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FULL SCALE STEEL PLATE SHEAR WALL: MCEER/NCREE PHASE II TESTS

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ABSTRACT

In Phase II of MCEER/NCREE cooperative project, a full scale two-story steel plate shear wall was obtained by replacing the buckled panels by new panels prior to submitting the specimen to further testing. To experimentally address the behavior of the repaired specimen in a new earthquake and the seismic performance of the intermediate beam in the first stage of Phase II, the specimen was tested under pseudo-dynamic loads equivalent to the first earthquake record considered in the Phase I tests. The specimen was subjected to cyclic testing to failure in the next stage of the Phase II tests to investigate the ultimate behavior of the intermediate beam and the cyclic behavior as well as the ultimate capacity of the specimen. It is shown that the repaired specimen can survive and dissipate significant amounts of hysteretic energy in a new earthquake without severe damages to the boundary frame or overall strength degradation. It is also found that the specimen had exceptional redundancy and exhibited stable force-displacement behavior up to the drifts of 5.2 % and 5.0 % at the first and second story respectively.

Introduction

A steel plate shear wall (SPSW) consists of infill steel panels surrounded by boundary frame members. These panels are allowed to buckle in shear and subsequently form a diagonal tension field. SPSWs are progressively being used as the primary lateral force resisting systems in buildings (Sabelli and Bruneau 2006). Past monotonic, cyclic and shaking table tests on SPSW in the United States, Canada, Japan, Taiwan and other countries have shown that this type of structural system can exhibit high initial stiffness, behave in a ductile manner and dissipate significant amounts of hysteretic energy, which make it a suitable option for the design of new buildings as well as for the retrofit of existing constructions (extensive literature reviews are available in Sabelli and Bruneau 2006 and Berman and Bruneau 2003a). Analytical research on SPSW has also validated useful models for the design and analysis of this lateral load resisting system (Thorburn et al.1983; Elgaaly et al.1993; Driver et al. 1997; Berman and Bruneau 2003b). Recent design procedures for SPSW are provided by the CSA Limit States Design of Steel Structures (CSA 2003) and the AISC Seismic Provision for Structural Steel Buildings (AISC 2005). Innovative SPSW

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designs have also been proposed and experimentally validated to expand the range of applicability of SPSW (Berman and Bruneau 2003; Vian and Bruneau 2005).

However, some impediments still exist that may limit the widespread acceptance of this structural system. For example, no research has directly addressed the replaceability of infill panels following an earthquake, and there remain uncertainties regarding the seismic behavior of intermediate beams in SPSW (intermediate beams are all the beams in a continuous SPSW except the top and bottom beams. This differentiation is needed because they are loaded differently by the yielding plates.). The latter problem was analytically addressed by Lopez Garcia and Bruneau (2006) using simple models, but experimental investigations on the behavior of intermediate beams, particularly for beams having reduced beam section (RBS) connections and composite concrete slabs, can provide much needed information on the behavior of this structural system and how to best design the intermediate beams.

To address the above issues with regards to SPSW performance, Phase II of the MCEER/NCREE project was developed. The testing program also investigated how to replace steel panels after a severe earthquake and how the repaired SPSW would behave in a second earthquake. This paper summarizes the Phase II tests conducted and observed ultimate behavior.

Specimen Description and Test Setup

The tests were carried out at the NCREE facility in Taipei, Taiwan. In Phase I of this project, a full scale two-story one-bay SPSW specimen was designed and fabricated. The specimen had equal height and width panels at each story. The infill panels and boundary frame members were sized based on the recommendations provided by Berman and Bruneau (2003). The RBS connection design procedure of the Federal Emergency Management Agency (FEMA) Document, FEMA 350 (FEMA, 2000) was used to detail the beam-to-column connections at top, intermediate and bottom level respectively. Beams and columns were of A572 Gr.50 steel members. Infill panels were specified to be SS400 steel, which is similar to ASTM A36 steel (Kuan 2005).

In order to investigate the seismic behavior of SPSW in severe earthquake and aftershocks in the Phase I tests, the SPSW specimen was tested under three pseudo-dynamic loads of progressively decreasing intensity. No fracture was found in the boundary frame in this phase, and it was deemed to be in satisfactory condition allowing for the replacement of infill panels for the subsequent phase of testing.

Fish plates were used along the boundary frame members to connect infill panels. The infill panels of Phase I were welded on one side of the fish plates and the new panels installed as part of Phase II were welded on the other side (after Phase I panels were cut-out). In the Phase I tests, the infill panels were restrained by horizontal restrainers to minimize the amplitude of the out-of-plane displacements of the panels that typically develop in SPSW at large inelastic drifts according to the design procedure proposed by Lin et al (2006).However, no restrainers were utilized in Phase II tests. Detailed information about specimen design and test results of Phase I tests is available in a companion paper, called “Phase I”, authored by Lin et al (2007).

The test setup is illustrated in Fig.1.The specimen was mounted on the strong floor. In-plane (north-south) servo controlled hydraulic actuators were mounted between the specimen and a reaction wall. Based on the ultimate strength of the specimen assessed using plastic analysis procedures (Berman and Bruneau 2003), three 1000kN hydraulic actuators were employed to apply earthquake load or cyclic load on the specimen at each story. Ancillary trusses (as part of the floor slab system) were used to transfer in-plane loads to the specimen at the floor levels. In order to avoid out-of-plane (east-west) displacements of the SPSW at floor levels, two hydraulic actuators were mounted at each floor level between the edge of the floor (ancillary truss) and a reaction frame. A vertical load of 1400 kN was applied by a reaction beam at the top of each column to simulate gravity load that would be present in the prototype. Each reaction beam transferred the load exerted from two vertical actuators mounted between the reaction beam and anchor rods pined to the strong floor.

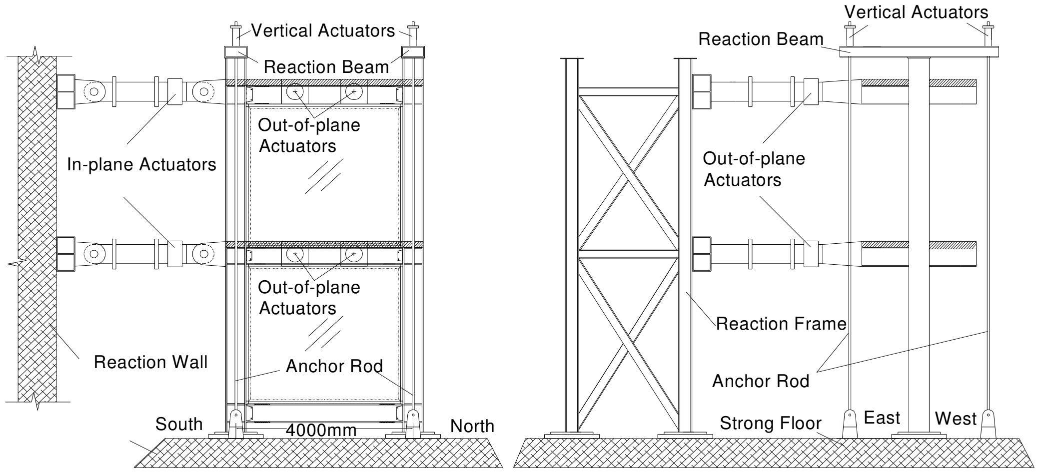


Figure 1. Test setup.

Phase II Pseudo-dynamic Test

In order to investigate how the repaired SPSW specimen would behave in a second earthquake in the first stage of Phase II, the specimen was tested under pseudo-dynamic loads corresponding to the Chi-Chi earthquake record (TCU082EW) scaled to a seismic hazard having a 2% probability of occurrence in 50 years (i.e. equivalent to the first earthquake record considered in the Phase I tests). This scaled earthquake record had a peak ground acceleration (PGA) of 0.63g and the peak pseudo acceleration (PSa) response of 1.85g at the fundamental period of 0.52 sec. The original ground motion record and the displacement histories at floor levels are shown in Fig.2.

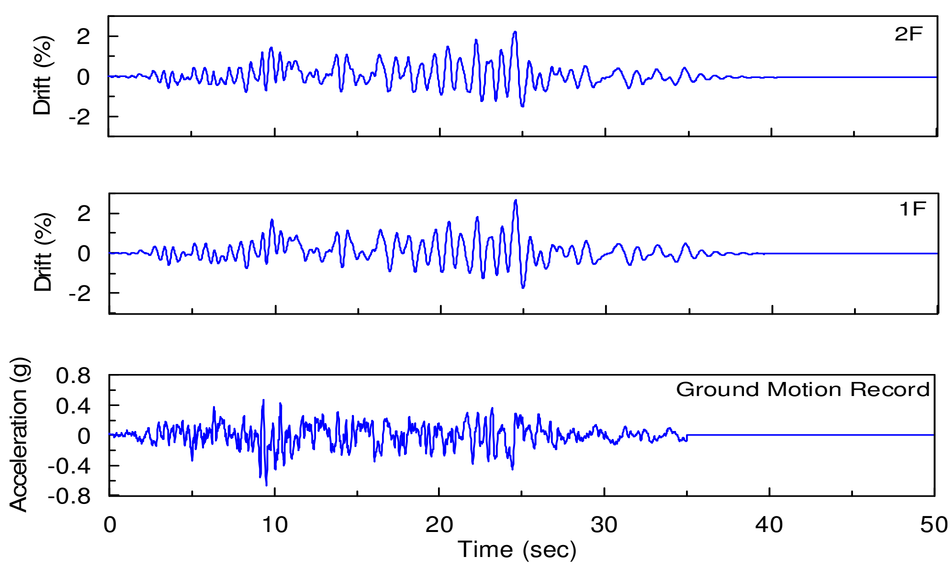


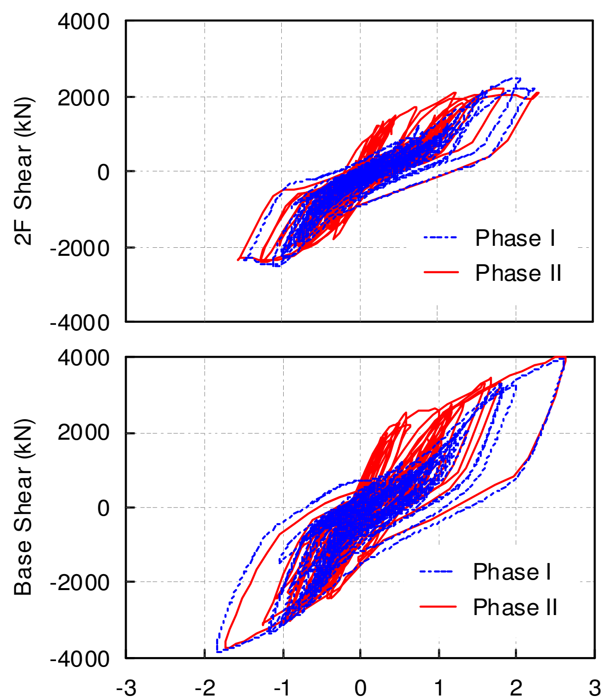
Figure 2. Ground motion record and displacement histories.

The SPSW specimen and hysteretic curves obtained from the Phase II pseudo-dynamic test, along with the counterpart results obtained from Phase I for the same level of excitation are shown in Figs.3 (a-b).

Drifts designated as "+"or "-" refer to loading in the north and in the south directions respectively. Observation of the hysteretic curves obtained from the Phase II shows that the first story dissipated more hysteretic energy than the second story. The infill panels buckled over both stories as anticipated, with maximum amplitude of out-of-plane deformations of 250 mm. Both the first and second story exhibited stable force-displacement behavior, with some pinching of the hysteretic loops as the magnitude of drifts increased, particularly after the development of a small fracture along the bottom of the shear tab at the north end of the intermediate beam at drifts of 2.6% and 2.3% at the first and second story respectively. After the pseudo-dynamic test, the boundary frame was in good condition (except for the aforementioned damage in the shear tab of the intermediate beam). There were notable plastic deformations at the column bases and RBS connections at all levels. Small fractures were found at the panel corners. All welds within the SPSW specimen were intact after the test. The buckled panels after Phase II pseudo-dynamic test are shown in Figs.4 (a-b) respectively.

Comparing the hysteretic curves from the Phase I and Phase II tests shown together in Fig. 3(b), the two specimens are found to behave similarly under the same strong ground motion except that the initial stiffness of the repaired specimen is higher than that of the original one. This is because the intermediate concrete slab suffered premature cracks and two anchor bolts fractured at the south column base at the time step of 9.5 sec and 24 sec of the first earthquake record in the Phase I tests respectively, as mentioned in the companion paper. The Phase I tests resumed after the specimen load transfer mechanisms were strengthened at those locations. The results shown in Fig. 3(b) for the specimen in Phase I are those obtained after the specimen was repaired due to the aforementioned failures. Therefore the infill panels had already experienced some inelastic deformation before these unexpected failures occurred.


(a) Specimen prior to Phase II tests



(b) Hystereses of Phase I and II pseudo-dynamic tests

Figure 3. Specimen and hystereses.

(b) 1F panel

(c) 2F panel

Figure 4. Buckled panels after Phase II pseudo-dynamic test.

Phase II Cyclic Testing to Failure

The next stage of the Phase II tests involved cyclic test of the SPSW specimen to investigate the ultimate behavior of the intermediate beam and the cyclic behavior as well as the ultimate capacity of the SPSW.

As mentioned earlier, the boundary frame members were still in good condition after the pseudo-dynamic test, except for a small visible fracture along the bottom of the shear tab at the north end of the intermediate beam. To correct this limited damage and get a better assessment of the possible ultimate capacity of SPSW, the damaged shear tab was replaced by a new one prior to conducting the cyclic test. A displacement-controlled scheme was selected for the cyclic test. Because the first mode response dominated the global response of the SPSW in the prior pseudo-dynamic test (although some higher mode effects were observed) and to allow testing both panels even if failure progressively develops at one of the two stories, a displacement constraint was exerted to keep the in-plane actuators displacing in a ratio corresponding to a first mode of response through-out the entire test. The arbitrary drift history is shown in Fig. 5 and is in the sprit of standard loading protocol. Since the specimen was pulled (to the south) to the maximum actuator stroke when peak drifts reached -3.2% and -3.0% at the first and second story respectively, the applied displacement history became unsymmetrical beyond that point, in that the peak drifts due to loading toward the south were kept at -3.2% and -3.0% at the first and second story respectively in all subsequent cycles while increasing displacements were still applied in the other direction.

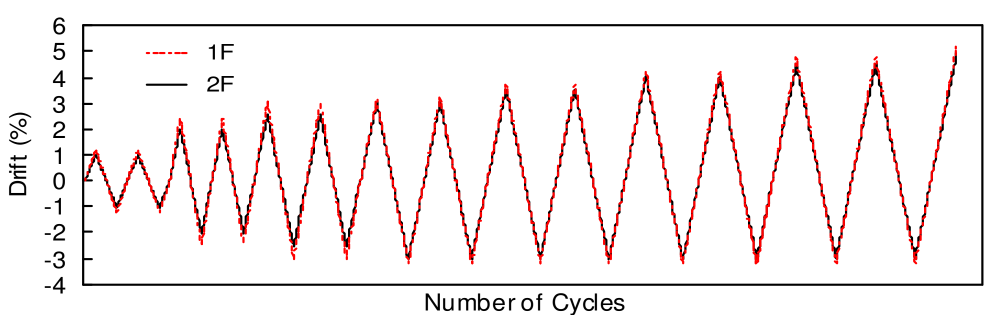


Figure 5. Cyclic drift histories.

The hysteretic curves resulting from the Phase II cyclic test, along with the results of the Phase II pseudo-dynamic test described in detail in the prior section, are shown in Fig. 6. Comparing the hysteretic curves in Fig. 6, it is observed the initial stiffness of the SPSW specimen in the cyclic test was smaller than that in pseudo-dynamic test. Because the previous pseudo-dynamic test stretched the infill panels up to the drifts of 2.6% and 2.3% at the first and second story respectively, the hysteretic loops exhibited pinching up to those drifts. Hysteretic loops were then full until drifts of 2.8% and 2.6% at the first and second story respectively in Cycle 7, when complete fracture occurred along the shear tab at the north end of the intermediate beam. This unexpected failure resulted in story shears reductions of 76 kN and 83 kN at the first and second story respectively mainly because the test was being conducted under displacement control rather than force control. A similar fracture developed along the shear tab at the south end of the intermediate beam when the specimen was pulled towards the reaction wall in this cycle.

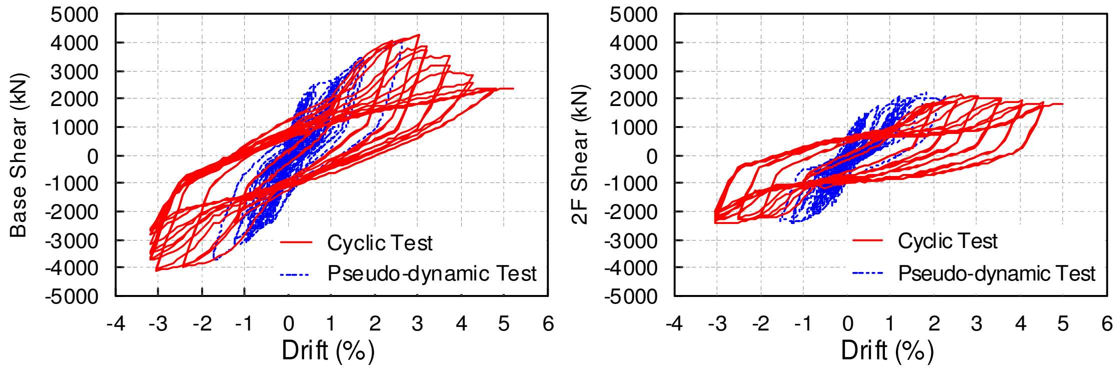


Figure 6. Hystereses of the Phase II tests

Figure 7. Ruptures at the north end of the intermediate beam.

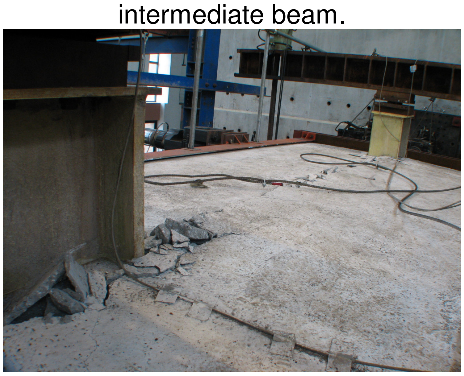


Figure 9. Crack at the top slab

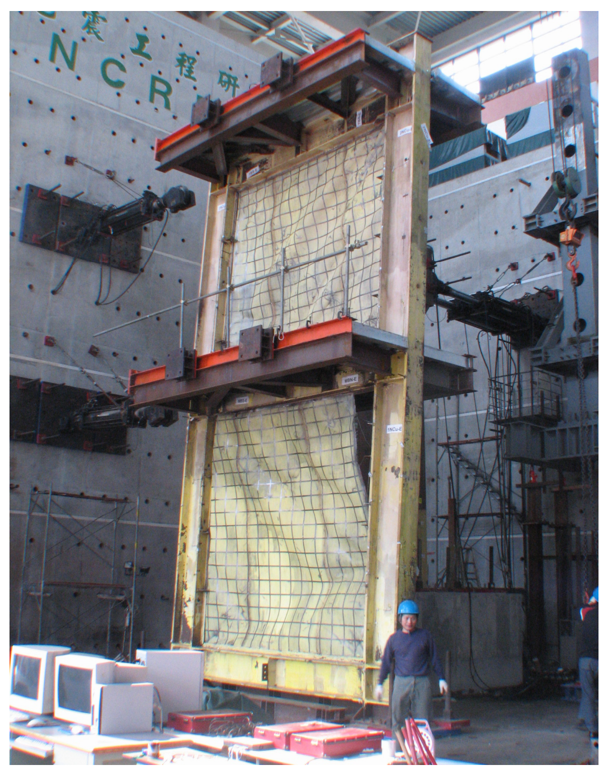


Figure 8. Fracture of the welds connecting the infill panels to fish plates.

Rupture of the shear tabs triggered fracture of the bottom flange at the north end of the intermediate beam. At drifts of 3.3% and 3.1% at the first and second story respectively in Cycle 9, the bottom flange at the north end of the intermediate beam fractured as shown in Fig.7. However, no fractures developed in the reduced beam flange regions of the intermediate beam. The welds connecting the infill panels to the fish plates around the north end of the intermediate beam also fractured over a substantial length to a more severe extent after the specimen experienced drifts of 5.2% and 5.0% at the first and second story respectively as shown in Fig.8. These events significantly changed the load path within the system. However, the SPSW specimen was still able to exhibit stable force-displacement behavior as evidenced by the hysteretic curves shown in Fig. 6, which demonstrates the redundancy of this kind of structural system. The cyclic test ended at drifts of 5.2% and 5.0% at the first and second story respectively, when sudden failure occurred in the load transfer mechanism, i.e. when a fatal longitudinal crack developed along the top concrete slab of the specimen as shown in Fig.9.

Conclusions

A repaired full scale two-story SPSW specimen was obtained by replacing the buckled panels installed in Phase I by new ones. It was subjected to pseudo-dynamic and cyclic testing in the Phase II tests, to experimentally address the replaceability of infill panels following an earthquake, the behavior of the repaired SPSW in a new earthquake, and the seismic performance of the intermediate beam.

The pseudo-dynamic test shows that a SPSW repaired by replacing the infill panels buckled in a prior earthquake by new ones can be a viable option to provide adequate resistance to the lateral loads imparted on this structure during new seismic excitations. The repaired SPSW behaved quite similarly to the original one. Testing showed that the repaired SPSW can survive and dissipate a similar amount of energy in the subsequent earthquake without severe damage to the boundary frame and without overall strength degradation.

Results from the cyclic test allowed to investigate the ultimate displacement capacity of the SPSW specimen. Though the hysteretic curves were pinched at the low drift levels due to the inelastic deformations that the infill panels experienced during the pseudo-dynamic test, and even though the strength of the SPSW dropped as the ends of the intermediate beam fractured, the SPSW structure exhibited stable force-displacement behavior and provided a significant hysteretic energy dissipation capacity, exhibiting substantial redundancy.

The columns and anchor beams, as well as top and bottom RBS connections performed as intended. However, the intermediate beams failed unexpectedly. The ends of the intermediate beams having RBS connections ultimately developed fractures in the shear tabs followed by fractures at the end of the bottom beam flange. No fractures developed in the reduced beam flange region. Further investigation is required to clarify the local behavior of intermediate beams in SPSW, to allow the development of a better understanding of how such intermediate beams should be designed.

Acknowledgments

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References

American Institute of Steel Construction (AISC), 2005. *Seismic Provisions for Structural Steel Buildings*. Chicago, Illinois.

Berman, J.W., and Bruneau, M., 2003a. Experimental Investigation of Light-Gauge Steel Plate Shear Walls for the Seismic Retrofit of Buildings, *Technical Report MCEER-03-0001*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, New York, USA

Berman, J.W., and Bruneau, M., 2003b. Plastic Analysis and Design of Steel Plate Shear Walls, *Journal of Structural Engineering*, 129(11), 1448-1456

Canadian Standards Association (CSA), 2005. *Limit States Design of Steel Structures.* CAN/CSA S16-01. Willowdale, Ontario, Canada.

Driver R.G., Kulak G.L., Kennedy, D.J.L., and Elwi A.E., 1997. Seismic Behavior of Steel Plate Shear Walls. *Structural Engineering Report No.215*, University of Alberta, Edmonton, Alberta, Canada.

Elgaaly M., Caccese V.,and Du C., 1993. Postbuckling Behavior of Steel-Plate Shear Walls under Cyclic Loads. *Journal of Structural Engineering,* 119 (2), 588-605.

Federal Emergency Management Agency (FEMA), 2000. *Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings*. FEMA 350. Prepared by the SAC Joint Venture for FEMA, Washington, D.C.

Kuan, M.L., 2005. "Discussion on ASTM A36 and CNS SS400." <*http://www.twce.org.tw/info/%A7%DE%AEv%B3%F8/367-3-1.htm*> (in Chinese).

Lin, C.H., Tsai, K.C., Lin, Y.C., Wang, K.J., Hsieh, W.D., Weng, Y.T., Qu, B., and Bruneau, M., 2006. The Sub-structural Pseudo-dynamic Tests of A Full-scale Two-story Steel Plate Shear wall. *Proceeding of the 4th International Conference on Earthquake Engineering. Paper No.155,* Taipei, Taiwan.

Lin, C.H., Tsai, K.C., Lin, Y.C., Wang, K.J., Qu, B., and Bruneau, M., 2007.Full Scale Steel Plate Shear Wall: NCREE/MCEER Phase I Tests. *Proceeding of the 9th Canadian Conference on Earthquake Engineering (in press)*. Ottawa, Ontario, Canada

Lopez-Garcia, D. and Bruneau, M., 2006. Seismic Behavior of Intermediate beams in Steel Plate Shear Walls, *Proceeding of the 8th U.S. National Conference on Earthquake Engineering, Paper No.1089*, San Francisco, CA.

Sabelli, R., and Bruneau, M., 2006. *Steel Plate Shear Walls (AISC Design Guide)*, American Institute of Steel Construction, Chicago, Illinois.

Thorburn, L.J., Kulak, G.L. and Montgomery C.J., 1983. Analysis of Steel Plate Shear Walls. *Structural Engineering Report No. 107*, Department of Civil Engineering, The University of Alberta, Edmonton, Alberta, Canada.

Vian D.and Bruneau, M., 2005. Steel Plate Shear Walls for Seismic Design and Retrofit of Building Structure, *Technical Report MCEER-05-0010*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, N.Y.