

A shake table investigation on interaction between buildings in a row

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Abstract. Pounding damage has been observed frequently in major earthquakes in the form of aesthetic, minor or major structural cracks and collapse of buildings. Studies have identified a building located at one end of a row of buildings as very vulnerable to pounding damage, while buildings in the interior of the same row are assumed to be safer. This study presents the results of a shake table investigation of pounding between two and three buildings in a row. Two steel portal frames, one stiffer and another more flexible, were subjected to pounding against a frame with eight other configurations. Three pounding arrangements were considered, i.e., the reference frame (1) on the right of the second frame, (2) in the middle of two identical frames, and (3) on the right of two identical frames. Zero seismic gap was adopted for all tests. Five different ground motions are applied from both directions (right to left and left to right). The amplification of the maximum deflection due to pounding was calculated for each configuration. The results showed that, for the stiffer building in a row, row building pounding is more hazardous than pounding between only two buildings. The location of the stiffer frame, whether at the end or the middle of the row, did not have much effect on the degree of amplification observed. Additionally, for all cases considered, pounding caused less amplification for stronger ground motions, i.e., the ground motions that produced higher maximum deflection without pounding than other ground motions.

Keywords: seismic pounding; structure-structure interaction; row of buildings

1. Introduction

Seismic pounding occurs when two adjacent structures or parts of a structure vibrate out of phase and the separation distance is too small to accommodate the relative closing displacement. Pounding causes the structures to exert repeated hammer like blows on each other which may cause minor non-structural or severe structural damage that may even lead to the complete collapse of buildings (Rosenblueth and Meli 1986). Surveys after almost all major earthquakes in urban areas have found the presence of damage due to pounding of buildings and bridges (Kasai and Maison 1997, Anagnostopoulos 1996, Palermo *et al.* 2011, Choww and Hao 2012). Several urban seismic vulnerability surveys have identified pounding as one of the major hazards (Jeng and Tzeng 2000, Bothara *et al.* 2008). Bothara *et al.* (2008) considered pounding as a critical structural weakness in the seismic assessment of Wellington city in New Zealand and found the

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presence of a large number of susceptible buildings in the city Central Business District (CBD). Jeng and Tzeng (2000) assessed the pounding vulnerability of Taipei City and found hundreds of mid-rise buildings susceptible to damage or even collapse from pounding. They have identified five building configurations that are prone to damage due to pounding. Though pounding can occur between any two buildings with insufficient gap, more damage have been observed when a building is: (i) adjacent to a more massive building, (ii) adjacent to a building with fewer stories, (iii) subject to eccentric pounding, (iv) at the end of a row of buildings, and (v) subject to mid-column pounding. A building is more vulnerable if it possesses more of these weaknesses.

The damage due to end building pounding had been identified as early as 1977 (Anagnostopoulos 1996). It is one of the most prevalent vulnerabilities as cities around the world are full of city blocks with rows of buildings in contact with each other, especially in CBDs (Jeng and Tzeng 2000, Bothara *et al.* 2008, Anagnostopoulos 1988, Cole *et al.* 2010). Thus, the subject has received considerable research attention. Anagnostopoulos (1988) conducted a numerical simulation on pounding of a row of buildings idealized as single degree of freedom systems, and concluded that the exterior structures in a row experience higher amplification of response than the interior structures. The response of interior structures were found to be amplified or reduced depending on whether their fundamental period was smaller or higher than the adjacent structures; stiffer structures typically receiving amplification and flexible structures undergoing reduction of response. The same stiffer structure in the middle of the row received less amplification than when they were placed externally. The study was one of the first to model energy loss during impact with a viscoelastic spring. Athanassiadou *et al.* (1994) carried out similar simulations including the effect of phase difference in ground motion due to the velocity of the seismic waves. They found that the stiffer structure, regardless of its position in a row, always suffered the most response amplification. The interaction between adjacent structures and their subsoil can also have a significant influence on the development of the relative displacement (Bi *et al.* 2011, 2013, Chouw 2002, 2008, Shakya and Wijeyewickrema 2009, Chouw and Hao 2005, 2008).

Anagnostopoulos and Spiliopoulos (1992) observed in numerical simulation of three multistory buildings that sometimes end building pounding produced higher response amplification than for middle building, but mostly the amplifications were comparable. There were even some cases where the amplification for interior building was higher. Ohta *et al.* (2006) analysed pounding between two and three buildings using finite element program SAP2000 and observed that, only in some cases for the same building response amplification as the end building in three buildings configuration is higher than in two pounding. The number of such cases was found to increase with a larger number of stories of participating structures.

From the observation in post-earthquake surveys and based on numerical studies, end building pounding is identified as more vulnerable than when the building is located in the middle of a row.

Cole *et al.* (2010) included external building in a row as one of the six configurations susceptible to pounding damage. Bothara *et al.* (2008) also considered end building in a row more vulnerable than those within the row. In contrast, damage survey from Christchurch 2011 showed several cases where the buildings in the middle of the row were badly damaged, while buildings at the end of the same row survived (Cole *et al.* 2011).

Several experimental studies on pounding of two building have been performed in the past. Papadrakakis and Mouzakis (1995) subjected two storey concrete frame structures to floor to floor pounding and found that structures nearest to their resonance amplify the displacement of the adjacent structure. Filiatrault *et al.* conducted experiments on pounding of unequal height steel structures to validate the performance of FE analysis software to predict pounding response. Chau

et al. (2003) studied the pounding between equal height steel structures. Rezavandi and Moghadam (2007) have experimentally evaluated the effectiveness of various mitigation measures in pounding of steel frames. The authors could not find any such experimental studies on three building pounding though it has often been numerically predicted as more hazardous than two building poundings. Similarly, no comparative experiments between two and three building pounding were found. To the authors' best knowledge an experimental validation of the conclusions derived from past numerical studies on row building pounding has never been reported.

In this work a parametric shake table study of pounding between two and three steel portal frames was conducted. The frames were subjected to five ground motions. A frame was designated as the reference frame and its response amplification were investigated with three configurations: (i) two building pounding (TBP), (ii) row building pounding (RBP) and (iii) end building pounding (EBP). The reference frame was kept at the centre of two other identical frames for RBP while it was placed to the right of the identical frames for EBP. The top displacements of the frames were measured, and the amplification of the maximum displacement is employed as the measure of severity of pounding. The impact forces have not been measured as the inclusion of any kind of force measuring device can alter the pounding force development and subsequently, response of the frames.

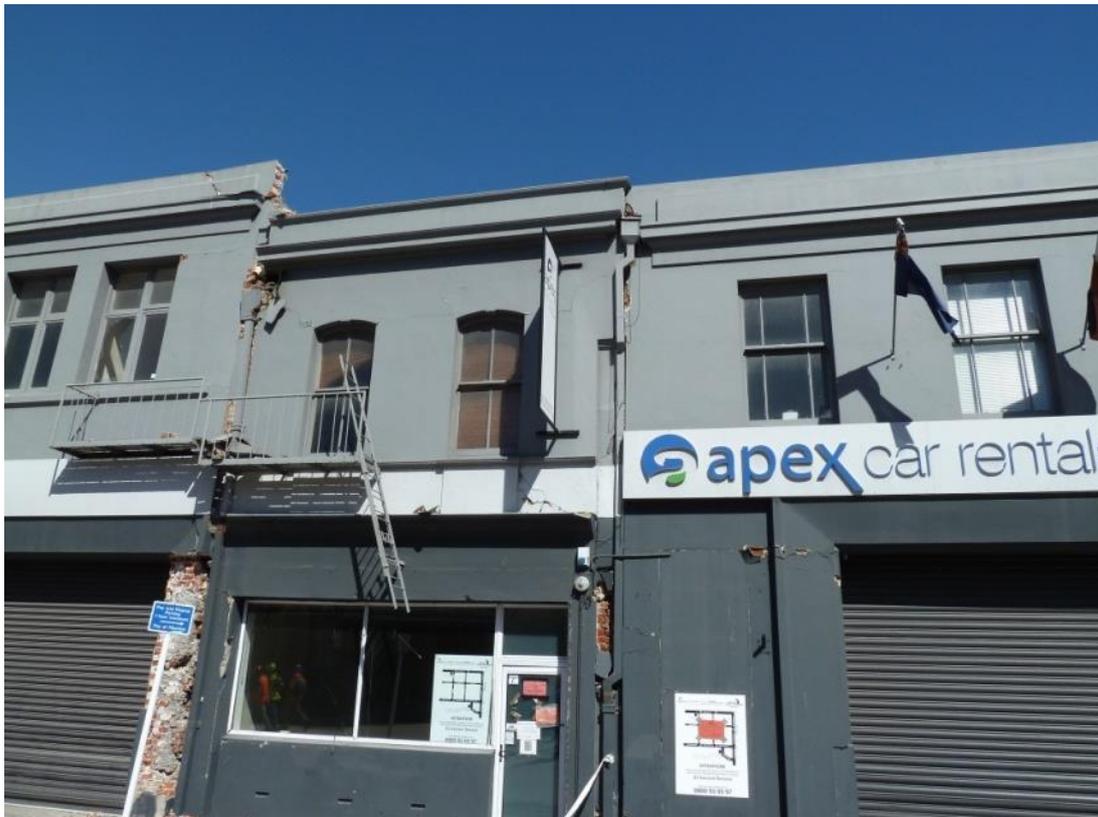


Fig. 1 Damage to adjacent buildings without gap observed in the 22 February 2011 Christchurch earthquake

2. Experimental setup

Fig. 2 shows steel frames fabricated for this study. The frames had three different column sizes: 50 x 3 mm, 75 x 3 mm, and 100 x 3 mm. The stiffness of these three different types of frames are displayed in Table 1. The inside dimension between the beams in Fig. 2 (Section A – A) varied according to the column size. The beams supported a 200 x 150 x 10 mm plate, which could be loaded with additional identical plates as shown by the dotted lines. Four mass setups were used for the test as shown in Table 2. The masses varied among different frames because the load plates were identical for all frames while the sizes of columns and horizontal bracings are different. The

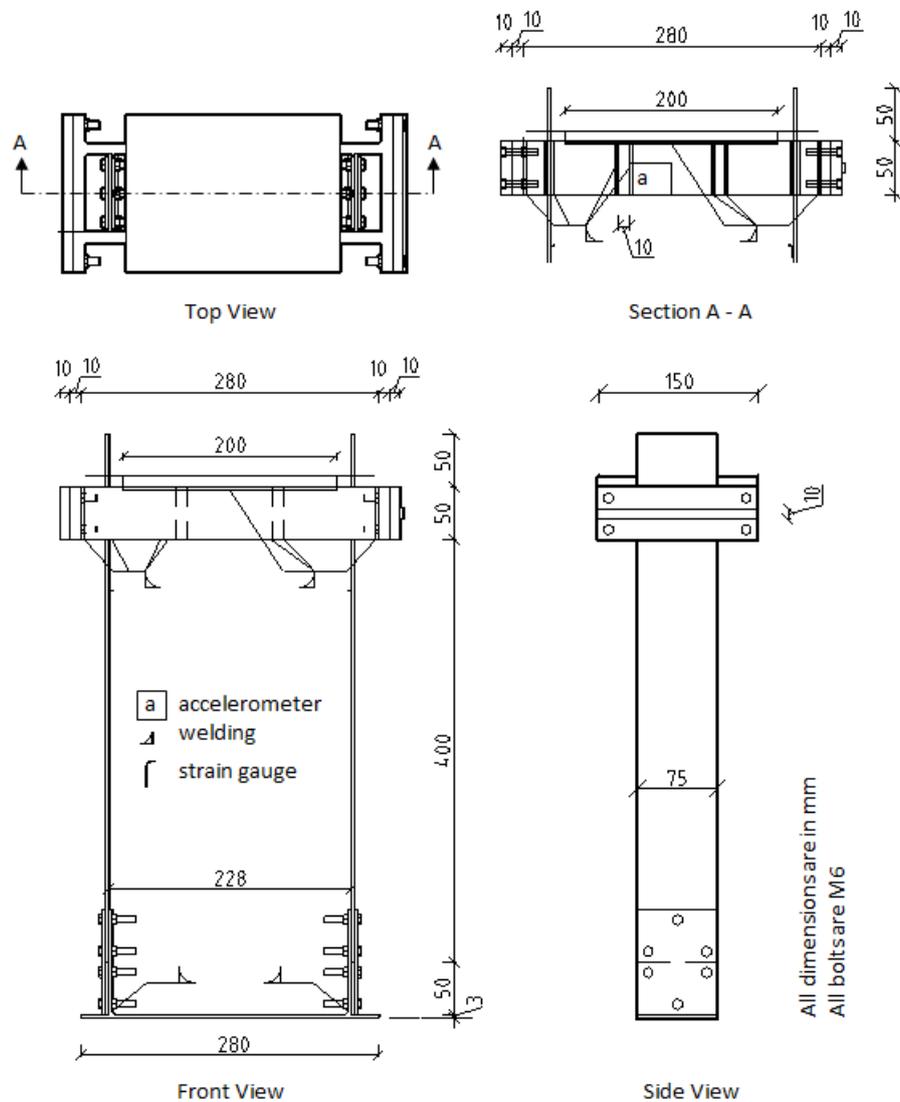


Fig. 2 Schematic drawing of the steel frame with 75 x 3 mm column

columns were connected to a separate base as shown in the expanded details on bottom left side of the figure, so that the details for all the columns in multi-storey frames would remain identical. There were two horizontal bracings between the beams. An accelerometer was attached to one of these bracings during the tests. A strain gauge is installed on each column just above the base joint. The strain gauges were calibrated against the displacement of each frame relative to its base.

A 150 x 10 x 3 mm steel strip is employed as the contact interface as shown in the details of the right end of the beam. The strip is glued and welded to a 150 x 50 x 10 mm steel plate, which is bolted to an identical plate welded to the beams. The top left end of the frame had a similar detail but did not have the 3 mm middle strip. Thus, a plain surface contact in 150 x 10 mm area was assumed when the frames were placed end to end.

Table 3 shows the fundamental period of the mass-frame combinations considered in the experiments. Snap back tests were conducted to determine the fundamental period and the damping constant of the structures. The actual periods of the frames were found to be within $\pm 2\%$ of the theoretical values.

Table 1 Stiffness ID for different frames

Stiffness ID	Column size (mm)	Lateral stiffness (N/m)
k1	50 x 3	5,926
k2	75 x 3	8,889
k3	100 x 3	11,852

Table 2 Masses considered

Mass ID	Additional load plates	Mass (kg)		
		k1	k2	k3
m0	0	8.04	8.59	9.14
m1	2	12.75	13.30	13.85
m2	4	17.46	18.01	18.56
m3	6	22.17	22.72	23.27

Table 3 Fundamental period T(s) for selected mass-frame combinations

Mass	Frame Stiffness		
	k1	k2	k3
m0	k1m0	k2m0	k3m0
	T = 0.23	T = 0.20	T = 0.17
m1	-	k2m1	-
	-	T = 0.24	-
m2	-	k2m2	-
	-	T = 0.28	-
m3	k1m3	k2m3	k3m3
	T = 0.38	T = 0.32	T = 0.28

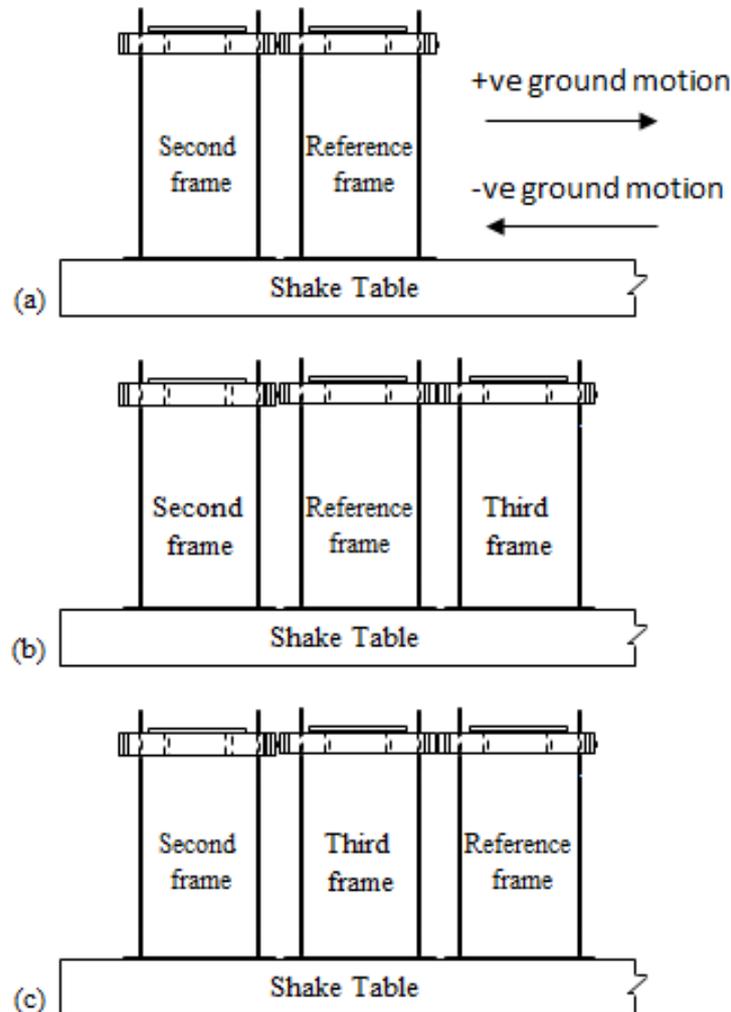


Fig. 3 Test setups for (a) Two building pounding (TBP), (b) Row building pounding (RBP), and (c) End building pounding (EBP)

The test setups are shown in Fig. 3. For two building poundings, the reference frame was subject to pounding against a second frame, as shown in Table 4. Thus, the second frame either had a same stiffness and different mass (e.g., Cases 3-6) or same mass and different stiffness (e.g., Cases 1, 7) as the reference frames. The Cases 2, 8, 9 and 15 were added so that both the reference frames were subjected to pounding with the same set of frames in all cases.

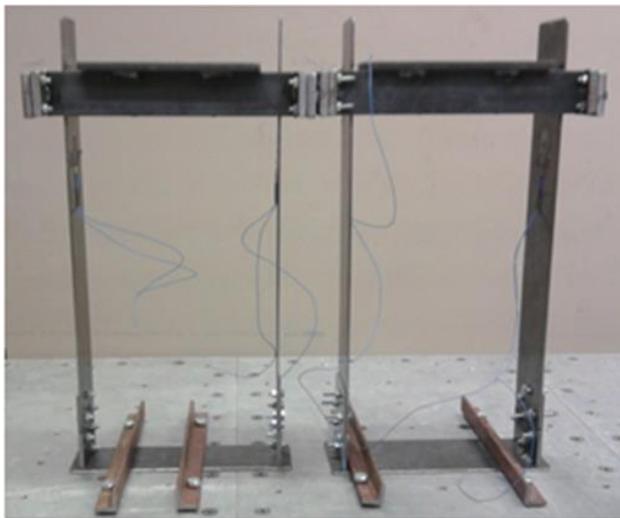
The second and third frames were identical for each EBP and RBP configurations. A three letter prefix will be added to the case number to identify the type of configuration being employed, e.g., when a k2m0 frame is pounding with a k2m3 frame, it will be called TBP6; when a k2m0 frame is between two k2m3 frames, it will be called RBP6; and when a k2m0 frame is on a side of the two k2m3 frames, it will be called EBP6. Fig. 4 shows the pounding arrangement for Case TPB7.

Table 4 Period ratio of the frames considered

Case	Configurations (from Table 3)	T_2 (s)	T_1/T_2 (-)
1	k2m0-k1m0	0.23	0.82
2	k2m0-k1m3	0.38	0.49
3	k2m0-k2m0	0.19	1.00
4	k2m0-k2m1	0.24	0.79
5	k2m0-k2m2	0.28	0.68
6	k2m0-k2m3	0.31	0.60
7	k2m0-k3m0	0.16	1.15
8	k2m0-k3m3	0.27	0.70
9	k2m3-k1m0	0.23	1.36
10	k2m3-k1m3	0.38	0.82
11	k2m3-k2m0	0.19	1.66
12	k2m3-k2m1	0.24	1.32
13	k2m3-k2m2	0.28	1.13
14	k2m3-k2m3	0.31	1.00
15	k2m3-k3m0	0.16	1.92
16	k2m3-k3m3	0.27	1.15

T_1 of reference frame 1 (k2m0) is 0.2 s

T_1 of reference frame 2 (k2m3) is 0.32 s



(a)



(b)



(c)

Fig. 4 Adjacent structure without seismic gap: (a) Frames on shake table, (b) pounding interface and (c) pounding elements without (top) and with middle strip (bottom)

Four selected time histories were applied to each configuration case, i.e., two artificial ground motions simulated based on the New Zealand design spectrum (NZDS 1 and NZDS 2) and two based on the Japanese design spectrum for hard soil condition (JDS 1 and JDS 2). The simulated time histories were scaled down so that the maximum ground displacement was ± 10 cm. In addition, the most well-known earthquake, the 1940 El-Centro ground motion (EIC) is also considered. The El-Centro ground motion was not scaled. The scaled displacement time histories are shown together with the El-Centro excitation in Fig. 5. Since the ground motion directions can have a significant effect on the pounding response, the ground motions were applied twice: once from left to right (termed positive direction) and secondly from right to left (termed negative direction). The zero separation was considered as past studies have found that the pounding response decreased as the gap size was increased. This is also a common configuration of buildings in CBDs of many big cities (see Fig. 1 and Chouw and Hao 2012). The displacement response of the frames was measured by the strain gauges placed on the columns just below the beams. After the tests were finished, the calibration was checked again, and no significant difference was found between the initial and final calibration factors. The pounding interface also did not show any indentation or other permanent deformation after any of the tests.

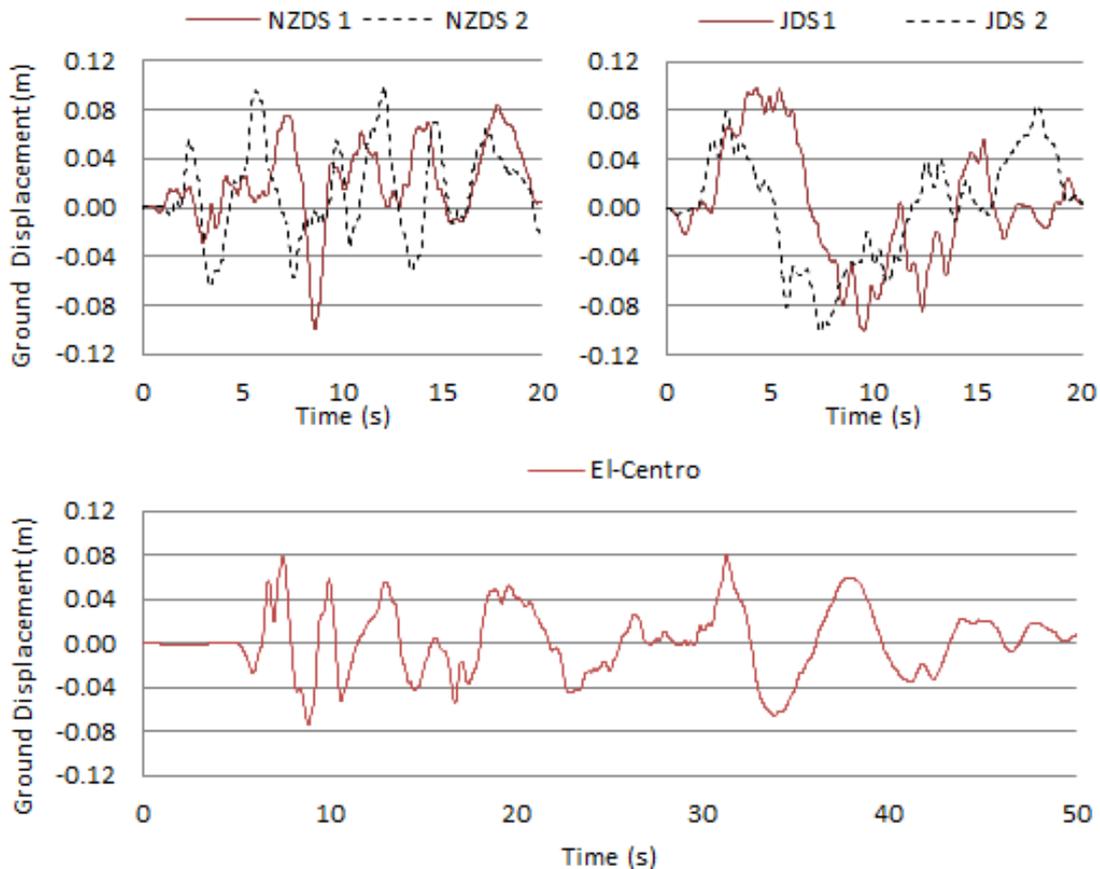


Fig. 5 Ground motions considered

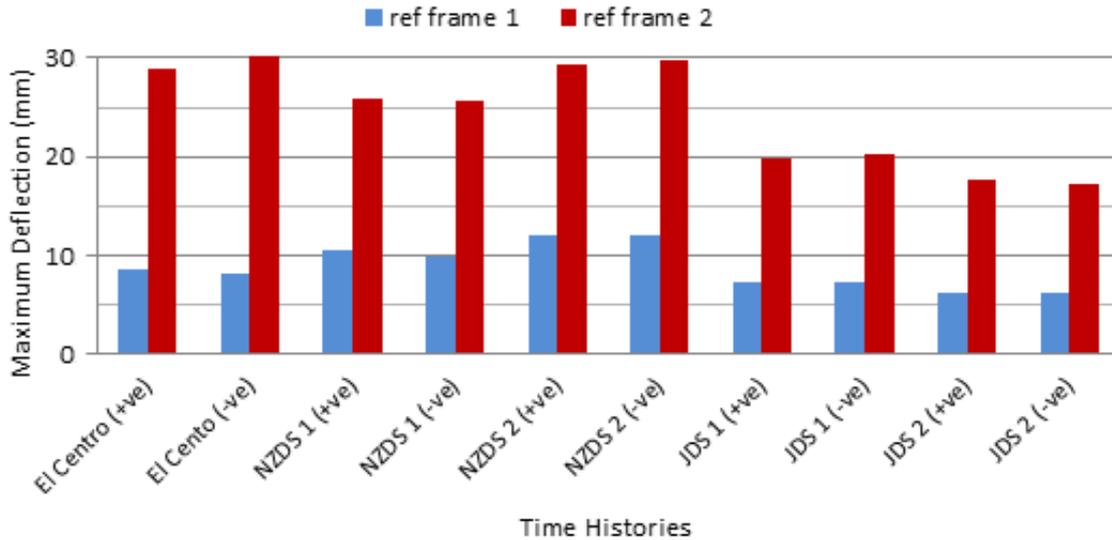


Fig. 6 Maximum deflection u_{max} of the reference frame without pounding

3. Results and discussion

The maximum deflections of the reference frames are shown in Fig. 6. For reference frame 1, the scaled NZDS loadings caused the maximum floor displacement while the scaled JDS loadings have the least floor displacement. The El-Centro ground excitation produced median deflection among the five time-histories in both directions. For reference frame 2, the El Centro ground motion caused the maximum floor displacement. Ideally, the deflection of the frame without pounding with the adjacent buildings should not be affected by the direction of the excitation but it can be seen that there is some slight effect likely due to inadvertent lack of symmetry in the model frames.

3.1. Two building pounding

For the study of pounding effects, a factor μ_{max}/u_{max} is used where μ_{max} and u_{max} are the maximum deflection of the reference frame with and without pounding with adjacent frames, respectively. Thus μ_{max}/u_{max} is the amplification of maximum deflection due to pounding. Fig. 7 shows μ_{max}/u_{max} of the reference frame 1. The results show that pounding not only amplified but also reduced the maximum deflection of the participating structures. Under the most demanding time history, NZDS 2, pounding reduced the maximum deflection in all configurations while for NZDS 1 the amplification was seen only for Case 2. For JDS and EIC time histories, the maximum μ_{max}/u_{max} occurred when the second frame was the most flexible (Case 2). It can be seen that, for a given frame pairing, the direction of ground motion can have a significant impact on μ_{max}/u_{max} . For example, μ_{max}/u_{max} for Case 6 under JDS 1 increased almost 20% when the ground motion direction was reversed.

The experimental results for reference frame 1 agree with the previous numerical studies that μ_{max}/u_{max} of reference frame 1 is highest when the other frame was most flexible (Case 2) and least

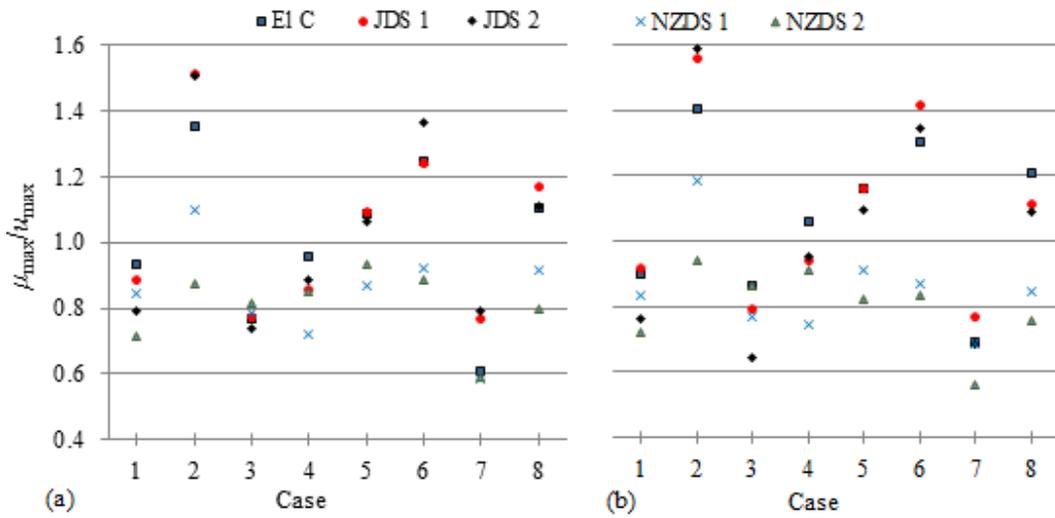


Fig. 7 μ_{\max}/u_{\max} of the reference frame 1 due to two building pounding under five ground excitations: (a) Positive and (b) negative directions

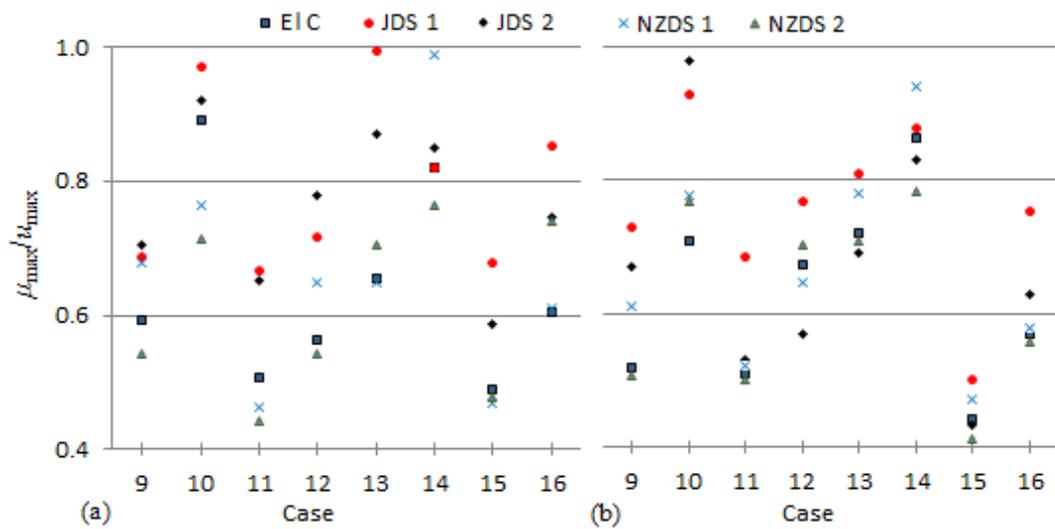


Fig. 8 μ_{\max}/u_{\max} of the reference frame 2 due to two building pounding under five ground excitation: (a) Positive and (b) negative direction

when the other frame was most stiff (Case 7). When both frames have the same stiffness, amplification increased with increase in mass (Cases 3 to 6). For the second frame of similar mass the amplification decreased with stiffness (cases 2, 6 and 8). μ_{\max}/u_{\max} was consistently less than one when T_1/T_2 was greater than 0.8 (Cases 1, 4 and 7). Some pounding was observed even when the second frame had the same mass and stiffness as the reference frame. It could be due to some slight difference in natural frequency even though every effort was made to keep the properties identical. Such pounding caused reduction in maximum displacement under all ground motions considered.

The results show that the displacement amplification due to pounding depends more on the fundamental periods of the two structures than on the mass. In Cases 6 and 8 the mass of the second frame was equal but T_2/T_1 in Case 6 was higher and so was μ_{\max}/u_{\max} . The displacement amplification was similar in Cases 5 and 8 which have second frames with different masses but almost equal fundamental periods.

Fig. 8 shows that the reference frame 2, for Cases 9 to 16, underwent reduction in displacement in all cases of TBP. The reduction was more for larger difference in period. For Case 10, where the second frame was more flexible, maximum displacement was reduced in both frames. Similar to the reference frame 1, some poundings were observed in Case 14 even though the two frames are nearly identical. The μ_{\max}/u_{\max} values also seemed to be affected by the u_{\max} of the frame. For instance, the deflection of reference frame 1 under El-Centro loading was similar to JDS earthquakes, and the μ_{\max}/u_{\max} values from the three ground motions were similar. While for reference frame 2, the u_{\max} and μ_{\max}/u_{\max} under EIC were similar to that produced by NZDS ground motions.

3.2. Row building pounding

The displacement amplification in RBP configurations is shown in Fig. 9. The results are presented only for the positive ground motion. It was observed that the displacement amplification was always greater than TBP for $T_1/T_2 < 1$ (Cases 1, 2, 4, 5, 6 and 8). The maximum increase is in Case 2, the μ_{\max}/u_{\max} of reference frame 1 under increased from 1.51 to 2.03. Similarly, when the reference frame was flexible than the adjacent frames (for instance Cases 9 and 11), the maximum deflection of reference frame was even more reduced in RBP (Fig. 9(b)) than in TBP configurations (Fig. 8).

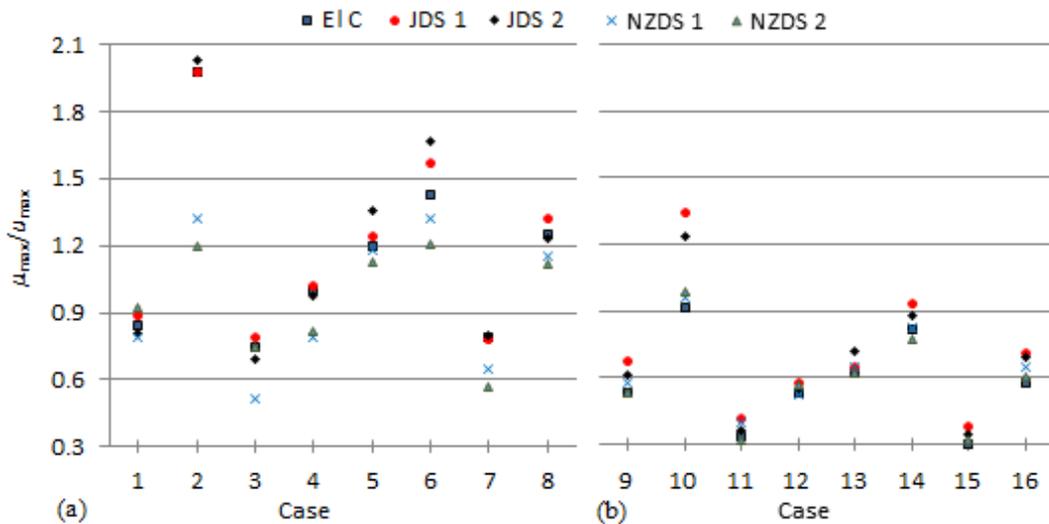


Fig. 9 μ_{\max}/u_{\max} due to row building pounding, with the reference frame in the middle: (a) Reference frame 1 and (b) reference frame 2

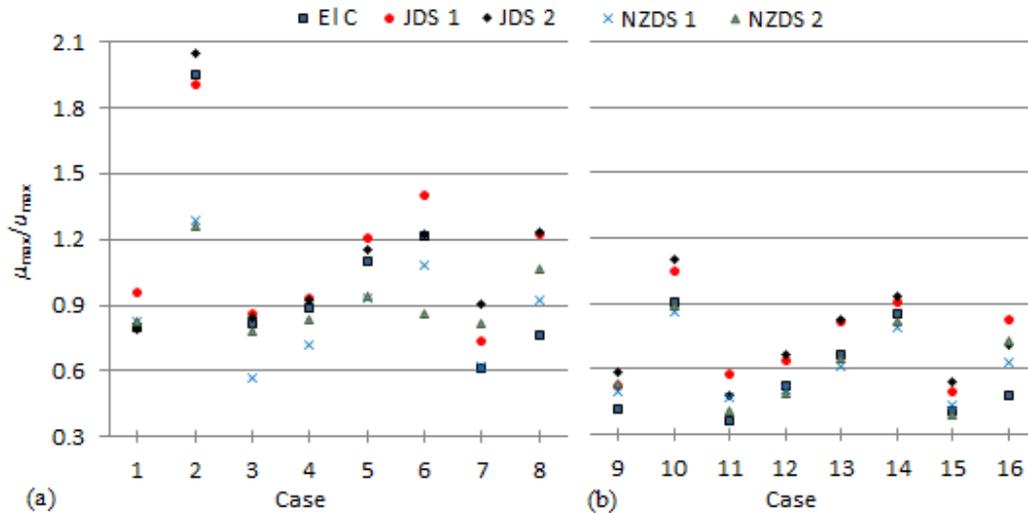


Fig. 10 μ_{max}/u_{max} due to end building pounding: (a) Reference frame 1 and (b) reference frame 2

3.3 End building pounding

Fig. 10 shows the displacement amplification of both reference frames due to EBP. It was observed that, except in a few isolated cases, the amplifications were similar to RBP. Thus pounding of three frame in a row seems to be always more severe for a stiffer structure irrespective of its position in the row. Similarly, the most flexible structure always had reduced displacement.

The displacement time history of reference frame 1 pounding against the most flexible frame (Case 2), under JDS1 ground motion in the three different configurations is presented in Fig. 11(a). The displacement of the reference frame was skewed to the positive in TBP and EBP but it was almost symmetric in RBP. The second frame, which was at the left end in all tests, had negative skew in all cases. The third frame also had comparable maximum deflection in both EBP and RBP. Even though the maximum deflections of the frame are similar in RBP and EBP, the frames attained higher peaks more often when they were placed at the end. The identical frames at the both ends in RBP had considerably different displacements (see the 2nd row, middle result and last row, 1st result of Fig. 11). The maximum displacement of flexible frame in Case 2 i.e., frame k1m3, was 38 mm without pounding. It can be seen that the pounding reduced the maximum deflection in all the cases, but the reduction was much more pronounced when it was placed at the centre.

The displacement response amplification of the stiffest frame under consideration, k3m0 under NZDS 2 and JDS1 ground motions are presented in Fig. 12. The amplification is high when pounding against reference frame 2 and low against reference frame 1. The pounding response is very low for NZDS ground motions compared to JDS excitations.

It is apparent from the results that pounding of three buildings is intrinsically more hazardous to the stiffer structure than two building pounding. The location of the stiffer structure whether at the end, or in the middle of the adjacent two frames, did not appear to have any bearing on the hazard posed. In many cases, the reference frame 1 suffered more displacement amplification in

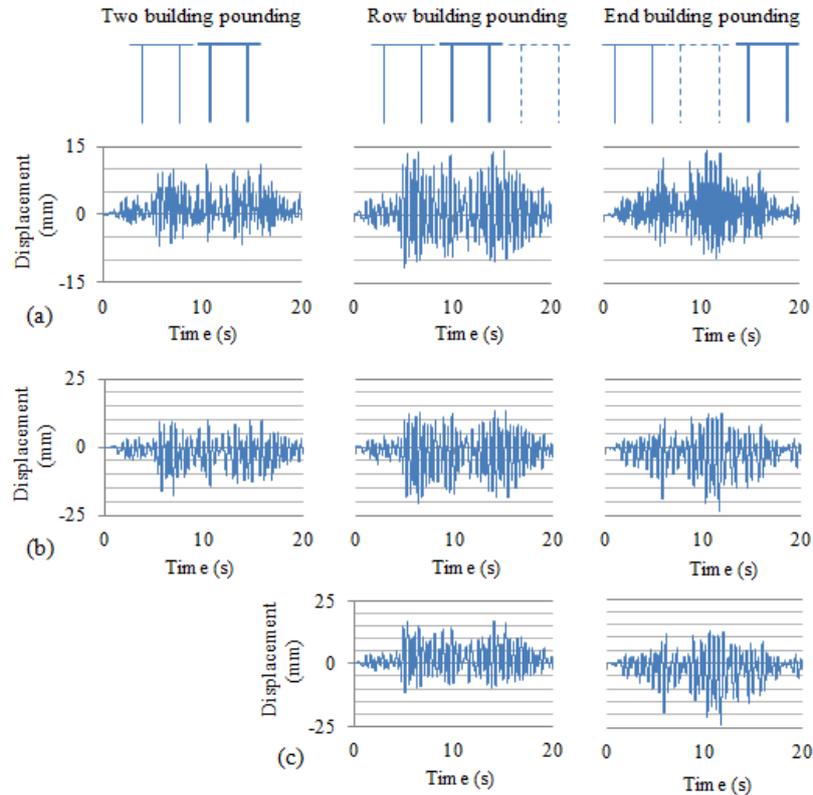


Fig. 11 Displacement time history under JDS loading for Case 2: (a) Reference frame 1 (bold frame in the top sketch), (b) 2nd frame (thin frame) and (c) 3rd frame (dashed frame).

RBP but there were several cases where EBP was more hazardous. Even when two stiffer frames were pounding with the more flexible reference frame, either one could have more amplification, dependent upon the ground motion, or even its direction (Fig. 12). When frames of similar time period suffered pounding, the displacement response was reduced in almost all cases

In all cases the response amplification due to NZDS earthquakes is much smaller than that from JDS or EIC. Except for a few isolated cases, the NZDS excitation induced pounding caused a reduction in maximum displacement. When the displacement was amplified, the amplification factor was always lower than in JDS. Amplification under El Centro seemed to depend upon the non-pounding response of the frame. The reference frame 1 had similar maximum deflection under EIC and JDS ground motion, and the μ_{\max}/u_{\max} values were also similar (Fig. 7), while for reference frame 2 both u_{\max} and μ_{\max}/u_{\max} under EIC ground motions are close to that from NZDS ground motions (Fig. 8). This suggests that, for the frames under consideration, pounding tends to cause more amplification in the frames that have lower u_{\max} , and less amplification when u_{\max} is higher. The behaviour may be related to the increased energy loss for higher velocity impact as observed in past studies (e.g., Jankowski 2008). Since no significant changes in calibration factor of strain gauges was found before and after the pounding experiments and no permanent deformation at the pounding location was observed, this behaviour cannot be attributed to plastic deformations.

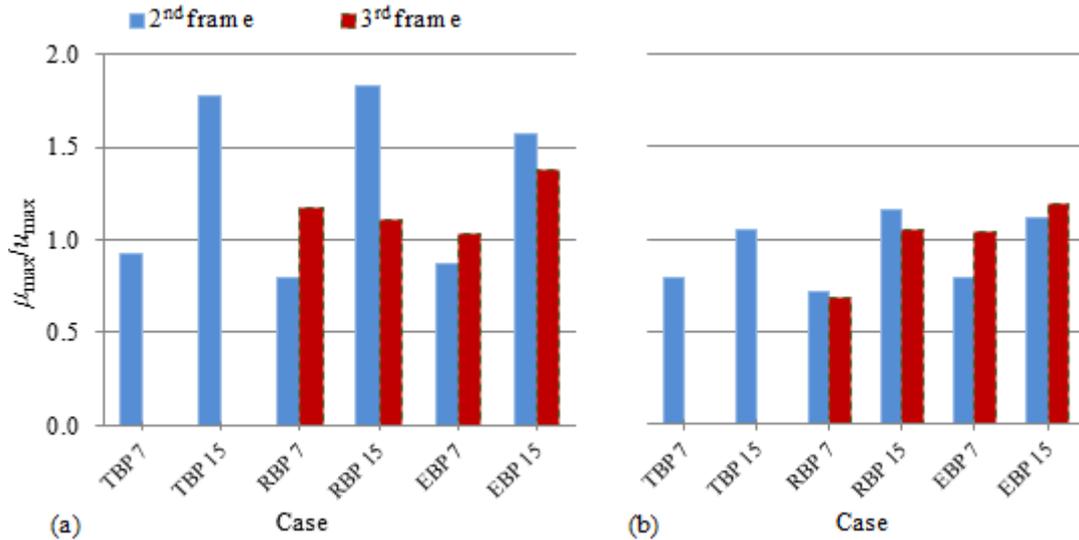


Fig. 12 μ_{\max}/u_{\max} of frame k1m0 in different pounding arrangements: (a) JDS2 and (b) NZDS2 ground motions

4. Conclusions

A parametric shake table investigation of pounding between two buildings and three buildings in a row was conducted. Five different ground motions were applied to steel portal frames with three different stiffness and four different masses. Two of the frames were selected as reference frames, and each was subjected to pounding against eight other frames. Each reference frame was subjected to pounding with two identical frames on either side and with the two identical frames on one side. The eight symmetrical configurations were termed row building pounding and the eight asymmetrical arrangements were for end building pounding. The displacement amplification ratio due to pounding was calculated by dividing their absolute maximum deflection by no-pounding deflection under the same time history. In total 480 tests were performed.

The following conclusions can be drawn:

- Pounding between a row of buildings is always more hazardous to the stiffer building than pounding between two buildings.
- The location of the stiffer building did not seem to have an effect. Thus, contrary to the accepted state of the art based mainly on numerical investigations, a stiff building in the middle of a row is not any safer than that at the end of the row. The relative hazard depends only on the ratio of fundamental period with respect to the adjacent structures of the row.
- When buildings of similar fundamental periods, i.e., with period ratio of 0.8 to 1.2 underwent pounding, the maximum displacement of all the buildings is always reduced.
- For the same frame arrangement, if a time-history produced higher maximum displacement of the stiffer frames without pounding, the amplification due to pounding was lower and vice-versa. This could be related to the observations from impact mechanics that higher velocity of impact can cause more energy loss.

The pounding force was not measured during the test; thus no conclusion can be drawn on the possible damage at the contact locations. With this caveat, the results strongly suggest that the

buildings located in the middle of a row should not be assumed safer than those at the end, even when the floor heights are same.

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