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SURVEY AND ANALYSIS OF BUILDING POUNDING DURING 1989 LOMA PRIETA EARTHQUAKE

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ABSTRACT

The results from a survey of pounding damage caused by the 1989 Loma Prieta earthquake is briefly discussed. Pounding was wide-spread over the San Francisco Bay area, and the total number of pounding incidences in the surveyed areas were more than 200 involving about 500 buildings. Analytical study of actual building pounding cases is also presented. The practical three-dimensional pounding analyses using elastic properties of the members are found to correlate to, and explain actual pounding damage.

KEYWORDS

Pounding; No-Pounding; Loma Prieta Pounding Survey; Damaged Buildings; SLAM-1; SLAM-2 ; Analytical Modeling; Global Response; Local Response; Damage Correlations; Stresses

INTRODUCTION

The collision of buildings, commonly called 'pounding' occurs during an earthquake when, due to their different dynamic characteristics, adjacent buildings vibrate out of phase and there is insufficient separation distance between them. The earthquake that struck Mexico City in 1985 has revealed the fact that pounding was present in over 40% of 330 collapsed or severely damaged buildings surveyed, and in 15% of all cases it led to collapse (Rosenblueth and Meli 1986, Bertero 1986).

This paper first explains the writers' survey and analysis of the pounding incidences during the 1989 Loma Prieta earthquake briefly. Pounding was wide-spread over the San Francisco Bay area, and the total number of pounding incidences in the surveyed areas were more than 200 involving about 500 buildings. Analysis of two actual pounding incidences are also discussed. They indicate that the practical three-dimensional computer program developed by the writers can predict the trends of the stresses and damage of the building members subjected to pounding.

1989 LOMA PRIETA POUNDING SURVEY

The writers surveyed the damage due to pounding in the San Francisco Bay area during the 1989 Loma Prieta Earthquake (Kasai and Maison 1991). This survey was compiled from data provided by: engineers, government officials and engineers, building owners, and block-by-block inspections performed by the writers. The database contains the input of about 90 interested parties and records more than 200 pounding occurrences involving more than 500 structures. Significant pounding was observed at sites over 90 km from the epicenter thus indicating the possible catastrophic damage that may occur during future earthquake having closer epicenters.

Pounding damage patterns were classified as follows: Type-1, major structural damage; Type-2, failure and falling of building appurtenances creating a life-safety hazard; Type-3, loss of building function due to failure of key mechanical, electrical or fire protection systems; and Type-4, architectural and/or minor structural damage. The following are some of the general survey findings and comments:

(1) The majority of reported cases are in urban areas including San Francisco, Oakland, Santa Cruz and Watsonville.

(2) Pounding typically involved multi-story buildings constructed prior to about 1930. They are typically of masonry construction with or without steel skeletal vertical load resisting systems. Very little consideration was given for separation between such buildings to preclude pounding. In many cases, they are in contact with each other.

(3) Fewer modern buildings suffered pounding. In such buildings, relatively larger separations exist. However, it is noted that many modern buildings having expansion joints suffered pounding due to small separations.

(4) There is evidence of correlation between occurrences of pounding and soft foundation soil conditions. This may be attributed to the more intense shaking typically reported for such soil conditions and/or from the possible settlement and rocking of the structures located on soft soils.

(5) Special pounding cases were also observed. They include; severe pounding at unsupported part (e.g., midheight) of columns or walls; pounding promoted by torsional behavior of building; and pounding between the buildings sharing a common wall.

(6) Older buildings that suffered Type-1 damage typically also had Type-2 damage (i.e., falling bricks). Modern buildings that pounded usually had Type-4 damage, and several of them also suffered Type-3 damage. The survey has relative distributions for damage Types 1 and 4 of 21% and 79%, respectively. Many of the present Type-4 damage cases will become damage Types 1, 2, and/or 3 when a future more severe earthquake affects the region. The Type-4 damage cases may be thought of as precursors for the major pounding damage yet to occur.

For detailed information regarding the pounding survey, see Kasai and Maison's paper (1991).

ANALYSIS OF DAMAGED BUILDINGS

The writers have conduced correlative pounding analyses of actual existing buildings damaged during the 1989 Loma Prieta earthquake. The analytical results are correlated with the observed damage of the buildings. Computer pounding analysis programs SLAM-1 and SLAM-2 (Maison and Kasai 1988, 1990) are used. The programs consider the buildings as three-dimensional (3D) multi-degree-of-freedom (MDOF) elastic systems. The SLAM-1 assumes that a building laterally collides with a rigid adjacent building and SLAM-2 considers that both buildings are flexible. The pounding is assumed to occur at a single floor level having a rigid diaphragm. The flexibility of local contact region is simulated by a spring with or without viscous damping. These programs use the stiffness and mass matrix formulated by SUPER-ETABS program (Maison et. al. 1983), and obtain time histories of global response of the buildings under pounding. The member force histories were also obtained by substituting back the global response histories into SUPER-ETABS. The SLAM program is found to simulate well the actual pounding responses of adjacent multistory building models tested using a shaking table (Filiatrault et al. 1995).

10-STORY BUILDING AND MASSIVE 5-STORY BUILDING.

Buildings and Analytical Modeling. Pounding between 10-story and 5-story buildings located in downtown San Francisco caused severe damage of the 10-story building. The damage of the 5-story building was of minor nature. Fig. 1 shows typical framing plans and elevations of the buildings. Pounding occurred in the vicinity of the 6th level of 10-story building as evidence by the observed local damage (Fig. 2). The 10-story building consists of 13 inches thick exterior masonry wall, with window openings as well as both exterior and interior steel moment resisting frames involving built-up columns. The 5-story building has moment resisting concrete frames and was retrofitted by adding steel concentric braces along three sides and a concrete shear wall on the fourth side of an atrium (Fig. 1). The 5-story building is very massive due to the heavy weight per unit area of the floors and very large plan dimensions compared to the 10-story building. It has total weight of 18,500 kips which is 4 times the weight of the 10-story building. The buildings are separated by a gap of 1.0 to 1.5 inches. The pounding location is eccentric with respect to the mass centers of the buildings (Fig. 1). As shown in Fig. 2, the masonry piers along south elevation of 10-story building developed large one-directional diagonal shear cracks above the pounding level.

Dynamic 3D-analyses are performed using SLAM-2. Determination of masonry elastic modulus involved a variety of trial analyses to determine its sensitivity on the analytical responses. The damping ratio was set to 5%. Local contact stiffness (Fig. 1) of 15,000 k/in was based on the in-plane stiffness of the floor slab in the vicinity of the contact point. The Loma Prieta ground motion measured at the basement of an 18-story building located in the vicinity of the study buildings is used for analysis, and its peak acceleration is about 0.15g in both X (i.e. east-west) direction and Y (i.e. north-south) direction (see Fig. 1).

The analytically obtained dominant vibration periods of 10-story building in X, Y, and rotational directions are 1.08, 0.58, and 0.59 seconds, and those of 5-story building 0.53, 0.33, and 0.69 seconds, respectively. Additional analyses were also conducted by hypothesizing that the 5-story building had not been retrofitted prior to the Loma Prieta event. Removing the retrofit steel braces and shear wall from the original 5-story building model, dominant vibration periods for the non-retrofitted 5-story building model in X, Y, and rotational directions are 1.00, 0.93, and 1.16 seconds, respectively. Thus, the lateral stiffnesses of the original building in X and Y directions, respectively, are only about 0.28 and 0.13 times those of the retrofitted building.

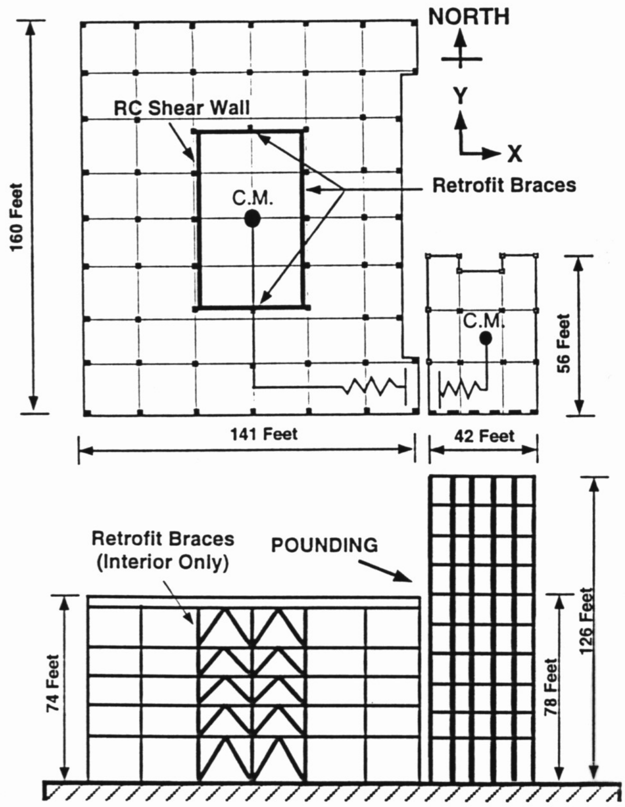


Fig. 1. Typical Framing Plans and Elevations of 10 and 5 Story Buildings.

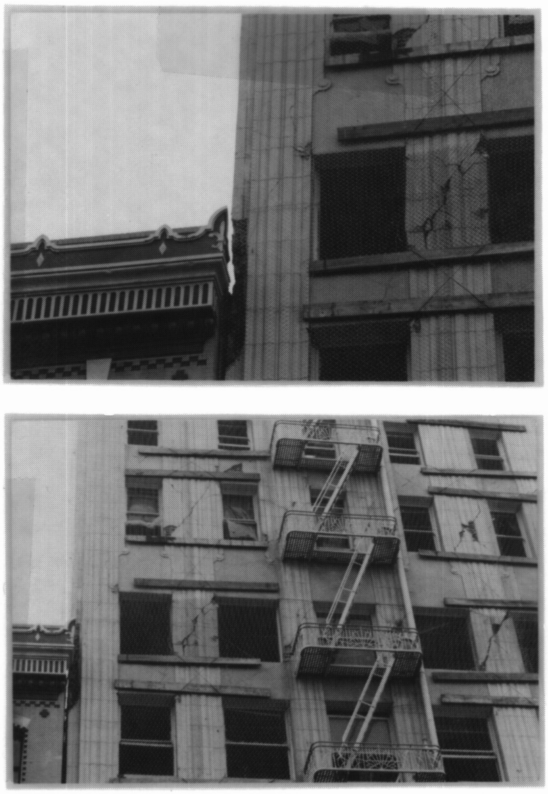


Fig. 2. Damage of Contact Region and Masonry Pier on South Face of 10-Story Building.

Global Responses. Fig. 3 plots X-direction shear force and torque for the 10-story building during the most severe impact with the retrofitted 5-story building. Note that positive torque is denoted as counter-clockwise direction. Before pounding the shear force and torque distribution varies smoothly along height. Pounding creates impact forces at the pounding level, making the shear and torque distribution highly non-uniform. Fig. 4 plots X-shear and torque time histories of 10-story building. Analysis results assuming no pounding are also shown for comparison. The sharp changes in shear and torque occur due to the impact forces at the pounding level. The plots indicate at least 5 major impacts during the earthquake, as well as much higher shear and torques compared to no-pounding case. Note, however, that except for the instances of pounding the shear and torque histories are similar to those of the no-pounding case.

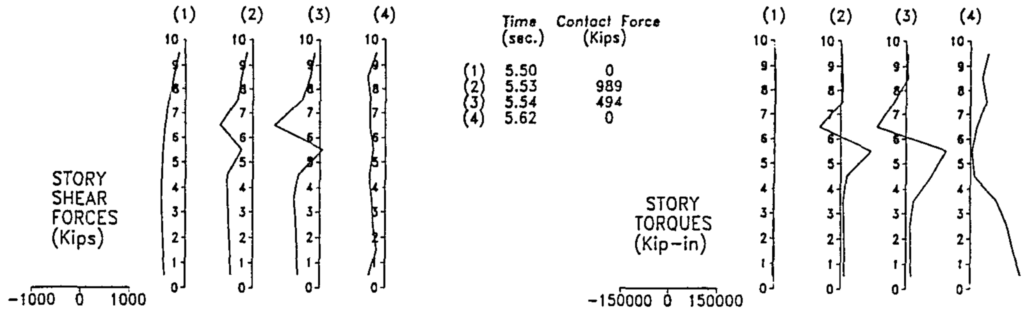


Fig. 3. X-direction Shear Force and Torque of 10-Story Building During the Most Severe Impact. (Pounding Against Retrofitted 5-Story Building)

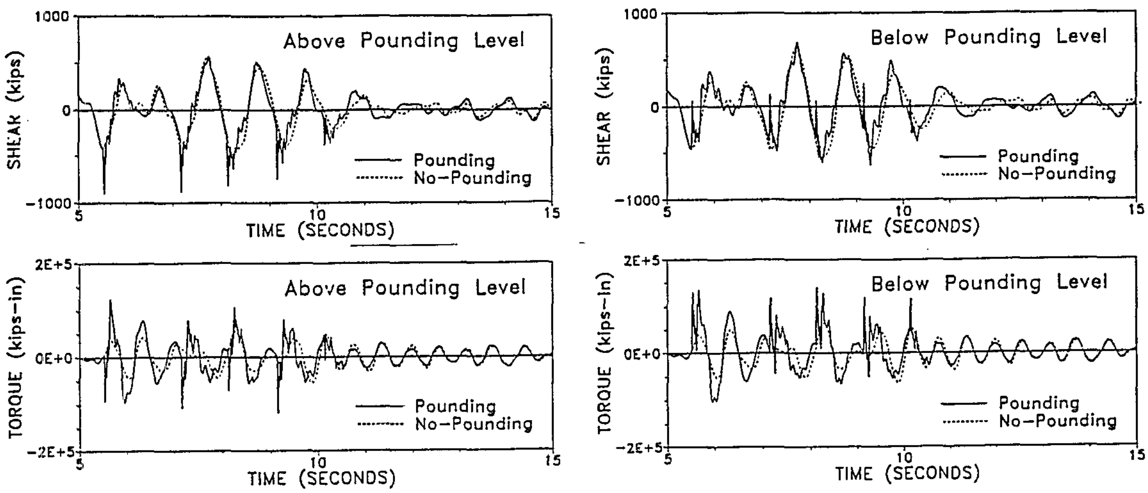


Fig. 4. X-direction Shear Force and Torque Time Histories for the 10-Story Building. (Pounding Against Retrofitted 5-Story Building)

Fig. 5 plots the envelops of the story X-displacement, X-shear, X-overturning moment, and torque of the buildings. Pounding and no-pounding cases are plotted considering either of the situations where the 5-story building is retrofitted or non-retrofitted. When 5-story building is retrofitted, pounding leads to slightly less maximum story displacements of both buildings compared with no-pounding case. However, when 5-story building is non-retrofitted, pounding produces displacements 30% less for the 5-story building and 70% more for the 10-story building compared with no-pounding case. Note that no-pounding displacements of the non-retrofitted 5-story building are about 3.6 times that of the retrofitted building.

When the 5-story building is retrofitted, pounding produces 100% to 200% more negative shear in the 10-story building at its story levels above the pounding location, compared with no-pounding case. Remarkably, when 5-story building is non-retrofitted, pounding produces 300% to 500% more negative shear of 10-story building compared with no-pounding case. Further, the torque of 10-story building due to pounding is larger than no-pounding case, and becomes significant when the 5-story building is non-retrofitted. Note also the increased pounding positive torque of the 10-story building below the pounding location. It is clear that pounding resulted in larger shear, overturning moment and torque of the 10-story building. Retrofitting of 5-story building reduced the seismic forces on 10-story building, thus it helped in reducing its further damage.

Local Responses and Correlations. Fig. 6 plots the peak shear stresses for damaged masonry pier of the 10-story building. The pier is on the south face, and is the second one from the west edge. This masonry pier as well as the fourth one from west edge do not involve steel columns, and they showed similar stresses, hence the following descriptions apply to these two piers. Additional plots depict the stresses obtained by suppressing the effect of global story rotation (i.e. twisting). The distribution of shear stress is similar to that of the story shear (Fig. 5). Pounding against retrofitted 5-story building increase negative shear stress above the pounding level, and the stress appears to be higher than the failure shear stress of 75 psi suggested by Freeman (1991). The large negative stress is also consistent with the one-directional diagonal cracks observed above the pounding level (Fig. 2).

It should be noted that the stress when suppressing the twisting effect, is smoothly distributed over the story height (Fig. 6). Thus, the twisting promoted the irregular distribution of the shear stresses. As shown earlier in Fig. 3, at the instance of pounding the negative torque developed above the pounding level increased the negative shear stress therein, and the positive torque below the pounding level decreased the negative shear stress therein. Note, however, that when the 5-story building is non-retrofitted, the above mentioned effect of torque is not significant in spite of the large positive torque developed (Fig. 3). This is because such large positive torque did not occur at the same time as the large negative story shear. The significant increase in the story shear directly resulted in extremely large masonry elastic stresses of about 300 psi, 4 times the suggested failure stress.

Although not shown, study was conducted on the four other south face piers having steel built-up columns. Their masonry shear stresses are about the half of those of the piers discussed above, consistent with only the slight damage of the piers observed. The east edge pier developed the cracks of a different pattern, and it could be due to the combined shear and tension stresses that must have increased due to pounding, as is evident from the overturning moment plots in Fig. 3. Unlike the south face, the north face of the building showed scattered cracks throughout the height. This can be explained by combining the shear stresses created by the story shear and torque in a similar way as explained above. Further analyses results appear to agree well with the observed damage of the 10-story building. They explain the complex mechanism of the pounding as well as its effect on member stresses of the two buildings. The elastic 3D-analysis of the buildings provides important information on the concentration of damage that can develop due to pounding. If the 5-story building had not been stiffened prior to the Loma Prieta event, the consequence of pounding could have been much more severe than the actual damage.

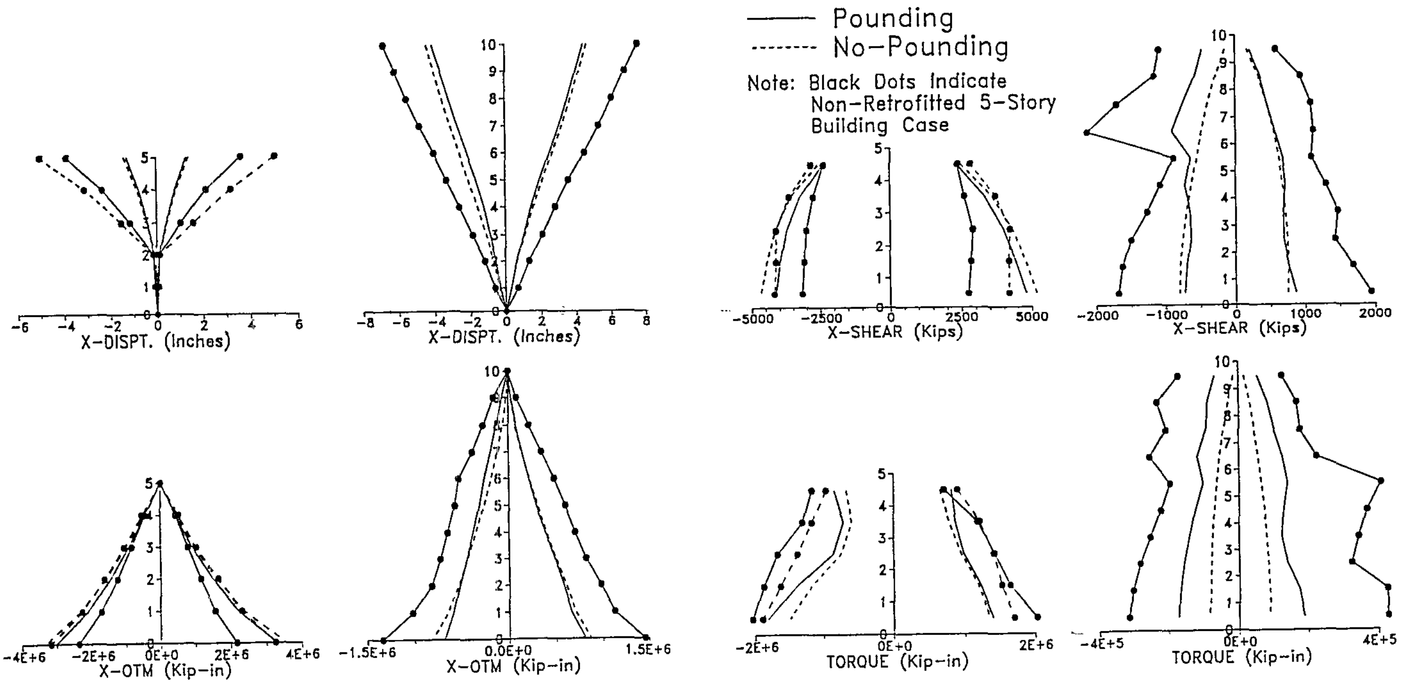


Fig. 5. Envelops of X-Displacement, X-Shear, X-Overturning Moment, and Torque of 10 and 5-Story Buildings.

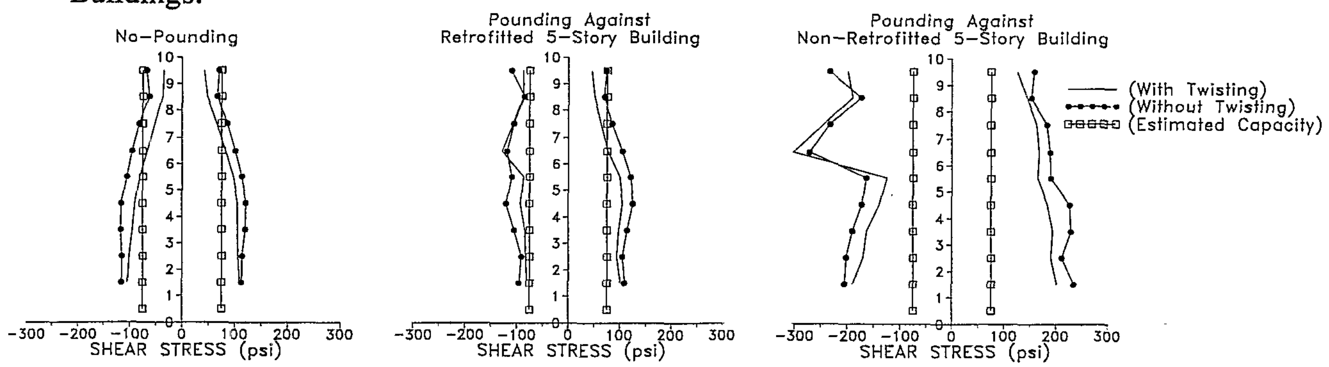


Fig. 6. Envelop of Shear Stresses for Damaged Masonry Pier of 10-Story Building.

IRREGULAR 7-STORY BUILDING AND 3-STORY BUILDING.

Buildings and Analytical Modeling. This case study investigates a 7-story building located in Oakland. The building is on a street corner and is adjacent to a relatively wide 3-story building. There is another 7-story building at the far corner of the street. Both 7 story buildings pounded at the both sides of 3-story building, as evidenced by the local contact damage at the 3rd level of the buildings. The 7-story building at the west side (Fig. 7) has suffered the most severe structural and non-structural damage, and its pounding response will be discussed herein. The 7-story building consists of moment resisting reinforced concrete frames and reinforced concrete shear walls at many locations. The concrete columns have the spiral reinforcements with small pitch, and they are believed to be ductile. Fig. 7 shows isometric view from north-east as well as typical floor framing plan showing the columns, walls, and beams. The building has a rectangular base plan and two setbacks in the plan towards north. One minor setback exists at mezzanine level and another major setback at the second floor level (named as 2nd story level), as a result of which the building takes the form of inverted "T" above the 2nd story level.

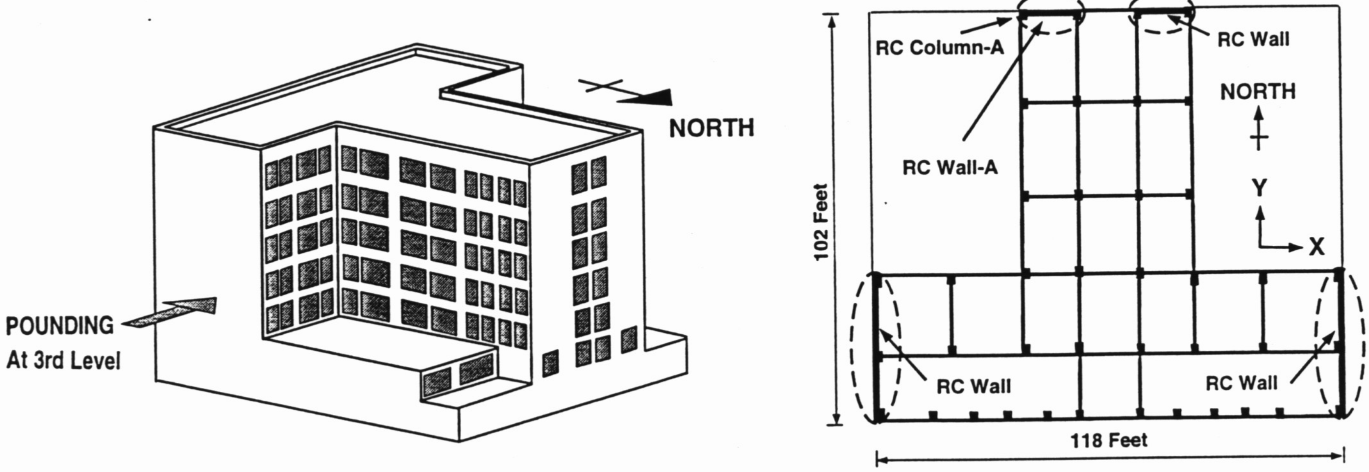
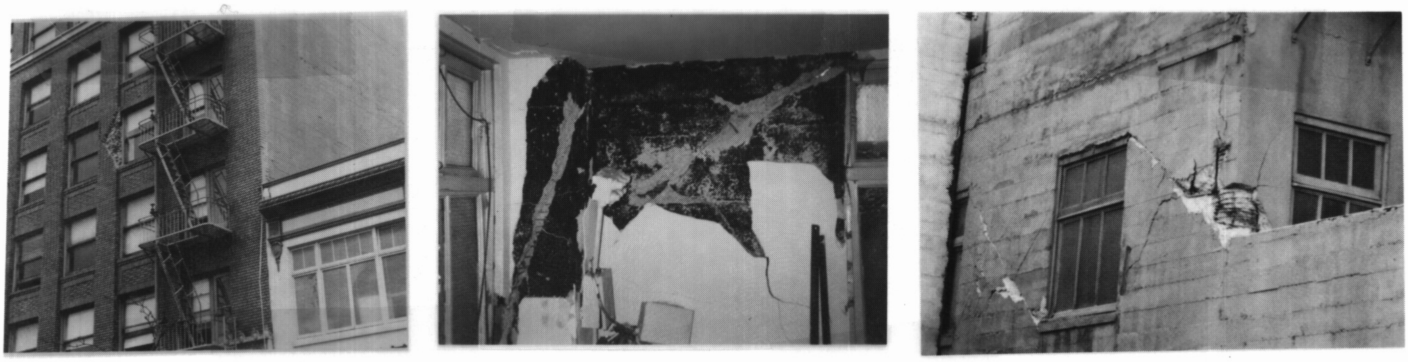


Fig. 7. Isometric View and Typical Floor Framing Plan of 7-Story Building.

Fig. 8 shows the pictures of the damaged elements, and their locations are shown in the typical floor plan (Fig. 7). Fig. 8(a) shows wide classical diagonal shear cracks developed in brick facade above pounding level along the south elevation of the building. The bricks are not structurally integrated into the structural framing. Fig. 8(b) shows the interior view of the damaged wall-A located at the stem of the "T" having the north frame running in east-west direction. The wall developed wide classical one-directional diagonal shear crack below 2nd floor level. The direction of the crack is consistent with the analytically obtained direction of shear stress, as will be explained. Column-A at the west edge of the wall-A suffered severe axial failure, which will be analytically interpreted. The column also failed in shear after the wall-A failed and lost shear resistance.



(a) (b) (c)

Fig. 8. Damage of 7- Story Building: (a) Brick Facade above Pounding Level, (b) Wall-A and Column-A (Inside View), and (c) Wall-A and Column-A (Exterior View).

In the analysis, the adjacent 3-story building is assumed to be rigid. This is because its motion is restricted by the 7-story buildings at its both sides. Series of analyses were conducted by assuming a variety of reductions in the concrete elastic modulus. The analytical model to be discussed in this paper uses 70% of the elastic modulus for most of the members and as low as 20% for the members that are analytically identified to be severely damaged. The gross section properties were used in determining the sectional properties. The fundamental vibration period in X-direction and Y-direction are 0.94 and 0.51 seconds. Damping of 5%, and contact stiffness of 50,000 k/in are used. 3D-dynamic analysis is performed using the SLAM-1 program, which considers the pounding of a flexible structure with adjacent rigid structure. The Loma Prieta ground motion recorded at a 2-story building in the vicinity of the building is used. The peak acceleration is 0.25g in X- (east-west) and 0.20g in Y- (north-south) directions, respectively.

<u>Global Responses.</u> Fig. 9 plots the envelop of X-direction gross responses along with the torque for the 7-story building for both the pounding and no-pounding cases. Pounding slightly amplifies positive shear and overturning moment above the pounding level, and significantly reduces them below the level. Pounding also reduces both negative shear and overturning moment throughout the story height. Positive torque acting in the counter-clockwise direction is significantly reduced below pounding level due to the impact force imposing clockwise torque to the lower levels.

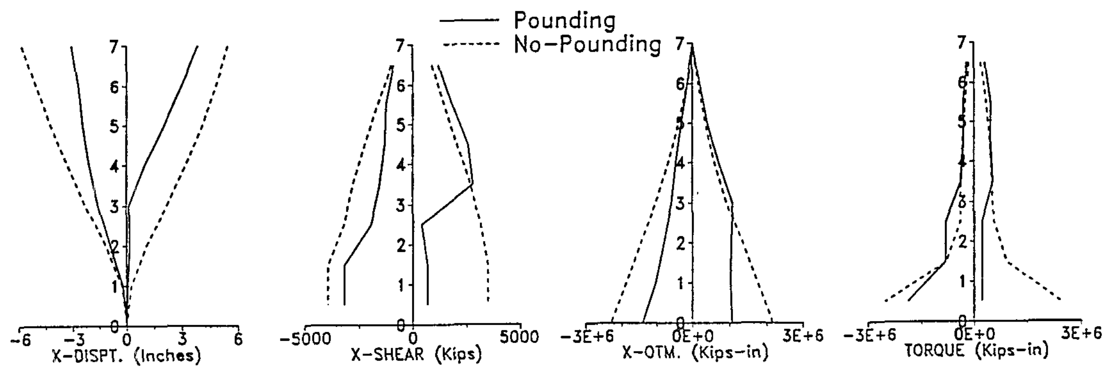


Fig. 9. Envelops of X-Displacement, X-Shear, X-Overturning Moment, and Torque of 7-Story Building.

<u>Local Responses and Correlations.</u> Fig. 10(a) plots the shear stress envelop for wall-A due to pounding and no-pounding. It also shows the ultimate stress of the reinforced concrete wall, taken as $10(f'_c)^{1/2}$, where f'_c (psi) is the compressive strength of the concrete (ACI 1989). Note that no-pounding case shows very large positive and negative shear stresses between the 1st and 2nd levels, and these stresses exceed the ultimate stress. This is due to the large window opening made at the region (see Fig. 7). In contrast, pounding case shows significantly reduced positive stresses of the wall below pounding level, similar to the distribution of story shear. This is especially beneficial for reducing the shear stresses at the region having the large openings (Fig. 7). On the other hand, negative shear stresses are still significant in pounding case, although they are smaller than those in no-pounding case. Thus, in the pounding case the positive shear stresses are well below the ultimate stress, but negative stress between the 1st and 2nd levels exceed the ultimate stress. This is consistent with the pattern and direction of severe crack developed in wall-A (Figs. 8(b) and (c)). Effect of torsion on the stresses of wall-A appears to be negligible (Fig. 10(a)). Note that, if the adjacent 3-story building had not existed and pounding had not occurred, the damage to the wall-A could have been much more serious as evidenced by the large positive and negative shear stresses shown in Fig. 10(a).

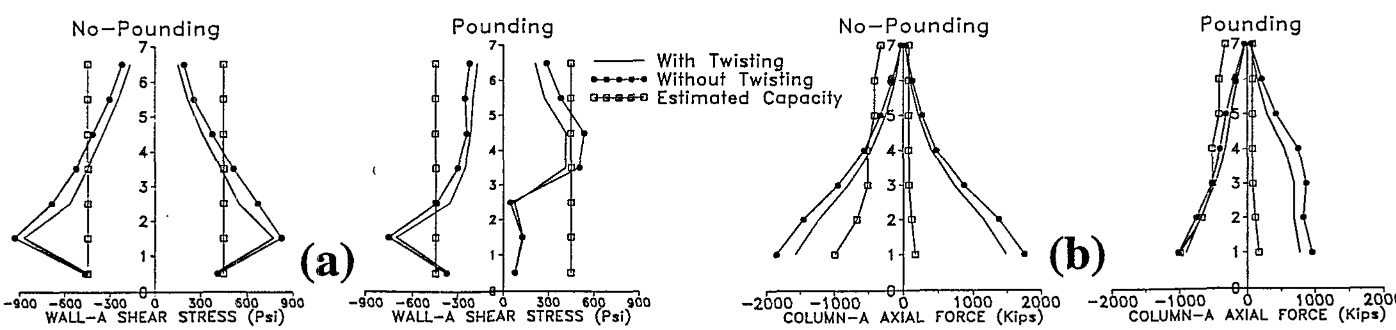


Fig. 10. (a) Shear Stress Envelop for Wall-A, and (b) Axial Force Envelop for Column-A.

As interpreted by the inspectors of this building, the failure of the column-A must have occurred soon after the wall-A lost its resistance against the negative shear. The failure of this column is complex, as suggested by the shear cracks developed in interior side of the column and concrete crushing and longitudinal reinforcing bar buckling (Fig. 8(c)). Substantial bending moment, shear, and axial force must have developed in this column. Fig. 10(b) plots axial force envelop for the column-A. It also shows the tensile capacity based on longitudinal reinforcing bar strength, and compression capacity based on the compression strengths of the bars and concrete (ACI 1989). Pounding resulted in significant reduction of both tension and compression axial forces below pounding level. But the tension axial forces are still much higher compared to the column capacity, suggesting significant yielding of the bars. Also, the compression force is almost same as the compression strength. These indicate that the failure of the concrete column was inevitable during the Loma Prieta event. Note, however, that if adjacent buildings had not existed, column-A could have suffered much more significant damage due to much larger compressive and tensile axial forces (Fig. 10(b)) as well as more significant shear and bending demands due to the damage of the adjacent wall-A that could have been more seriously damaged under the no-pounding situation (Fig. 10(a)).

These analyses indicate that, in contrast to the pounding of 10-story building discussed in the previous section, pounding of the 7-story building helped in reducing its damage as compared to the no-pounding case.

CONCLUSIONS

(1) The practical three-dimensional pounding analysis computer programs using elastic properties of members can explain both local and global response of adjacent buildings studied herein, and they can be employed to perform case-by-case analyses of other pounding situations.

(2) Damage of 10-story building was supplemented due to pounding against the adjacent massive and retrofitted 5-story building. Above the pounding level, shear stress in a particular direction significantly increased. The damage could have been even more significant if the 5-story building had not been retrofitted.

(3) Pounding helped in reducing the damage of the 7-story building of an irregular configuration. Relatively low pounding location lead to minor increase of member stresses above the pounding level, and significantly reduced stresses below the pounding level.

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