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SEISMIC RESPONSE MITIGATION OF TOWER-PODIUM STRUCTURE USING PASSIVE FRICTION DAMPER: EXPERIMENTAL INVESTIATION

C.L. NG[1] and Y.L. XU[2]

SUMMARY

The application of control devices in coupled building systems has been recently recognized as an effective alternative for seismic protection. In consideration of passive control approach, most of previous studies focused on application of fluid dampers, and the buildings in the coupled systems were in similar structural configuration. This paper reports an experimental investigation to demonstrate the control effectiveness of passive friction damper as a coupling device that implemented in a scaled tower-podium structure tested on a shake table at The Hong Kong Polytechnic University. The passive friction damper was designed in such a way that the normal clamping force could be changed independently of frequency and amplitude. Dynamic characteristics of the test models in the cases of uncoupled and rigidly connected were first identified, which were followed by seismic simulation tests. The effects of coupling configurations including the cases of uncoupled, rigid coupled and passive controlled were evaluated. Passive control force level and ground motion on control performance were also examined. Installation of friction damper showed positive results in reduction of absolute acceleration and interstory drift responses of both buildings. Rigidly connecting tower and podium structures, in contrast, revealed its inherent tendency in amplifying the response of tall structure in particular.

INTRODUCTION

Increasing population together with growing social and commercial activities but limited land available is one of the common problems encountered in most modern countries, which consequently leads to more and more medium- and high-rise buildings either built closely to each other or in complex form. Among various construction forms, medium/high-rise building constructed with podium structure is a popular engineering scenario, by which a large open space for commercial uses, for instances, car parking, shopping arcade, restaurants or hotel lobbies, at ground level can be achieved. Presence of the podium, whose lateral stiffness may be much larger than that of the coupled medium- or high-rise buildings, leads

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[1] PhD. Student, Dept. of Civil and Structural Engrg., The Hong Kong Polytechnic University. Email: 01900362r@polyu.edu.hk
[2] Chair Professor of Structural Engineering, Director of Research Center for Urban Hazards Mitigation, Dept. of Civil and Structural Engrg., The Hong Kong Polytechnic University. Email: ceylxu@polyu.edu.hk

to an abrupt change in the lateral stiffness of the tall structure at the top level of the podium. Qu and Xu [1] studied the case of connecting podium structure to medium-rise structure with elastic link and smart damper, from which seismic responses of the tall structure at the upper level above connections were shown to be significantly amplified for the case of rigid connection, and regarded as “whipping effect”. Such a problem cannot be easily solved through modifying the structure form in a conventional way. Though numerous studies in recent decades demonstrate that various control technologies, such as passive, active and semi-active have high effectiveness on the protection of structures from large seismic events through employment of dissipative devices within a building, coupling buildings has recently been shown to be a viable alternative for the protection of adjacent structures. Interaction control by coupling buildings may be considered to be superior to conventional control of individual building in a sense that vibration of the two buildings can both be suppressed on one hand, and better reservation of internal spaces for architectural use can be provided on the other hand.

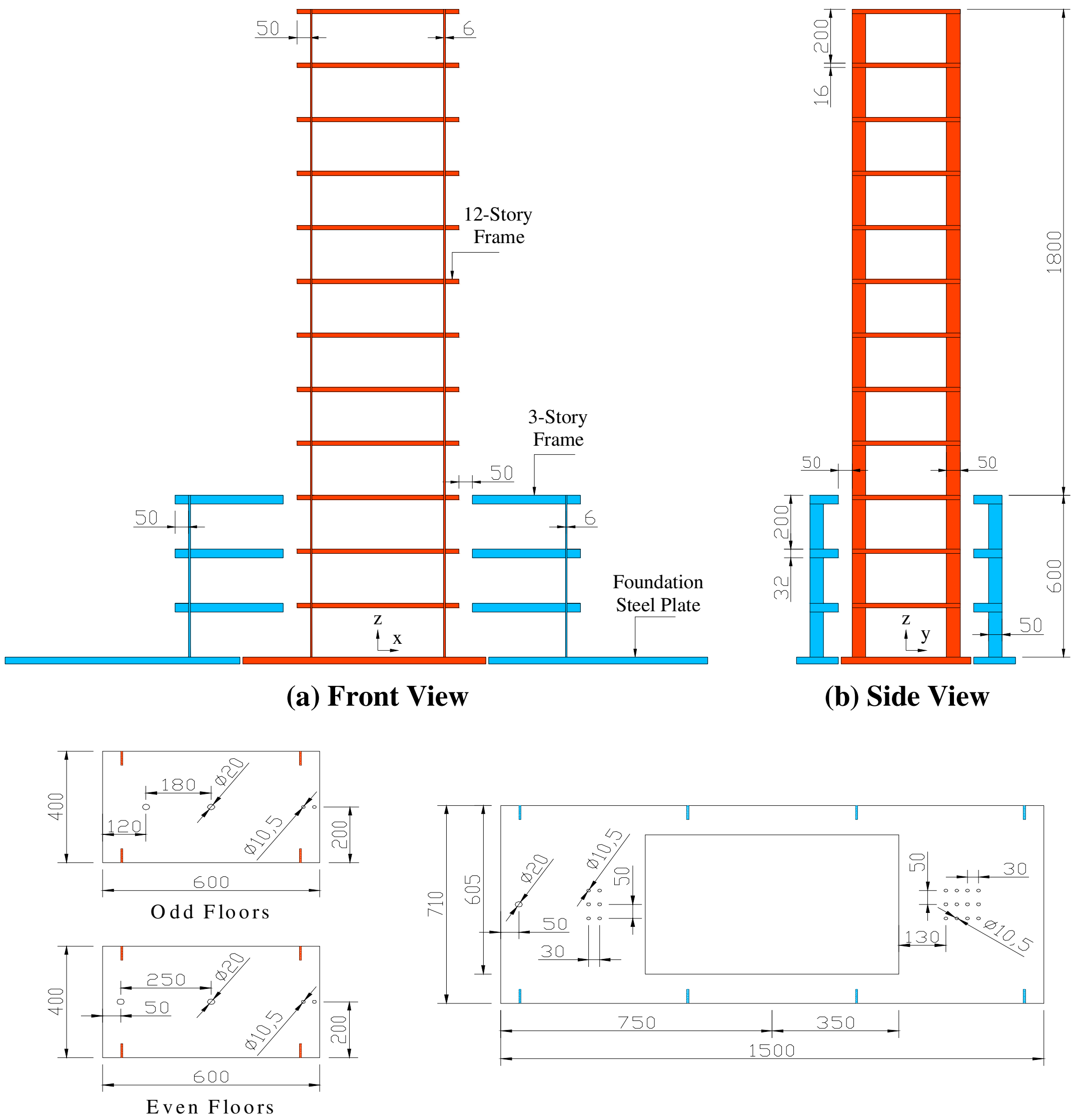
Researches on coupled building control have evaluated the effectiveness of different control strategies for the coupled low-, medium- and high-rise structures since 1990’s. With regard to the passive control approach (Luco et al. [2, 3]; Zhang et al. [4]; Yang et al. [5] and Xu et al. [6-8]), series of studies conducted either on numerical or experimental basis have reported positive results in response mitigation due to seismic excitations. Considering that passive friction damper is a well-recognized effective and stable device to suppress dynamic response of structures in many engineering practice, this paper presents an experimental investigation to explore the possibility of using friction dampers to link a podium structure to a tall building so as to alleviate structural vibration of both buildings. Passive friction damper of new configuration was designed and manufactured. Shaking table tests on a scaled 12-story tall building and a 3-story podium structure for three cases were performed in the study: (1) both structures were uncoupled; (2) both structures were rigidly coupled with rigid links; and (3) a single coupling link, using frictional damped device, interconnects the structures at the top floor of 3-story structure. Effectiveness of the passive friction damping system was evaluated through the comparison of seismic response between the conventional building, the rigid connected building, and the passive friction damped building.

EXPERIMENTAL SETUP

Experimental arrangement for system identification and seismic simulation tests of passive control included components of coupled building model, passive friction damper, shaking table, and data acquisition unit.

Coupled Building Model

The coupled building system was developed and constructed in the Structural Dynamics Laboratory of The Hong Kong Polytechnic University, which is comprised of a slender 12-story frame and a relatively stiff 3-story frame as shown in Figure 1. These two dissimilar shear building models represent a medium-rise and low-rise coupled building system, and denoted as “tower (or building 1)” and “podium (or building 2)” structure, respectively. The 12-story steel frame was constructed from 12 rigid floor plates in 600 mm × 400 mm × 16 mm and 4 equal sized rectangular strips with the cross section of 6 mm × 50 mm for columns. The tower structure is 2400 mm tall and the interstory height is 184 mm. The rigid floor of 3-story steel frame is 1500 mm × 710 mm in plan and 32 mm thick. A rectangular opening with 700 mm × 500 mm was made at the center of each floor plate, so that the 12-story building could be arranged inside the middle of the 3-story building and separated by a distance of 50 mm for all sides. Same sized rectangular strips as 12-story building were used for columns of 3-story building with 8 columns in total and 4 equally spaced on each side. The podium structure is 600 mm tall and the interstory height is 168 mm. The columns and steel floor plates of both buildings were connected with full butt weld and each building was respectively supported on a 25 mm thick steel base plate, and thus 12- and 3-story frame were fixed firmly on seismic simulator with 6 and 8 high tensile strength bolts, respectively. The story mass of buildings, including the mass of columns, is approximately $m_1 = 31.78$ kg and $m_2 = 182.28$ kg. All columns are 435 MPa high strength steel with 210 GPa modulus of elasticity. The 6 mm × 50 mm rectangular column strips were arranged in a manner that the first natural frequency of each building was appeared in x-direction, and building motion was also restricted in the x-direction, and therefore the coupled building system was effectively reduced to planar frames in the x-z plane. It is worth to point out that each steel floor plate was, without doubt, highly rigid in horizontal direction compared with columns, and therefore both frames are deemed to be a shear type model. To achieve a rigid coupling between tower and podium structures, high strength steel box-section tubes were used to link the first three floors of two buildings along the middle line of the buildings in the x-direction. Each coupling link was fixed on each sides of building by 2 high strength bolts. The assumption rigid connection was verified by examining the measured relative displacement between buildings on each floor.



(a) Front View (b) Side View

(c) Floor Dimension of 12-Story Building (d) Floor Dimension of 3-Story Building

Figure 1. Configuration of Coupled Building Model

Frictional Dissipative Device

Implementation of a passive friction damper (PFD), which was deigned and manufactured in the Structural Dynamics Laboratory of The Hong Kong Polytechnic University, was placed on the third floor of podium structure interconnecting the tower structure (see Figures 2a-b). The PFD is comprised of a clamping unit with a spring being compressed by an adjustable screw to provide specified normal load on friction material. A ring-shaped static load cell is attached between the spring and the friction material to quantify normal clamping force applied. Only stains are measured and clamping force is indirectly obtained. Brake lining pad is utilized as friction material because its friction coefficient is rather inherently independent of displacement and velocity. A steel plate is placed between a pair of friction pad material, and thus frictional sliding force is induced when the plate is moving due to the relative motion of coupled building. The control force was measured by a load sensor, and the slippage displacement of PFD was obtained with a linear variable differential transformer (LVDT) rigidly mounted between two structures on the third floor.

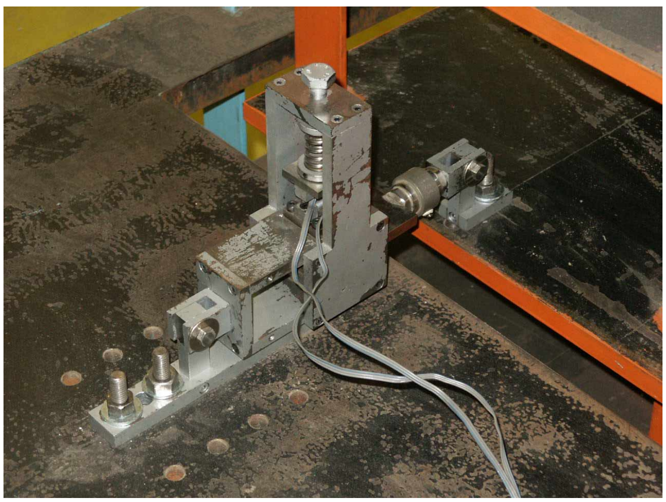


Figure 2a. Passive Friction Damper

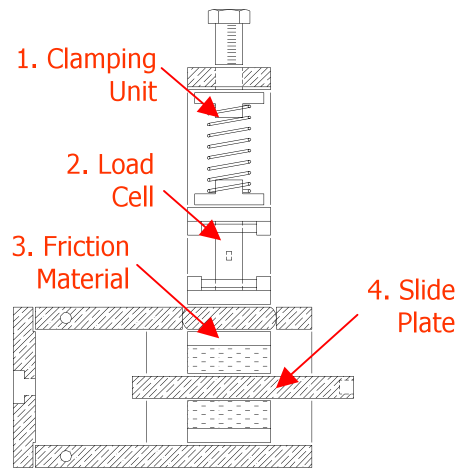


Figure 2b. Components of PFD

In order to appraise the performance on stability and reliability of the passive friction damper, the PFD specimen was first tested under displacement and forcing frequency control using a MTS hydraulic testing machine of type 8520 before implemented into the coupled building system. Influence on damper performance was assessed by a series of dynamic cyclic tests in which the parameters considered are as follows: excitation frequency (1, 3, 5 Hz), displacement amplitude (1, 3, 5 mm) and frictional sliding force (30, 50, 70, 90 N). The data were all measured at sampling rate of 200 Hz over sampling period of 2 minutes, and each parameter was varied while keeping the values of all the other parameters constant. One of the key objectives of the dynamic testing of frictional device is to examine its frequency dependency. The hysteretic behaviour over a range of excitation frequency and displacement amplitude of damper was tested and the test results with the displacement amplitude at 5 mm are shown in Figure 3a. Having sliding force at the level of 50 N, the hysteretic behaviour clearly demonstrates a high independency of excitation frequency for all displacement amplitudes. The same phenomenon was also observed for other sliding force levels. The effect of the displacement amplitudes was studied, in which excitation frequency was set at 1, 3 and 5 Hz, and sliding force level was varied from 30 to 90 N by step increment of 20 N. The measured force-displacement loops at 5 Hz are shown in Figure 3b. Again, stability of hysteretic loop is remained over the range of displacement amplitudes. As a whole, the hysteretic behaviour of PFD is both displacement and frequency independent. The use of the Coulomb law for friction modeling is thus deemed to be justified for the later analytical simulation of coupled building system in passive control.

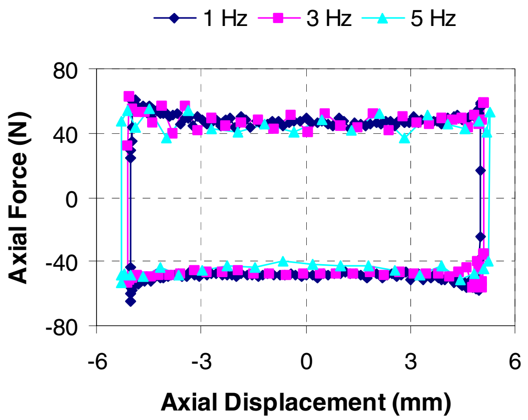


Figure 3a. Effect of excitation frequency

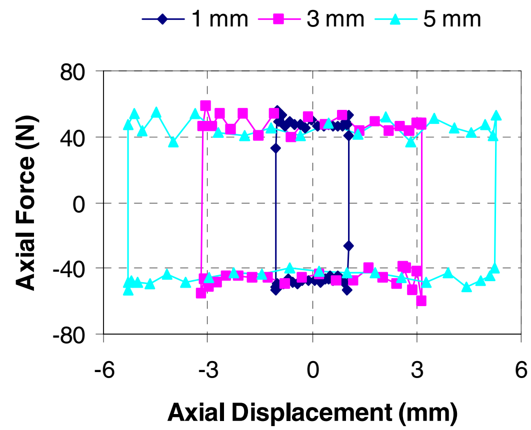


Figure 3b. Effect of Displacement Amplitude

Earthquake Simulator

A uniaxial earthquake simulator, that was designed and built by MTS Corporation, is housed in the Structural Dynamics Laboratory of The Hong Kong Polytechnic University. The 3 m × 3 m table is driven by a hydraulic actuator-servovalve assembly, by which the table is capable to reach a maximum displacement of ±100 mm and a maximum acceleration of ±1 g with a 10-ton of proof specimen mass. The nominal operational frequency range of the simulator is 0-50 Hz. Close-loop feedback either on control of displacement or acceleration can be employed to enhance the stability of table and the repetition of motion simulation.

Data Acquisition System

To develop high quality data acquisition, a sixteen-channel data-acquisition system was employed. The data acquisition system consists of twelve accelerometers associated with twelve charge amplifiers, a NI PCI-6052E data acquisition board, a LabVIEW Full Development System software package and PC computer works in Win-2000 operating system. High sensitivity PCB capacitive DC accelerometers, model 4370, produced by Brüel & Kjær North America Inc. are used for response measurement of the building stories and ground motion, the accelerometers have sensitivity of 100 pC/g. Charge amplifiers of model type 2635 produced by Brüel & Kjær are incorporated before the A/D converter quantizing the continuous signal to a digital representation in terms of finite number of bits, by which the effect of signal noise is therefore minimized. Lower and upper cut-off frequencies adopted for each channel are 0.2 Hz and 100 Hz, respectively. The NI PCI-6052E data acquisition board is a high-performance, high-precision multifunction board featuring 333 kS/s sampling rate, 16-bit of A/D resolution and ±0.05 to ±10 V input range. The NI PCI-6052E can measure at most 16 single-ended analog inputs in either differential or referenced input mode. It also features analog and digital triggering capability, as well as two 24-bit, 20 MHz counter/timers, and eight digital I/O lines. 16-bit analog outputs are also available for the NI PCI-6052E DAQ devices. Layout of sensors and arrangement of acquisition unit are shown in Figure 4.

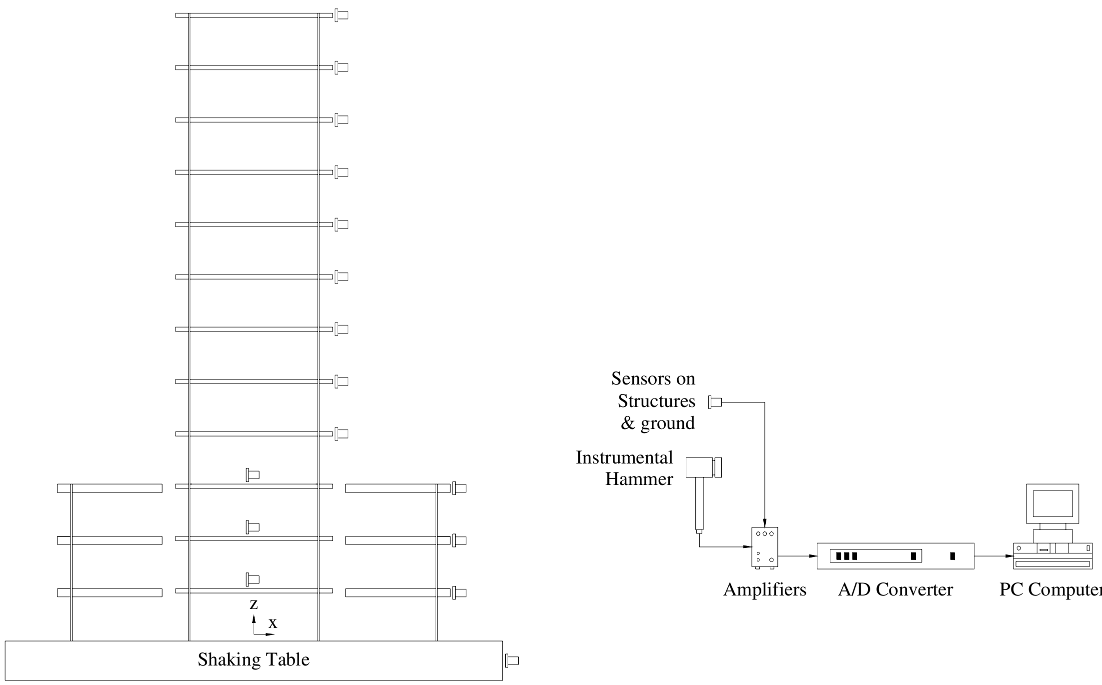


Figure 4. Schematic of Experimental Setup

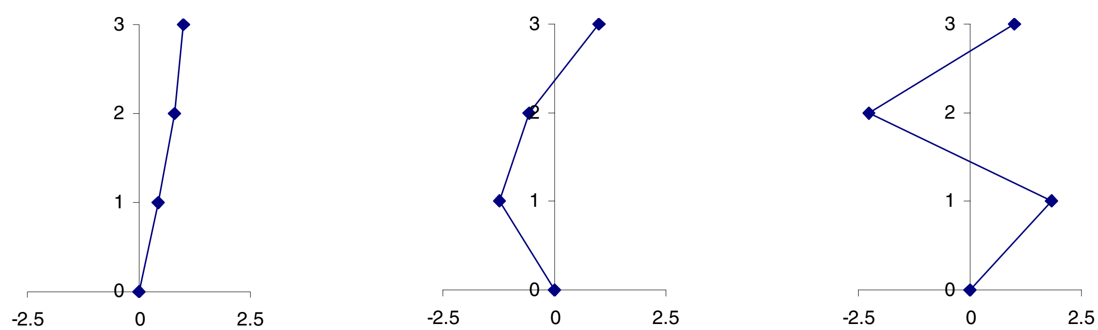
SYSTEM IDENTIFICATION OF TEST MODELS

The natural frequencies, mode shapes and modal damping ratios of the building models were identified experimentally for two cases: (1) the tower and podium structures are uncoupled; and (2) both building models are rigidly connected. The approach for obtaining parameters of dynamic characteristics was first by exciting the system over a wide frequency range of interest which can be done by either using an instrumental hammer randomly impact the test structure or inputting a band-limited white noise ground acceleration $\ddot{x}_g$, and first method was adopted in this study. Note that impulsive force was applied at top floor of buildings in either test case. Transfer functions were determined consequently by performing Fourier transform based on the recorded acceleration responses building floors as well as the impact force time series. All parameters including natural frequencies, mode shapes and modal damping ratios could be identified from the measured transfer function, and the bandwidth method was adopted to determine the modal damping ratios. The first three natural frequencies and modal damping ratios for different test cases are listed in Table 1, and the corresponding mode shapes are also displayed in Figure 5. It is noted that discrete time signals is sampled at 500 Hz which is more than twice the interested frequency component presented in sampled signal, and thus fulfills Nyquist sampling theory to reduce the effect of aliasing. To minimize the amount of distortion of the discrete spectral density functions due to spectral leakage, the sampled finite duration signals can be multiplied by a window function of hanning before the FFT is performed. The effects of noise and nonlinearities in the results are also reduced by 75% overlapping of each collection of signal to increase the number of average to 15 and attain a high frequency resolution at 0.061 Hz.

From Table 1, the lowest frequency (4.52 Hz) of coupled system indicates that it is between the values of fundamental frequencies for each of the uncoupled structures. The ratios of the uncoupled frequency to the coupled one for the first three modes of tall and short buildings are 0.81, 1.08 and 1.20 for the tall building and 2.19, 2.68 and 2.57 for the short building respectively when both models are in rigid connection at the lowest three floors. It should be mentioned that this ratio could act as a control feasibility/capability index indicating the potential effectiveness of interaction control approach towards each structure. The less of ratio approaching to unity, the higher control efficiency regarding that mode can be expected. In general, for the case of tower-podium structure, there always is a large difference in dynamic characteristics between each uncoupled building, where tall building normally possesses a much lower frequency for the first several modes, and thus this control scenario is suitable for this building configuration. Additionally, a vibratory mode cannot be controlled by a coupling link which is placed at a node of that mode as reported by Fukuda *et al* [9]. It can be observed from the first three vibratory modes shown in Figure 5b that third floor is not located as a nodal position among all three modes. Therefore, linking the structures with PFD on third floor is an appropriate decision.

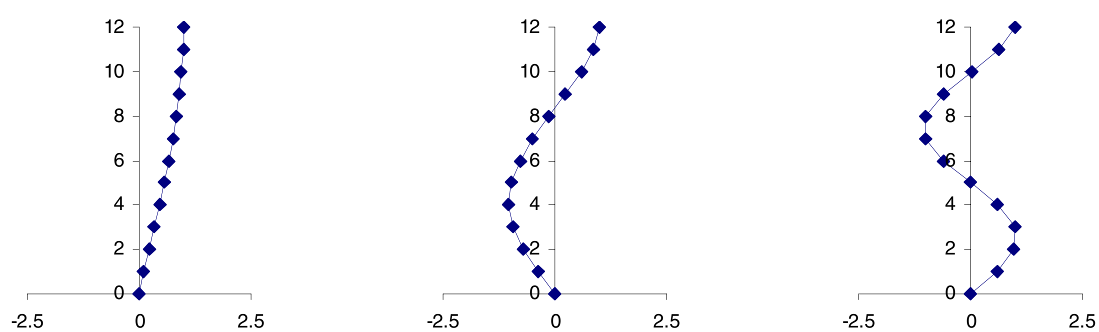
Table 1. Identified Dynamic Properties of Building Models

Mode No.	Building System Configuration		
	Uncoupled 3-Story Frame	Uncoupled 12-Story Frame	Rigidly Connected 12-3-Story Frame
	(a) Natural Frequency (Hz)		
1^{st}	9.89	3.66	4.52
2^{nd}	27.83	11.23	10.38
3^{rd}	40.28	18.80	15.63
	(b) Modal Damping Ratio (%)		
1^{st}	0.66	1.43	1.10
2^{nd}	0.26	0.61	0.53
3^{rd}	0.16	0.48	0.58



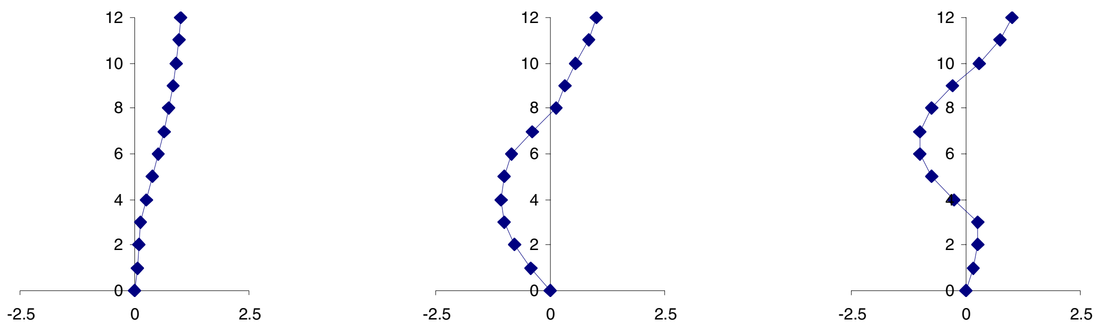
Mode 1 (9.89 Hz) Mode 2 (27.83 Hz) Mode 3 (40.28 Hz)

(a) 3-Story Building Model



Mode 1 (3.66 Hz) Mode 2 (11.23 Hz) Mode 3 (18.80 Hz)

(b) 12-Story Building Model



Mode 1 (4.52 Hz) Mode 2 (10.38 Hz) Mode 3 (15.63 Hz)

(c) 12-3-Story Coupled Building Model

Figure 5. Mode Shapes of the 12-Story and 3-Story Frames

EVALUATION OF EXPERIMENTAL RESULTS

Ground Motion

The coupled building system was subjected to 4 simulated earthquakes, the intensities of which were scaled to 0.15g and the durations of which were scaled by a factor of 1/2 and 1/3. The time histories of historical earthquakes were derived from the following sources: (1) El Centro. The N-S component recorded at the Imperial Valley Irrigation District substation in El Centro, California, earthquake of May 18, 1940; (2) Hachinohe. The N-S component recorded at Hachinohe City during the Tokachi-oki earthquake of May 16, 1968; (3) Northridge. The N-S component recorded at Sylmar Country Hospital parking lot in Sylmar, California, during the Northridge, California, earthquake of January 17, 1994; and (4) Kobe. The N-S component recorded at the Kobe Japanese Meteorological Agency (JMA) station during Hyogo-ken Nanbu earthquake of January 17, 1995.

Response Time Histories

It is worth to notice that only acceleration responses were measured, floor displacements were obtained through twice integrations of the acceleration time series. To avoid significant long-period signal distortions which may be produced in velocities and displacements derived by integrating acceleration time series, a scheme developed by National Strong-Motion Program (NSMP)[†] for semi-automated processing of digital data was employed to address this problem.

Evaluation of Control Performance

Response evaluation of the coupled building system is considered with 3 cases including uncoupled, rigidly connected and passively controlled. Two indices were used as evaluation criteria related to the building responses. The two criteria are based on rms interstory dirft ratio (J_1), rms acceleration ratio (J_2):

$$J_1 = \frac{\max\limits_i \left[d_i^{rms} \right]}{d_{1,\,uncoupled}^{rms}}, \quad J_2 = \frac{\max\limits_i \left[\ddot{x}_i^{rms} \right]}{\ddot{x}_{n,\,uncoupled}^{rms}} \tag{8}$$

over the range of floor number i for the tall and short buildings, where $d_i(t)$ and $\ddot{x}_i(t)$ are the interstory drift and absolute acceleration time histories respectively of the ith floor, and $d_{1,\,uncoupled}^{rms}$ and $\ddot{x}_{n,\,uncoupled}^{rms}$ denote the rms interstory drift at the 1st floor and the rms absolute acceleration at the top floor of the

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† http://nsmp.wr.usgs.gov

uncoupled structures. The seismic response ratios in accordance with the evaluation criteria for different coupled building configurations are presented in Tables 2 and 3.

Table 2. Response Ratios for Coupled Building System under 1/2 Time-Scaled Earthquakes

Evaluation Indices	Earthquake	Coupled Building Configuration				
		Rigid	Passive (25 N)	Passive (45 N)	Passive (65 N)	Passive (85 N)
J_1 Maximum RMS Drift	El Centro	0.7431 (1.7774)	0.6658 (0.8162)	0.5387 (0.7629)	0.4557 (0.7836)	0.3885 (0.8036)
	Hachinohe	0.7517 (1.3679)	0.6539 (0.7516)	0.4944 (0.7120)	0.4173 (0.7403)	0.3570 (0.7876)
	Northridge	**1.3776** (2.1354)	**0.8369** (0.8504)	**0.7237** (0.7655)	**0.6466** (0.7788)	**0.5889** (0.7901)
	Kobe	0.9402 (**3.3026**)	0.7993 (**0.9661**)	0.6781 (**0.9722**)	0.5876 (**0.9995**)	0.5053 (**1.0337**)
J_2 Maximum RMS Acc.	El Centro	1.0373 (0.8205)	0.7107 (0.8071)	0.5820 (0.7269)	0.5186 (0.7236)	0.4794 (0.7215)
	Hachinohe	1.0714 (0.6667)	0.7213 (0.7782)	0.5793 (0.6756)	0.5266 (0.6905)	0.4869 (0.6940)
	Northridge	**1.7411** (0.9162)	0.8116 (0.8461)	**0.7470** (0.7181)	**0.6876** (0.7089)	**0.6750** (0.7238)
	Kobe	1.2822 (**1.2648**)	**0.8553** (**0.9094**)	0.7402 (**0.8933**)	0.6467 (**0.8883**)	0.5698 (**0.9629**)

1. Value in parenthesis represents the response ratio of 3-story building.
2. Bolded value represents the maximum value among four seismic motions.

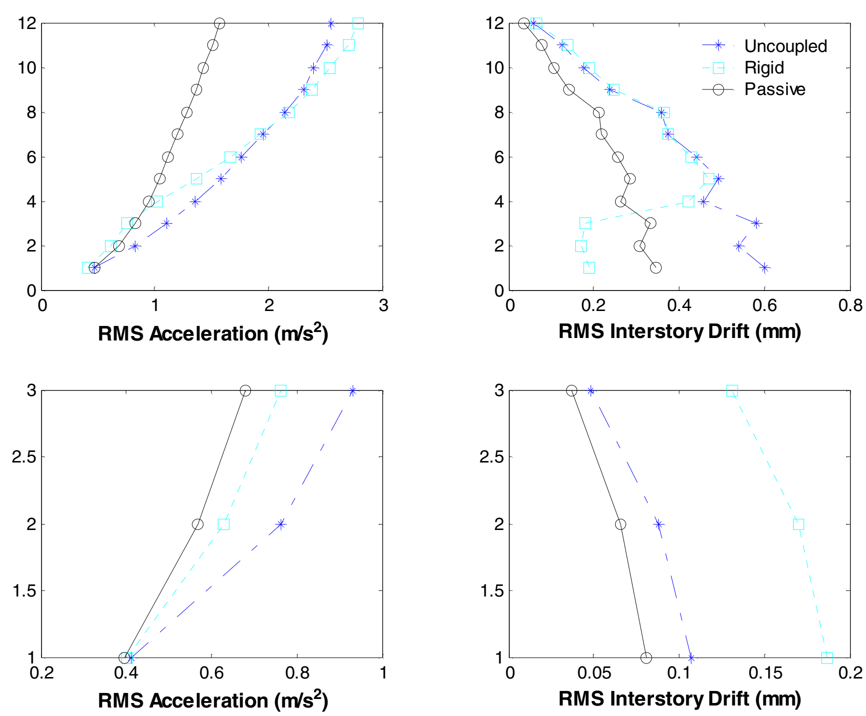
Table 3. Response Ratios for Coupled Building System under 1/3 Time-Scaled Earthquakes

Evaluation Indices	Earthquake	Coupled Building Configuration				
		Rigid	Passive (25 N)	Passive (45 N)	Passive (65 N)	Passive (85 N)
J_1 Maximum RMS Drift	El Centro	**1.1667** (1.6282)	0.7000 (0.8503)	0.5873 (0.7767)	0.5237 (0.8184)	**0.4521** (0.8387)
	Hachinohe	0.5977 (1.0936)	**0.7514** (0.7891)	**0.6138** (0.7234)	0.5142 (0.7544)	0.4284 (0.7359)
	Northridge	0.6257 (0.7287)	0.5845 (0.7696)	0.4156 (0.6609)	0.3345 (0.6260)	0.3074 (0.6374)
	Kobe	1.0373 (**2.388**)	0.7169 (**0.9903**)	0.5892 (**0.9697**)	**0.5244** (**0.9148**)	0.4460 (**0.8519**)
J_2 Maximum RMS Acc.	El Centro	**1.5702** (0.8260)	0.6873 (0.8053)	0.6183 (0.7261)	0.5982 (0.7368)	**0.5946** (0.7614)
	Hachinohe	0.8846 (0.7074)	**0.7555** (0.8091)	**0.6477** (0.7225)	**0.5984** (0.7111)	0.5781 (0.6921)
	Northridge	0.9675 (0.5054)	0.6599 (0.7758)	0.5307 (0.6627)	0.4610 (0.6157)	0.5181 (0.5699)
	Kobe	1.3472 (**0.9877**)	0.7445 (**0.9896**)	0.6236 (**0.9408**)	0.5586 (**0.9184**)	0.5021 (**0.8614**)

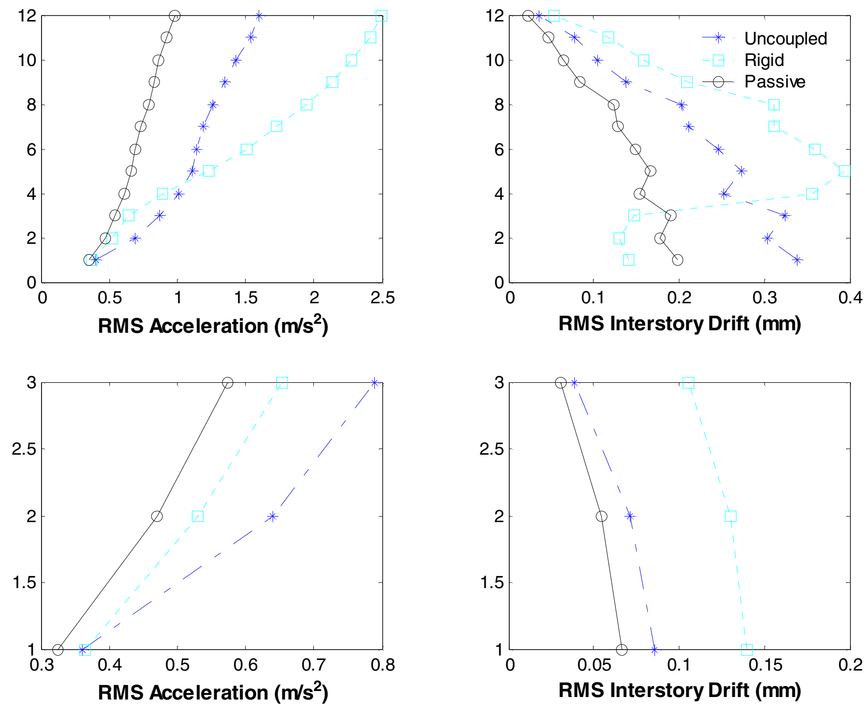
1. Value in parenthesis represents the response ratio of 3-story building.
2. Bolded value represents the maximum value among four seismic motions.

In the case of rigidly coupled building, it is clearly indicated that the rms accelerations of short building are slightly reduced, whereas results in a significant trade-off of acceleration response of tall building. Consider the rms story acceleration of the tall building, there is a on average 19% increase, and over 50% and 70% amplification is observed under 1/2 time-scaled Northridge and 1/3 time-scaled El Centro earthquake, respectively. Additionally, rigid coupling is definitely unfavorable to the podium structure in view of the interstory drift, and any reduction in interstory drift of tower building is really uncertain. Therefore, rigidly connecting the buildings is not a beneficial configuration for the coupled building system as a whole. The passive control is clearly able to reduce the maximum rms acceleration responses by 15-40% and the maximum rms interstory drifts by 16-55% for the tall building. Responses reduction of the short building is less considerable, and there are not more than 14% and 15% drop in maximum rms accelerations and maximum rms interstory drifts, respectively. The relatively smaller responses attenuation is because of the comparatively heavy floor mass and high floor stiffness. It is worthy to point out that under some seismic ground motions, a drop in responses reduction for podium structure is noted for large passive control force although further responses reduction for tower structure can be achieved. Thus the level of control force should be designed with awareness to balance the control performance of both buildings.

The acceleration and interstory drift response profiles of each building in rms magnitude over the case of uncoupled, rigidly coupled and passively controlled (45 N) are shown in Figure 6. Increasing proportion of higher mode participation is observed in acceleration response under earthquakes with time scale factor of 1/3, while first mode is dominant for 1/2 time-scaled earthquake excited acceleration response. It is clear the passive control strategy is capable in reducing either low or high mode of acceleration response of the coupled building system in particular to the tower structure. Similar ability in interstory drift reduction also indicates that for all time-scaled historical record earthquakes, alleviation of the interstory drift at the base of the tall building is shown to be very promising. Note that performance of passive control strategy in controlling interstory drift of podium structure is less obvious, and the possible reason as mentioned before is due to the high floor stiffness and the low ratio of maximum control force to floor weight (5%).



(a) 1/2 Time Scale Factor



(b) 1/3 Time Scale Factor

Figure 6. RMS Response Profiles of Coupled Building System under 0.15g El Centro Earthquake

CONCLUSIONS

Coupled building systems employed with friction dampers in passive control approach were investigated experimentally. The frictional damping device connected 12- and 3-story building model on top floor of the short building, and the measurements of the structures were the absolute accelerations of building stories. The buildings were subjected to four simulated historical earthquakes that were magnitude- and time-scaled, whereby the rms responses for the passive control case were compared to that of uncoupled and rigidly connected building systems. The results showed that the seismic resistance performance of the two steel frames was enhanced by the implementation of passive friction damper, whereas an adverse effect on structural responses of rigidly coupled building was observed. Therefore, rigid connection is not necessarily a good seismic design scenario. The types of ground motion did affect the seismic responses of the buildings but only slightly altered the damper control performance.
Selecting control force level at 15% of the tall building's floor weight, the passive control approach reduced the rms magnitude of interstory drift and acceleration by 16-55% and 15-40%, respectively, for the tall building subjected to the four simulated earthquakes. Considering podium structure, 3-29% reduction in rms interstory drift and 6-34% reduction in rms acceleration were obtained. Proper design in the magnitude of passive control force is advised because there will be a trade-off in vibration reduction of the podium structure for further structural suppression of the tall building.

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