Shake Table Testing of a Self-Centering Post-Tensioned Steel Frame

D. Wang[1] and A. Filiatrault[2]

[1] *Structural Engineer, DCI-Engineers Inc., Seattle, WA, USA*
[2] *Professor, Dept. of Civil, Structural and Environmental Engineering, The State University of New York at Buffalo, Buffalo, NY, USA*
Email: dwang@dci-engineers.com, af36@buffalo.edu

ABSTRACT :

This paper describes seismic shake table tests on a 3-story, 2-bay, steel plane frame model incorporating novel Self-Centering Post-Tensioned (SCPT) connections. This system, unlike traditional welded steel frames, implements high strength post-tensioned strands along with sacrificial yielding elements in each beam-to-column connection and is particularly appealing for important buildings from an initial investment stand point. In order to increase the scale of the specimen as much as possible, the plane test frame was sandwiched between existing gravity supported masses. This allowed the seismic testing of a three-story structure incorporating exterior and interior SCPT connections over the three stories. For comparison purposes, a second conventional fully-welded Steel Moment Resisting Frame (SMRF) with similar beam and column sections as those of the SCPT frame was also tested. Those two test models were subjected to various ground motions of increasing intensities. The results of the tests indicated that the displacement response of the SCPT frame was very similar as that of the fully-welded SMRF but the acceleration response was reduced. While the beams of the SMRF yielded under the largest seismic excitations, the energy dissipation mechanism of the SCPT frame was limited to the Energy Dissipating (ED) bars without inelastic deformations of the beams and columns. This result suggested that significant saving would be associated with the repair of the SCPT building by replacing the ED bars compared to the replacement of the damaged beams or columns in the SMRF building after earthquakes.

KEYWORDS: Self-Centering, Post-Tensioned, Moment-resisting Frame, Shake Table

1. INTRODUCTION

In the last thirty years, an increase in mid- to high-rise buildings has been observed because of the increased needs of space by expanding urban areas. Steel Moment Resisting Frame (SMRF) constructions are most widely used in mid- and high-rise buildings in North America. Prior to the 1994 Northridge earthquake in California, it was believed that SMRF represented one of the most adequate forms of earthquake-resistant construction, as evidenced by very few failures or severe damaged observed in SMRFs and steel structures in general following earthquakes. In fact, prior to 1994, the only severe damage reported in steel structures, occurred in the Pio Suarez building in the 1985 Mexico City Earthquake (SAC 1995).

However, in the 1994 Northridge Earthquake in Los Angeles, over 100 SMRF structures experienced brittle beam-to-column connection fractures as a result of this Richter magnitude 6.8 seismic event (SAC 1995). Therefore, after the Northridge earthquake, a significant research effort was conducted to provide new design procedures and guidelines to insure a good seismic performance of SMRFs. At the same time, other parallel research efforts were carried out to develop new types of steel structures as alternatives to SMRF. One of those structures is the Self-Centering System (SCS). The SCS, involving post-tensioned and energy dissipating elements or other special devices with self-centering property, exhibits nonlinear softening behavior, ductility and energy dissipation. Re-centering forces are provided by post-tensioned systems or other special devices to return the structure to or near to its original position and eliminate or diminish residual deformations after earthquakes.

Different types of SCS have been studied analytically and experimentally mostly in the last fifteen years. Those SCS includes Shape Memory Alloys (SMA) dampers, Energy Dissipating Restraint (EDR), SHAPIA damper, rocking self-centering connections, Post-Tensioned Energy Dissipating (PTED) connections, Post-tensioned Friction Damped Connections and others. (Wang, 2007). All of those configurations were proven to possess good seismic performance. However, to the knowledge of the authors, the experimental research of SCS implemented in an entire Steel Moment Resisting Frame (SMRF) has not been conducted. All of the previous experimental works associated with SCS in SMRF were concentrated on subassemblies such as beam-column joints or base-column joints. There is a need to experimentally investigate the seismic performance of a complete steel frame using SCS.

In this paper, shake table testing with two 1/3 scale steel frames: SMRF and Self-Centering Post-Tensioned (SCPT) frame, is carried out to evaluate the seismic performance of each system. The displacement response, acceleration response and the energy dissipation of the two frames are compared and improved detailing of the SCPT connections is suggested.

2. TEST SPECIMEN DESIGN

2.1 Prototype Design

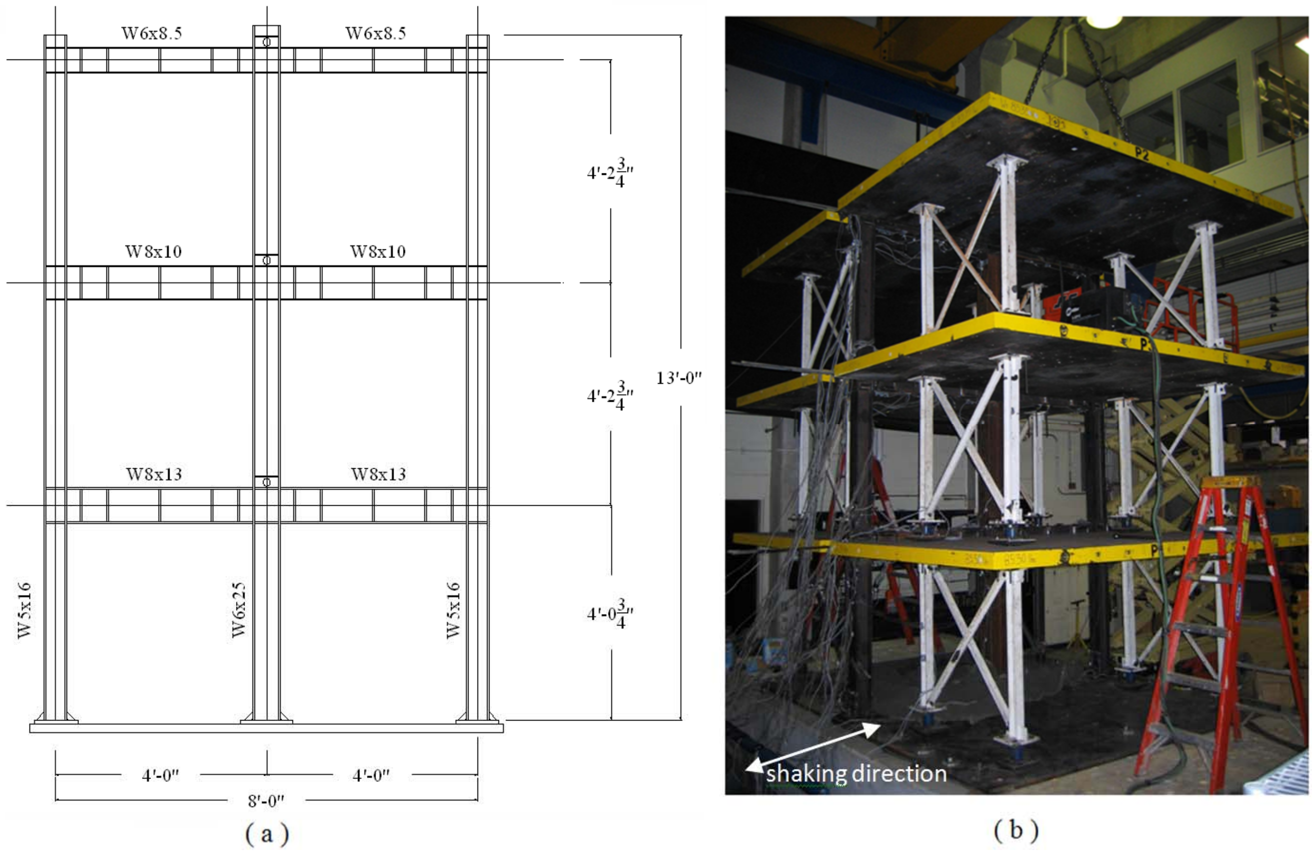
The prototype building associated with the test specimen is considered to be the MCEER West Coast Demonstration Hospital termed as WC70 (Yang et al, 2002). However, due to the limitations of dimensions and gravity capacities of the shake table, it was difficult to design a whole-frame assembly model scaled exactly from the WC70 structure. Therefore, the prototype was re-designed based on the international building code (IBC 2003). To achieve the similar seismic behavior of the WC70 frame, the re-designed prototype building was assumed to be a small outpatient building with the same structural characteristics and location as the WC70 frame. The gravity load per unit area in the new prototype structure was identical to that of the WC70 frame.

2.2 Model Scaling and Specimen Description

The prototype needed to be scaled as a model compatible with the requirements of the shake table. Considering those limitations of the shake table as well as the added artificial mass, two fundamental Scaling Factors (SF) were determined as the length and mass SF and were made equal to 3 and 11.7, respectively. Thereafter, other SFs such as time, acceleration, force and etc. were calculated based on the above two factors. Then, the SMRF model was obtained by scaling the prototype with those SFs. The design of the SPCT connections was based on favoring the reduction of accelerations over the reduction of displacements (Wang, 2007).

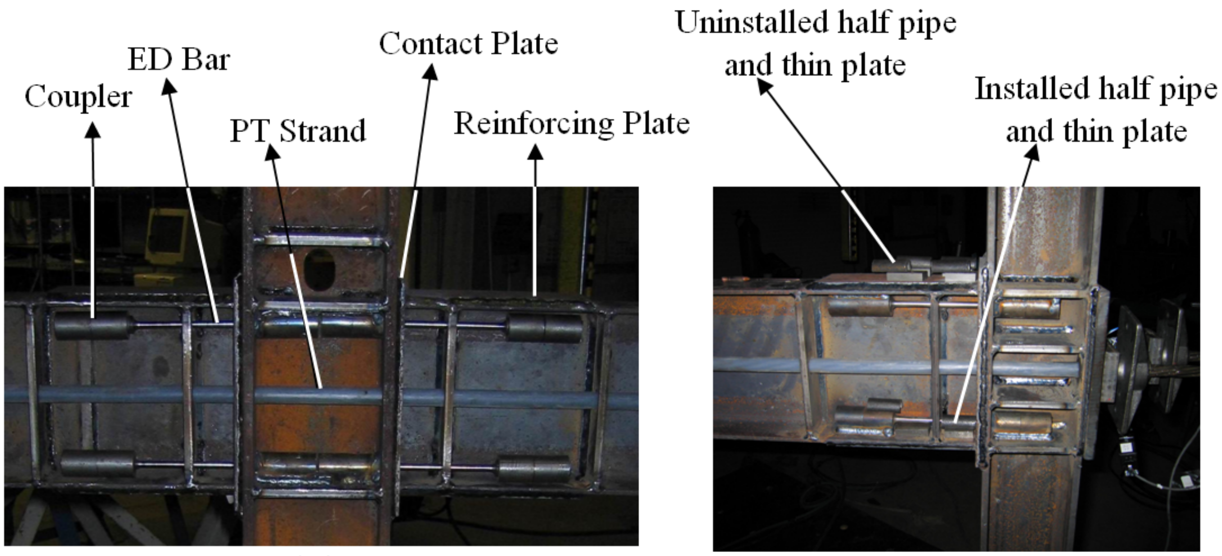
As shown in Fig. 1(a), the frame models are 2-bay 3-story SMRF and SCPT frames with the same dimensions. Fig. 1(b) indicates that a Floor Mass Simulator (FMS) is used for the artificial mass simulated for the seismic weight. Due to the rocking support design, the FMS performs as a pin-based structure in the shaking direction providing no lateral stiffness and its bracing system resists the deformations in the transverse direction. Each frame model (SMRF and SCPT frame) was installed between two frames of the FMS and was connected to the FMS by circular steel bars through the central column at each level.

The SMRF frame incorporated the conventional fully welded connections, i.e. the web and flange of beams were directly welded to the flange of columns, while the SCPT frame model used SCPT connections without any welding between the beams and columns. As illustrated in Fig. 2, Post-Tensioned (PT) strands were installed along the beam to provide a “clamping” force for the beam-column connections. The PT strands also provided a re-centering capability to the structural system under lateral earthquake loading. Also, four Energy Dissipating (ED) bars were welded to each beam-column connection. Each ED bar was connected to a threaded mechanical coupler. When gap openings between the beam-column interfaces occur in the large lateral deformation, the ED bars will yield in tension and compression and absorb energy during seismic shaking. In the SCPT model, the two 0.6 or 0.5 inch DYWIDAG System International (DSI) PT strands in different levels were used and initially post-tensioned to 37, 30 and 27 kips in level 1, 2 and 3, respectively and the #6 DSI bars with 75 ksi yielding strength machined to 5/16 or 9/32 inch diameter were installed as the ED bars.



(a) (b)

Figure 1 SMRF and SCPT frame model (a) elevation view (b) frame with FMS installed on the shake table



(a) (b)

Figure 2 SCPT beam-column connections (a) interior connection (b) exterior connection

3. TEST PROTOCOLS

The ground motion used in the shaking table testing was selected among an ensemble of 25 synthetic MCEER simulated earthquake records (Wanitkorkul and Filiatrault, 2005) with a 10% probability of exceedance in 50 years for a high seismic zone in Southern California, since the prototype structure was designed for a life safety performance. Two series of shake table tests were conducted for both the SMRF and SCPT frame models. In the first series, the selected ground motion was used, while in the second series, the direction of the ground motion was reversed (polarity). Different amplitudes from 25% to 150% of the selected ground motion were used during the tests. Low level white noise tests were also conducted after each seismic test and used to monitor the changes in dynamic characteristics of the test frames such as their natural frequencies and mode shapes. This white noise excitation had 0.05g amplitude and a wide frequency band (0.5 - 50Hz). Sine-sweep and snap-back tests were also conducted to double check the system information.

4. EXPERIMENTAL RESULTS

4.1 Absolute Acceleration Response

As reported by Kircher (2003), the damage to nonstructural components led to 50% of the $18.5 billion total loss during the 1994 Northridge earthquake. Recall that the prototype building of the SCPT model was designed as a steel frame hospital, which would have many acceleration-sensitive nonstructural components. Therefore, it is very important to reduce the acceleration response in order to decrease the damage of the acceleration-sensitive nonstructural components and the corresponding economic loss after seismic hazards.

The peak floor absolute acceleration responses of two models for different-intensity ground motions are shown in Fig. 3. In those bar charts, the peak accelerations of the SCPT model are always lower than those of the SMRF model. For the low-intensity (25%-100%) ground motions, the acceleration in the SCPT model is reduced by up to 37% compared to that in the SMRF model, while this reduction reaches up to 41.8% after the 100%-intensity excitations. The large reduction in the acceleration response demonstrates the excellent seismic performance of the SCPT structure.

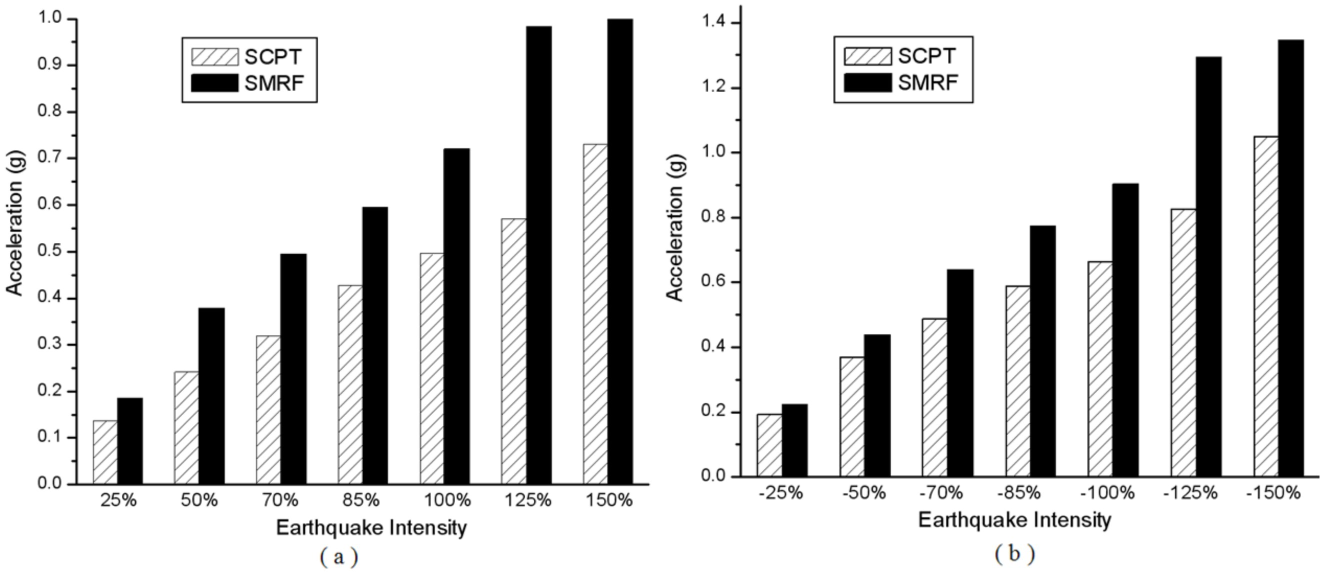


Figure 3 Peak floor absolute acceleration response (a) at 2nd level (b) at 3rd level

4.2 Inter-Story Drift Response

The displacement response of the two model frames is presented by comparing their inter-story drifts. As illustrated in Fig. 4, it can be observed that the maximum inter-story drifts of the two model structures for the 100%-intensity ground motions are lower than 1%. Considering the maximum inter-story drift for life safety is 2.5% according to FEMA (FEMA 356, 2000), those two model structures performed beyond this performance level. It is also indicated that the drift values of the SCPT models are similar or slightly higher compared to those in the SMRF models in the corresponding intensity seismic tests.

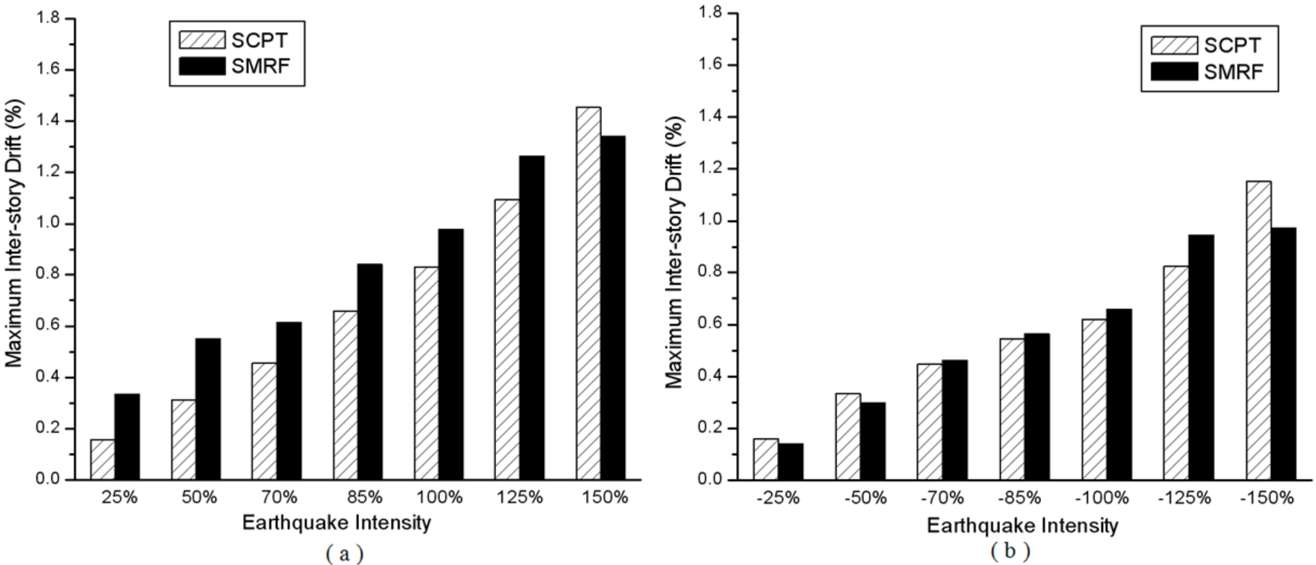
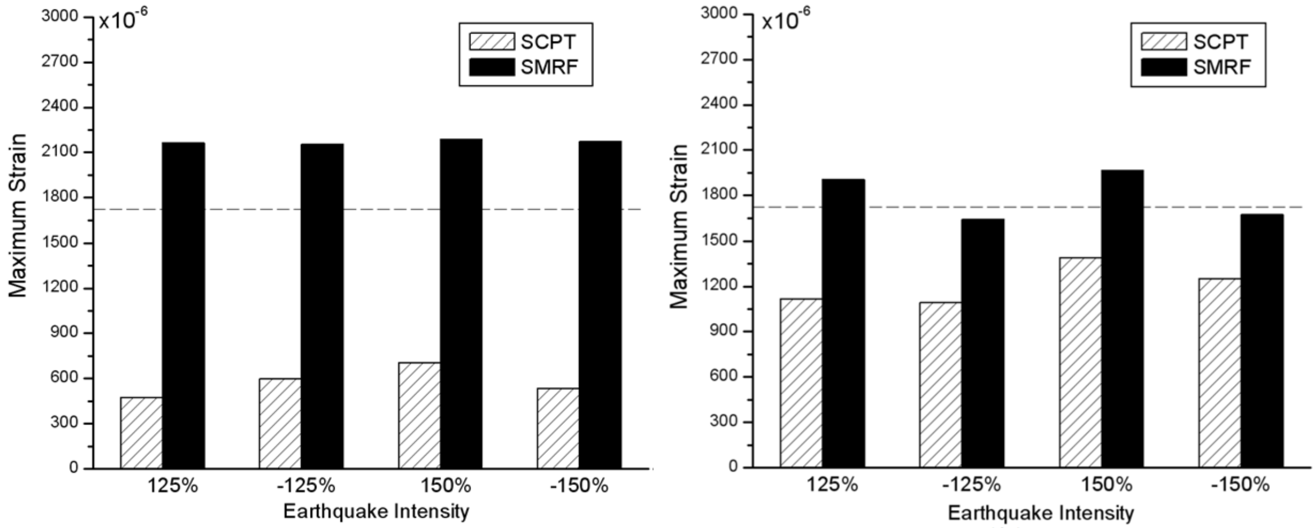


Figure 4 Maximum inter-story drift (a) at 1st level (b) at 3rd level

4.3 Strain Response for Beams and Columns

The yielding strain of the steel with 50ksi yielding strength used for the beams and columns in the SCPT and SMRF models was 1724x10^{-6} with the corresponding elastic modulus 29000ksi. If the strain on the exterior surface of the beam or column flanges exceeds the yielding strain, the beam or column can be considered to begin yielding. Through checking the strain time histories in different seismic tests, it was found that no yielding occurred in the beams or columns of the two models for the seismic tests from 25% to 100% intensity, which demonstrates that the good seismic behavior of both models.

As seen in Fig. 5 (a), the strains in the beams of the 1st and 2nd floor of the SMRF model exceed the yield limit, while no yielding occurs in those same beams of the SCPT model. As expected, the ED bars yielded instead of the beams as a result of the gap opening in the beam-column joints. These results demonstrate that under the high intensity (125% and 150%) earthquake hazards, the SCPT structure performed better since no damage occurred to the main structural components. The cost associated with repair of the main structural components is much more than the cost of replacing the ED bars. Fig. 5 (b) shows that the base columns of the SMRF model yielded slightly, while still no yielding occurred in the SCPT structure, which is in accordance with the conclusion based on the performance of the beams. For the weak-beam-strong-column design philosophy of the SMRF building, the columns should not begin yielding before most of the beams yields. However, the two exterior columns were reduced from a w5x19 to w5x16 section due to the material availability, which decrease the yielding strength of the columns. Also, the beams in the 1st and 2nd floor of the SMRF model had yielded. Considering these two facts, the slight yielding of the columns can be acceptable based on the weak-beam-strong-column theory.



(a) (b)

Figure 5 Maximum strains (a) of the beam in the 2nd level (b) of the base exterior column

4.3 Energy Evaluation

The energy dissipation capacity is an important index to represent the seismic performance of a structure during earthquakes. According to the theory of structural dynamics, the energy balance equation is given by:

$$E_K + E_D + E_F = E_I \tag{4.3.1}$$

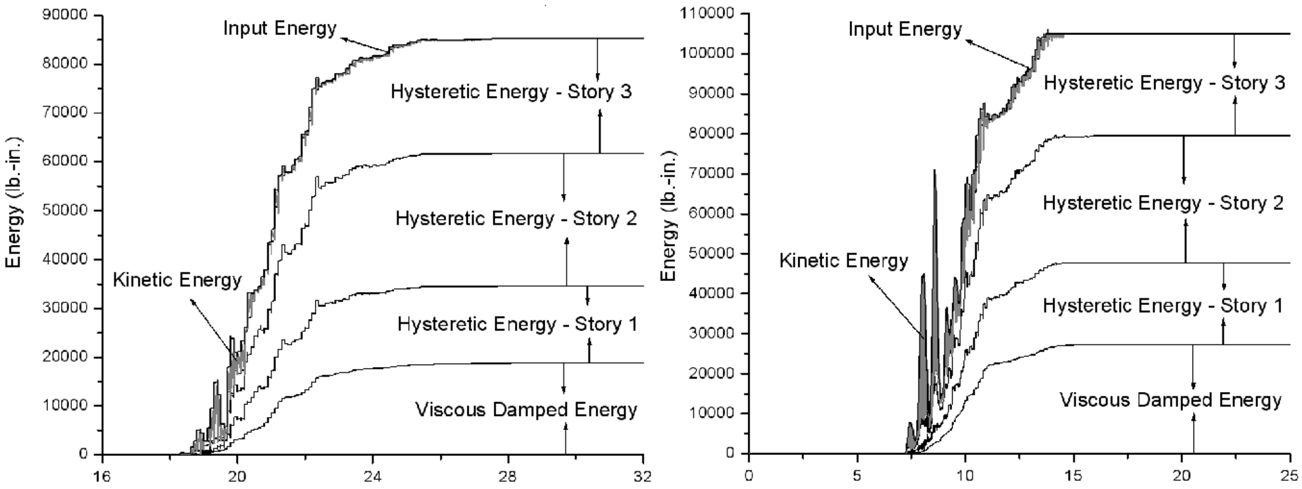
Where E_I is input energy; E_K is kinetic energy; E_D is viscous damped energy; E_F is hysteretic energy including unrecoverable and recoverable strain energy.

Based on Eq. (4.3.1), the absolute energy equation for the multi-story building subjected to an earthquake excitation was derived as follows (Uang and Bertero, 1990):

$$\frac{1}{2}[\dot{v}_a]^T M[\dot{v}_a] + \int[\dot{v}_r]^T Cd[v_r] + \int[f_s]^T d[v_r] = \int(\sum_{i=1}^{N} m_i \ddot{v}_{ai})dv_g \tag{4.3.2}$$

Where $[\dot{v}_a]$ is absolute velocity vector; M is diagonal mass matrix; $[\dot{v}_r]$ is relative velocity vector; C is damping matrix; d is derivative sign; $[v_r]$ is relative displacement vector; $[f_s]$ is restoring force vector; m_i is lumped mass of the i-th floor; $\ddot{v}_{ai}$ is absolute acceleration at the i-th floor; v_g is ground displacement.

In Eq. (4.3.2), the kinetic energy, viscous damped energy and the input energy can be directly calculated by the obtained experimental results while the hysteretic energy may be calculated as the result of E_I-E_K-E_D. The velocity time histories, used to calculate the kinetic and viscous damped energy, were obtained by derivatives of the corresponding floor displacement time histories. The damping matrix used in this calculation for the two models were obtained from snap-back tests. The energy distribution time history during the 125%-intensity test for the two models is shown on Fig. 6 (a) and (b), respectively. The label “hysteretic energy –story i” means the hysteretic energy absorbed by the whole structure due to the input energy generated by the motion of the i-th floor mass. The hysteretic energy includes the recoverable and unrecoverable strain energy. At the end of the excitation, it represents only the unrecoverable energy as a result of inelastic deformations of the structures. Comparing these two figures, it is found that the peak kinetic energy in the SMRF model is larger than that in the SCPT model, which suggests that the velocity response of SCPT is lower. The reduction of velocity response can decrease the lost to the velocity-sensitive nonstructural components during earthquakes.



(a) (b)

Figure 6 Energy time history (a) SCPT model (b) SMRF model

The conventional engineering idea is that the higher energy dissipation the better seismic performance while as shown in table 4.1, it is found that the total hysteretic energy of the SCPT model is lower than that of the SMRF model. However, the seismic performance of SCPT model is better than the SMRF model as shown in the previous sections from global and local responses. The reason is that although lower hysteretic energy is dissipated by the SCPT model compared to the energy dissipated by the SMRF model, the input energy flowing into the SCPT model is also reduced. It should be pointed out that the hysteretic energy absorbed in the SMRF model was due to the inelastic deformation of the beams and columns, which cause damage to the structural components. For the SCPT model, the ED bars played a sacrificial role without any damage to the beams and columns.

Table 4.1 Energy Distribution (lb-in)

	Test No.	Input Energy	Viscous Damped Energy	Hysteretic Energy			
				1st floor	2nd floor	3rd floor	total
SCPT Model	PE6A	85155	18757	15798	27148	23452	66398
SMRF Model	ME06A	105008	27323	20404	31674	25607	77685

4.4 Shear Transfer Issues

Under the large intensity earthquake excitations (e.g. 125% or 150% intensity), vertical movements of the beam ends in the SCPT model were observed through the video cameras installed on the 1st floor. These vertical movements of the beam were unexpected since the calculation of the initial model indicated that the shear transferred from the beam to column could be resisted by the friction between the beam and column interfaces. Those unexpected vertical movements were the result of insufficient friction. The reduction of this friction is believed to be due to the fact that the contact surface between the beam and column were not sand blasted. This result indicates that the sand blasting of the contact interface or, alternatively, the inclusion of vertical supports such as slotted shear tab connections may be necessary for SCPT connections.

5. CONCLUSIONS

This paper experimentally evaluated the seismic performance of two steel plane frame models. The first model incorporated novel Self-Centering Post-Tensioned (SCPT) connections, while the second model was a conventional fully-welded Steel Moment Resisting Frame (SMRF). Based on the experimental results obtained, the following conclusions can be drawn:

i) The reduction in acceleration response and similar displacement response of the SCPT model, compared to those in the SMRF model, confirmed the good seismic performance of the SCPT model.

ii) Unexpected vertical movements of the beam ends in the SCPT model was observed under the severe seismic excitations, which suggested that the beam ending surfaces connected to the columns should be sand blasted to increase the friction or additional shear tabs should be installed to the beam-to-column joints to supply more shear resistance.

iv) The strain responses indicated that after severe earthquakes, the beams of the SMRF structure were damaged by yielding while only the ED bars yielded in the SCPT model without any damage to the beams. These results verified that it would cost much more to repair the beams of the SMRF structure compared to the SCPT building.

v) The energy analysis suggested that although the energy dissipated by the SCPT model was less than that absorbed by the SMRF model, the total energy flowing to the SCPT model was still less than that traveling to the SMRF model. This phenomenon may be associated with the better seismic performance of the SCPT model. The different hysteresis energy distribution (i.e. in yielding beams of SMRF model and in yielding ED bars of SCPT model), again demonstrated the lower repair cost for the SCPT model.

ACKNOWLEDGEMENTS

This research was supported financially by the Earthquake Engineering Research Centers Program of the National Science Foundation (NSF) under Award Number EEC-9701471 to the Multidisciplinary Center for Earthquake Engineering Research (MCEER). Any opinions, findings, conclusions and recommendations presented in this dissertation are those of the authors and do not necessarily reflect the views of the sponsor.

The authors also gratefully acknowledge DYWIDAG-Systems International for providing the post-tensioning systems tested and the technical staff of the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University at Buffalo for their support in the execution of the shake table tests.

REFERENCES

1. FEMA 356 (2000). “Prestandard and Commentary for the Seismic Rehabilitation of Buildings”, Federal Emergency Management Agency, November, 2000.
2. IBC (2003). International Building Code, International Code Council, Inc., USA
3. Kircher, C. A. (2003) “It Makes Dollars and Sense to Improve Nonstructural System Performance”, Proc. ATC 29-2 Seminar on seismic Design, Performance, and Retrofit of Nonstructural Components in Critical Facilities, Newport Beach, California, October 23-24, 2003.
4. SAC (1995) “Steel Moment Frame Connection”, Advisory No. 3, Report No. SAC 95-01, Sacramento, CA.
5. Uang, C.-M. and V.V. Bertero (1990), Evaluation of seismic energy in structures, Earthquake Engineering and Structural Dynamics., 19, 77-90.
6. Wang, D. (2007). Numerical and Experimental Studies of Self-Centering Post-Tensioned Steel Frames, Ph.D. Dissertation, The State University of New York at Buffalo, NY 14260
7. Wanitkorkul, A. and Filiatrault, A. (2005) “Simulation of Strong Ground Motions for Seismic Fragility Evaluation of Nonstructural Components in Hospital”, Technical Report: MCEER-05-2005 Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo, NY.
8. Yang, T. Y. and Whittaker, A. (2002). “MCEER Demonstration Hospitals”, Department of Civil, Structural and Environmental Engineering, University at Buffalo. (available online at http://civil.eng.buffalo.edu/hospital/)