13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 2176

EXPERIMENTAL AND THEORETICAL SIMULATIONS OF SEISMIC TORSIONAL POUNDINGS BETWEEN TWO ADJACENT STRUCTURES

K.T. CHAU[1], X. X. WEI[2], C.Y. SHEN[3], and L.X. WANG[4]

SUMMARY

Most existing buildings in regions of moderate seismicity are built without seismic provisions, and very often asymmetric structural form is used. Seismic torsional pounding between two adjacent asymmetric buildings can lead to serious consequences. Thus, both shaking table tests and theoretical analysis for torsional pounding between two adjacent structures of different natural frequencies and damping ratios subject to different combinations of stand-off distance and seismic excitations are conducted in the present study. Two steel towers with adjustable dynamic characteristics are tested on the shaking table at The Hong Kong Polytechnic University. The center of mass of both towers may differ from the center of stiffness such that torsional response of both buildings can be induced even subject to one-direction ground excitation. Subjected to sinusoidal excitations, poundings between the two towers could appear as either periodic or chaotic. The torsional responses are found more chaotic than the translational responses. A type of periodic group poundings was observed for the first time (i.e. a group of non-periodic poundings repeating themselves periodically). A new theoretical formulation is being derived for such system by incorporating nonlinear Hertz impact, and much theoretical works remain to be done.

INTRODUCTION

Pounding between adjacent structures is a commonly observed phenomenon during major earthquakes. Poundings may cause both architectural as well as structural damages and, in some cases, it may lead to collapse of the whole structure. For example, pounding was observed in over 15% of the 330 collapsed or severely damaged structures when an earthquake struck Mexico City in 1985 [1-2]. During the 1989 Loma Prieta earthquake, there are over 200 pounding occurrences involving more than 500 buildings in San Francisco, Oakland, Santa Cruz, and Watsonville [3]. In general, poundings can be developed

[1] Professor and Associate Head, Department of Civil and Structural Engineering, The Hong Kong Polytechnic University, Hong Kong, China. Email: cektchau@polyu.edu.hk
[2] Professor, Department of Applied Mechanics, School of Science and Technology, Beijing Institute of Technology, Beijing, China. Email: cexxwei@bit.edu.cn
[3] Engineer, Earthquake Engineering Research Center, Guangzhou University, Guangzhou, China. Email: shency@163.net
[4] PhD student, Department of Civil and Structural Engineering, The Hong Kong Polytechnic University, Hong Kong, China. Email: celixin.wang@polyu.edu.hk

between high-rise buildings, between low-rise buildings, as well as between high-rise and low-rise buildings during strong earthquakes. Adjacent structures with different dynamic characteristics and with different floor levels are more conducive to seismic poundings.

Many investigations have been carried out on pounding damage caused by past earthquakes [1-8], on mitigation of pounding hazards [9-12], and on modeling of pounding between structures [13-28]. Pounding between adjacent structures is a very complex phenomenon, which may involve plastic deformation, local crushing as well as fracturing at the contact. These nonlinear deformations are not easy to be incorporated into the modeling of pounding. Therefore, idealizations and assumptions have inevitably been used in theoretical models [13-28]. For example, structures have been idealized as rigid barriers, single-degree-of-freedom oscillators or multi-degree-of-freedom oscillators; poundings between structures have been modeled by linear dashpot-spring system or nonlinear impact model. Despite of these simplifications, theoretical analyses have been valuable in providing insight into the pounding mechanisms [13-28].

Among these analytical analyses, the pounding model by Davis [13], which incorporates the nonlinear Hertzian impact, deserves more deliberation. In particular, Davis [13] models poundings as nonlinear impacts between a single-degree-of-freedom oscillator and a rigid barrier. This model is considered more realistic because nonlinear contact has been incorporated. Actually, poundings between adjacent structures are seldom linear. Employing the model by Davis [13], Pantelides and Ma [29] considered poundings between a damped single-degree-of-freedom structure and a rigid barrier when they are subject to the ground motions of the 1940 El Centro earthquake, the 1971 San Fernando earthquake, the 1989 Loma Prieta earthquake, and the 1994 Northridge earthquake. More recently, Chau and Wei [30] extended the model of Davis [13] to consider poundings as nonlinear impacts between two single-degree-of-freedom (SDOF) oscillators. There are relatively few experiments performed to check the validity of these theoretical models in applying to actual poundings between buildings observed in the field. Only three shaking table tests have been performed to validate theoretical models for seismic poundings. In particular, Papadrakakis and Mouzakis [31] have performed shaking table experiments on pounding between two two-storey reinforced concrete buildings with zero-gap separation, subject to sinusoidal and random motions; and their experimental results are compared to the predictions by the Lagrange multiplier method [25]. Filiatrault et al. [32] have conducted shaking table tests for poundings between adjacent three- and eight-storey-steel frames subject to the time history of the 1940 El-Centro earthquake, and the experimental results are compared to the predictions by two computer programs. Chau et al. [33] have conducted shaking table tests on pounding between two towers and the results compare well with the theory proposed by Chau and Wei [30].

However, none of the above mentioned studies considered the potential of torsional pounding. There are relatively few studies on torsional pounding. Hao and Shen [34] considered the required separation to avoid torisonal pounding between two adjacent asymmetric structures. In a recent study by Zhu et al. [35] for three-dimensional pounding between adjacent bridge girders, torsional pounding was included in the analysis. However, much remains to be learnt about the torsional pounding phenomenon. For example, the experiments done by Zhu et al. [35] are of very small scale (the model mass of about 2kg). Therefore, in this paper, a more comprehensive study including both shaking table tests and theoretical studies will be considered. Many of these works are still ongoing, therefore, only the preliminary results are reported here. Nevertheless, our experimental studies should provide a yardstick for future torsional pounding studies.

THEORETICAL STUDIES

Theoretical formulation

Figure 1 shows a model for two asymmetric structures containing offset masses. Both structures are supported by four columns with bending stiffness of k_A for tower A and k_B for tower B. The center of mass and center of stiffness are denoted by M_A and S_A respectively on Fig. 1. The eccentricities of the center of mass from the center of stiffness along the *x*- and *y*- directions are e_{Ax} and e_{Ay} respectively. For each tower, three degrees of freedom are assumed (e.g. two translational displacements u_A and v_A and one rotation θ_A for Tower A).

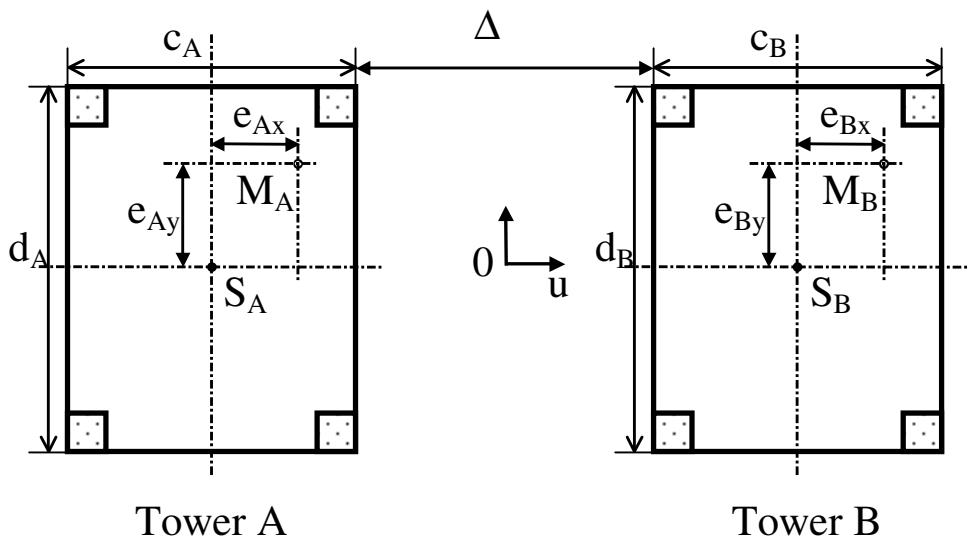

Fig.1 Schematic diagram for the two adjacent towers

The separation of the two towers is denoted by Δ. The center-to-center distance between the two columns along the *y*-direction is d_A; and that along the *x*-direction is c_A. Similar definitions also apply to Tower B. The origin of the coordinate system for the deflections of the two towers is set at the center between the two towers. If the structures are subject to ground motion along the *x*-direction, the equations of motion for Tower A can be written as:

$$\ddot{u}_A + 2\xi_{Ax}\dot{u}_A + \omega_{Ax}^2 u_A + e_{Ay}\omega_{Ax}^2\theta_A + F_{ABx}/m_A = -\ddot{u}_g \tag{1}$$

$$\ddot{v}_A + 2\xi_{Ax}\dot{v}_A + \omega_{Ax}^2 v_A - e_{Ax}\omega_{Ax}^2\theta_A + F_{ABy}/m_A = 0 \tag{2}$$

$$\ddot{\theta}_A + 2\xi_{A\theta}\dot{\theta}_A + e_{Ay}\omega_{Ax}^2 u_A - e_{Ax}\omega_{Ax}^2 v_A + \omega_{A\theta}^2\theta_A + T_{AB\theta}/m_A = 0 \tag{3}$$

where m_A is the mass of Tower A, ω_{Ax} is the natural frequency of the tower (=$4k_A/m_A$), $\omega_{AB\theta}$ is the torisonal natural frequency, ξ_{Ax} is the corresponding damping ratio, F_{ABx} and F_{ABy} are the pounding force components along the *x*- and *y*-directions, and $T_{AB\theta}$ is the pounding torsional force between the two towers. The ground acceleration along the *x*-direction is given as $\ddot{u}_g$. Similar equations of motion can also be written for Tower B. As formulated by Chau and Wei [30], the pounding force can be estimated from Hertz contact law. Due to the page limitation, the full details will not be given here. If there is no pounding between the towers, the pounding interactions will be set to zeros. Consequently, the two towers will oscillate independently, and there will be no nonlinear coupling between two sets of differential equations for each tower and the resulting equations can be solved easily.

Numerical results

There are various ways to solve the resulting coupled differential equations for Towers A and B. The most straightforward approach is to rewrite the second order coupled differential equations as first order differential equations and solve them by Runge-Kutta method with error control. It was found that special cares for numerical integration are needed when torisonal poundings occur, due to highly nonlinear interactions between the towers. Without going into the details, we only report one particular numerical example here. Figure 2 shows the numerical simulations of torsional pounding between two adjacent buildings.

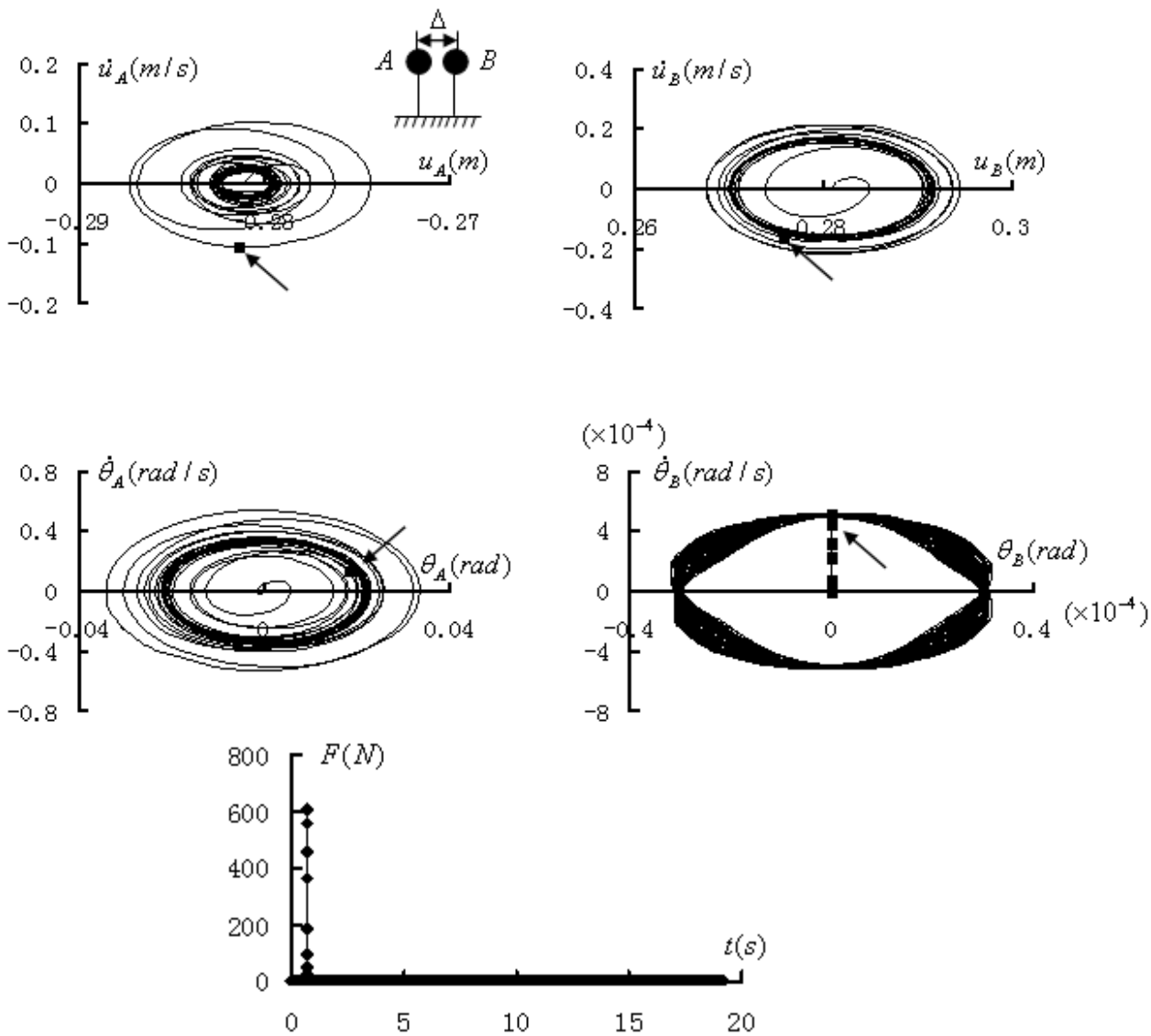

Fig.2 Results of numerical simulations for torsional pounding between two towers shown in Fig. 1. The position of the pounding is indicated by arrows in the diagram.

The parameters used in this numerical calculation are: $m_A = 173.2kg$, $m_B = 145kg$, $f_A = 2.54Hz$, $f_B = 2.83Hz$, A_g =2.5mm, $f = 2.5Hz$, Δ =12mm, $e_{Ay} = -112mm$, $e_{By} = 0$, $e_{Ax} = e_{Bx} = 0$, where A_g is

the magnitude of the sine wave input while *f* is the frequency of the input. As seen in Fig. 2, a pounding occur at about 1 second then the two towers oscillate independently. Rotational oscillations of Tower B are induced only after the impact, and they are predominantly periodic. Note also that the structural response before and after the impact is not the same. It seems that torsional pounding may lead to a new and different response, depending on the energy transfer between the two towers. Much more numerical computations are being conducted and will be reported in our later publication. The main purpose of this example is to illustrate the potential complexity of structural response induced by torsional poundings.

EXPERIMENTAL STUDIES

Experimental set-up

The pounding experiments are performed on a MTS uniaxial seismic shaking table of size 3m×3m at the Hong Kong Polytechnic University (see Fig. 3). It is capable of producing a maximum horizontal acceleration of 1g at a maximum load of 10 tons. The maximum velocity and displacement can be up to 0.5m/s and ±10cm respectively. The actuator is controlled by a 469DU digital seismic table controller, and the working frequency of the table ranges from 1 to 50 Hz. The shaking table can simulate motions with control in displacement, velocity or acceleration (i.e. a three-variable-control). The displacement control is primarily for low frequency range, velocity control for middle frequency range, and acceleration control for high frequency range. The maximum overturning moment that can be restrained by the bearing of the table is 10 ton-m.

Fig. 3 Set-up of the two towers on the shaking table at PolyU

Each of the two steel towers shown in Fig. 3 is constructed by 4 columns of rectangular hollow section of 50mm (length) ×30mm (width) × 4mm (thickness). The columns are arranged in a rectangular pattern with a spacing of 400mm along the shaking direction, and a spacing of 600mm along the orthogonal direction. The shorter side of the 4 columns is aligned with the shaking direction. The height of column is 2m. Two horizontal tie bars of rectangular cross section 50 (mm) ×5 (mm) are hinged at about the mid height along shaking direction (see Fig. 3), and the diameter of screw is about 12mm. Two pairs of contact are arranged along the side of top slab of the two towers, such that torsional pounding can be recorded. Four columns are welded to the top plate of size 1000mm × 550mm ×15mm and to a base steel plate of

size 1000mm × 600mm ×15mm. Two rectangular notches are drilled on the bottom plates at the base of the structures, such that bolts (the diameter of screw is 22mm) can be used to tie down the towers and the separation Δ between the two towers can be adjusted accordingly. To ensure the rigidity at the connections between the columns and the bottom and top plates, triangular steel plates of 50×50×15 are welded to all the joints. In addition to the tie-down bolts, the bottom plate is clamped to the shaking table. Different additional masses have been added to the top plate of the two towers to simulate different natural frequencies, and their positions can be adjusted to simulate different torsion frequency of the two towers. In order to adjust the mass on the top plate accurately and efficiently, a track along the orthogonal direction is installed (see Fig. 4). Accelerometers are attached to the center of mass of each tower below the top slab, so that time histories of all displacement, velocity and acceleration can be recorded. In addition, to record the torisonal rotations two accelerometers are used to record the displacement and acceleration near the edge on the top plate of each tower. Both displacement and acceleration at the surface of the shaking table is also recorded in order to double check the response of the shaking table.

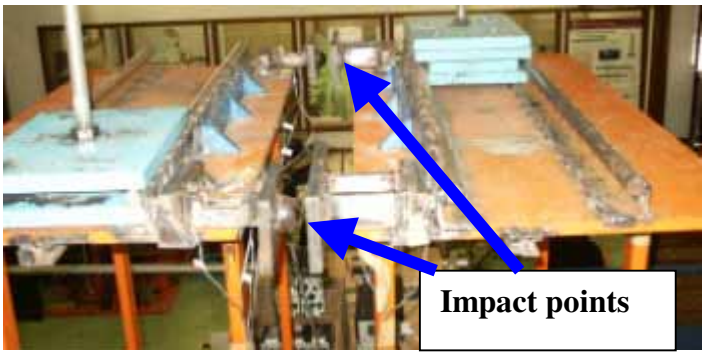

Fig. 4 The layout of the two impactors of semi-sphere at the top plates of the two towers

To simulate the pounding, a steel semi-sphere of 50mm diameter is welded to a steel plate of 150×100×15 (mm), which is further connected to the upper steel platform through two steel bars on which strain gauges are attached to measure the pounding force (see Fig. 4). More specifically, 4 TML strain gauges model PFL-20-11 are installed along these steel bars to record the impact force induced by torsional poundings. Micro-strain down to 1με can be measured. The Young's modulus, Poisson ratio and the yield stress of the steel sphere and the plate are estimated as $E = 2.06\times10^5\,\text{MPa}$, $\nu = 0.28$, and $p_0 = 215\text{MPa}$ respectively. A sampling rate of 2020 data per second is used to record data for each channel. When only the natural frequency is wanted, a sample rate of 100 data /second is found sufficient.

Experimental results

A series of shaking table experiments have been conducted to investigate the phenomenon of torsional poundings. Figure 5 shows the phase diagrams of both translations (first row) and rotations (second row) of both Towers A and B. The phase diagrams for velocity and displacement given in the first row are for the edge accelerometers marked as "S" in Fig. 5.

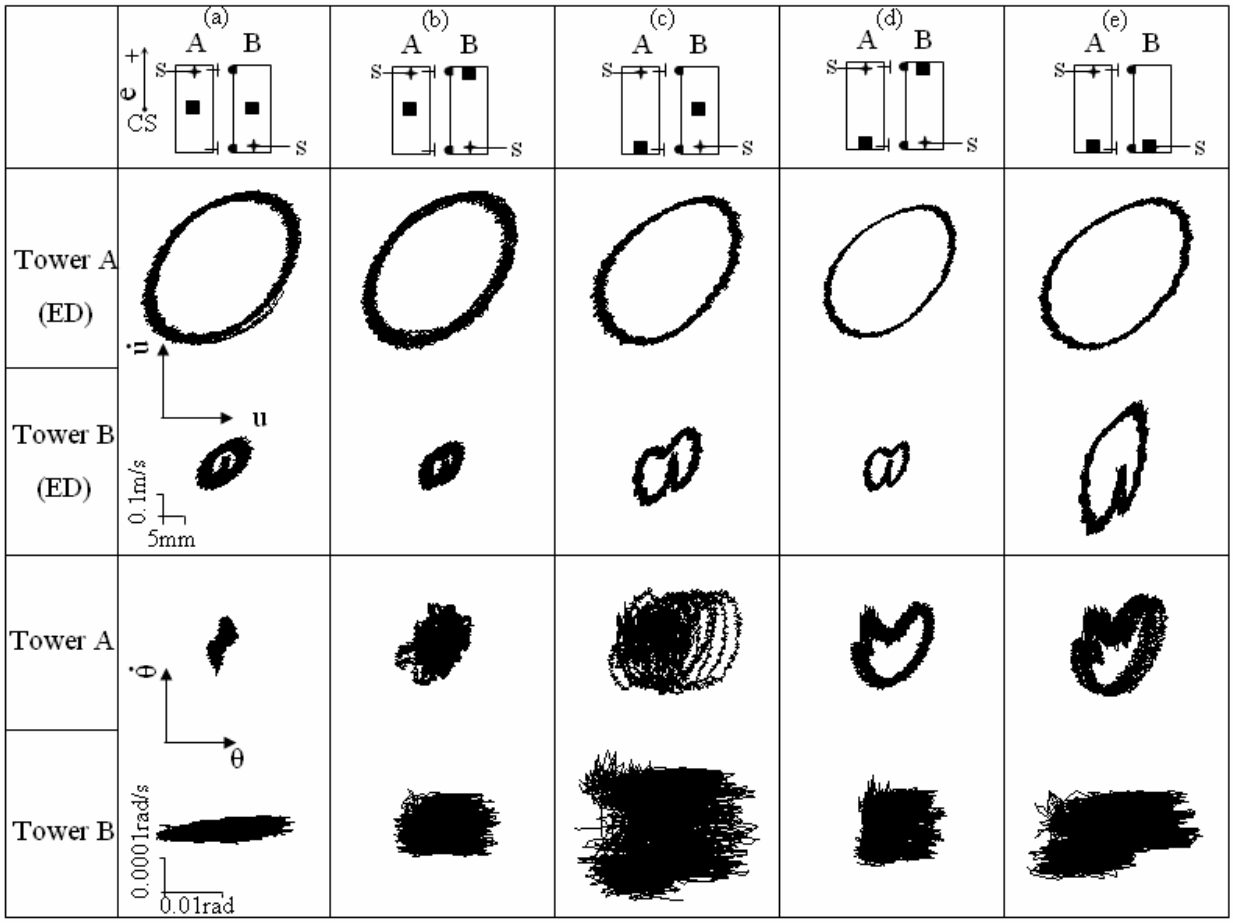

Fig. 5 A compilation of various testing results when the masses on the top of the two towers were shifted. The first row is the phase diagrams of translational velocity versus displacement of the two towers, whereas the second row is the phase diagrams of rotation rate versus rotation for the two towers.

More specifically, the mass of tower A is 173.2 kg whereas that for tower B is 145 kg with the corresponding natural frequencies being 2.54Hz and 2.83Hz respectively. The damping ratio of towers A and B are 5.2% and 4.7% respectively. The initial separation distance between the two towers is Δ =12mm. The input ground motion is a harmonic sine wave with magnitude of A_g =2.5mm, and frequency of $f = 2.5Hz$. Since these systems are not perfectly symmetric as slight imperfections are inevitable in experiments, rotational responses are observed even for case (a) (the first column). The translational responses are roughly periodic whereas those for rotations are rather chaotic. Pounding between the two structures were also observed (indicated by the jumps in the translational phase diagram of Tower B). The natural torsional vibrations of Towers A and B are 7.03 and 7.13Hz respectively. In case (b), the center of mass of Tower B was shifted by 70mm, which yields a natural torsional frequency of 6.64Hz. The torsional responses appear to be much larger than case (a), but they remain chaotic. Case (c) corresponds to a mass shift of −112mm in Tower A, the heavier tower. The pattern of periodicity is much more apparent for this case. The rotational responses of both towers further increase while the rotational response of Tower A indicates a shifting of periodic oscillations whereas that of Tower B remains chaotic. Case (d) is for a mass shift of −112mm for Tower A and 70mm for Tower B. The periodic solutions for both rotational and translational motions become much more apparent. The torsional response of Tower A becomes periodic while Tower B remains chaotic. Case (e) is for a mass

shift of −112mm for Tower A and −70mm for Tower B. The structural responses are quite similar to that for case (d) except that the magnitude of responses is much larger in this case. It should be noted that the damping ratio of both Towers actually change slightly when the experiments continue. Presumably some plastic deformation may be resulted in the towers because of pounding, but such effects are quite small and should not be significant.

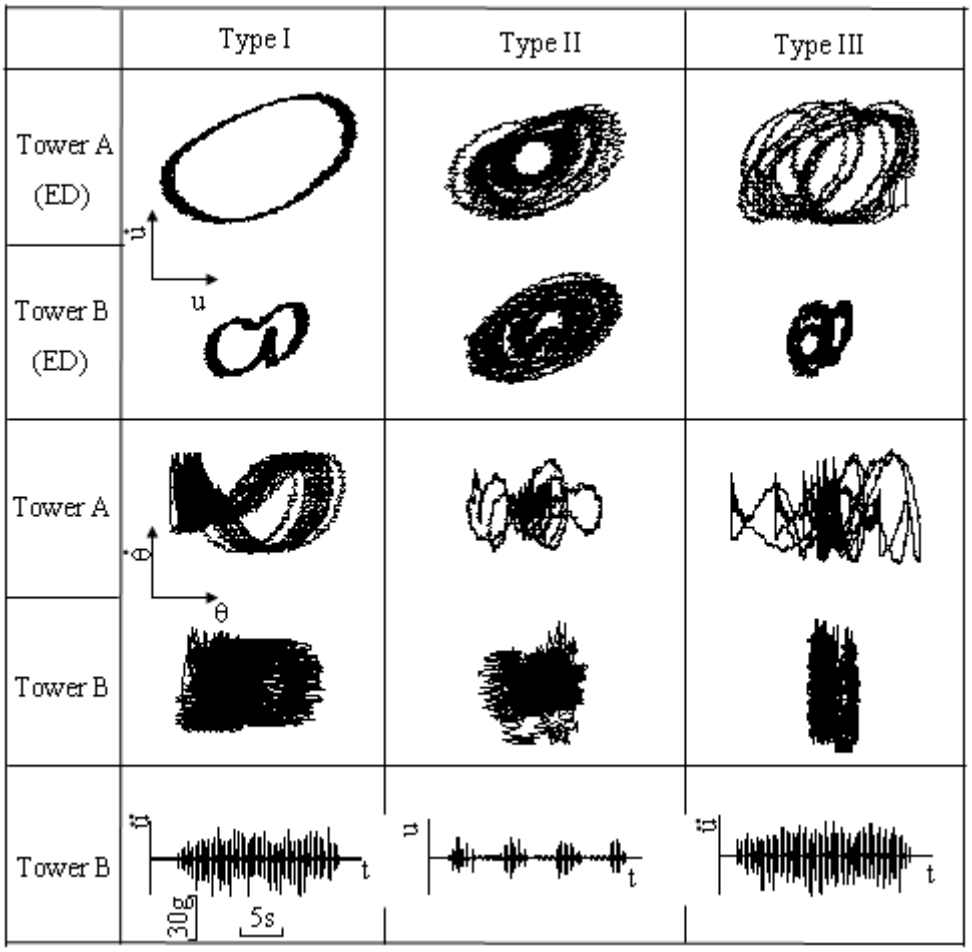

Fig. 6 Three different types of pounding behaviors have been observed: (I) Periodic: (II) A group of periodic response; (III) Chaotic response.

Figure 6 shows the case of three different types of pounding results: (I) Periodic: (II) A group of periodic response; (III) Chaotic response. The parameters of this set of experiments are: A_g=1.2mm, $m_A = 173.2\text{kg}$, $f_1^A = 2.54\text{Hz}$, $\zeta_t^A = 2.1\%$, $f_t^A = 5.86\text{Hz}$, $e_y^A = -112\text{mm}$; $m_B = 145\text{kg}$, $f_1^B = 2.83\text{Hz}, \zeta_1^B = 4.9\%$, $f_t^B = 6.15\text{Hz}$, $\zeta_t^B = 1.1\%$, δ=16.1mm and $e_y^B = +70\text{mm}$. The only differences for responses type I , II and III are the frequency of the input ground motion: type I: f_g=2.52Hz; type II: f_g=2.72Hz; type III: f_g=2.62Hz. Therefore, torsional pounding are highly ground-motion-dependent. The time histories on the last row of Fig, 6 clearly illustrate the effect of group response in Type II. This type of response has never been reported either experimentally or theoretically. Much more effort is needed to further investigate this peculiar phenomenon.

CONCLUSIONS

A comprehensive study on torisonal pounding between adjacent buildings has been initiated. Both theoretical analyses and shaking table tests are being conducted at the Hong Kong Polytechnic University. Our preliminary numerical results demonstrate that the structural responses of a building may change abruptly or drastically as a result of seismic pounding. Our shaking table tests also demonstrate that structural responses (both translational and torsional) increase with the torsional properties of the buildings. Both periodic and chaotic torsional responses can be observed. It seems that the response of the lighter building is more conducive to the appearance of pounding-induced-motions and may be more vulnerability to pounding damage. This effect can be viewed as a result of energy transfer between two systems. Three different types of poundings are observed and they depend stronylu on the frequency content of the ground motion. They are the periodic, periodic of a group of responses and chaotic responses. Much more theoretical analyses and experiments need to be done in investigating this complicated problem.

ACKNOWLEDGEMENT

The work described in this paper was fully supported by grants from the Research Grants Council of the Hong Kong Special Administrative Region, China (Project No. PolyU 5035/02E) and from Polytechnic University through the "Area of Strategic Development (ASD) in Mitigation of Urban Hazards" (Project No. A226). Technical support on shaking table test from Mr. T.T. Wai and Mr. C.F. Cheung is acknowledged.

REFERENCES

1. Anagnostopoulos SA. “Building pounding reexamined: How serious a problem is it.” Proceedings of the 11th World Conference on Earthquake Engineeri*ng*, Acapulco, Mexico, 1996; Paper No. 2108.
2. Rosenblueth E, Meli R. “The 1985 earthquake: causes and effects in Mexico City.” Concrete International, American Concrete Institute 1986; 8(5): 23-24.
3. Kasai K, Maison BF. “Building pounding damage during the 1989 Loma Prieta Earthquake.” Engineering Structures 1997; 19:195-207.
4. Anagnostopoulos SA, Spiliopoulos KV. “An investigation of earthquake induced pounding between adjacent buildings.” Earthquake Engineering and Structural Dynamics 1992; 21:289-302.
5. Bertero VV. “Observations of structural pounding.” Proc. Int. Conf.: The Mexico City Earthquakes-1985. (Eds Cessaro MA, Romero EM), ASCE, Mexico City, 1987; 264-278.
6. Anagnostopoulos SA. “Pounding of buildings in series during earthquakes.” Earthquake Engineering and Structural Dynamics 1988; 16:443-456.
7. Anagnostopoulos SA. “Earthquake induced pounding: State of the art.” *Proceedings of the 10th European Conference on Earthquake Engineering*, Vienna, Austria, 1995; 2:897-905.
8. Kasai K, Jagiasi AR, Jeng V. “Inelastic vibration phase theory for seismic pounding mitigation.” Journal of Structural Engineering ASCE 1996; 122:1136-1146.
9. Westermo BD. “The dynamics of inter-structural connection to prevent pounding.” Earthquake Engineering and Structural Dynamics 1989; 18:687-699.
10. Jankowski R, Wilde K, Fujino Y. “Pounding of superstructure segments in isolated elevated bridge during earthquakes.” Earthquake Engineering and Structural Dynamics 1998; 27:487-502.
11. Penzien J. “Evaluation of building separation distance required to prevent pounding during strong earthquakes.” Earthquake Engineering and Structural Dynamics 1997; 26:849-858.
12. Filiatrault A, Cervantes M. “Separation between buildings to avoid pounding during earthquakes.” Canadian Journal of Civil Engineering 1995; 22:164-179.

13. Davis RO. 1992. “Pounding of buildings modeled by an impact oscillator.” Earthquake Engineering and Structural Dynamics 1992; 21:253-274.
14. Jing HS, Young MH. “Random response of a single-degree-of-freedom vibro-impact system with clearance.” Earthquake Engineering and Structural Dynamics 1990; 19:789-798.
15. Jing HS, Young MH. “Impact interactions between two vibration systems under random excitation.” Earthquake Engineering and Structural Dynamics 1991; 20:667-681.
16. Hahn GD, Valenti MC. “Correlation of seismic responses of structures.” Journal of Structural Engineering ASCE 1997; 123:405-413.
17. Stavroulakis GE, Abdalla KM. “Contact between adjacent structures.” Journal of Structural Engineering ASCE 1991; 117:2838-2850.
18. Lin JH. “Separation distance to avoid seismic pounding of adjacent buildings.” Earthquake Engineering and Structural Dynamics 1997; 26:395-403.
19. Wolf JP, Skrikerud PE. “Mutual pounding of adjacent structures during earthquakes.” Nuclear Engineering and Design 1980; 57:253-275.
20. Wada A, Shinozaki Y, Nakamura N. “Collapse of building with expansion joints through collision caused by earthquake motion.” Proceedings of the 8th World Conference on Earthquake Engineering San Francisco, California 1984; 4:855-862.
21. Miller RK, Fatemi B. “An efficient technique for the approximate analysis of vibro-impact.” Journal of Vibrations, Acoustics, Stress, and Reliability in Design ASME 1983; 105:332-336.
22. Maison BF, Kasai K. “Analysis for type of structural pounding.” Journal of Structural Engineering ASCE 1990; 116:957-977.
23. Maison BF, Kasai K. “Dynamics of pounding when two buildings collide.” Earthquake Engineering and Structural Dynamics 1992; 21:771-786.
24. Sun H, Song Y, Liu W. “Contact-impact dynamic response analysis of structures.” Computational Structural Mechanics and Applications 1994; 11:31-42 (in Chinese with English abstract).
25. Papadrakakis M, Apostolopoulou C, Zacharopoulos A, Bitzarakis S. “Three-dimensional simulation of structural pounding during earthquakes.” Journal of Engineering Mechanics ASCE 1996; 122:423-431.
26. Hao H, Zhang SR. “Spatial ground motion effect on relative displacement of adjacent building structures.” Earthquake Engineering and Structural Dynamics 1999; 28:333-349.
27. Jeng V, Kasai K. “Spectral relative motion of two structures due to seismic travel waves.” Journal of Structural Engineering ASCE 1996; 122:1128-1135.
28. Filiatrault A, Cervantes M, Folz B, Prion H. “Pounding of buildings during earthquakes: A Canadian perspective.” Canadian Journal of Civil Engineering 1994; 21:251-265.
29. Pantelides CP, Ma X. “Linear and nonlinear pounding of structural systems.” Computers and Structures 1998; 66:79-92.
30. Chau KT, Wei XX. “Pounding of structures modelled as nonlinear impacts of two oscillators.” Earthquake Engineering and Structural Dynamics 2000; 30:633-651.
31. Papadrakakis M, Mouzakis H. “Earthquake simulator testing of pounding between adjacent buildings.” Earthquake Engineering and Structural Dynamics 1995; 24:811-834.
32. Filiatrault A, Wagner P, and Cherry S. “Analytical prediction of experimental building pounding.” Earthquake Engineering and Structural Dynamics 1995; **24**:1131-1154.
33. Chau KT, Wei XX, Guo X, Shen CY. “Experimental and theoretical simulations of seismic pounding between adjacent structures.” Earthquake Engineering and Structural Dynamics, 2003; 32(4): 537-554.
34. Hao H, Shen J, “Estimation of relative displacement of two adjacent asymmetric structures.” Earthquake Engineering and Structural Dynamics, 2001; 30: 81-96.
35. Zhu P, Abe M, Fijino Yozo, “Modelling three-dimensional non-linear seismic performance of elevated bridges with emphasis on pounding of girders.” Earthquake Engineering and Structural Dynamics, 2002; 31: 1891-1913.