Proceedings of the Ninth Pacific Conference on Earthquake Engineering
Building an Earthquake-Resilient Society
14-16 April, 2011, Auckland, New Zealand

Development of pounding model for adjacent structures in earthquakes

S. Khatiwada, N. Chouw and J.W. Butterworth

Department of Civil and Environmental Engineering, University of Auckland, New Zealand.

ABSTRACT: Building pounding, a frequently recurrent problem in strong earthquakes, occurs when there is inadequate separation between adjacent structures. Several models have been proposed for the calculation of the resultant impact force, and its effect on the participating structures. This study analyzes two impact models, viz. the elastoplastic impact model by van Mier et al and the nonlinear viscoelastic impact element proposed by Jankowski and proposes a new impact model. The proposed viscous elastoplastic impact element combines all three properties of viscosity, elasticity and plasticity in an impact element for the first time. The plastic effect may be due to the material yielding at the contact location of the participating structures. A sample numerical investigation is presented for the seismic pounding between two adjacent buildings due to the 1940 North South El-Centro ground motions. The results show that the time history of the roof displacement of the participating structures is significantly different and the maximum displacement is reduced when the new model is employed when compared to the results obtained from numerical simulations using a nonlinear viscoelastic impact element.

1 INTRODUCTION

Seismic pounding is defined as the collision of adjacent buildings during earthquakes. The principal reason for seismic pounding is insufficient separation between the buildings. The phenomenon is mostly observed in old buildings that were constructed before the advent and popularity of earthquake resistant design principles. Although many current building codes specify a minimum seismic gap, it is still sometimes inadequate as the codes necessarily lag behind current research and fail to include the effect of other parameters that affect the structural deformation. The gap is often seen as a waste of prime real estate by developers and has been reduced in some newer versions. It also seems to unfairly penalize a property owner whose neighbour has already constructed a building at the boundary of their properties, as the new owner will have to provide the gap to accommodate the relative deformation of both buildings. A different problem is posed by existing buildings which were designed with sound structural methodology but before the prevalence of earthquake-resistant designs. Numerous cities in the world have their major streets outlined by a row of adjacent buildings built with no separation. The problem has been observed in the seismic assessment surveys of many cities e.g. Taipei and Wellington (Jeng and Tzeng, 2000; Bothara et al., 2008) and authors' survey of damaged buildings in Christchurch CBD due to the M6.3 earthquake on 22 February 2011 (see Figure 1).

The most significant manifestation of pounding hazard was reported in Mexico City, due to the 1985 Mexico earthquake where 15% of all cases led to collapse (Rosenblueth and Meli, 1986) though this statement was later revised, and pounding was estimated to have led to significant damage or collapse of 3% to 4.5 % of total buildings that suffered such damages (Anagnostopoulos and Karamaneas, 2008). This publication has often been cited as the turning point for the accelerated research in building pounding, though some studies had been published prior to the 1986 article by Rosenblueth and Meli. The study raised the awareness in the engineering community for the need to assess the pounding forces, so as to better design new buildings, and retrofit the older constructions to withstand a possible seismic collision with adjoining structures.

Studies of seismic pounding typically employ the so-called lumped mass model, where the building floors are assumed as diaphragms with lumped seismic masses. The numerical models are further simplified by employing two-dimensional analysis, and again, by coalescing the 2D frame model into

Figure 1. Pounding induced damage of buildings in Christchurch CBD

a single column system and representing the lumped mass by nodes on the column. The pounding effects are simulated by the laws of stereo-mechanics (Goldsmith, 2001) or by a collision element introduced between the lumped masses that are at the same level in the adjoining buildings (Fig. 2). There have also been studies that employ wave theory (Cole, 2010) to calculate the pounding forces or that combine both buildings as a single unit during collisions (Chouw, 2002). However, the collision element approach is the one most commonly employed by researchers.

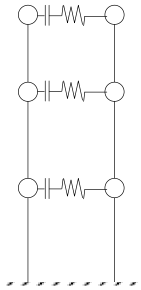

Figure 2. Lumped Mass model with a gap and an elastic spring in series

The collision element can be defined as a link between the two masses that is activated when the gap between the two lumped masses becomes zero or less. All types of collision element have at least two properties: a gap or an opening which is the distance between the surfaces of colliding masses, and a stiffness value (k) which is the spring constant of a linear spring. It will impart equal and opposite forces on both structures when they collide. This pounding model will result in an unrealistic, perfectly elastic contact and to avoid this drawback Anagnostopoulos (1988 and 2004) proposed a linear link that has both stiffness as well as a damping value to simulate the energy lost during collisions. This model has been further revised to include the nonlinear effects inherent in the Hertz contact law (Muthukumar and Desroches, 2004 and 2006; Jankowski, 2005 and 2006)). Jankowski (2005) and Muthukumar and Desroches (2006) provided a succinct analysis of these different collision elements (not repeated here).

There have also been various experimental studies on pounding (van Mier et al., 1991; Zhu et al., 2002; Chau et al., 2003). Van Mier et al. (1991) proposed an elastoplastic impact element model based on the experimental results.

The current study is based primarily on two previous studies, namely, by van Mier et al. (1991) and Jankowski (2005).

This paper addresses the two aforementioned pounding models in conjunction with the experimental results presented by van Mier et al. (1991). A new model is proposed to explain the discrepancies between the experimental research and the results from the existing impact elements. A sample numerical investigation is presented and the difference in results from the existing and the proposed impact elements is discussed.

2 POUNDING MODELS

2.1 Elastoplastic impact element (van Mier et al., 1991)

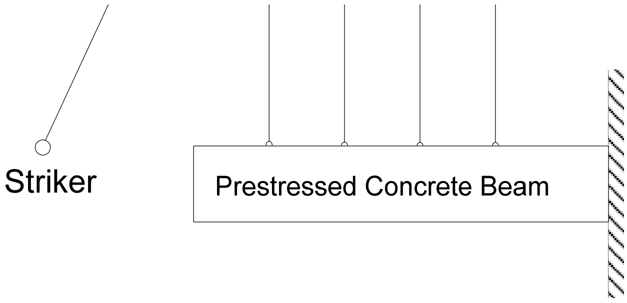

Figure 3. Experimental setup (simplified) by van Mier et al. (1991)

Van Mier et al. (1991) enumerated the results of various experiments conducted under the same experimental setup (Fig. 3). The study recorded the pounding force time history on impacts between a pendulum concrete striker and a fixed prestressed concrete beam where the striker had different contact surface geometry, mass and initial impact velocity values. Of particular interest, for the purpose of the current paper, are the results which compared the variation due to the striker mass (570 kg and 290 kg), and showed the differences due to the initial impact velocity (0.5 m/s and 2.5 m/s). The shapes of the smoothed contact force time histories are as shown in Figure 4.

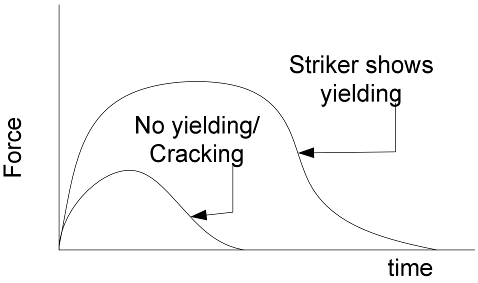

Figure 4. Simplified experimental contact force time history.

The results showed that the peak force remains almost identical, for particular striker geometry, even when the mass is doubled, though the duration of the peak force is longer for the higher mass. The slight difference can be attributed to the inherent variance in concrete even when cast under identical conditions and sometimes even in the same batch. This suggests an elastoplastic behaviour as deduced and used by van Mier et al. to formulate the elastoplastic impact model. Similarly, when the impact velocity of the 570 kg mass is 0.5 m/s rather than 2.5 m/s, there is no plateau in the curve, which suggests that the impact impulse was too small to induce plasticity.

The paper proposes the following equations for the collision forces, which are elastic-ideally plastic:

$$F(t) = K_e \delta^{\frac{3}{2}}(t) \qquad \text{(Elastic loading and unloading)}$$
$$F(t) = K_p \delta^{n}(t) \qquad \text{(Plastic region)} \qquad (1)$$

The article acknowledges that the unloading stiffness could differ from the loading stiffness, but adopts the same values for simplicity. Van Mier et. al. did not propose the values of the stiffness parameters for general application for all structures. Van Mier et al. (1991) employed no viscous parameter in their elastoplastc impact model, and this seems to be a weakness, as is apparent from all the loading curves. The loading curves produced by the first part of Equation 1 will be concave shaped but the loading curves seen in the actual experiments are all convex in shape.

2.2 *Nonlinear viscoelastic impact element (Jankowski, 2005)*

Jankowski (2005) proposed a nonlinear viscoelastic impact element, composed of a nonlinear elastic element stiffness $\bar{\beta}$ in parallel with a nonlinear damper that only becomes active during the approach period. The "approach-only" dashpot tries to emulate the experimental observations that most of the energy loss during impact occurs during approach (Goldsmith, 2001). The pounding force is given by:

$$\begin{aligned} F(t) &= \bar{\beta}\,\delta^{3/2}(t) + \bar{c}(t)\dot{\delta}(t) && \text{for } \dot{\delta}(t) > 0 \\ F(t) &= \bar{\beta}\,\delta^{3/2}(t) && \text{for } \dot{\delta}(t) \le 0 \qquad (2) \end{aligned}$$

Where $\bar{\beta}$ is the impact stiffness parameter, $\dot{\delta}(t)$ is the velocity of the left hand mass relative to the right hand mass, and $\bar{c}$ is the instantaneous damping of the impact element, given by

$$\bar{c} = 2\bar{\xi}\sqrt{\bar{\beta}\sqrt{\delta(t)}\,\frac{m_1 m_2}{m_1 + m_2}} \qquad (2a)$$

where m_1 and m_2 are the masses of the colliding bodies, and the damping ratio $\bar{\xi}$, according to Jankowski (2006), is given by:

$$\bar{\xi} = \frac{9\sqrt{5}}{2}\,\frac{1-e^2}{e\,(e\,(9\pi - 16) + 16} \qquad (2b)$$

where e is the coefficient of restitution.

The parameter $\bar{\beta}$ differs from the Hertz's coefficient k_h, due to the inclusion of the viscous parameter and has to be derived iteratively.

Jankowski (2005) proved that, a posteriori, the iterative procedure can produce a close fit to the experimental time histories, but it suffers from two drawbacks. First, the value of parameter $\bar{\beta}$ can only be derived iteratively if the collision force time history is available after the event, and second, the relation can explain the results when the pounding force is in the elastic range proposed by van Mier, but not when it is in the elastoplastic range, i.e. it cannot replicate the plateau in the elastoplastic curve. The proposed model will address this second limitation of the nonlinear viscoelastic model.

2.3 *Viscous elastoplastic impact element*

This study proposes a viscous elastoplastic impact model. The model combines the elastoplastic and nonlinear viscoelastic impact elements and describes the pounding force as:

$$\begin{aligned} F(t) &= \bar{\beta}\,\delta^{3/2}(t) + \bar{c}(t)\dot{\delta}(t) && \text{for } \bar{\beta}\,\delta^{3/2}(t) + \bar{c}(t)\dot{\delta}(t) < F_E \quad \dot{\delta}(t) > 0 \\ &= F_E && \bar{\beta}\,\delta^{3/2}(t) + \bar{c}(t)\dot{\delta}(t) \ge F_E \quad \dot{\delta}(t) > 0 \\ &= \bar{\beta}\,\delta^{3/2}(t) && \bar{\beta}\,\delta^{3/2}(t) < F_E \qquad \dot{\delta}(t) \le 0 \\ &= F_E && \bar{\beta}\,\delta^{3/2}(t) \ge F_E \qquad \dot{\delta}(t) \le 0 \qquad (3) \end{aligned}$$

Where F_E is the yield strength of the material or structural element at the contact point. The parameter should be deduced judiciously from the study of the model. For instance, if there is mid-column pounding, the value of F_E is the force that will cause the failure in column. Similarly, if the impact is on a corner of the slab, F_E is the force that will crush that particular corner, and if the collision is on the face of a beam, F_E should be able to cause flexural failure or material crushing failure at that particular surface of contact.

The parameters $\bar{c}$ and $\bar{\xi}$ are the same as those of the nonlinear viscoelastic impact element and are given by:

$$\bar{c} = 2\bar{\xi}\sqrt{\bar{\beta}\sqrt{\delta(t)}\,\frac{m_1 m_2}{m_1 + m_2}} \qquad (3a)$$

where m_1 and m_2 are the masses of the colliding bodies, and the damping ratio $\bar{\xi}$, according to Jankowski (2006), is given by:

$$\bar{\xi}=\frac{9\sqrt{5}}{2}\frac{1-e^2}{e\,(e\,(9\pi-16)+16} \tag{3b}$$

where e is the coefficient of restitution.

The model assumes that, once the pounding force exceeds the elastic capacity of the material or the structural element at the contact location, there will be yielding at that location or element, and any additional force will not be transferred to the global structure. In short, there is a short-lived failure mechanism at the contact location which will last until the value of viscoelastic contact force, $(\bar{\beta}\,\delta^{3/2}(t)+\bar{c}(t)\dot{\delta}(t))$ exceeds F_E. For simplicity, it is assumed that the unloading curve after yielding is the same as in the nonlinear viscoelastic impact model, although there is a significant chance that the unloading stiffness will be altered due to the change in material or section properties (chipping off, spalling and dents). During seismic pounding of buildings, this limitation also applies to the loading curves of any subsequent pounding at the same location.

The viscous elastoplastic impact element model tries to reflect the material behaviour at the contact point by imposing a condition of yielding if the stress at the contact area exceeds the elastic strength of the material or structural element. When the elastic strength is exceeded, any extra energy will be dissipated by the local plastic degradation while the force up to the elastic limit will be transferred to the global structure. This is to ensure that the model reflects the global structural behaviour after the onset of local plastic yielding. If the structural element is a redundant one, it may change the global stiffness character slightly but if the location is a critical one, for instance any location in a determinate structure, it may induce immediate structural failure.

3 NUMERICAL INVESTIGATION

An example of the application of the proposed pounding model to obtain the pounding force activated between a flexible frame structure and an adjoining ‘rigid’ masonry structure with which it shares a common wall is considered (Fig. 5). Such building configurations are common in South Asian countries. The rigid building is an older construction, built up to the boundary line between two properties and the flexible structure is a newer construction, whose outer columns have been pushed back from the boundary line to avoid load asymmetry at the isolated footings. The owner of the newer building has increased the living area by joining his building to the adjacent building, and foregoing the construction of a wall at the boundary line. The slab and beams have 0.6 m cantilever projections to the boundary, and the slab is slightly above the slab of the neighbouring building, such that the beam of the flexible building is in contact with the slab of the rigid building.

The flexible building has nine reinforced concrete columns of size 225 mm x 225 mm. The stiffer building has three 225 mm thick longitudinal masonry walls and four 225 mm thick transverse walls. Thus the masonry building is very stiff and can be assumed to move as a rigid body with the ground. Both buildings have reinforced concrete slabs.

The lumped masses at the floor levels are 44.36 t and 82.60 t respectively for the flexible and stiffer buildings. The masonry characteristics are taken from Ip (1999). The fundamental periods of the two buildings are found to be 0.208 sec and 0.007 sec.

The proposed impact element still contains the drawback that the impact stiffness parameter $\bar{\beta}$ has to be iteratively defined but for this particular problem, it can be seen that the contact area is a rectangle, bounded by sharp edges, and thus nearly constant with little geometric variation due to deformation. Thus the impact stiffness parameter can be assumed to be equal to the axial stiffness of the projecting slab, i.e. 4.2 x 10^7 kN/m.

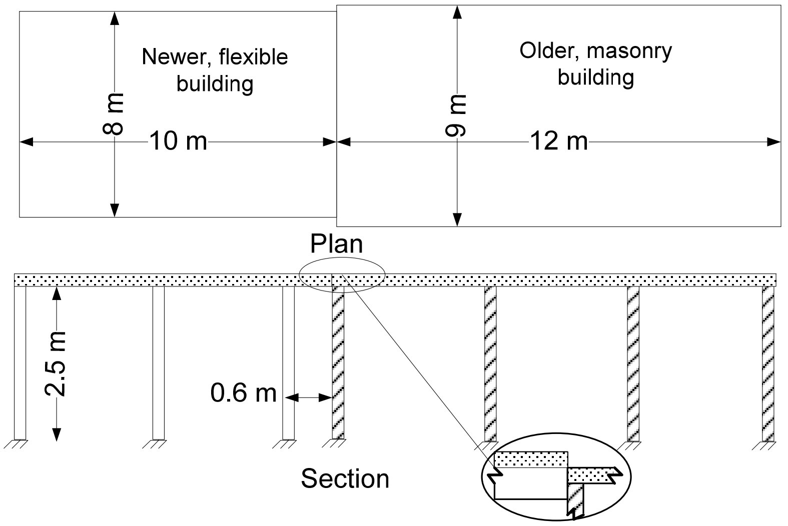

Figure 5. Adjacent buildings under investigation

The restitution coefficient has been calculated for each impact, from the relationship of the concrete vs. concrete impact, as given by Jankowski (Jankowski 2010),

$$e = -0.0070\, v^3 + 0.0696\, v^2 - 0.2529\, v + 0.7929 \tag{4}$$

where v is the relative velocity at the initiation of impact.

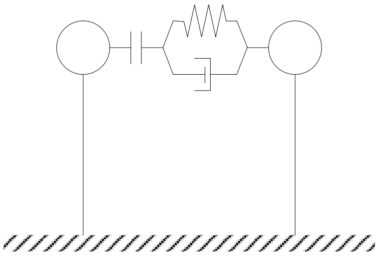

Figure 6. Simplified models of the adjacent buildings

The model is analyzed with 6 s of the 1940 North-South El Centro ground acceleration time history (Fig. 7) downloaded from Pacific Earthquake Engineering Research Center and linearly scaled to the maximum ground acceleration of 0.5 g. The analysis is performed using both the nonlinear viscoelastic and viscous elastoplastic models.

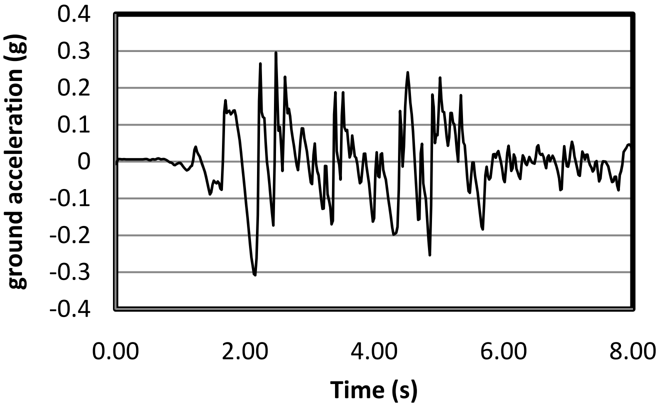

Figure 7. El-Centro ground motion

The analysis includes the effect of the local plasticity on the global structural response only. The gap widening due to permanent compression of participating surfaces is ignored. This will introduce some errors, as the gap size widens as the contact area yields, an effect which is not incorporated in this analysis. Similarly, the impact of strain rates on the local yielding may be significant for concrete.

The numerical analysis was conducted with an in-house program. The accuracy of the program was verified by comparing the results of a linear visco-elastic analysis with that from SAP2000. The results, including the effect of stiffness, damping and local plasticity are presented in Figure 8.

It is apparent that the relative displacement between the structures is reduced as the plasticity effects are considered. The two graphs are identical until 2.4 seconds at which instant the pounding force is large enough to induce plasticity. The graphs then diverge and the largest relative displacement in the viscous elastoplastic model is, as expected, less than that obtained using the nonlinear viscoelastic model. Since the right hand side masonry structure in this numerical study is relatively rigid, the relative displacement time history is nearly identical to the roof displacement time history for the flexible frame structure and so the latter is not reproduced here.

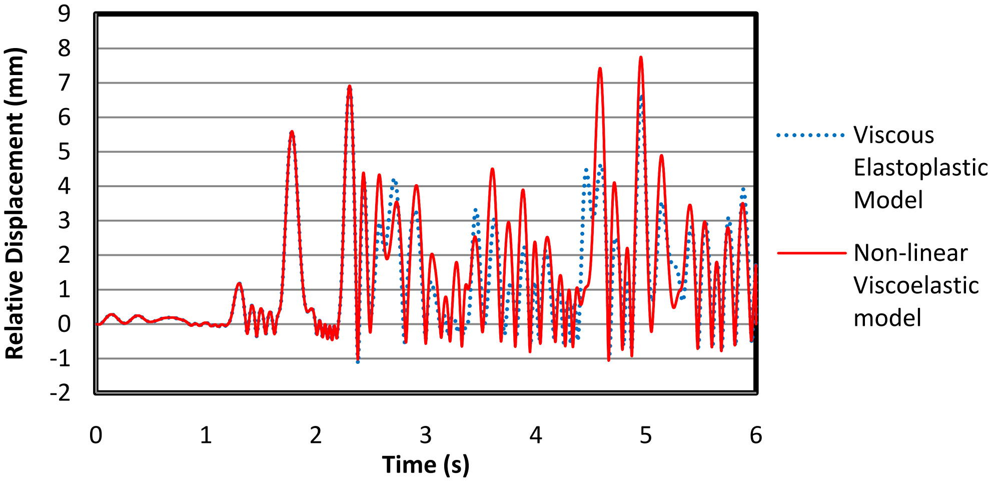

Figure 8. Relative displacement time histories

4 CONCLUSIONS

This paper studied two impact elements, namely, an elastoplastic impact element proposed by van Mier et al. (1991) and a nonlinear viscoelastic impact element proposed by Jankowski (2005). A new impact element is proposed, combining the parameters and ideas of the two earlier models into one coherent model.

A sample numerical study has been presented to illustrate the difference in the non-linear viscoelastic model and the proposed viscous elastoplastic model. The effect of pounding on the relative displacement of a reinforced concrete building adjacent to a relatively rigid masonry building with no separation gap, and a common wall between them is considered. The proposed impact element highlights the effects of local yielding, and shows that the relative displacement between the two floors, and as a result, the floor displacement response of the flexible frame structure is reduced if the plasticity effects at the contact location are considered.

Further studies may include the effect of strain rate on concrete deformation and the effect of the gap widening on the pounding force development. This may help in explaining the damage propagation in the participating structures.

ACKNOWLEDGEMENT

The authors would like to thank the anonymous reviewer for the constructive comments and the University of Auckland for providing the first author with the International Doctoral Scholarship.

REFERENCES

Anagnostopoulos, S. A. (1988). Pounding of buildings in series during earthquakes. *Eathquake Engineering and Structural Dynamics* **16**: 443-456.

Anagnostopoulos, S.A. (2004). Equivalent viscous damping for modeling inelastic impacts in earthquake pounding problems. *Earthquake Engineering and Structural Dynamics* **33**(8): 897-902.

Anagnostopoulos, S.A. and Karamaneas C.E. (2008). Collision shear walls to mitigate seismic pounding of adjacent buildings, *Proceedings.of the 14th World Conference on Earthquake Engineering*, *October 12-17, 2008,* Beijing, China

Bothara, J.K., Jury R.D., Wheelers K. And Stevens C. (2008). Seismic Assessment Of Buildings In Wellington: Experiences And Challenges. *Proceedings of the 14th World Conference on Earthquake Engineering, October 12-17, 2008,* Beijing, China

Chau, K. T., Wei, X. X., Guo, X., And Shen, C. Y. (2003). Experimental and theoretical simulations of seismic poundings between two adjacent structures. *Earthquake Engineering and Structural Dynamics* **32**(4): 537-554.

Chouw, N. (2002). Influence of soil-structure Interaction on pounding response of adjacent buildings due to near-source earthquakes. *JSCE journal of Applied Mechanics* **5**: 543-553.

Goldsmith, W. (2001). *Impact: the theory and physical behaviour of colliding solids*, Dover Pubns.

Cole, G., Dhakal, R., Carr, A., and Bull, D. (2010). An investigation of the effects of mass distribution on pounding structures. *Eathquake Engineering and Structural Dynamics.* DOI: 10.1002/eqe.1052

Ip, F. (1999). *Compressive Strength and Modulus of Elasticity of Masonry Prisms*, ME Thesis, Carleton University.

Jankowski, R. (2005). Non linear viscoelastic modelling of earthquake induced structural pounding. *Earthquake Engineering and Structural Dynamics* **34**(6): 595-611.

Jankowski, R. (2006). Analytical expression between the impact damping ratio and the coefficient of restitution in the non linear viscoelastic model of structural pounding. *Earthquake Engineering and Structural Dynamics* **35**(4): 517-524.

Jankowski, R. (2010). Experimental study on earthquake-induced pounding between structural elements made of different building materials. *Earthquake Engineering and Structural Dynamics* **39**(3): 343-354.

Jeng, V. and Tzeng, W. (2000). Assessment of seismic pounding hazard for Taipei City. *Engineering Structures* **22**(5): 459-471.

Muthukumar, S. and Desroches, R. (2004). Evaluation of impact models for seismic pounding. *Proceedings of the 13th world Conference on Earthquake Engineering, August 1-6, 2004* Vancouver, B. C., Canada.

Muthukumar, S. and DesRoches, R. (2006). A Hertz contact model with non linear damping for pounding simulation. *Earthquake Engineering and Structural Dynamics* **35**(7): 811-828.

Rosenblueth, E. and Meli, R. (1986). The 1985 earthquake: causes and effects in Mexico City. *Concrete International* **8**(5): 23-34.

van Mier, J.G., Pruijssers A., Reinhardt, H.W. and Monnier, T. (1991). Load Time Response of Colliding Concrete Bodies. *Journal of Structural Engineering* **117**(2):354-374

Zhu, P., Abe, M. and Fujino, Y. (2002). Modelling three dimensional non linear seismic performance of elevated bridges with emphasis on pounding of girders. *Earthquake Engineering and Structural Dynamics* **31**(11): 1891-1913.