spectra match	PGA (g)	PGV (m/sec)	PGD (cm)	Specific energy density (cm^2/sec)	Arias intensity (cm/sec)	Housner intensity (cm)	Period Ts (sec)
San Simeon, CA. / RSN3994_36153090	6.52	Real	0.13	13.10	7.72	497.3	0.21	40.29	0.13
		Match	0.38	33.80	7.93	1133.6	1.27	123.0	0.38
Morgan Hill, USA / RSN457_ G03000	6.19	Real	0.19	11.02	3.12	500.3	0.34	48.6	0.19
		Match	0.52	26.20	4.32	856.3	1.42	118.8	0.48
Christchurch, NZ / RSN8124_RHSCN86W	6.20	Real	0.29	33.52	16.99	1479.6	1.13	100.56	0.29
		Match	0.45	37.20	18.14	1667.6	1.82	140.41	0.45
L'Aquila, Italy / RSN4481_FA030YLN	6.30	Real	0.52	35.91	4.47	565.9	1.37	91.60	0.51
		Match	0.63	43.62	3.68	714.3	2.08	111.69	0.62
Loma Prieta, USA / RSN811_ WAH090	6.93	Real	0.65	38.12	5.91	1487.2	6.27	128.35	0.64
		Match	0.60	35.35	5.69	1299.5	5.90	119.80	0.58
Imperial Valley, USA / RSN160_H-BCR140	6.53	Real	0.60	46.75	20.22	2655.4	3.97	174.64	0.59
		Match	0.57	40.62	15.79	1721.5	2.92	126.76	0.56
Bam, Iran / RSN4040_BAM-L	6.60	Real	0.81	124.12	33.94	7989.2	7.83	389.31	0.79
		Match	0.65	64.89	20.07	2802.4	4.80	147.44	0.64
Kobe, Japan / RSN1106_KJM000	6.90	Real	0.83	91.11	21.11	7581.8	8.38	363.11	0.82
		Match	0.45	36.94	12.33	2054.2	2.29	139.71	0.44
Chi-Chi, Taiwan / RSN1231 _CHY080-N	7.62	Real	0.86	93.16	41.66	10247.2	6.41	395.38	0.85
		Match	0.53	37.16	39.54	3786.0	1.88	141.20	0.52



(a) Real ground motion records (b) Matched ground motion records

Fig. 1 Response spectra of the various earthquakes along with the design response spectrum (ECP 2008)

is not less than the corresponding ordinate of the target response spectrum. Where T_1 is the fundamental period of the structure system. According to ASCE7-10 section 17.6.3.4 (ASCE 2010): the average value of the measured response parameter of interest is permitted to be used for design, if seven or more pairs of ground motions are used for the response-history analysis, if fewer than seven pairs of ground motions are used, the maximum value of the response parameter of interest shall be used for design.

2.3 Building structural and seismic design

The building construction industry in Egypt had broadly used medium-rise RC buildings having twelve stories, the height limit authorized by the local authorities in most regions. These buildings are constructed with diverse patterns and structural systems. Three models for typical buildings with three, six and twelve stories are selected as shown in Fig. 2. The buildings have story height 3 *m* for all floors and bay width 5 m in both directions. Concrete with compressive strength $f_c = 30$ MPa, unit weight $\gamma_c = 25\ \mathrm{kN/m^3}$, modulus of elasticity $E_c = 24$ GPa, Poisson's ratio $\upsilon = 0.2$ and reinforcing steel with yield strength $f_y = 360$ MPa are used for analysis and design. The design process requires the determination of the loads that act on the RC buildings. The gravity loads include dead loads and live loads; and lateral loads include earthquake loads. The dead loads take account of the own weight of the structural components, the weight of flooring cover (1.5 kN/m^2) and panel wall loads intensity of 10 kN/m′ on all beams. A live

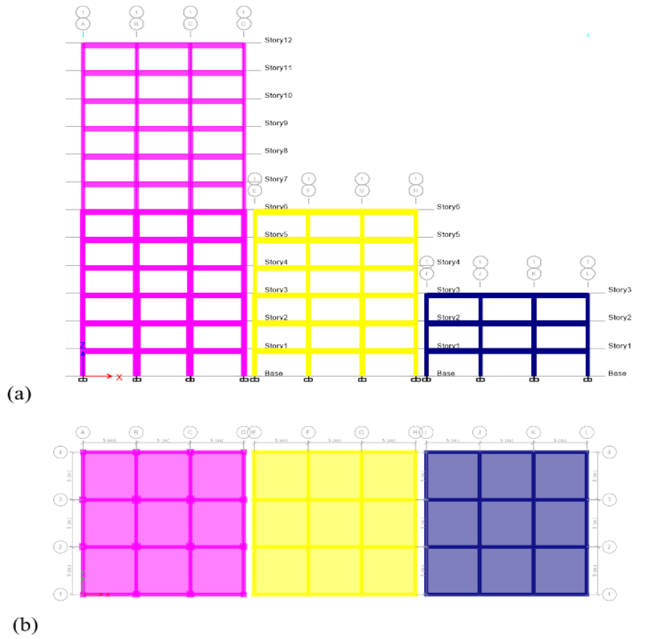


Fig. 2 Three-, six- and twelve- story buildings: (a) Elevation, (b) Typical floor plan

load of 2 kN/m^2 is selected for the residential buildings.

The seismic design of the studied buildings has been done according to ECP-201 (ECP 2008), with design parameters of: importance factor γ = 1; earthquake zone (5B) based on Egyptian zoning system; peak ground acceleration PGA = 0.3 g; Type 1 design response spectrum; soil class (D) and soil factor S = 1.8. The reduction factor, *R* = 5, is selected for MRF buildings. All structural elements of the buildings are designed, where the floor has slab-beam system with 0.15 m slab thickness and 0.3 × 0.7 m dropped beam. The dimensions and reinforcement of column elements for the studied buildings are shown in Table 2. The core philosophy of the Egyptian Seismic Design Code provisions (ECP 2007, Abdel Raheem *et al*. 2018) with respect to the design requirements for RC buildings is in full compliance with the related EC8 (ECS 2004). The code provisions for seismic resistant design of structures allow for serviceability limit state through a damage limitation under normal service conditions, whilst ultimate limit state provisions are included. The capacity design rules are adopted, where the brittle failure or other harmful failure mechanisms (plastic hinges in columns, shear failure of structural elements, failure of beam-column joints, yielding of foundations) shall be prohibited, through the definition of the design actions in selected regions from equilibrium conditions. For the MRF structural systems, the capacity design condition should be fulfilled at all beam-column joints

$$\sum M_{RC} \geq 1.3 \sum M_{Rb} \tag{1}$$

2.4 Building finite element modelling

The seismic pounding among three aligned adjacent RC-MRF buildings with 3-, 6-, 12-stories during seismic events is investigated. A three-dimensional (3D) finite element (FE) model has been developed as shown in Fig. 2 and 3D nonlinear time-history analyses have been performed. 3D FE models of the studied buildings are adopted to consider the significance of the accidental torsion requirement in Section 12.8.4.2 of ASCE 7-10 for buildings. The accidental torsion provisions require application of +/-5% offset of the Centre of mass in each of two orthogonal directions to compute a torsional moment, thus increasing the base shear seismic design demands. The finite element software ETABS (CSI 2016) has been used to perform the dynamic analysis utilizing a set of nine-ground motion records to excite the buildings model. Rayleigh damping is adopted, the coefficients multiplying the mass and stiffness matrices are calculated based on carefully selected frequencies of the studied buildings. The mass and stiffness coefficients $\alpha = 0.3603$ and $\beta = 0.0025$ are determined from specified damping ratio $\zeta = 5\%$ and natural frequencies of first vibration mode of 12-story building and second vibration of the 3-story building ($\omega_i = 4.01$ and $\omega_j = 35.30$). The total seismic mass is calculated as dead load plus an additional 25% of live load based on the ASCE7-10 (2010) in Section 12.7.2 for the effective seismic weight of the building used for seismic based shear calculation "*a minimum of 25 percent of the floor live load shall be included*". The practice on buildings subjected to earthquakes shows that masonry infill walls completely modify the behavior of bare frames due to increased initial stiffness and low deformability. It is difficult to predict the masonry infill effect on the frames members; as different failure modes can occur either in the masonry or in the surrounding frame. Thus, due to several uncertainties regarding the infill layout as non-structural elements, openings through infill wall, complications in modeling infill wall-frame interaction, the infill effects are hard to be quantified and usually ignored in structural design.

2.5 Structural impact model

To simulate pounding between adjacent buildings, the gap between the buildings is modelled by using compression only gap element as shown in Fig. 3. A linear damper is introduced to overcome the drawback of the linear viscoelastic model and simulate the energy dissipation (Komodromos *et al*. 2007, Polycarpou and Komodromos 2010, Jankowski 2010). The pounding force of impact model F_I is determined as

$$F_I = \begin{cases} k_G \delta + c\dot{\delta} & \delta \geq G \\ 0 & \delta < G \end{cases} \qquad \delta = u_i - u_j - G, \qquad \dot{\delta} = \dot{u}_i - \dot{u}_j \tag{2}$$

Where δ and $\dot{\delta}$ are the relative displacement and velocity between colliding structural elements. k and c are the stiffness and damping for the gap element, respectively. u_i, u_j and $\dot{u}_i$, $\dot{u}_j$ are the displacement and velocity of the element's nodes *i*, *j* and *G* is the separation gap.

Numerous researches have been investigated the different possibilities for determination of the gap element stiffness. Watanabe and Kawashima (2004) have performed

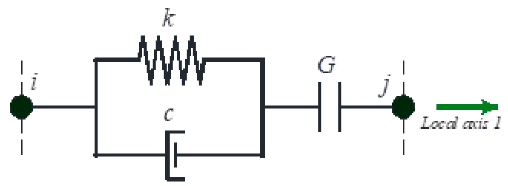


Fig. 3 Viscoelastic impact model

a numerical simulation to lighten the suitable stiffness of impact spring and the time interval of numerical integration based on the wave propagation theory, it concluded that the impact stiffness can be defined as the axial stiffness of the contact bodies, a gap element with stiffness equal to the axial stiffness of floor at the impact level is integrated (Wada *et al*. 1984, Maison and Kasai 1992). Anagnostopoulos (1988) proposed gap element with twenty times amplification factor multiplied with the lateral stiffness of the stiff SDOF system. In current study, the impact stiffness of the gap element K is determined as the greater value of either the axial stiffness of the collided floors or the lateral stiffness of the stiffer building at the impact level (Kawashima and Shoji 2000, Unjoh *et al*. 2003, Abdel Raheem 2009, Guo *et al*. 2012, Shrestha *et al*. 2013).

$$K = \gamma \frac{EA}{b} \quad \text{or} \quad \gamma \frac{3EI}{h^3} \tag{3}$$

direction, I is the moment of inertia of equivalent cantilever Where, A is the area of the impact surface, E is the modulus of elasticity, and b is building width in the impact model of the stiffer building, h is the height building up to the impact level. A sensitivity analysis is done for the selection of the value of impact stiffness; on which the stiffness amplification factor is determined, $\gamma = 50$. Energy dissipation during contact is accounted through damping constant c.

3. Code-specified separation to avoid pounding

The minimum code-specified separation of adjacent buildings necessitates that all structures be detached from neighboring structures. Separations should take into consideration the maximum inelastic displacement response δ_M, where $\delta_M = 0.7R\Delta_s$ in which R is the force modification factor that represents the inherent over-strength and global ductility capacity of the lateral load resisting system, and Δ_s is design inter-story drift resulting from the design seismic forces. Seismic codes provisions and design regulations worldwide state minimum separation distances to be implemented among adjacent buildings, to prevent pounding, which is clearly equal to the relative displacement demand of the two conceivably colliding structural systems (UBC 1997, ICC 2003, Garcia 2004, Abdel Raheem 2006). The minimum separation distance could be given by either ABSolute sum (ABS) or Square Root of Sum of Squares (SRSS) or Double Difference

Table 3 Minimum required separation distance between two adjacent structures

Design code	Code provisions for separation distance between adjacent buildings
NBCC (NRCC 2005)	$\delta_M = \delta_{M1} + \delta_{M2}$ the ABsolute Sum (ABS) of the peak displacements of two adjacent buildings. δ_M is the separation distance between two structures. δ_{M1} and δ_{M2} are the peak lateral displacement of the adjacent building 1 and 2.
UBC97 (ICBO 1997) IBC (2003)	$\delta_M = \sqrt{{\delta_{M1}}^2 + {\delta_{M2}}^2}$ the square root of sum of squares (SRSS), δ_{M1}andδ_{M2} = Peak displacement response of adjacent structures. The adjacent buildings located on the same property line (UBC97: Clause 1633.2.11, IBC: Clause 1620.4.5).
EC8 (ECS 2004)	The quadratic combination of the maximum peak displacements of adjacent buildings.
IS1893 (IS 2007)	R times the sum of the calculated story displacements using design seismic forces to avoid damage of the two structures when the two units deflect towards each other. When the two buildings are at the same elevation levels, the factor R may be replaced by R/2. (Clause 7.12.3), R is the response reduction factor.
IBC (2009) ASCE (2010)	$\delta_M = \frac{C_d}{I}\delta_{\max}$: where C_d is the deflection amplification factor, which depends on the seismic force resisting system, $\delta_{\max}$ represents the peak relative displacement between adjacent buildings calculated from the elastic analysis and I is the importance factor determined
FEMA273 (FEMA 1997)	Separation distance between adjacent structures shall be less than 4% of the building height and above to avoid pounding. (Clause 2.7.4, Clause 2.11.10)
NBC (2003)	This minimum distance not be lower than 2/3 of the sum of the maximum displacement of adjacent blocks nor lower than Separation distance (cm) = 3 + 0.004 (h-500). (Clause3.8.2), h is the height of structure (cm).
ECP201 (ECP 2008)	Each building separated from its neighbor shall have a minimum clear space from the property boundary, other than adjoining a public space, either by 2.0 times the computed deflections or 0.002 times its height whichever is larger, and in many cases, not less than 2.5 cm. Parts of the same building or buildings on the same site which are not designed to act as an integral unit shall be separated from each other by a distance of at least 2.0 times the sum of the individual computed deflections or 0.004 times its height whichever is larger, and in many cases, not less than 5.0 cm.

Combination (DDC) (Abdel Raheem 2014). The ABS and SRSS rules provide unreasonably conservative separation distances that are extremely hard to be successfully executed, particularly when the adjacent structures have close matching vibration characteristics. The Double Difference Combination (DDC) rule is a more logical approach for evaluation of the critical required separation distance, which is almost equivalent to the peak relative displacement response (Jeng *et al*. 1992, DesRoches and Muthukumar 2002). Three various criteria to estimate the separation required to avoid seismic pounding between structural systems were inspected. None of the criteria assessed is completely perfect as in none of them gives separations that are reliably correct or somewhat conservative. Observations indicate that there is still a need to adequately characterize the correlation between displacement responses of nonlinear systems.

The majority of building codes suggest separation distances based on maximum lateral displacements of each building or height of buildings in order to provide safety gap size between them. The Canadian code considers the simplest approach in which the absolute sum (ABS) of the peak displacements of two adjacent buildings (NRCC 2005). The edition of 1997 of the Uniform Building Code; UBC97 (ICBO 1997) and the edition of 2003 of the International Building Code (IBC 2003) suggest the formula of the square root of sum of squares (SRSS). The quadratic combination of the maximum peak displacements has also been employed in EC8 (ECS 2004). The peak relative displacement between adjacent buildings is modified for determination the seismic gap distance with deflection amplification factor and importance factor (IBC 2009, ASCE 2010) as illustrated in Table 3. A number of codes specify the minimum seismic gap using some ways which are independent from the dynamic characteristics of structures. The Federal Emergency Management Agency suggests the determination of the minimum seismic gap as a percentage of the height of buildings without any computations of the peak displacements in order to prevent their pounding during earthquakes (FEMA 1997). Some of the regulations suggest calculating the minimum seismic gap based simultaneously on peak structural displacements and height of structures. The Indian code for seismic design recalls the simple sum of peak displacements of adjacent buildings to be the base for calculating the minimum seismic separation gap together with a response reduction factor (IS 2002). The regulations from the Egyptian and Peru codes for seismic design use values of the peak displacements of two adjacent buildings as well as the heights of structures as guides (NBC 2003, ECP 2008). Although numerous recent codes require a minimum seismic separation gap, it is as yet insufficient as codes essentially lag behind the recent research (Abdel Raheem 2014, Jankowski and Mahmoud 2015). The relative separation demand u_{Rel} that is calculated as the peak of the relative displacement time history response of adjacent buildings is a more realistic approach.

4. Numerical results and discussion

4.1 Natural vibration analysis

In most structural design, empirical building period formulas are used to initiate the design process (Kwon and Kim 2010). The determination of the vibration characteristics of a building can be obtained by experimental methods with observation of the dynamic in-situ behavior through in-situ ambient noise measurements or using analytical modelling based on the mechanical properties of the components, including all elements contributing either to the mass or stiffness of the system. The vibration characteristics for the studied adjacent buildings in terms of fundamental period and vibration modes as gained from the structural analysis using finite element models and empirical expression in the ECP-201 and other international building codes (ICBO 1997, ICC 2003, ECS 2004, NRCC 2005, ECP 2008) are listed in Table 4. The fundamental periods of the three building models based on ECP-201 (2008) are 1.102, 0.655, 0.390 sec, whereas the fundamental period based on FE approach are 1.566, 0.897, 0.533 sec, which reaches 142, 137, 137%

Table 4 Free vibration period characteristics of studied buildings

Period analytical and building-code expressions		Fundamental period (sec)		
		12-Story	6-Story	3-Story
3D FE model vibration analysis	1st lateral vibration mode	1.566	0.897	0.533
	Torsional Vibration mode	1.369/0.522	0.820	0.503
	2nd lateral vibration mode	0.577	0.314	0.178
	3rd lateral vibration mode	0.335	0.184	0.113
ECP-201 (ECP 2008)	$T = 0.075H^{3/4}$	1.102	0.655	0.390
ECP-201 (ECP 1993)	$T = 0.1\ N$	1.200	0.600	0.300
IBC (ICC 2003)	$T = 0.073\ H^{3/4}$	1.073	0.638	0.379
UBC (UBC 1997)	$T = 0.049\ H^{3/4}$	0.720	0.428	0.255
EC8 (ECS 2004)	$T = 0.075\ H^{3/4}$	1.102	0.655	0.390
NBCC (NRCC 2005)	$T = 0.05\ H^{3/4}$	0.735	0.437	0.260

*H = the building's height measured from the base; and N = the number of stories for 12-story, 6-story and 3-story buildings that introduced in the code provisions. Hence it is clear that the code formulas have a significant defect in the calculation of vibration period which is considered the main parameter for lateral force procedure.

4.2 Separation gap among adjacent buildings

The common provision of building codes recommends a minimum separation gap based on maximum lateral displacements of each building to prevent pounding among adjacent structures. Although building codes take care of this problem, building designers are often reluctant to implement the required separation between buildings to eliminate pounding. To accomplish an adequately safe structural functioning throughout seismic hazards, an accurate seismic design should consider the relative displacements estimated using a nonlinear time history analysis. The peak value of displacement time history responses of the no-pounding case for 12-story, 6-story and 3-story buildings (u_{12}, u_6, u_3) are listed in Tables 5-7. The peak value determines the maximum displacement for standalone building at the potential level of impact with adjacent buildings in different alignment configurations. The nonlinear dynamic time history analysis for three different alignment configurations of three adjacent buildings in series has been studied as shown in Fig. 4: (a) Configuration I (12-6-3); (b) Configuration II (3-12-6); and (c) Configuration III (6-3-12). The critical separation distance, u_{Rel} is calculated as the peak value of the relative displacement time history response of all possible alignment configurations "I, II, III" of adjacent three buildings in series under various input excitation, and compared to the code-specified separation based on ABS and SRSS rules.

The minimum required separation distance between adjacent structures; u_{Rel} is calculated as the peak value of the relative displacement time history responses at all the potential pounding levels and all the possible potential alignment of the adjacent buildings. The required separation is calculated for the no-pounding case, where the interaction between adjacent buildings of all configurations due to

Table 5 Peak values of the relative displacement (m) between 12-Story and 6-Story models at 6th level of impact

Earthquake	u_{12}	u_6	$u_{Rel_{12\&6}}$	ABS	$SRSS$	$u_{Rel_{12\&6}}/ABS$	$u_{Rel_{12\&6}}/SRSS$
San Simeon	0.079	0.115	0.144	0.194	0.140	0.742	1.032
Morgan Hill	0.077	0.141	0.165	0.218	0.161	0.757	1.027
Christchurch	0.109	0.122	0.162	0.231	0.164	0.701	0.990
L'Aquila	0.092	0.131	0.163	0.223	0.160	0.731	1.018
Loma	0.085	0.152	0.180	0.237	0.174	0.759	1.034
Imperial Valley	0.092	0.147	0.211	0.239	0.173	0.883	1.217
Bam	0.101	0.122	0.168	0.223	0.158	0.753	1.061
Kobe	0.090	0.171	0.227	0.261	0.193	0.870	1.175
Chi-Chi	0.107	0.148	0.168	0.255	0.183	0.659	0.920
Maximum	0.109	0.171	0.227	0.261	0.193	0.883	1.217
Average	0.092	0.139	0.176	0.231	0.167	0.762	1.053
Standard deviation	0.011	0.018	0.026	0.020	0.016	0.072	0.091

Table 6 Peak values of the displacements for 12-Story and 3-Story models at 3rd level of impact

Earthquake	u_{12}	u_3	$u_{Rel_{12\&3}}$	ABS	$SRSS$	$u_{Rel_{12\&3}}/ABS$	$u_{Rel_{12\&3}}/SRSS$
San Simeon	0.033	0.077	0.076	0.110	0.084	0.691	0.907
Morgan Hill	0.032	0.077	0.056	0.109	0.083	0.514	0.672
Christchurch	0.045	0.082	0.081	0.127	0.094	0.638	0.866
L'Aquila	0.042	0.065	0.076	0.107	0.077	0.710	0.982
Loma	0.052	0.074	0.068	0.126	0.090	0.540	0.752
Imperial Valley	0.051	0.071	0.077	0.122	0.087	0.631	0.881
Bam	0.048	0.077	0.059	0.125	0.091	0.472	0.650
Kobe	0.046	0.064	0.085	0.110	0.079	0.773	1.078
Chi-Chi	0.055	0.075	0.066	0.130	0.093	0.508	0.710
Maximum	**0.055**	**0.082**	**0.085**	**0.130**	**0.094**	**0.773**	**1.078**
Average	**0.045**	**0.074**	**0.072**	**0.118**	**0.087**	**0.608**	**0.833**
Standard deviation	**0.008**	**0.006**	**0.010**	**0.009**	**0.006**	**0.105**	**0.147**

Table 7 Peak values of the relative displacements for 6-Story and 3-Story models at 3rd level of impact

Earthquake	u_{12}	u_6	$u_{Rel_{12\&6}}$	ABS	$SRSS$	$u_{Rel_{12\&6}}/ABS$	$u_{Rel_{12\&6}}/SRSS$
San Simeon	0.072	0.077	0.105	0.149	0.105	0.705	0.996
Morgan Hill	0.071	0.077	0.108	0.148	0.105	0.730	1.031
Christchurch	0.074	0.082	0.119	0.156	0.110	0.763	1.077
L'Aquila	0.075	0.065	0.091	0.140	0.099	0.650	0.917
Loma	0.083	0.074	0.104	0.157	0.111	0.662	0.935
Imperial Valley	0.070	0.071	0.112	0.141	0.100	0.794	1.123
Bam	0.071	0.077	0.099	0.148	0.105	0.669	0.945
Kobe	0.086	0.064	0.119	0.150	0.107	0.793	1.110
Chi-Chi	0.076	0.075	0.107	0.151	0.107	0.709	1.002
Maximum	**0.086**	**0.082**	**0.119**	**0.157**	**0.111**	**0.794**	**1.123**
Average	**0.075**	**0.074**	**0.107**	**0.149**	**0.105**	**0.719**	**1.015**
Standard deviation	**0.006**	**0.006**	**0.009**	**0.006**	**0.004**	**0.055**	**0.076**

pounding is neglected. Hence the introduced separation distance is valid for all studied configurations and represents the minimum gap that could be provided to avoid pounding. The Canadian code considers the simplest approach in which the absolute sum (ABS) of the peak displacements of two adjacent buildings (NRCC 2005). The edition of 1997 of the Uniform Building Code; UBC97 (ICBO 1997) and the edition of 2003 of the International Building Code (IBC 2003) suggest the formula of the square root of sum of squares (SRSS). The complete quadratic combination (CQC) of the maximum peak displacements has also been employed in EC8 (ECS 2004). While the ABS and SRSS approached for the determination of the minimum required separation distance between two adjacent structures requires only the peak displacement responses, the CQC requires the peak values in addition the time displacement responses to define the cross correlation coefficients. A measure index of the calculated relative displacement ration to either ABS or SRSS is calculated to investigate the actual required separation distanced based on nonlinear time history analysis relative the minimum suggested separation gap in different international seismic design codes (ICBO 1997, ECS 2004, NRCC 2005). Since the absolute sum (ABS) method considers complete out-of-phase response of the adjacent buildings, the ratio of u_{Rel} to the sum of u_A and u_B could be considered as a degree of out-of-phase of adjacent buildings, which depends on adjacent building vibration and input earthquake excitation characteristics. The out-of-phase displacement among buildings is obviously detected because of different vibration periods of the adjacent buildings. The closing and opening peak displacements are important to decide the level of prejudiced response of the pounding system. Thus, seismic pounding between adjacent buildings may cause unseemly damages albeit every standalone structure might have been designed perfectly to resist the hit of realistic earthquake actions. The separation distance u_{Rel} obtained based on nonlinear time history analysis is compared with the corresponding estimate that is based on ABS and SRSS response combination rules suggested in several seismic design codes as illustrated in Tables 5-7. The average required separation distance at the potential 6th level of impact reaches 0.176 ± 0.014 m between 12-story and 6-story buildings and with maximum required separation of 0.227 m that is 88% and 122% to the code defined minimum required gap distance based on ABS and SRSS, respectively. While the average required separation distance at the potential 3rd level of impact reaches 0.072 ± 0.010 m between 12-story and 3-story buildings and with maximum required separation of 0.085 m that is 77% and 108% to the code defined minimum required gap distance based on ABS and SRSS, respectively. Furthermore, the average required separation distance at the potential 3rd level of impact reaches 0.107 ± 0.009 m between 6-story and 3-story buildings and with maximum required separation of 0.119 m that is 79% and 112% to the code defined minimum required gap distance based on ABS and SRSS, respectively. Moreover, the code-prescribed width of the separation joint could be insufficient when the fundamental periods of the adjacent buildings are close to the excitation frequency due to resonance phenomenon.

4.3 Effect of gap size on response demands

The nonlinear dynamic analyses have been carried for four different gap sizes "G" of 2 cm, 6 cm and 12 cm in addition to in-contact adjacent buildings; G = 0. The magnification in response demands of adjacent buildings depends on natural vibration period of each building and their ratio besides the dominant frequency of input excitation. In addition to that alignment configuration of buildings, whether exterior building exposed to one-side impact or interior building exposed to two-sided impact. Table 8 shows the peak displacement responses at pounding levels for configuration II for in-contact "$G = 0$" adjacent buildings under selected input earthquakes and compared to no pounding case. For exterior 3-story and 6-story buildings, the seismic pounding reduces the peak displacement response demands of building in both impact and rebound directions and the level of reduction depends on the input excitation. The displacement peak responses in the impact direction are significantly decreased about 50-

Table 8 Peak displacement response (m) at pounding level (Configuration II, G = 0)

Model		3-Story building		12-Story building				6-Story building	
Impact level		3rd level				6th level			
Response direction		Rebound	Impact direction		Rebound		Impact direction		Rebound
Kobe	NoPounding	-0.059	0.064	-0.046	0.046	-0.089	0.090	-0.171	0.156
	Pounding	-0.058	0.032	-0.043	0.037	-0.093	0.078	-0.078	0.128
	%	**-2**	**-50**	**-7**	**-20**	**4**	**-13**	**-54**	**-18**
L'Aquila	NoPounding	-0.065	0.061	-0.040	0.042	-0.084	0.092	-0.131	0.105
	Pounding	-0.063	0.027	-0.035	0.045	-0.098	0.093	-0.057	0.092
	%	**-3**	**-56**	**-13**	**7**	**17**	**1**	**-56**	**-12**
San Simeon	NoPounding	-0.057	0.077	-0.033	0.030	-0.079	0.072	-0.113	0.115
	Pounding	-0.052	0.019	-0.033	0.027	-0.086	0.061	-0.070	0.092
	%	**-9**	**-75**	**0**	**-10**	**9**	**-15**	**-38**	**-20**

75% and 38-56% compared to that of no-pounding case for 3-story and 6-story buildings, respectively. While the displacement peak responses in the opening/rebound direction are slightly decreased about 2-9% and 12-20% compared to that of no-pounding case for 3-story and 6-story buildings, respectively. For the interior 12-Story building with two sided-impacts at 3rd and 6th levels, the displacement response demand decreases due to pounding in the impact direction with maximum 15% of that no-pounding case. The displacement response demand at 6th story level increases in the rebound direction with maximum 17% of that no-pounding case, while the response demands at 3rd level could increase 7% or decrease about 20% of that no-pounding case; depending on the input excitation.

Table 9 shows the peak acceleration responses at pounding levels for configuration II under different earthquakes and compared to no pounding case. For the exterior buildings at end of the row of adjacent buildings are exposed to one-sided impacts and as a rule experience acceleration response magnification in the opening direction that can be very significant. The magnification effect ranges from 156% to 360% for 3-story building at the top floor depending on separation gap size and input excitation, while for 6-story building, it ranges 391% to 654% at the 6th story level of impact. For 12-story interior building, in contrast, is exposed to two-sided impacts that can cause amplifications of acceleration response that could ranges from 218% to 547% relying upon the level of impact, separation gap size and input excitation characteristics.

Fig. 5 presents the displacement responses envelops for different spacing sizes that confirms the trend of impact effect on the displacement response demands of the adjacent building in configuration II of buildings alignment. The peak of story displacement response depends on the input excitation characteristics and gap size, enlarging separation gap width is most likely effective to eliminate contact when the separation is adequately wide. A gap size of 6 cm is sufficient to significantly reduce the impact effect at 3rd level between 3- and 12-story adjacent buildings, while a gap size of more than 12 cm is required to significantly reduce the impact effect at 6th level between 6- and 12-story adjacent buildings. The seismic pounding provides displacement restrains on the impacting side, but may amplify displacement responses on the other side, particularly the response of 12-stroy building at the height

Table 9 Peak acceleration response at pounding level (Configuration II) (m/s^2)

Model		3-Story building		12-Story building				6-Story building	
Impact level		3rd level				6th level			
Response direction		Rebound	Impact direction		Rebound		Impact direction		Rebound
Kobe	No pounding	-9.56	9.73	-7.29	6.35	-7.34	7.10	-12.53	10.83
	Pounding G = 0	-26.14	10.27	-26.83	22.00	-36.08	15.51	-9.74	50.90
	Pounding G = 2	-32.12	11.67	-22.33	28.56	-36.06	17.89	-10.66	46.61
L'Aquila	No pounding	-10.34	8.95	-8.36	7.46	-8.92	8.35	-10.17	11.24
	Pounding G = 0	-37.25	11.07	-20.48	33.19	-45.39	20.34	-9.09	56.87
	Pounding G = 2	-16.15	9.77	-32.71	20.26	-38.90	18.50	-9.25	43.97
San Simeon	No pounding	-9.43	11.07	-6.24	5.97	-6.98	7.60	-9.01	9.26
	Pounding G = 0	-27.51	9.54	-22.58	23.17	-37.60	26.81	-9.70	55.87
	Pounding G = 2	-30.24	11.38	-22.78	15.13	-38.15	28.28	-10.50	60.60

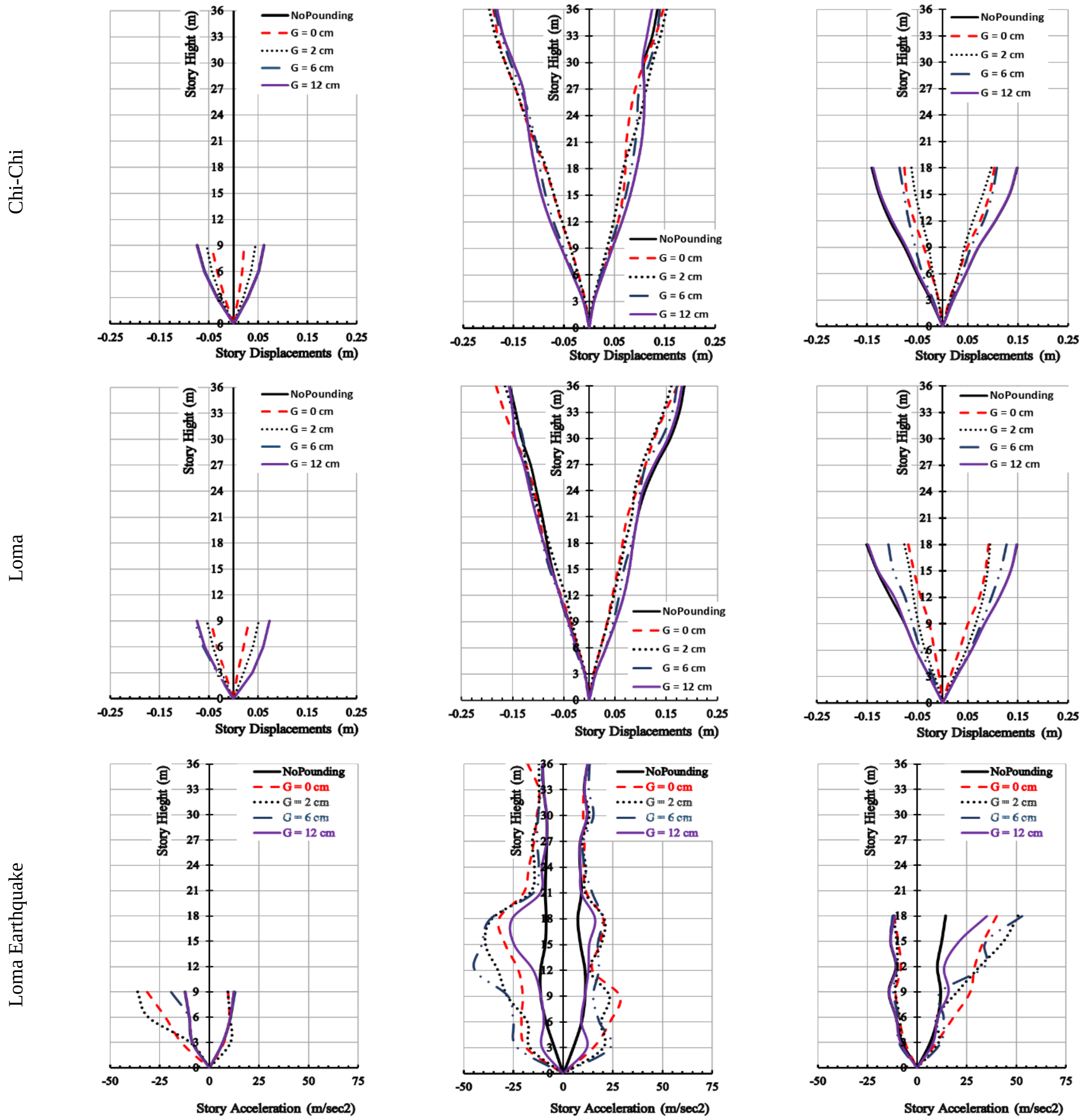
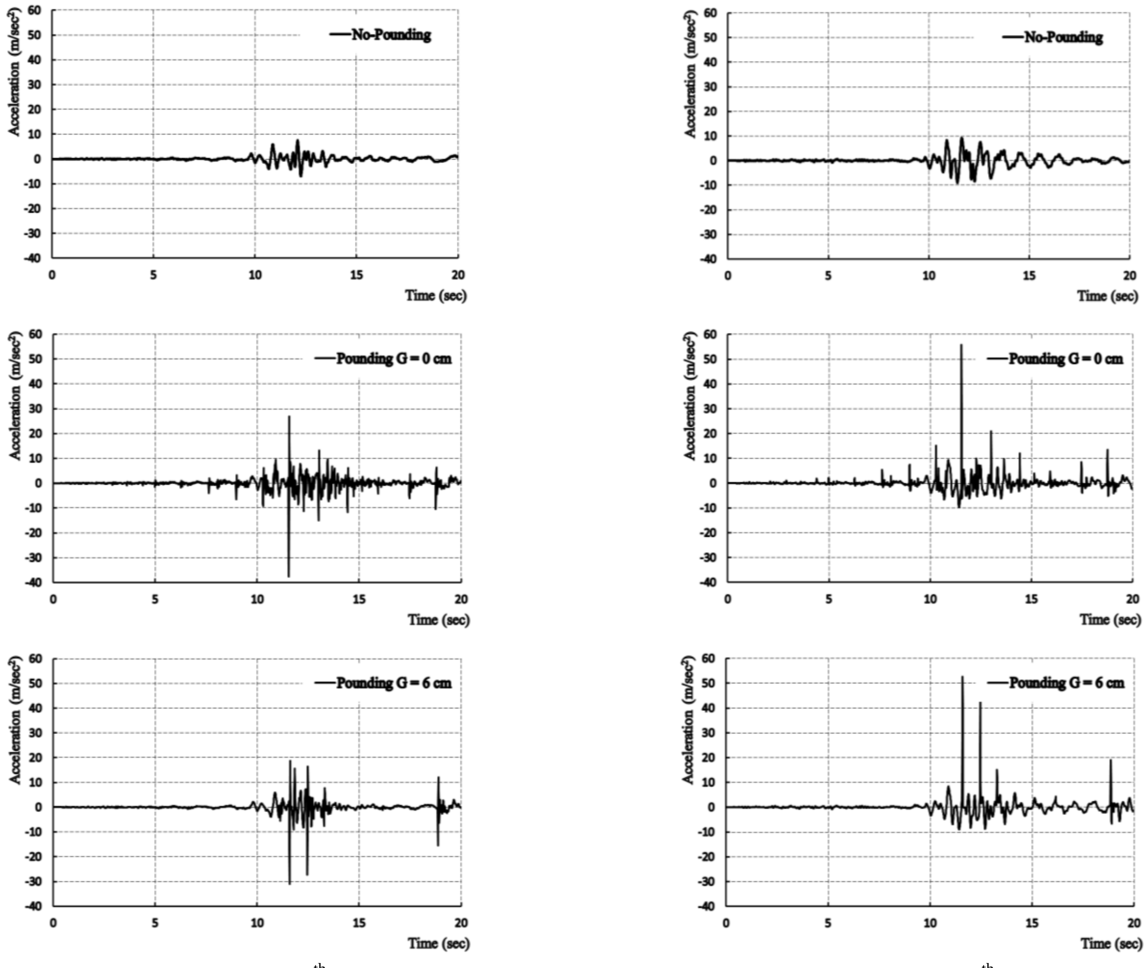


Fig. 5 Displacement and acceleration responses envelops for different spacing sizes (Configuration II)

levels above the impact level. Furthermore, the maximum responses in the short building are decreased in the impact and rebound directions. It can be concluded that in the shorter building pounding results in reduction of displacements in all stories while in the taller building generally the response decreases in the lower levels but only slightly increases in the upper ones. The pounding effect of the impact at the 6^{th} level is more significant than that of impact at 3^{rd} level.

Considering that losses due to non-structural components have consistently been reported to be far greater than those resulting from structural damage, it is imperative to consider maximum story horizontal accelerations. Modern code design provisions evaluate the maximum story horizontal accelerations to design the non-structural systems and their connections to the main structure. Nevertheless, the pounding phenomenon between adjacent buildings is not taken into account, which generally leads to higher values of the accelerations in comparison with the case of well-separated buildings. This characteristic can be observed in Figs. 5 and 6, which depicts the story horizontal acceleration envelopes for buildings in contact with different gap sizes and no-pounding case and acceleration response time history under the San Simeon earthquake for different gap sizes. Fig. 5 comprises the envelope responses under Loma earthquake; it is evident that buildings subjected to pounding generally present higher story acceleration in comparison with no



(a) 12-Story building (6th level) (b) 6-Story building (6th level)

Fig. 6 Acceleration time histories under the San Simeon earthquake for different gap sizes (Configuration II)

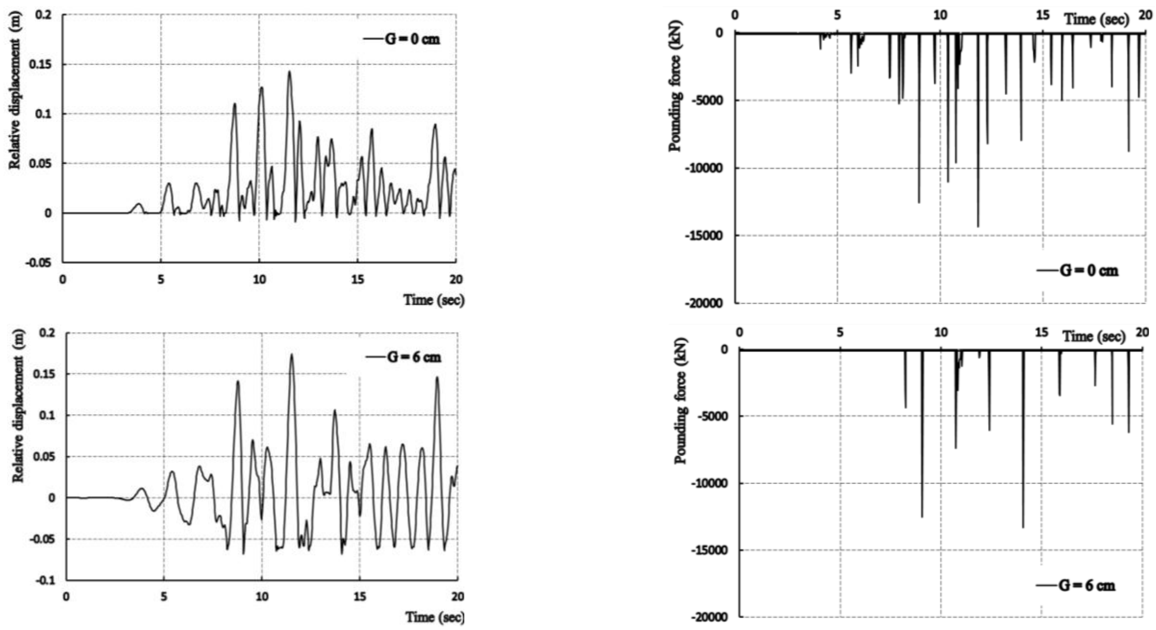
pounding case. Therefore, it is obvious that the maximum story horizontal accelerations of buildings are strongly affected by the seismic gap between the collided buildings. The acceleration response of high-rise building at the height levels below the impact levels is significantly amplified at both directions due to two-sided impact, the response gets its maximum values at pounding of in-contact building and with small gap size of 2 cm and decrease effectively with the increase of gap size, while the response of the floors at the height levels above the impact level is slightly affected. Furthermore, the maximum responses in the low rise building are significantly increased in the rebound directions over the whole height of building, while the response in the impact direction is slightly affected due to one-side impact for the exterior building of adjacent in series alignment buildings.

Fig. 6 presents the acceleration response time histories of the colliding buildings at the potential top level of the 6-story and 3-story buildings for different gap sizes, under San Simeon earthquake record. The acceleration response is amplified due to collision among the adjacent buildings and can gain several times that from no-pounding case. The most evident change in the graphs is that there are upsurges in negative accelerations for 3-story building and in positive accelerations for 6-Story building due to the configuration arrangement, while for 12-Story building the increase occur in positive and negative accelerations. For in-contact pounding; G = 0, the peak negative acceleration at top level of 3-story building is as high as -27.51 m/sec^2 at 13.38 seconds. It is nearly three times greater related to no pounding acceleration which is only -9.43 m/sec^2 at 11.07 seconds. The peak positive acceleration produced in 6-story building during collision is as much as 55.87 m/sec^2 at 11.55 seconds. It is about 6 times greater than the peak acceleration for no pounding case which is only 9.26 m/sec^2 at 11.65 seconds. While for 12-story building, the crowning negative acceleration at sixth level (critical pounding level) is as high as -37.6 m/sec^2 at 11.53 seconds. It is 5.4 times higher related to no pounding acceleration which is only -6.98 m/sec^2 at 12.26 seconds, and the greatest positive acceleration at third level is as high as 23.17 m/sec^{2}at 11.65 seconds. It is 3.9 times higher related to no pounding acceleration which is only 5.97 m/sec^2 at 11.85 seconds.

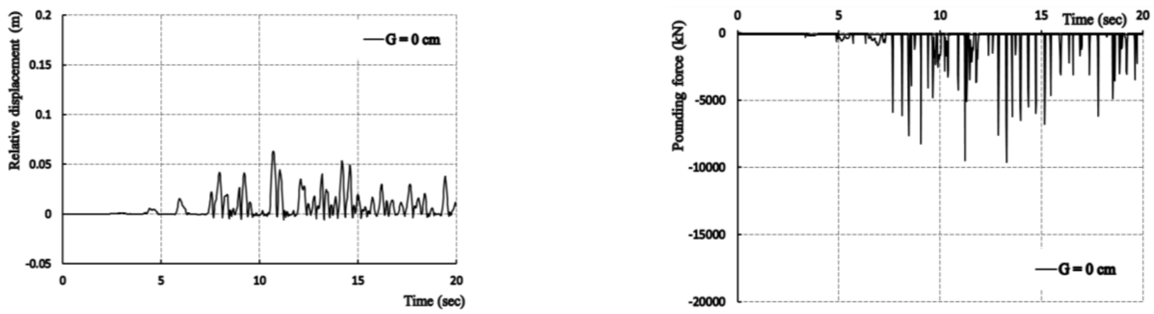
Table 10 presents peak pounding force induced under Chi-Chi earthquake for different gap sizes. The pounding between 3-story building and 12-story buildings at 3rd story level displays higher value of the impact force for gap size G = 2 cm, even greater than the case of in-contact alignment G = 0. Furthermore, the potential impact is extended over all stories for the in-contact case, with lighter

Table 10 Peak pounding force induced under Chi-Chi earthquake for different gap sizes (kN) (Configuration II)

Story	Impact force between 3-&12- Story buildings				Impact force between 12-&6- Story buildings			
	G = 0	G = 2	G = 6	G = 12	G = 0	G = 2	G = 6	G = 12
Story 6	-	-	-	-	13032	8425	12130	10425
Story 5	-	-	-	-	12499	7542	9879	4723
Story 4	-	-	-	-	10272	9631	6369	0
Story 3	8054	10872	1642	0	6004	7911	0	0
Story 2	6136	5808	0	0	4832	2003	0	0
Story 1	3327	0	0	0	2007	0	0	0



(a) 6th level pounding between 12- and 6-story buildings



(b) 3rd level pounding between 6- and 3-story buildings

Fig. 7 Displacement and pounding force response time histories under Kobe earthquake, Configuration II

impact at lower stories. The pounding between 6-story building and 12-story buildings at 6th story level displays higher value of the impact force for gap size G = 6 cm that is close the case of in-contact alignment G = 0, and higher than that of case for G = 2 cm. In general, it is noticed that maximum relative displacement, the impact force is rapidly increasing and then slightly decreases with further reduction or increase in the separation. The ratio of the offered seismic gap to the maximum relative displacements between adjacent buildings for each earthquake input excitation appears to play an important role in the severity for a range of the separation gap near the middle third of of the structural pounding and its consequences. Fig. 7 shows the sequence of impact force and relative displacement time history responses at the top levels of the 6-story and 3-story buildings for different gap size (G = 0, 6 cm) under Kobe earthquake, since the top level experiences the most critical condition. For the relative displacement time history, negative values depict opening relative displacements, while positive values result from the event of strong

Table 11 Peak story shear at pounding level (Configuration II) (kN)

Model		3-Story building		12-Story building				6-Story building	
Response level		At base level		3rd level	6th level	At base level		At base level	
Response direction		(+)	(-)	(+)	(-)	(+)	(-)	(+)	(-)
Kobe	No Pounding	4578	-5303	5454	-5927	7847	-7274	7946	-7810
	Pounding G = 0	5041 (10%)	-2589 (-51%)	6892 (26%)	-7971 (34%)	7987 (2%)	-9509 (31%)	3751 (-53%)	-8084 (4%)
	Pounding G = 2 cm	4590 (0.2%)	-5252 (-1%)	6795 (25%)	-7573 (28%)	7942 (1%)	-8047 (11%)	6408 (-19%)	-7034 (-10%)
L'Aquila	No Pounding	5315	-4597	5367	-5910	6140	-6770	7198	-7281
	Pounding G = 0	4902 (-8%)	-2429 (-47%)	7556 (41%)	-7243 (23%)	8119 (32%)	-7728 (14%)	2832 (-61%)	-6959 (-4%)
	Pounding G = 2 cm	5307 (-0.1%)	-4294 (-7%)	7131 (33%)	-6669 (13%)	8512 (39%)	-8845 (31%)	4084 (-43%)	-5326 (-27%)
San Simeon	No Pounding	4621	-5604	4596	-3903	5127	-5146	7643	-6870
	Pounding G = 0	4696 (2%)	-1990 (-64%)	6472 (41%)	-10112 (159%)	7273 (42%)	-8357 (62%)	3162 (-59%)	-7119 (4%)
	Pounding G = 2 cm	4128 (-11%)	-4341 (-23%)	6852 (49%)	-9711 (149%)	9256 (81%)	-7717 (50%)	4936 (-35%)	-7342 (7%)

impacts causing the pounding. The occurring of pounding develops larger opening relative displacements. The acceleration response variation at the impact level of the 6-story and 3-story buildings during collision between adjacent buildings in series under various earthquakes is determined. Pounding is a severe load condition that could result in unexpected magnitude and short duration acceleration spikes, which consecutively cause larger damage to building contents. An abrupt stopping of velocity at the impact level results in great and quick acceleration pulses in the opposite direction. The adjacent buildings tend to pound together in several different times if the separation gap gets narrower.

Table 11 shows the peak story shear responses at impact levels and at base for configuration II under different earthquakes and compared to no pounding case. For the exterior buildings, the story shear response demands are significantly reduced up to 53% of that no-pounding case. While for the interior buildings, in contrast, are exposed to two-sided impacts that the story shear responses are significantly increased either at the base or above the impact levels. The magnification effect reaches 149%, 260% and 181% at the 3rd and 6th story level of impact and at base depending on separation gap size and input excitation. An appropriate gap distance for the adjacent structures should be determined regarding the characteristics of an expected earthquake along with the properties of structures. Damage potential due to pounding not only governed by the magnitude of the collision force, but also by the recurrence number of strong impacts. Although the increase of separation gap from zero to 2 to 6 to 12 cm develops larger opening relative displacements but in contrast it has the capability for decreasing impact effects and could decrease the number of pounding's event. Furthermore, enlarging separation gap width is most likely effective to eliminate contact when the separation is adequately wide.

Likewise, pounding can magnify the global response of interacting adjacent buildings. Pounding generates an acceleration response and shear force response demands at different story levels that might be greater than those from no-pounding case, as shown in Fig. 8, however the peak of story displacement response depends on the input excitation characteristics and position of buildings alignment in a row. In Configuration II, for the both exterior buildings (3-story and 6-story buildings), top level pounding decreases the peak shears over the entire building height except for the top story near the impact level gets larger values. Whilst for the interior 12-story building, pounding magnifies shear above pounding level as well as acceleration at the level of collision. The sway of the higher building is suddenly limited by the shorter building and it experiences high story shear forces above the pounding level. The seismic responses of adjacent buildings in series are affected adversely by the pounding. To evaluate the pounding effects on seismic response of buildings in series, interior and exterior building should be differentiated, the first exposed to two-sided impacts and the second to one-sided impacts. Due to pounding, the maximum variation in shear forces of the higher building is always monitored in the inter-story above the top floor of the shorter adjacent building. For shorter adjacent buildings, the seismic vibrations reduced considerably; consequently, the severity of the probable pounding is reduced. The higher adjacent building experienced more severe seismic demand due to pounding than the shorter building. The pounding effect that primarily increases the story shear response is the de-acceleration that occurs when the adjacent buildings collide. However, the duration of the collisions is small. As pounding happens; the building experiences high impact forces and acceleration spikes at the instant of contact. The peak of acceleration response due to pounding could attain 10 times more than

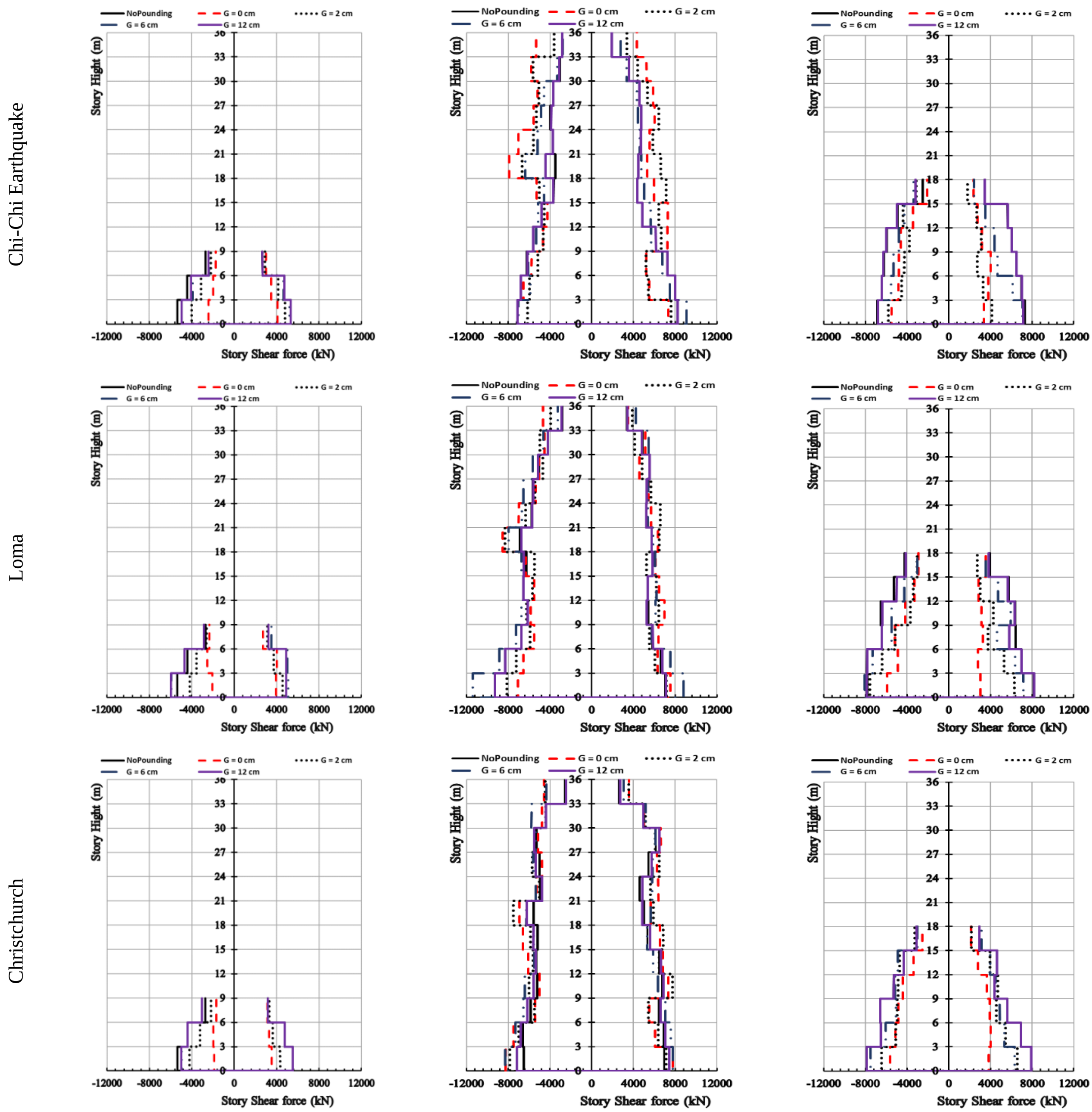


Fig. 8 Story shear response envelops for different spacing sizes (Configuration II)

that of no-pounding case, which are within the range viewed in experimental results (Guo *et al*. 2009). The energy balance analysis confirms that pounding, in addition to the local damage it usually causes, can increase or reduce the structural response, depending on the vibration characteristics of the adjacent structures.

4.4 Alignment configurations of buildings effects

Many of the buildings that survived after the earthquake have the benefit of being located between two buildings and behave as a unique building that has superior performance than those of the standalone building. The interior position of building among adjacent buildings reduces the potential damaging effects of the seismic pounding. As a short building is located between two high-rise buildings, the vibration amplitude of the short building is reduced and its effect on the two adjacent buildings is decreased as could be illustrated in configuration III (6-3-12). The displacement response demands are significantly reduced for 3-story interior building, slightly affected the response of the 6-story exterior building by increasing in the rebound direction and decreasing in the impact direction. While the response of 12-story building is almost not affected. In configuration I (12-6-3), the displacement response demands are significantly reduced for the 6-story interior building and the 3-story exterior short building, while the 12-story exterior high building has an increase of the response over the height above the impact level in the rebound direction, and response decreases in the impact

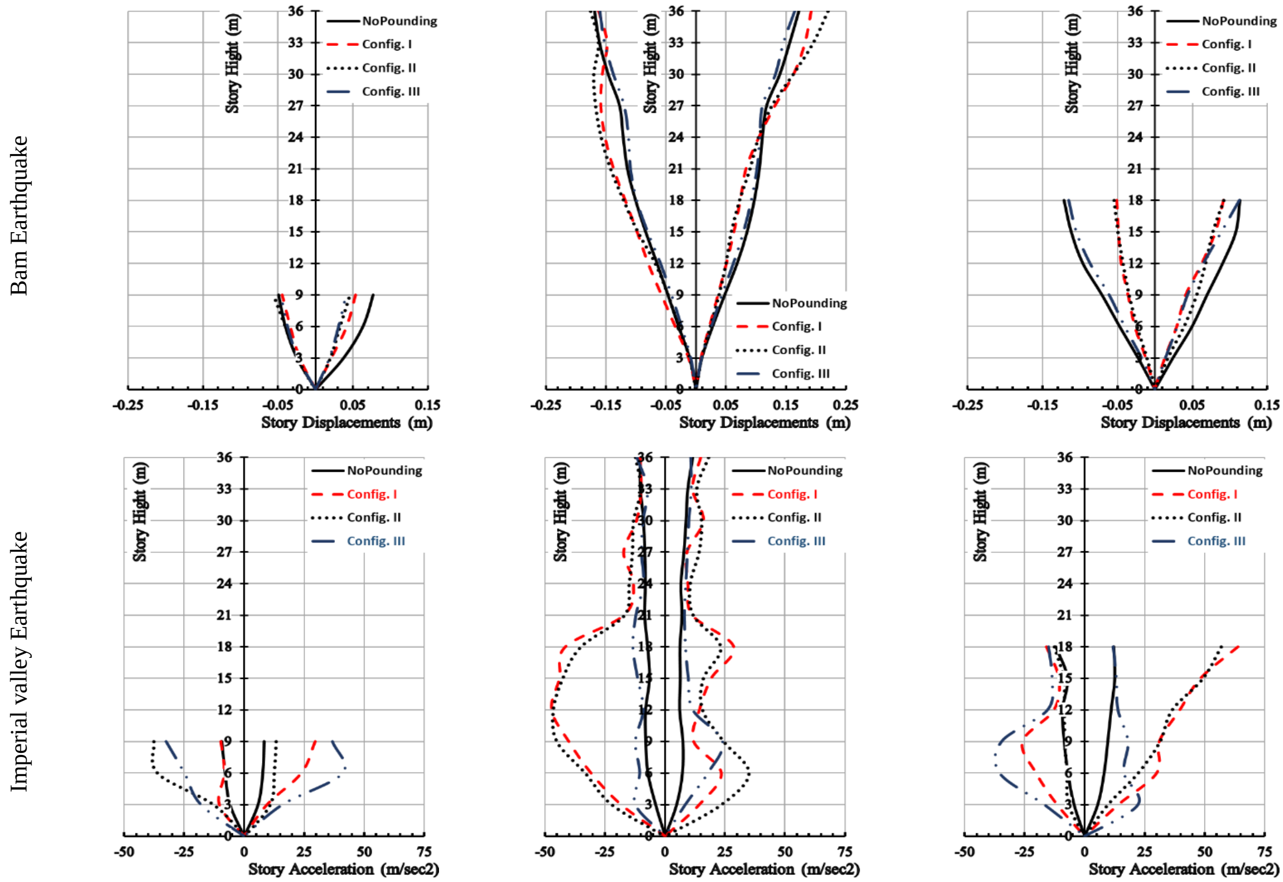


Fig. 9 Story displacement/acceleration responses envelops for different Configurations of adjacent buildings (G = 2 cm)

direction. In configuration II (3-12-6), the displacement response demands are significantly reduced for the 3- and 6-story exterior buildings, while the 12-story interior high building has an increase of the response over the height above the impact level in the rebound direction, and response decreases in the impact direction, the impact effect is dominated by the impact with 6-story building.

To evaluate the pounding effects on seismic response of buildings in series, interior and exterior building should be differentiated, the first exposed to two-sided impacts and the second to one-sided impacts. The magnification in the response buildings is extremely serious for cases with highly out-of-phase buildings. Two-sided pounding magnifies the stiff building response, and decreases the flexible building response. Due to pounding, the maximum variation in shear forces of the higher building is always monitored in the story above the top floor of the shorter adjacent building. For shorter adjacent buildings, the seismic vibrations reduced considerably; consequently, the severity of the probable pounding is reduced. Figs. 9-10 show the response envelopes of adjacent buildings for different configurations under several earthquake records. The peak displacement responses depend on the input excitation characteristics and alignment position of building in series. Comparing pounding-involved and independent vibration responses for the adjacent buildings in series for different configurations shows that the 12-story building is more influenced by pounding because it acts as a stopper for the external buildings. Although the 12-story has long period and higher amplitude of motion and the 3- and 6-story shorter buildings have relative short periods, the 12-story building has relative high stiffens at the level of impacts. In configuration II, pounding has increased the peak absolute displacement of the middle high building above the impact level, as compared to the no pounding case. Whereas, it has decreased the peak displacement of the left and right relative short buildings.

In configuration I, pounding effect has decreased the mean and maximum peak displacement responses of the 6-story interior building and the 3-story exterior building by about 50, 44% and 35%, 34% as compared to the no pounding case, respectively. Whereas, the mean and maximum peak displacement responses of the 12-story exterior buildings could increase by about 11%, 14% in the rebound direction and decrease 22%, 19% in the impact direction. In configuration II, the mean and maximum peak displacement responses of the 12-story interior building increased by about 11 and 19% in the rebound direction and decrease 20 in the impact direction as compared to the no pounding case. Whereas, it has decreased the mean and maximum peak displacement responses of the left and right exterior buildings by about 38%, 28% and 51%, 47% for 3-story and 6-story building, respectively.

An abrupt change of velocity direction at the impact level results in great and high acceleration pulses in the opposite direction. The acceleration response has high magnitude and short duration floor acceleration spikes, which in sequence cause foremost damage to building

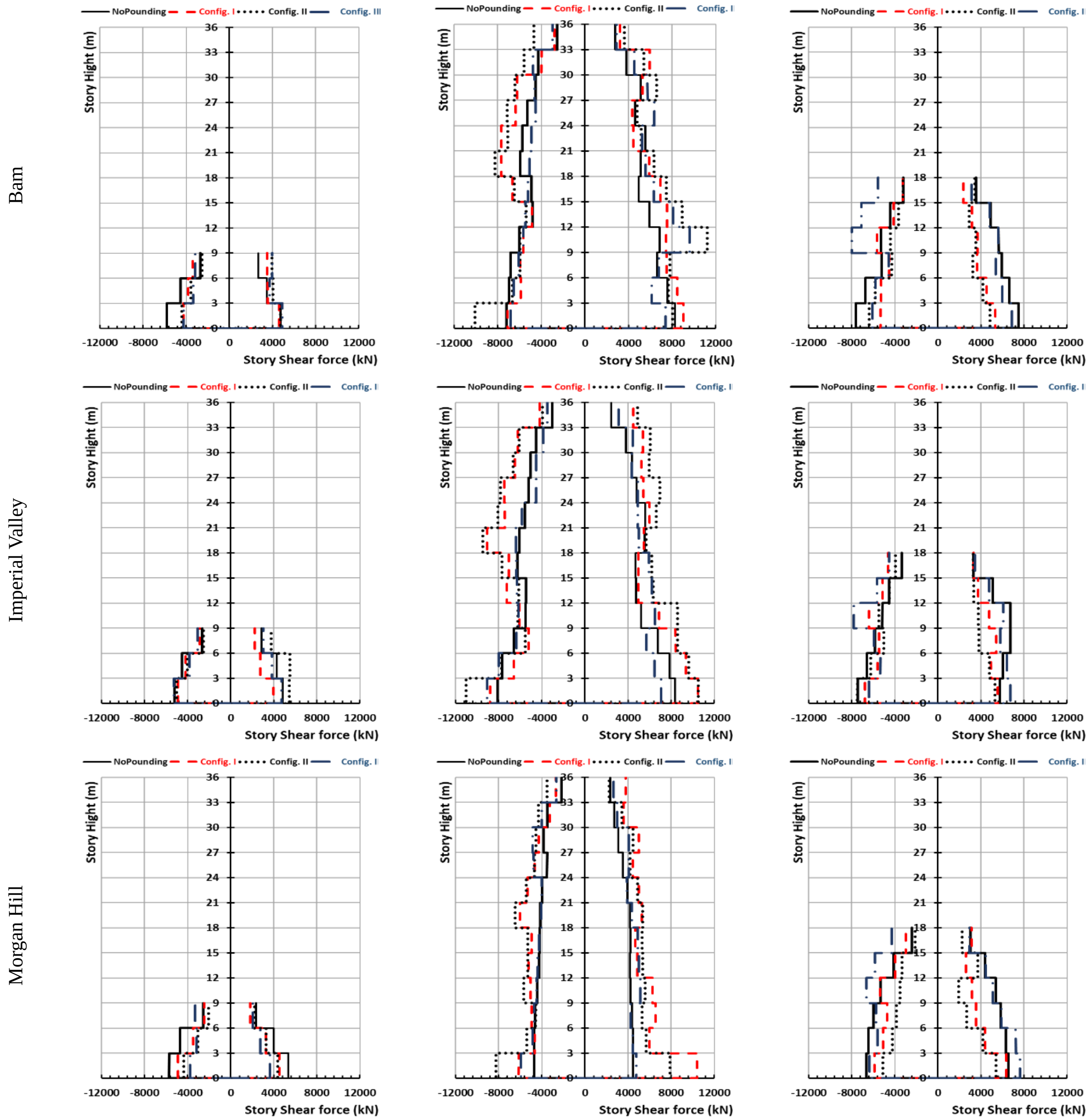


Fig. 10 Story Shear envelops for different Configurations of adjacent buildings (G = 2 cm)

contents. In configuration III, the 3-story short building is located between 6- and 12-story high-rise buildings, the vibration amplitude of the short building is decreased and acceleration response is increased and its influence on the 12-story adjacent building is negligible. The response of 6-story is significantly amplified below the impact level for the acceleration response, story shear above the impact level in the rebound direction. In configuration II, when a 12-story high-rise building is located between 3-story and 6-story buildings, its acceleration response is increased at the height levels below the collision level. At the levels over the collision level, no significant increase is observed in the responses. While, the mean and maximum acceleration responses of low rise building are slightly changes either increase or decrease in the impact direction and significantly increased in the rebound direction all over the building height. In configuration I, when a 6-story medium-rise building is located between 12-story and 6-story buildings, its acceleration response is increased at the height levels below the collision level. At the levels over the collision 3rd level, no significant change is observed in the responses compared to the no pounding case. While, the mean and maximum acceleration responses of 3-story low rise building are slightly changes either increase or decrease in the impact direction and significantly increased in the rebound direction all over the building height. The mean and maximum acceleration responses of 12-story building are slightly changes either increase or decrease above the impact level and significantly increased in the both directions below the impact level.

For the 12-Story building, pounding amplifies story shear response above impact level twice as much in configuration I, where it is in the left end of straight alignment as exterior building with one sided-pounding to 6-Story building. While in configuration II, the seismic responses of 12-story building as interior building with two sided-pounding are significantly increase for acceleration and shear force response demands. As the12-Story building located in the right end as exterior building with one-sided pounding to 3-Story building in configuration III, the shear force response is not affected by pounding. The shear response of 6-Story building is increased by pounding in all configurations, but when it located in the right end, configuration II, the response amplification become less than other cases. The pounding has significant effects on the peak of story shear for 3-Story building as it has internal alignment and subjected to two-sided pounding. Seismic collision of 3-story buildings decreases the mean shear force demand over all stories below the collision level and improve the behavior of structure for the different configuration either exterior with one-sided impact or interior with two-sided impact. However, in the case of 6-story buildings, story shear demands are decreased for the interior alignment in configuration I and exterior alignment in configuration II but the response is increased significantly, especially at the height level in which the collision is occurring (3^{rd} level) and above for exterior alignment in configuration III. In the case of 12-story buildings, story shear demands are increased for the interior alignment in configuration II above the impact levels in the opposite direction of impact, and slightly affect below the impact levels. The response is increased significantly, especially at the height level in which the collision is occurring and above for exterior alignment in configurations I and III in rebounding direction. It is observed that the stiffer structure; 12-story building, irrespective of its relative alignment position, undergone the most story drift and shear force response magnification.

5. Conclusions

Seismic pounding is an extremely nonlinear phenomenon and a severe load case that could be a source of major structural damages. Therefore, an assessment of the seismic pounding hazard to the adjacent buildings is superficial in future building code calibrations. Thus, this study targets are to draw useful recommendations for code calibration through a numerical simulation approach for the evaluation of the pounding risks on adjacent buildings during their functional life. A numerical simulation is formulated to estimate the pounding effects on the seismic response demands of three adjacent buildings in series with different alignment configurations. Three adjacent buildings of 3-storey, 6-storey and 12-storey MRF buildings are combined together to produce three different alignment configurations; these configurations of adjacent buildings are subjected to nine ground motions that are absolutely compatible with the design spectrum. The nonlinear time-history is performed for the evaluation of the response demands of different alignment configurations of the adjacent buildings using structural analysis software ETABS. Various response parameters are investigated such as displacement, acceleration, story shear force responses, impact force and hysteretic behavior. Based on the obtained results, it has been concluded that the severity of the seismic pounding effects depends on the vibration characteristic of the adjacent buildings, the input excitation characteristic and whether the building has interior or exterior alignment position, thus either exposed to one or two-sided impacts.

Based on the obtained results, it has been concluded that the severity of the pounding effects on the response of adjacent buildings in series depends on the vibration characteristics of the adjacent buildings, the input excitation characteristics, separation gap size, height ratio and the alignment position of the building in series: whether interior building with potential two-sided impacts or exterior building with potential one-sided pounding. It is noticed that for a range of the separation gap near the middle third of maximum relative displacement, the impact force is rapidly increasing and then slightly decreases with further reduction or increase in the separation. The ratio of the offered seismic gap to the maximum relative displacements between adjacent buildings for each earthquake input excitation appears to play an important role in the severity of the structural pounding and its consequences. Moreover, the pounding hazard of adjacent buildings could be amplified as the periods of buildings approach the dominant period of input excitation.

Pounding may occur at different floor levels, allowing the activation of multiple contact locations along the height of the buildings. The vertical location of potential pounding extensively affects the distribution of story peak responses through the building height. It is observed that the stiffer structure; 12-story building, irrespective of its relative alignment position, undergone the most story drift and shear force responses magnification. The acceleration response of high-rise building at the height levels below the impact levels is significantly amplified at both directions due to two-sided impact, the response gets its maximum values at pounding of in-contact building and with small gap size of 2 cm and decrease effectively with the increase of gap size, while the response of the floors at the height levels above the impact level is slightly affected. Furthermore, the maximum responses in the short buildings are significantly increased in the rebound directions over the whole height of building, while the response in the impact direction is slightly affected due to one-side impact for the exterior building. An abrupt change of velocity direction at the impact level results in great and high acceleration pulses in the opposite direction. The acceleration response has high magnitude and short duration floor acceleration spikes, which in sequence cause foremost damage to building contents. Moreover, the pounding hazard of adjacent buildings could be amplified as the periods of buildings approach the dominant period of input excitation. The seismic pounding provides displacement restrains on the impacting side, but may amplify displacement responses on the other side, particularly the response of 12-stroy building at the height levels above the impact level. Furthermore, the maximum responses in the short building are decreased in the impact and rebound directions.

The interior position of building among adjacent buildings reduces the potential damaging effects of the seismic pounding. As a short building is located between two high-rise buildings, the vibration amplitude of the short building is reduced and its effect on the two adjacent buildings is decreased. As a result, the response of high-rise building at the impact level is reduced. Additionally, the maximum responses in the short building are also reduced. As high-rise building is located between two short buildings, its response is decreased at the height levels below the impact level. The maximum responses of short building are reduced. The story shear in the higher buildings increases above pounding level as well as acceleration at the level of impact. The story shear response demand decreases in the shorter of the adjacent buildings over the entire building height with the exception of the stories in the vicinity of the impact.

The pounding has a considerable effect on the story shear response of the higher building in the stories upper than roof of the shorter structure. It is observed that pounding can make the story shear in the stories just higher than roof of the shorter building to surpass those of the lower ones. The time lag of the impact of the interior building with the right and left exterior buildings and different levels of impact reduce the impact interaction effect on the response demands of adjacent buildings in series. Synchronized impact at different levels of impact could maximize the adjacent building interaction and impact effects. Although pounding may sometimes reduce the overall structural response of short buildings and thus be considered beneficial, more often it will amplify the response significantly of the relative higher building irrespective the position of the building in the alignment configurations of adjacent buildings in series. The differences in height, period, the period ratio and relative alignment of adjacent buildings seem to be the crucial factors that affect the response of pounding buildings. Therefore, it is highly recommended to introduce into the codes conditions and provision for the assessment of the minimum required seismic separation and the pounding risk of buildings. Although some of the findings will be case study specific, many of the findings are highly relevant to many other adjacent buildings. Continued research is urgently needed in order to provide the engineering design profession with practical means to evaluate and mitigate the extremely hazardous effects of pounding.

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