The influence of pounding on member demands in low rise buildings

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ABSTRACT: While pounding between insufficiently separated buildings is commonly acknowledged to occur during earthquakes, very little information on pounding's loading effects are currently available. This paper presents a numerical study of two Wellington buildings with various separations that result in pounding. Both buildings are modelled as 1960's, three storey buildings with concrete moment resisting frames. Member shear and ductility demands are recorded and compared to each building's demands when the buildings are sufficiently separated to prevent pounding. Additionally, the collision force at the point of contact is compared between models. The effects of soil structure interaction are also investigated for pounding models. Pounding is found to increase or decrease ductilities and interstorey shears depending on the specific configuration. Interstorey shears induced by pounding are found to exceed 'no pounding' configurations by up to 35%. The implications and likely loadings due to pounding for low rise Wellington buildings are also discussed and tentatively quantified.

1 INTRODUCTION

While building pounding is known to occur during earthquakes (Bertero 1986; Kasai and Maison 1997; Cole et al. 2011b), few effective tools are available for the prediction of pounding-caused building damage (Cole et al. 2010). Recent studies (Cole et al. 2011a) have shown that detailed modelling of colliding diaphragms is necessary when collisions occur between adjacent building diaphragms. Damage resulting from building collision can be categorised into local damage and global damage. Local damage occurs in the area immediately surrounding the point of impact on each building and is directly related to the collision force. Global damage can occur throughout a building and is a result of the collision's momentum transfer, which changes the velocity of both buildings.

In this paper an example building configuration, with properties similar to many New Zealand low-rise buildings, is modelled in detail. This building model is used to illustrate how pounding may affect low-rise buildings in terms of local and global building damage. The influence of building separation is investigated, and the effects of soil-structure interaction are presented for the considered model. It is hoped that the presented damage characteristics will provide a useful contextual background for design engineers that wish to mitigate pounding-related damage in similar building configurations.

2 MODELLING DESCRIPTION

2.1 Analysis properties

Building modelling was performed using *Ruaumoko3D* (Carr 2007), a non-linear time history programme developed at the University of Canterbury. Uniform modal damping (Wilson and Penzien 1972) was adopted in all modelling. This is because the more commonly adopted Raleigh damping causes high levels of damping in the high frequency modes. In these analyses, the high frequency modes are important because they are excited during collision. Pounding analyses are also sensitive to the adopted integration time step. In these analyses, the time step is set at 10^{-4} seconds. This was determined by reducing the time step until the equations of motion conserved the total model energy over the duration of the ground motion. At a time step of 10^{-4} seconds, an acceptable energy loss of 1.4% was recorded. Building beams are modelled using Modified Takeda law with $\alpha = 0.5$ and $\beta = 0$ as recommended by Dong (2003), with the post elastic stiffness set at 1% of the initial stiffness. Columns are modelled using the revised Takeda hysteresis, with the cracking moment set at 60% of yield moment, and the post elastic stiffness set in the same manner as the beam hysteresis. Curvature ductility limits have been adopted from Walker and Dhakal (2009), which recommends a beam curvature limit of $\mu_\phi = 9$ and column curvature limit of $\mu_\phi = 12$ for limited ductile concrete members.

2.2 Building selection

Two 1960's three-storey buildings were selected for modelling. These buildings are both currently in use in Wellington. The buildings can be characterised as buildings with favourable geometry (such as low total height, no major building irregularities) but primitive earthquake restraint systems (masonry infilled panels). The buildings were selected because they represent a common pounding risk observed in New Zealand towns and cities. In reality, the two modelled buildings are not actually located adjacent to one another, so the results presented here represent a strictly theoretical risk.

While the existing buildings selected for modelling have infilled panels, the panels are assumed to contribute negligible stiffness to each building's response. This assumption is made because buildings of this era typically cast the concrete columns prior to building the infill masonry panels. This decision resulted in gaps between the masonry and the surrounding column, which isolates the masonry panels. The presented models are thus assumed to act as reinforced concrete frame structures. It would be possible to develop much more detailed panel models which allow the activation of masonry panels once the specified gap is closed; however such an approach would require lengthy development time that is not currently available to the authors.

The adopted building configuration is presented in Figure 1. Soil-structure interaction is also modelled, but omitted from the figure for clarity. Note that while some adjacent floors are not at exactly the same height, a completely horizontal collision force is assumed. This is because the differences in floor height are less than the depth of the perimeter beams of these walls.

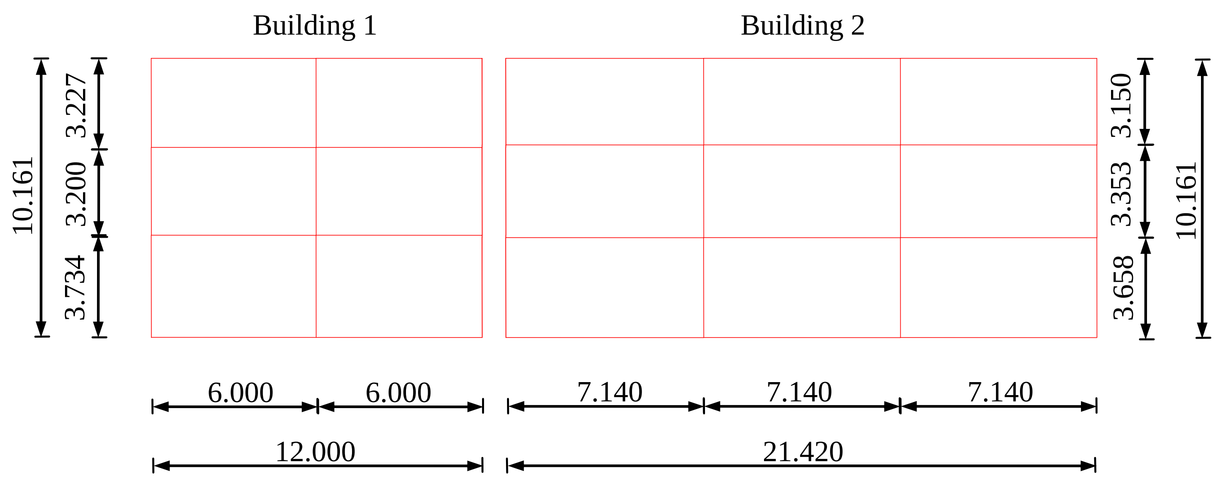

Figure 1 Analysed model configuration (Elevation)

Soil structure interaction is simulated using the fundamental lumped parameter model (Wolf 1994). This model simulates the movement of a rigid disk on an elastic halfspace using viscous dampers, elastic springs and additional masses. Soil properties were obtained through from bore logs of one of the selected buildings, and conversations with practising geotechnical engineers from Beca, Carter, Hollings and Ferner Ltd, who are familiar with Wellington soils.

2.3 **Ground motion records**

Three excitation records are used in the pounding analyses; El Centro (Imperial Valley, 1940), Tabas (Iran, 1978), and La Union (Mexico, 1985). These particular records are adopted because they are considered to possess similar characteristics to a major rupture of the Wellington fault. The records have been scaled according to NZS1170.5:2004. This scaling requires selection of a soil class. The soil class (type C) was selected based upon the soil type of Te Aro, a central Wellington suburb with many buildings with pounding potential. The ground motions are scaled for the 1/25 year event. This corresponds to a 'service level' motion in the current New Zealand standard (NZS1170.5 2004) and is roughly equivalent to the 'ultimate' design criteria when the buildings were constructed in the 1960s (Fenwick and MacRae 2009). In the early 1960s, buildings were constructed using the 1935 New Zealand building code, which used the working stress design method. Fenwick and MacRae converted the working stress method to an equivalent limit state formulation to allow comparison to current New Zealand codes. They found that the 1935 code produced an equivalent horizontal seismic shear of $0.104W_t$, where W_t is the seismic weight of the considered structure. The 1/25 year horizontal seismic shear in the current New Zealand standard is $0.1W_t$.

2.4 **Adjusted parameters**

In the presented analyses, building separation is modelled in 5 mm increments between 0 and 15 mm. Additionally, a building separation sufficient to prevent pounding is performed to provide the 'no pounding' response. Each of these separations is modelled for six ground motions: El Centro +, El Centro -, Tabas +, Tabas -, La Union + and La Union -. The +/- designation denotes the direction of the ground motion. Reversing the ground motion changes when the buildings first come into contact and therefore generates a unique response.

3 **RESULTS**

Modelling results are presented in this section. Many presented figures state normalised separations and normalised parameters recorded in the analyses. These normalisations are performed in the following manner:

- building separations are normalised by the building separation required to prevent pounding for the considered record. This separation is found by performing an analysis with a large separation (for example, 10 metres) for each ground motion. The relative separation is then calculated by subtracting Building 1's displacements from Building 2's displacements in each time step. The maximum recorded value of relative separation corresponds to the minimum separation required to prevent pounding.
- individual parameters are normalised by the result obtained from the corresponding analysis without pounding.

Building displacements, global building damage, and local building damage are each considered separately.

3.1 **Displacement sensitivity to pounding**

Figure 2 presents the displacement sensitivity of Level 3 (the roof) of both buildings to pounding. Each building's left and right roof displacement envelope is shown. Rightward movement of Building 1 or leftward movement of Building 2 can cause collision. Each ground motion record is identified in the figure legend. These names are abbreviated from that stated in Section 2.4

The amplifications presented in Figure 2 show the influence of pounding on each building's performance. A value of 1.2 indicates that pounding has increased the considered building's displacements by 20%. The normalised separation indicates initial building separation. When the normalised separation is zero, then the buildings are touching at the beginning of the excitation. When the normalised separation is 1.0, then the buildings have sufficient separation to avoid pounding. The figure shows that both buildings' displacements can be either increased or decreased by considerable margins (+40% and -35%). Building 1's rightward movement is reduced due to the presence of Building 2. However, Building 2's rightward movement frequently increases despite the presence of Building 1. This is a result of the change in dynamic properties of Building 2 due to pounding. The influence of pounding can be observed to be loosely linearly correlated with building separation. As separation increases, displacement amplification (or de-amplification) reduces. While displacement amplification is useful for understanding the pounding process, it does not directly indicate building damage. This is considered in the following sections.

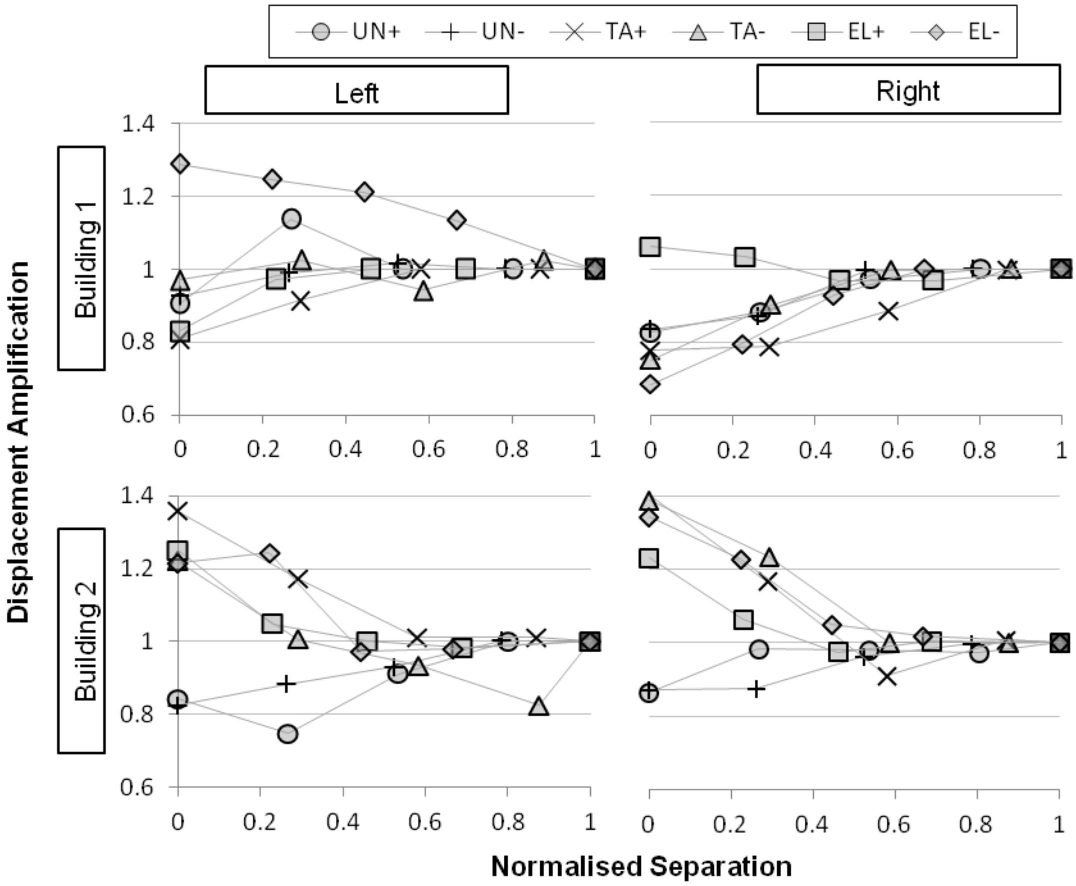

Figure 2 Level 3 displacement envelopes normalised by no contact displacement envelopes

3.2 Global damage sensitivity to pounding

A normalised shear amplification summary of all thirty analyses is presented in Figure 3. Each line presents the maximum interstorey shears for one analysis, normalised by the maximum interstorey shears of the corresponding 'no pounding' analysis. This figure adopts a different format to show the amplification of shear force at all three building levels. The amount of building separation is not identified for each record. Instead, all shear magnifications are shown on a single plot of each building in order to view the range of the results.

The nature of the amplifications differs between the buildings. Building 1's shear amplifications regularly increase with increasing storey; however, Building 2's shear amplifications remain almost constant. The lower amplification at Level 1 of Building 1 is attributed to column yielding at this level, which restricts shear amplification. While the maximum normalised displacement magnifications (Figure 2) are larger than that of the shear magnifications, comparison of averaged values reveals more sensitivity within the shear results. On average, both buildings' roof shears are magnified by 10%. A maximum increase in shear of 35%, and maximum decrease of 10% was recorded in the analyses.

Member curvature ductilities show similar trends to the interstorey shears, but are generally found to be more sensitive to system changes. This sensitivity is attributed to the significantly lower post elastic stiffnesses in each member. Figure 4 presents the maximum ductility observed in all column members during a given record. Beam ductilities present similar trends with a maximum recorded ductility of 3. Note that ductilities less than 1.0 are not recorded in Ruaumoko, so some data points are missing. All ductilities remain within the acceptable capacities stated in Section 2.1. Decreasing building separation generally reduces column ductility demand in Building 1, while usually slightly increasing Building 2's column demand.

Figure 3 Normalised shear demand over each building's height. Mean values indicated with dashed black lines

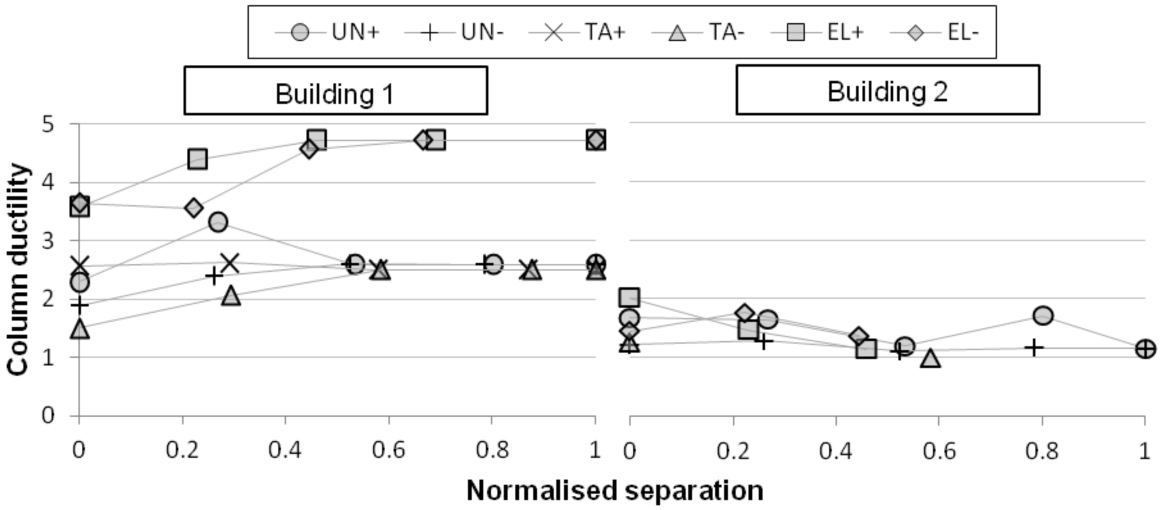

Figure 4 Effect of building separation on maximum column curvature ductility

3.3 Local damage sensitivity to pounding

Figure 5 presents the maximum collision force recorded at each building level during each pounding record. The left section of the figure presents the raw data from these records, while the right section normalises these values. Since collision force cannot be normalised by the non-existent 'no pounding' collision force, values are instead normalised by the collision force recorded with zero building separation. The normalisation of displacements also changes in this figure. Displacement normalisation is performed in the same manner as described in Section 3, however the minimum separation is calculated independently for each floor. Therefore, while a single analysis has only one building separation, it is normalised by three different 'no pounding' separations at the three floor levels. This new form of separation normalisation is included here because the floors' responses are shown to be strikingly similar.

The largest collision forces are not recorded at the third floor of the buildings. This is because the first and second floors are considerably heavier than the roof level. The light weight construction of many low rise buildings' roofs may be a significant mitigating factor for buildings of this type.

The relationship of the normalised collision force is starkly different from that observed with previous parameters. Displacement, shear and ductility amplifications decrease approximately linearly with increasing separation. However, collision force initially increases with increasing separation. This is because increased separation allows the buildings to generate larger relative velocities before collision, which increases the collision force (Cole et al. 2011a). At separations as great as 80% that required to prevent pounding, collision force values are approximately as large as when zero building separation is present.

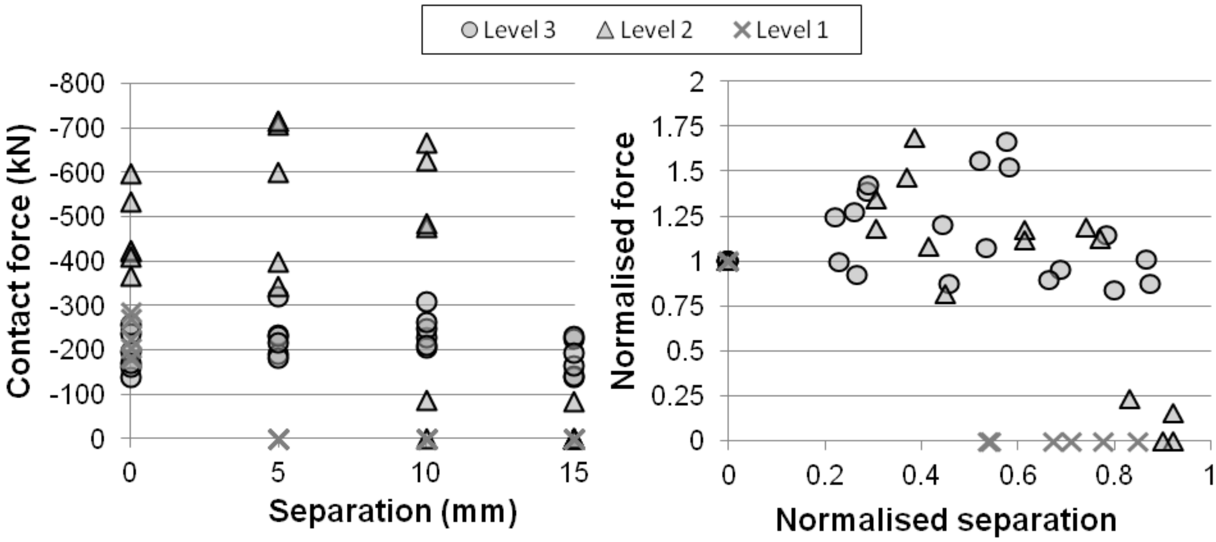

Figure 5 Maximum recorded contact force in terms of building separation. Left: raw data. Right: collision force normalised by the ground motion's maximum collision force at zero separation.

Level 1 and Level 2 report zero maximum collision forces for separations that are less than the minimum separation required for no pounding in the considered ground motion (i.e. less than a normalised separation of 1.0). This is because the buildings' displacement responses change in the pounding models, which sometimes cause less displacement to occur at these lower levels.

3.4 **Influence of Soil Structure Interaction (SSI)**

Finally, the significance of soil structure interaction (SSI) is briefly assessed. To reduce the required computation, only 0 mm, 10 mm and 'no pounding records' were analysed. Figures presented in this section normalise recorded values of the model without SSI (model *NoSSI*) by the corresponding values for the model presented in the previous sections with SSI (model *Default*).

Figure 6 presents the displacement amplification at all three levels when the SSI foundation models are removed. The lower floors' displacements are deamplified more than the higher floors. Removing SSI from the model can reduce the buildings' displacement envelopes by up to 40%. SSI thus has a significant effect on the displacement of the presented model.

Interstorey shears are not presented here due to the space constraint. However, remarkable insensitivity is observed in the interstorey shears to the SSI modelling. Average shear de-amplifications of less than 3% are recorded. This suggests that the additional displacement is primarily due to foundation flexibility, rather than an increase in the buildings' spectral acceleration. A change in spectral acceleration could have been caused by the period shift resulting from the SSI modelling. Yielding of members at either end was found to occur only in isolated columns and is not considered to have significantly contributed to the reported shear insensitivity.

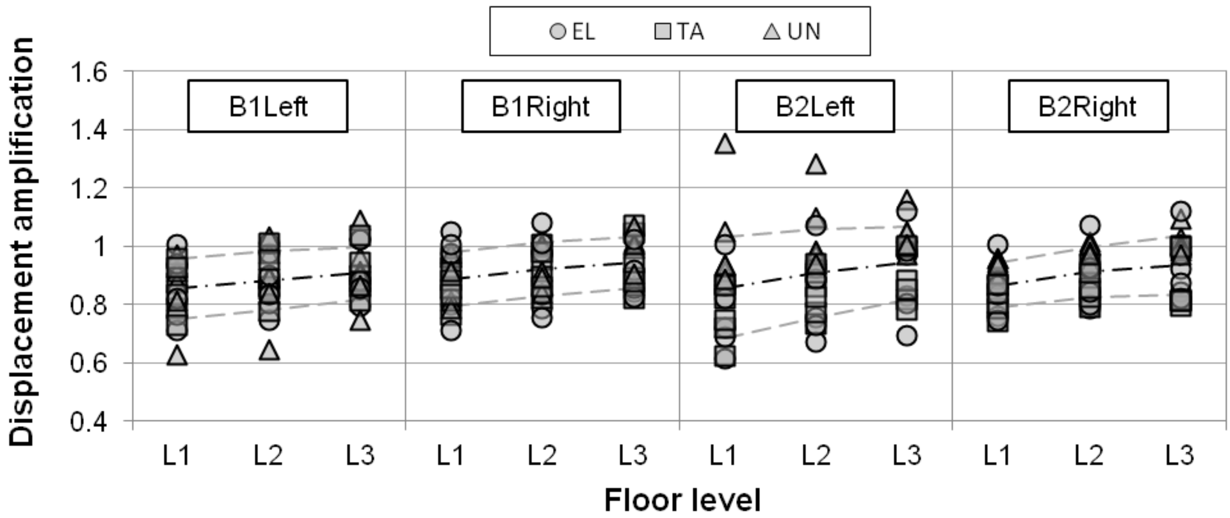

Figure 6 Displacement ratio of *NoSSI*/*Default*. The black dotted line indicates mean values, while grey lines show mean ± one standard deviation.

Curvature ductility amplification (Figure 7) is significantly more sensitive and can be either amplified or deamplified by SSI. In order to obtain sufficient data for this plot, it was necessary to obtain member curvatures for elements that were not yielding. This is because multiple configurations did not yield in the *NoSSI* analyses. Removing SSI has caused a transfer of curvature demand in the building from the columns to the beams. This is attributed again to the foundation flexibility. The ductility amplification between *NoSSI* and *Default* is approximately constant for the 10 mm and the no pounding analyses, but changes significantly for 0 mm separation.

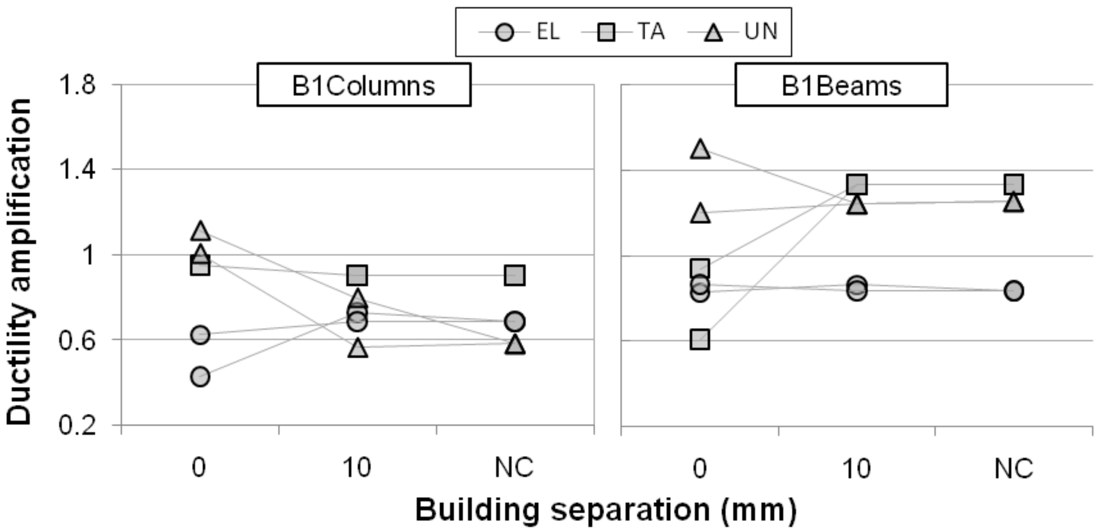

Figure 7 Ductility amplification of *NoSSI/Default.* NC refers to the no contact (or no pounding) analysis

Finally, contact force increases of up to 20% and decreases of up to 40% were also observed. Collision force is observed to be more sensitive to SSI effects than interstorey shear, but less sensitive than the recorded displacements.

4 CONCLUSIONS

The following conclusions are drawn from the analyses performed in this paper. These results have been obtained from only one building configuration. Other building configurations may respond in a substantially different manner those described here.

- Interstorey shears were observed to increase by an average of 10%, and a maximum of 35%, at roof level when pounding occurred. Lower amplifications of shear were observed at lower floor levels. Member ductility demands were also found to increase due to pounding.

- As building separation increases, collision force was observed to increase by up to 70%. At roof level, a separation of 85% of the separation required to avoid any building contact still causes a very similar force magnitude to buildings without any separation.
- The behaviours of collision force and interstorey shear with increasing separation were found to be fundamentally different. If both damage measures are of interest, their responses must be evaluated separately.
- Soil structure interaction has a significant effect on low-rise building pounding response. The effects of SSI must be considered if detailed pounding modelling is to be performed.

5 ACKNOWLEDGEMENTS

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REFERENCES:

Bertero, V. V. (1986). *OBSERVATIONS ON STRUCTURAL POUNDING*. The Mexico Earthquakes—1985: Factors Involved and Lessons Learned, Mexico City, Mexico, ASCE, New York, NY, USA.

Carr, A. J. (2007). *Volume 3: User manual for the 3 Dimensional Version Ruaumoko 3D*. Christchurch, University of Canterbury.

Cole, G., Dhakal, R., Carr, A. and Bull, D. (2011a). An investigation of the effects of mass distribution on pounding structures. *Earthquake Engineering & Structural Dynamics* **40**(6): 641-659.

Cole, G. L., Dhakal, R. P., Carr, A. J. and Bull, D. K. (2010). Building pounding state of the art: Identifying structures vulnerable to pounding damage. *New Zealand Society for Earthquake Engineering Annual Conference (NZSEE 2010)*. Wellington, New Zealand**:** P11.

Cole, G. L., Dhakal, R. P., Carr, A. J. and Bull, D. K. (2011b). Case studies of observed pounding damage during the 2010 Darfield earthquake. *Pacific Conference on Earthquake Engineering (PCEE)*. Auckland, New Zealand**:** paper 173.

Dong, P. (2003). Effect of varying hysteresis models and damage models on damage assessment of r/c structures under standard design level earthquakes obtained using a new scaling method. *Civil Engineering*. Christchurch, University of Canterbury. **PhD**.

Fenwick, R. and MacRae, G. (2009). COMPARISON OF NEW ZEALAND STANDARDS USED FOR SEISMIC DESIGN OF CONCRETE BUILDINGS. *BULLETIN OF THE NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING* **42**(3): 087-203.

Kasai, K. and Maison, B. F. (1997). Building pounding damage during the 1989 Loma Prieta earthquake. *Engineering Structures* **19**(3): 195-207.

NZS1170.5 (2004). Structural Design Actions. *Part 5: Earthquake Actions - New Zealand*. Wellington, Standards New Zealand.

Walker, A. F. and Dhakal, R. P. (2009). Assessment of material strain limits for defining plastic regions in concrete structures. *Bulletin of the NZ Society of Earthquake Engineering* **42**(2): 86-95.

Wilson, E. L. and Penzien, J. (1972). Evaluation of Orthogonal Damping Matrices. *International Journal of Numerical Methods in Engineering* **4**: 5-10.

Wolf, J. P. (1994). *Foundation vibration analysis using simple physical models*. Englewood Cliffs, NJ, PTR Prentice Hall.