Stessa 2009 – Mazzolani, Ricles & Sause (eds)

Seismic design of boundary frame members of steel plate shear walls

B. Qu
Department of Civil and Environmental Engineering, California Polytechnic State University, San Luis Obispo, CA, USA

M. Bruneau
Department of Civil, Structural and Environmental Engineering, University at Buffalo, Buffalo, NY, USA

ABSTRACT: Steel Plate Shear Walls (SPSWs), which consist of infill steel panels surrounded by columns, called Vertical Boundary Elements (VBEs), and beams, called Horizontal Boundary Elements (HBEs), are rapidly becoming an appealing alternative lateral force resisting system for building structures in the United States, Canada, Mexico, Japan, Taiwan and other countries. This paper presents results of some recent research to expand the range of applicability of SPSWs with emphasis on improving the understanding of seismic performance of the boundary frame members. Following a brief review of the observed failure of the intermediate HBE from the MCEER/NCREE testing program, the models developed to investigate the behavior of HBEs are presented, followed by design recommendations. Then, analytical models to prevent the in-plane shear yielding and to estimate the out-of-plane buckling strength of VBEs are developed, followed by a review of past experimental data to investigate if the previously observed VBE failures were due to excessive VBE flexibilities or other causes. It is shown that the existing VBE flexibility requirement specified in the current design codes is uncorrelated to satisfactory VBE performance. The proposed analytical models predict the performance of previously tested SPSWs that correlates well with the experimental observations.

1 INTRODUCTION

Steel Plate Shear Walls (SPSWs) consist of unstiffened infill steel panels surrounded by columns, called Vertical Boundary Elements (VBEs), on both sides, and beams, called Horizontal Boundary Elements (HBEs), above and below. These infill steel 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 & Bruneau 2006).

Past tests on SPSWs 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 (Berman & Bruneau 2003). Analytical research on SPSWs has also validated useful models for design and analysis of this lateral load resisting system (Thorburn et al. 1983; Driver et al. 1997; Berman & Bruneau 2003). Recent design procedures for SPSWs 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 designs have also been proposed and experimentally validated to expand the range of applicability of SPSWs (Berman & Bruneau 2003, Vian & Bruneau 2005).

However, some impediments still exist that may limit the widespread acceptance of SPSWs. For example, little information exists on the behavior and design of boundary frame members in SPSWs, particularly the intermediate HBEs having reduced beam section (RBS) connections and VBEs. Note that intermediate HBEs are those to which are welded infill steel panels above and below, by opposition to anchor HBEs that have steel panels only below or above. This paper briefly presents results of some recent research that further addresses the above pressing concerns.

2 BEHAVIOR AND DESIGN OF INTERMEDIATE HBES

2.1 *MCEER/NCREE testing*

A full scale two-story one-bay SPSW specimen was fabricated in Taiwan and a two-phase experimental program (Phase I and II tests) was conducted at the laboratory of NCREE. The specimen with equal height and width panels at each story was measured 8000 mm high and 4000 mm wide between boundary frame member centerlines. HBE and VBE were of A572 Gr.50 steel members. Infill panels were specified to be SS400 steel which is similar to ASTM A36 steel in this case. The RBS connection design procedure proposed by FEMA 350 (FEMA 2000) was used to detail the HBE-to-VBE connections at top, intermediate and bottom levels respectively. The infill panels were designed to be 3 mm and 2 mm thick at the first and second story respectively. Prior to the Phase II tests, the buckled infill panels were removed and replaced by new panels.

The specimen was mounted on the strong floor. In-plane (south-north) servo controlled hydraulic actuators were mounted between the specimen and a reaction wall. Three hydraulic actuators were employed to apply in-plane (south-north) lateral load on the specimen at each story. Two hydraulic actuators were used to avoid out-of-plane (east-west) displacement at floor levels. A vertical load of 1400 kN was applied by a reaction beam at the top of each column to simulate the gravity loads. The specimen is shown in Figure 1.

In Phase I, the specimen was tested under three pseudo-dynamic loads using the Chi-Chi earthquake record (TCU082EW) scaled up to levels of excitations representative of seismic hazards having 2%, 10% and 50% probabilities of exceedances in 50 years, subjecting the wall to earthquakes of progressively decreasing intensity. No fracture was found in the boundary frame and it was deemed to be in satisfactory condition allowing for the replacement of infill panels. The buckled infill steel panels were replaced by new ones prior to submitting the specimen to the subsequent phase of testing. Detailed information about the results from the Phase I tests are presented elsewhere (Lin et al. 2007).

In the first stage of Phase II, the specimen was tested under pseudo-dynamic load corresponding to the Chi-Chi earthquake record (TCU082EW) scaled up to the seismic hazard of 2% probability of occurrence in 50 years which was equivalent to the first earthquake record considered in the Phase I tests (Qu et al. 2007). Figure 2 shows the plastic deformations at the ends of the intermediate HBE observed during the test. As shown, the center of the yielded zone, which can be deemed to be the location of the lumped plastic hinge, moved toward the VBE face. This observation is different from those for a beam having RBS connections in a conventional moment frame, in which plastic behavior of the flange usually concentrates at the center of the RBS (i.e. where the beam flange is reduced most severely).

The next stage of Phase II tests involved cyclic test on the SPSW specimen in order to investigate the ultimate behavior of intermediate HBE. A displacement-controlled scheme was selected for the cyclic test. The ultimate behavior of intermediate HBE was found to be a complete fracture occurred along the shear tab at the end of the intermediate HBE followed by the complete fracture at the bottom flange as shown in Figure 3. However, no fractures developed in the reduced beam flange regions of the intermediate HBE.

2.2 *Moment demand at VBE faces*

Although many effects may have contributed to the unexpected failure at the ends of the intermediate HBE in the MCEER/NCREE SPSW specimen, flexural strength deficiency at the VBE face is a factor worthy of investigation. The original design of the intermediate HBE assumed that all inelastic beam action concentrates at the RBS centers and used a simple free body diagram as shown in Figure 4 to calculate the flexural demand at VBE face. In the free body diagram, L represents the span of the HBE, d represents the depth of the HBE, e represents the distance between plastic hinge to VBE face, distributed loads (i.e. ω_{ybi}, ω_{xbi}, ω_{ybi+1}, and ω_{xbi+1}) represent the infill panel yield forces; P_R and P_L represent axial forces at the right and

Figure 1. MCEER/NCREE specimen.

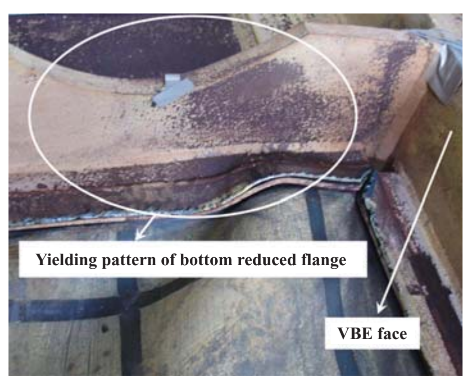

Figure 2. Yielding pattern at the end of intermediate HBE.

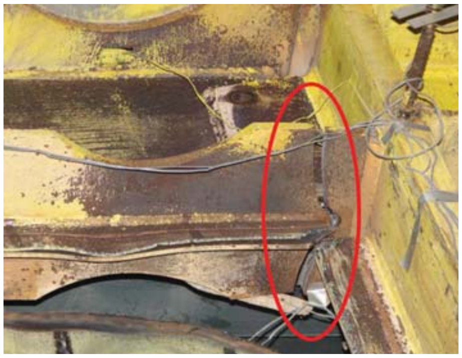

Figure 3. Ruptures at the end of the intermediate HBE.

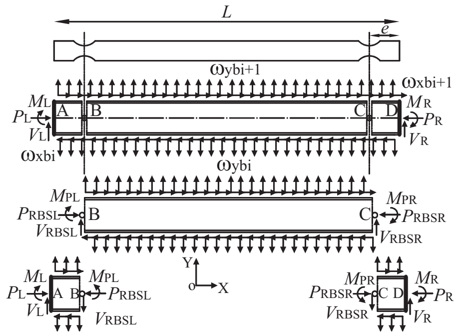

Figure 4. Free body diagram of intermediate HBE.

left ends of the HBE; M_R and M_L represent moment demands at the right and left VBE faces; V_R and V_L represent shear forces at the right and left VBE faces; P_{RBSR} and P_{RBSL} represent axial forces at the right and left plastic hinges; V_{RBSR} and V_{RBSL} represent shear forces at the right and left plastic hinges; and M_{RBSR} and M_{RBSL} represent the plastic moments at the right and left plastic hinges, respectively. For analysis purpose, the HBE is divided into three segments, the middle segment between two plastic hinges, and the right and left segments outside of the plastic hinges.

Using a static analysis procedure, one can obtain the following flexural demands at the right VBE face

$$M_R = M_{RBSR} + V_{RBSR}e + \frac{(\omega_{ybi} - \omega_{ybi+1})e^2}{2} - \frac{(\omega_{xbi} + \omega_{xbi+1})de}{2} \quad (1)$$

The free body diagrams shown in Figure 4 produce reasonable results for beams having RBS connections in conventional moment frame. However, they may be inadequate for intermediate HBEs having RBS connections in SPSWs. The yielding pattern at the end of intermediate HBE shown in Figure 2 suggested that the center of the yielded zone, which can be deemed to be the location of lumped plastic hinge, moves towards the VBE face rather than occurs at the RBS centers. This effect can be ascribed to the presence of large axial and shear forces that vary along the HBE, and the presence of vertical stresses in HBE web due to infill panel forces (Qu & Bruneau 2008).

For design purpose, it is recommended to assume that the actual plastic hinge moves toward the VBE face and has a plastic section modulus, Z_{RBS}, equal to the average of the plastic section moduli of the unreduced part of the HBE and that at the RBS center (i.e. Z and Z_{center}, respectively), which is :

$$Z_{RBS} = \frac{Z_{center} + Z}{2} \quad (2)$$

the plastic moment at the plastic hinge is reduced by the axial and shear forces in the HBE, and the vertical stresses in HBE web. This effect can be considered by incorporating the cross-section plastic moment reduction factor, β_{RBSR}, into the determination of moment resistances of plastic hinges:

$$M_{RBSR} = \beta_{RBSR} R_y f_y Z_{RBS} \quad (3)$$

where β_{RBSR} can be determined by using the procedure proposed by Qu & Bruneau (2008), R_y is the ratio of expected to nominal yield stress, and f_y is the yield strength of the intermediate HBE.

Using the above method to account for the actual location and strength of plastic hinges, the free body diagram shown in Figure 4 and equation (1) remain valid. Noted that the predicted moment demands should compare with the available strengths at the VBE faces.

2.3 *Examination of the intermediate HBE in MCEER/NCREE specimen*

Using the recommendations proposed in the prior section for checking the adequacy of flexural strength at VBE face, the intermediate HBE of the MCEER/NCREE specimen was redesigned. Assuming the material has a yield strength of 346 MPa, the new intermediate HBE was determined to be a W24 × 76 member. The cross-section properties and flange reduction geometries of the redesigned and original members are summarized in Table 1.

A preliminary assessment was made by comparing the design moment demands and available flexural strengths at the VBE faces. For comparison purpose, results of both the redesigned and original members are provided in Table 2.

As shown in the above table, the flexural strength of the original HBE at the right VBE face is smaller than the demand. This would explain the unexpected failure (i.e. fractures at the HBE ends) observed during MCEER/NCREE tests as shown in Figure 3. By

Table 1. Summary of cross-section properties and flange reduction geometries.

HBE	d (mm)	b_f (mm)	t_f (mm)	t_w (mm)	a* (mm)	b* (mm)	c* (mm)
original	350	250	19	11	135	230	48
redesigned	607	228	17.3	11.2	160	486	57

* flange reduction parameters described in FEMA 350.

Table 2. Design demands and available strengths at VBE faces.

HBE	Left VBE face		Right VBE face	
	demand ($kN•m$)	strength ($kN•m$)	demand ($kN•m$)	strength ($kN•m$)
original	660	774	748	571
redesigned	809	951	876	891

comparison, the strengths of the redesigned HBE are greater than the demands, which indicate the SPSW designed per the recommendation proposed here would have likely not suffered from the observed premature failure.

3 BEHAVIOR AND DESIGN OF VBEs

3.1 *Current design requirements*

The early Canadian provisions for SPSWs (i.e. CSA S16-95, CSA 1994) required VBEs to be designed as beam-column using a conventional strength-based approach. This approach was challenged by the results of tests on quarter-scale SPSW specimens (i.e. Specimens SPSW2 and SPSW4) by Lubell et al. (2000), in which the VBEs designed using the strength-based approach exhibited either significant "pull-in" deformation or undesirable premature out-of-plane buckling. Some have ascribed these failures to insufficient VBE stiffness. If VBEs deform excessively, they may be unable to anchor the infill panel yield forces. A non-uniform diagonal tension field may then develop and solicit the VBEs inconsistently to the design assumptions.

To ensure adequately stiff VBEs, CSA S16-01 (CSA 2000) introduced the flexibility factor, ω_t, proposed in previous analytical work of plate girder theory, as an index of VBE flexibility. Noting that the Lubell et al. specimens had flexibility factors of 3.35, and that all other known tested SPSWs that behaved in a ductile manner had flexibility factors of 2.5 or less, CSA S16-01 empirically specified an upper bound of 2.5 on ω_t. Note that this requirement can be converted into the VBE flexibility requirement presented in the current design codes (Qu & Bruneau 2008).

In design, the intent is that the aforementioned flexibility limit prevents excessively slender VBE. However, beyond the empirical observations and analogy to plate girder theory, no work has investigated whether the significant inward inelastic deformations of VBEs observed previously were directly caused by excessive VBE flexibilities or due to other causes, such as shear yielding at the ends of VBEs. In addition, no theoretical research has established a relationship between ω_t and the out-of-plane buckling strength of VBE as part of SPSW behavior.

To better understand the above issues, analytical models for preventing VBE shear yielding and for estimating the out-of-plane elastic buckling strength of VBEs are presented along with the reassessment of the Lubell et al. specimens.

3.2 *In-plane shear yielding*

As mentioned earlier, the significant "pull-in" deformation of VBE observed during the tests on the single-story specimen (SPSW2) by Lubell et al. (2000) as shown in Figure 5 was a milestone event that led to the current limit specified for the flexibility of VBEs in SPSWs (AISC 2005 and CSA 2000). This undesirable performance was ascribed to the insufficient VBE stiffness. However, VBE shear yielding is another important factor that may result in significant inelastic VBE deflections. At the time of this writing, no literature has reported or checked whether this specimen had encountered VBE shear yielding.

To have a better understanding of the observed significant inward deformations in VBEs, an analytical model for estimating VBE shear demand is proposed using the free body diagram shown in Figure 6. Conservatively, assuming that the moments applied at the top and bottom ends of the VBE are equal to their expected nominal plastic moments, one can obtain the following estimate of VBE shear demand from equilibrium:

$$V_{u-design} = \frac{2R_y f_y Z_c}{h_{si}} + \frac{\omega_{xci} h_{si}}{2} + \frac{\omega_{yci} d_{ci}}{2} \tag{4}$$

where d_{ci} and Z_c are the depth and plastic section modulus of VBE, ω_{xci} and ω_{ybi} are horizontal and vertical components of infill panel yield forces along VBE. Note that equations for calculating ω_{xbi} and ω_{ybi} are available in Berman & Bruneau (2008). In design, the shear demand obtained from (4) should be compared to the VBE shear strength.

To validate the above analytical model for estimating VBE shear design force and check whether VBE shear yielding had occurred in SPSW2, using the published geometries and material properties, the strip model of SPSW2 was developed and a pushover analysis was performed in SAP2000. Note that 20 strips were used for the infill plate. Steel was modeled as an elasto-perfectly

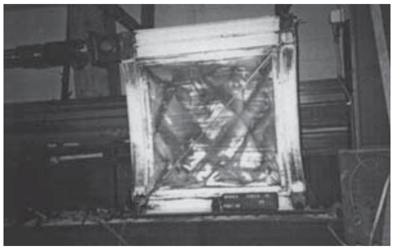

Figure 5. Deformation and yield pattern of SPSW2 (from Lubell et al. 2000).

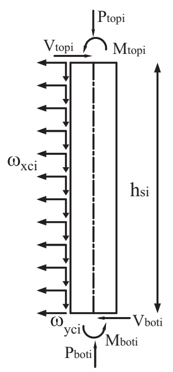

Figure 6. In-plane free body diagram of VBEs.

plastic material. Plastic hinges accounting for the interaction of axial force and flexure were defined at the ends of VBEs. The maximum VBE shears from this analysis was found to be 107 kN and the shear design force predicted from equation (4) was 113 kN. Therefore, the developed analytical model, although slightly conservative, can be used for design purpose.

On the other hand, the shear strength of the VBEs was found to be 75 kN that is smaller than the maximum VBE shear demand obtained from pushover analysis in SAP2000 (i.e. 107 kN). This result demonstrates that VBE shear yielding occurred in that specimen during the tests, resulting in the significant in-plane VBE deflection due to inelastic shear deformations. Yielding pattern of the VBE webs further confirms this point. As indicated by the flaked whitewash shown in Figure 5, the VBE web yielded uniformly at the VBE ends as opposed to the yielding pattern usually observed in flexural plastic hinges, indicating significant inelastic shear deformations. Note that the axial force in the VBEs can also affect the yielding pattern of VBE webs. However, the axial force developed in the VBEs is insignificant in this single-story case.

3.3 *Out-of-plane buckling*

Besides the aforementioned excessive pull-in deformations, another undesirable behavior of VBE is out-of-plane buckling, which has been observed during the tests on a quarter-scale four-story SPSW specimen (i.e. SPSW4) by Lubell et al. (2000).

To better understand the VBE out-of-plane buckling behavior, on the basis of the energy method, the corresponding criteria that define the buckling limit state under the boundary conditions illustrated in Figure 7 can be expressed as a combination of *m* and *n* equal to unity:

$$\begin{aligned} & m+\frac{n}{2}=1 \text{ when Case (A)} \\ & \frac{\mathrm{m}}{4}+\frac{n}{8}=1 \text{ when Case (B)} \\ & \frac{\pi^2}{21}m+\frac{\pi^2}{56}n=1 \text{ when Case (C)} \\ & \frac{\pi^2}{21}m+\frac{5\pi^2}{168}n=1 \text{ when Case (D)} \end{aligned} \tag{5}$$

where *m* and *n* are the generalized external forces and can be respectively obtained by normalizing the concentrated force applied at the top of the VBE (i.e. P_{topi}) and the resultant infill panel yield force along the VBE (i.e. $\omega_{\text{yci}}h_{\text{si}}$), by the Euler buckling load of a simply supported VBE without any intermediate loads along its height. Namely, *m* and *n* can be determined as

$$m=\frac{P_{topi}}{\left[\dfrac{\pi^2 EI_{yi}}{h_{si}^2}\right]} \text{ and } n=\frac{\omega_{yci}h_{si}}{\left[\dfrac{\pi^2 EI_{yi}}{h_{si}^2}\right]} \tag{6}$$

where I_{yi} is moment of inertia of the VBE taken from the weak axis. Note that derivation of the above criteria is available in Qu & Bruneau (2008).

For a given load combination (i.e. a pair of *m* and *n*) and boundary conditions, if left-hand side of the corresponding criterion is greater than 1, the VBE is expected to encounter out-of-plane buckling.

A closer look at the Lubell et al. specimen and the buckled shape of its VBE reveals that Case C boundary conditions were present (i.e. bottom end of the VBE was fixed to the ground while the top end was pinned in the out-of-plane direction). To better understand this, the VBE deflection traced from the specimen is superposed to those corresponding to cases B and C boundary conditions in Figure 8. Comparing the deflected shapes confirms that the VBE end conditions correspond to those of Case C. Accordingly, applying Criterion C provides a value of 1.066 greater than 1.0, indicating the expected occurrence of VBE out-of-plane buckling. This suggests that out-of-plane buckling of the VBEs in the Lubell et al. specimen is uncorrelated to the flexibility factor.

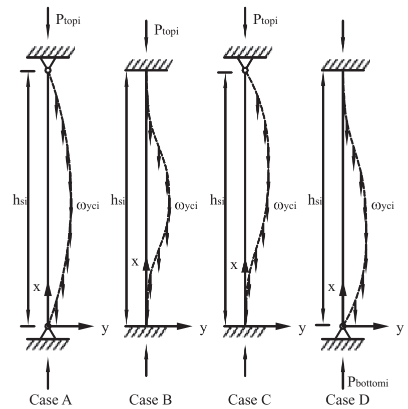

Figure 7. Considered boundary conditions.

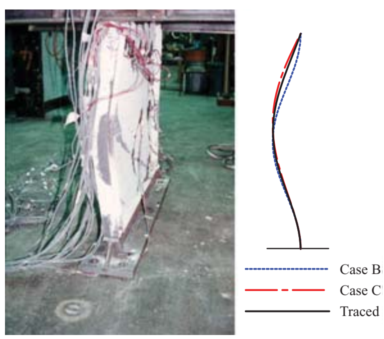

Figure 8. Out-of-plane buckling of bottom VBE (Photo: Courtesy of Ventura C.E.).

4 CONCLUSIONS

Based on the observation of the yielding pattern and failure mode of the intermediate HBE in MCEER/NCREE specimen, recommendations to estimate the moment demand at the end of the intermediate HBE having RBS connections in SPSWs have been proposed. A design procedure based on these recommendations uses simple free body diagrams and is able to prevent the observed premature failure of the HBE.

It is shown that the existing limit on ω_t is uncorrelated to satisfactory in-plane and out-of-plane VBE performance. Alternatively, the proposed analytical model for in-plane VBE shear demands, from which predicted performance correlates well with past experimental results, can be used to ensure desirable VBE behavior. Future analytical and experimental research should investigate whether in-plane buckling equations similar to those used for out-of-plane buckling are necessary for use in the interaction equations to calculate the beam-column strength of VBEs, and whether other concerns may justify retaining the use of ω_t factor to achieve satisfactory seismic performance of VBEs in SPSWs.

ACKNOWLEDGEMENTS

This work was supported by the EERC Program of NSF under Award Number ECC-9701471 to MCEER. However, any opinions, findings, conclusions, and recommendations presented in this paper are those of the writers and do not necessarily reflect the views of the sponsors.

REFERENCES

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

Berman, J.W., and Bruneau, M. (2003). 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. (2008). Capacity Design of Vertical Boundary Elements in Steel Plate Shear Walls. *AISC Engineering Journal* Vol.45, No.1.

CSA (1994). *Limit States Design of Steel Structures.* CAN/CSA S16-95. Willowdale, Ontario, Canada.

CSA (2000). *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.

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.

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, *9th Canadian Conference on Earthquake Engineering*, Ottawa.

Lubell, A.S., Prion, H.G.L., Ventura, C.E., and Rezai, M. (2000). Unstiffened Steel Plate Shear Wall Performance under Cyclic Loading. *Journal of Structural Engineering*, Vol.126 No.4.

Qu, B., Bruneau. M., Lin, C.H., Tsai, K.C., and Lin, Y.C. (2007). Full Scale Steel Plate Shear Wall: NCREE/MCEER Phase II Tests, *9th Canadian Conference on Earthquake Engineering*, Ottawa.

Qu, B., and Bruneau. M. (2008). Seismic Behavior and Design of Boundary Frame Members of Steel Plate Shear Walls, *Technical Report MCEER-08-0012*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, N.Y.

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, New York, USA.