# RECENT DEVELOPMENT FROM RESEARCH ON STEEL PLATE SHEAR WALLS

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**ABSTRACT**

Steel Plate Shear Walls (SPSWs) are rapidly becoming an appealing alternative lateral force resisting system for building structures in high seismic areas. This paper presents results of some recent research to expand the range of applicability of SPSWs. Emphasis is on improving the understanding of seismic and blast performance of SPSWs (in a multi-hazard perspective). Preliminary results of an investigation into the behavior of SPSWs with infill panels designed to resist various percentage of the specified lateral load are also presented.



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## INTRODUCTION

A steel plate shear wall (SPSW) consists of infill steel panels surrounded by boundary beams and columns. These panels are allowed to buckle in shear and subsequently form diagonal tension fields. The selection of SPSW as the primary lateral force resisting system in buildings has become more popular in recent years, and SPSW have been used increasingly as practicing engineers discover the benefits of this option. Understanding of the seismic behavior of thin plate SPSW has significantly improved in recent years. Yet, much research remains to be done to address a number of questions related to analysis, behavior, and design. Recent experimental and analytical work conducted at the University at Buffalo has investigated some of these aspects. This paper provides an overview of this work. In particular, the paper focuses on:

- Full scale experimental research on a two-story SPSW developed to generate knowledge on the behavior of intermediate beams, their performance when having reduced beam sections (RBS) and composite behavior, as well as on the replaceability of buckled steel plates and performance of the so repaired SPSW following an earthquake.

- Preliminary experimental results on the blast resistance of SPSW.

- Investigations into the SPSWs for which the combined resistances of infill plates and boundary frame are designed to share various percentage of the specified lateral load.

## TESTING ON FULL-SCALE TWO-STORY SPSW

Extensive monotonic, cyclic and shaking table tests on SPSW in North America have shown that SPSW 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 the retrofit of existing constructions (Timler and Kulak, 1983; Tromposch and Kulak, 1987; Cassese et al., 1993; Elgaaly et al., 1993; Driver et al., 1998; Rezai, 1999; Lubell et al., 2000; Berman and Bruneau, 2005; 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. To address the above issues, a two-phase experimental program was developed under the collaborative efforts of MCEER and NCREE. This section summarizes the tests conducted and observed ultimate behavior with emphasis on Phase II of this program.

A full scale two-story SPSW specimen was fabricated and tested at the laboratory of NCREE, Taipei, Taiwan. 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. The infill panels and boundary frame members were sized based on the recommendations provided by Berman and Bruneau (2003). 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 in this case. The RBS connection design procedure (FEMA350, 2000) was used to detail the beam-to-column connections at top, intermediate and bottom level respectively. Prior to Phase II tests, the infill panels buckled in Phase I tests were removed using flame-cut and replaced by new panels. Fish plates were used along boundary frame members to connect infill panels. The infill panels of Phase I were weld on one side of the fish plates and those of Phase II on the other side. The test setup is illustrated in Figure 1.

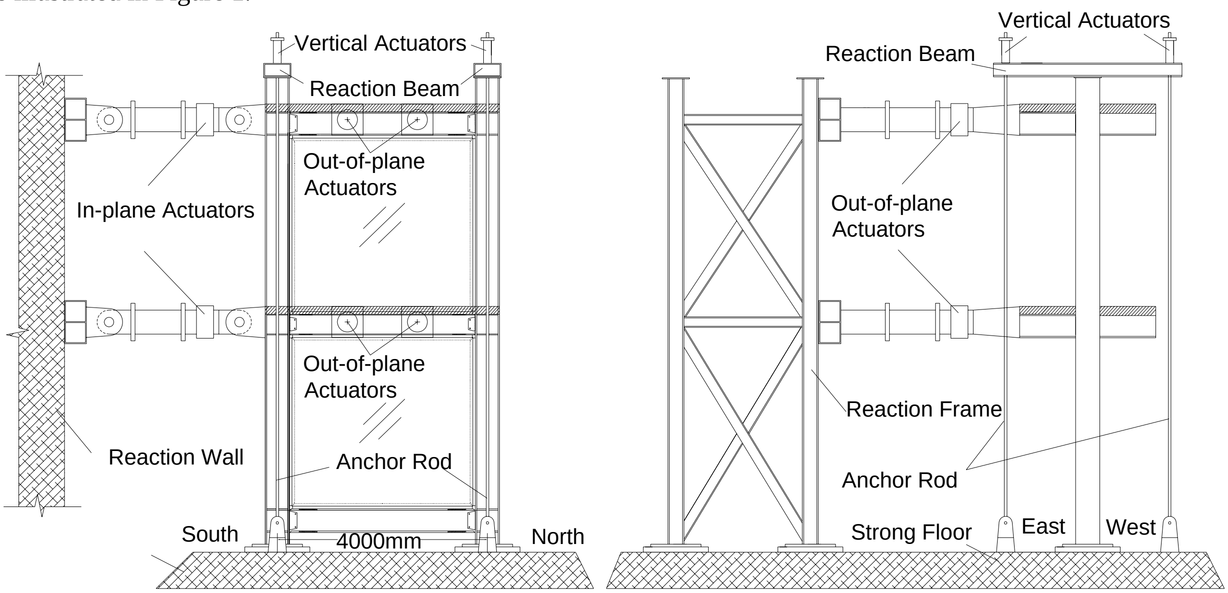
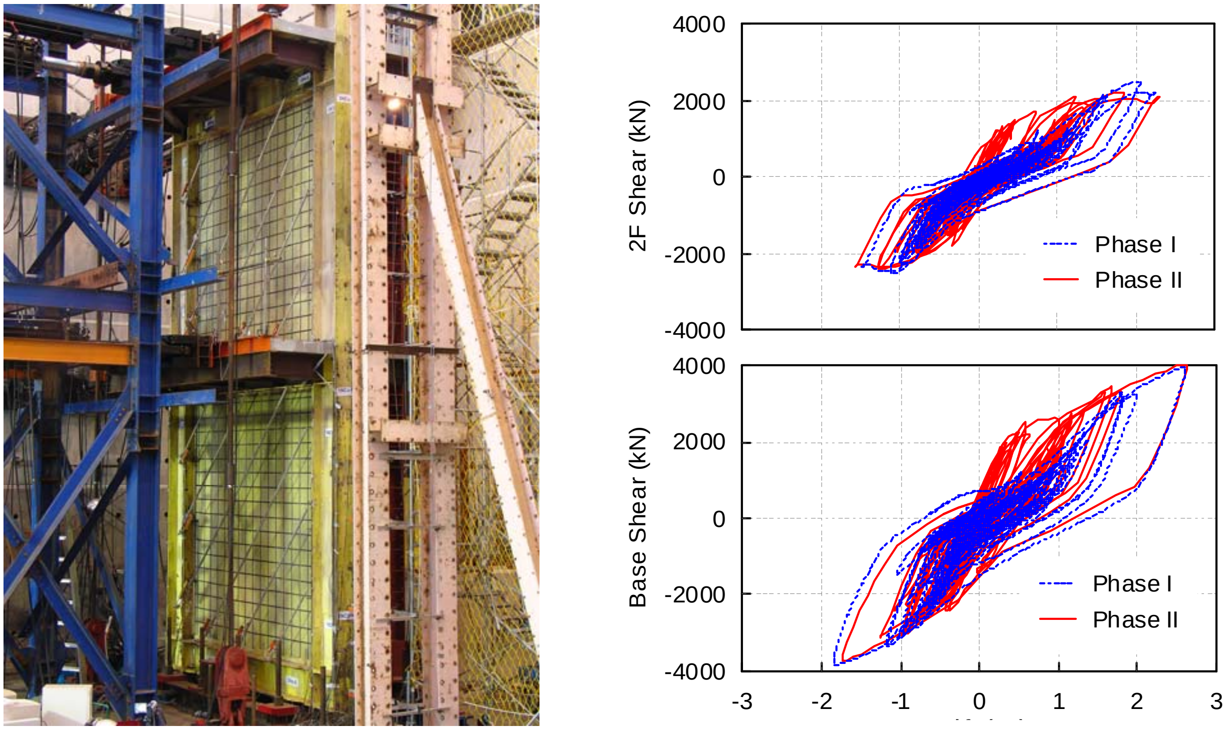



*Figure 1. Test setup*

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. Despite the numerous ancillary calculations that checked the adequacy of the specimen, the intermediate concrete slab and the south column base suffered premature failures at the time step of 9.5 sec and 24 sec of the first earthquake record respectively. The tests resumed after the specimen was strengthened at these locations. The SPSW specimen behaved similarly to the Phase II pseudo-dynamic test described in greater length below. No fracture was found in the boundary frame and it was deemed to be in satisfactory condition allowing for the replacement of infill panels for the subsequent phase of testing.

In order to investigate how the repaired SPSW would behave in a second earthquake 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 after the buckled panels were replaced by new panels.

The specimen prior to Phase II tests and hysteretic curves from Phase II pseudo-dynamic test along with the counterpart results from Phase I are shown in Figure 2. Observation of the hysteretic curves obtained from Phase II shows that the first story dissipated more hysteretic energy than the second story. Both the first and second story exhibited stable displacement-force 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. All welds within the SPSW specimen were intact after the test. Comparing the hysteretic curves from the Phase I and Phase II tests shown together in Figure 2b, 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 results shown for the specimen in Phase I are those obtained after the specimen was repaired due to the unexpected failures mentioned in Phase I tests. Therefore the infill panels had already experienced some inelastic deformation before these unexpected failures occurred.

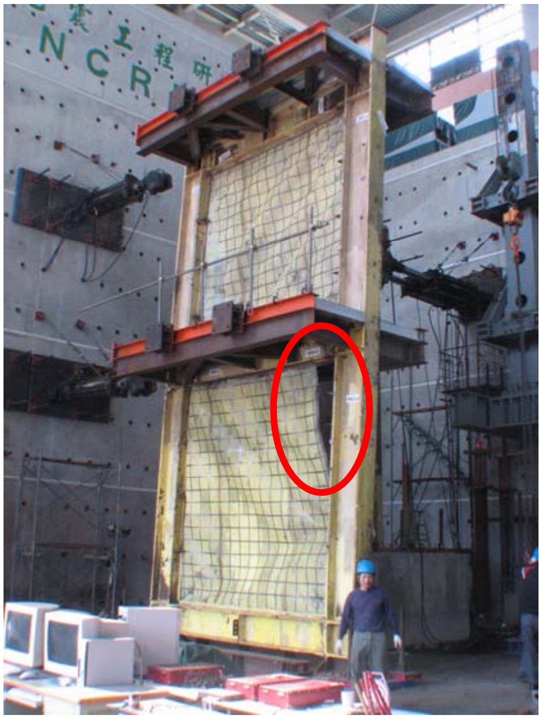


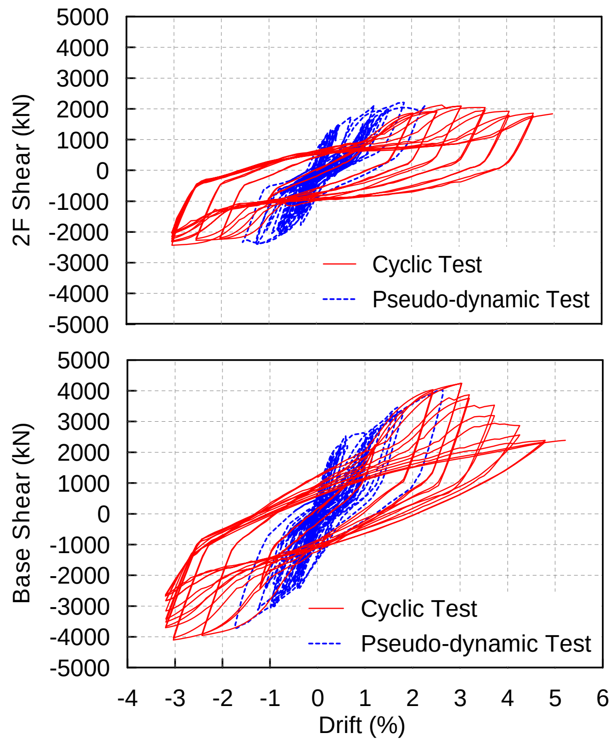

*(a) Specimen prior to Phase II tests* *(b) Hystereses of Phase I and II pseudo-dynamic tests*

*Figure 2. Specimen and hystereses*

The next stage of Phase II tests involved cyclic test on the SPSW specimen in order to investigate the ultimate behavior of intermediate beam and the cyclic behavior and ultimate capacity of SPSW system after sever earthquakes. As mentioned in the observations of Phase II pseudo-dynamic test, the boundary frame members were in good condition after the pseudo-dynamic test except for a small fracture was found 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.

The specimen after Phase II tests and hysteretic curves resulting from the Phase II cyclic test, along with the results of Phase II pseudo-dynamic tests, are shown in Figure 3. 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 specimen 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. A similar fracture developed along the shear tab at the south end of the intermediate beam when the specimen was pulled towards to the reaction wall in this cycle.






*(a) Specimen after Phase II tests* *(b) Hystereses of Phase II tests*

*Figure 3. Specimen and hystereses*

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 Figure 4. 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. These events significantly changed the load path within the system. However, the SPSW specimen was still able to exhibit stable displacement-force behavior as evidenced by the hysteretic curves shown in Figure 3b, 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 a 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 Figure 5. More information about the testing results of Phase I and II is presented elsewhere (Lin et al 2007 and Qu et al 2007)

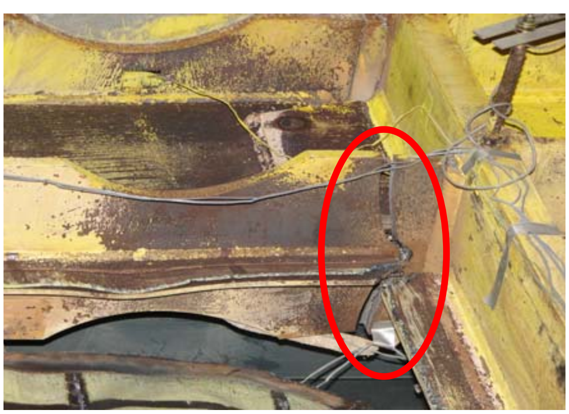


*Figure 4. Ruptures at the north end of intermediate beam*

*Figure 5.Crack at the top slab*

## PRELIMINARY EXPERIMENTAL RESULTS ON THE BLAST RESISTANCE OF SPSW

In recent years some engineers have advocated the use of SPSW in buildings and other structures to resist out-of-plane blast loading. An example of one such structure is presented in Sabelli and Bruneau (2006). Although advanced analysis of these systems, i.e., nonlinear finite element analysis, suggests SPSWs might be capable of resisting substantial out-of-plane impulsive loads and sustaining large inelastic deformation, there has been no experimental validation of the analytical results. To gain an improved understanding of the behavior of SPSWs subjected to out-of-plane blast loading an experimental investigation was conducted. For this study, reduced scale models were fabricated based on a representative prototype SPSW designed to resist seismic loading following the AISC Seismic Provisions (2005).

Two 0.4 scale single story single bay SPSWs were designed and fabricated for blast testing. Each SPSW consisted of two VBEs (W310x79), two HBEs (S200x34) with RBS and a 1.9 mm thick infill plate measuring 1372 mm by 1270 mm connected to the boundary frame through fishplates (L76x51x4.8). The VBE elements were specified ASTM A992 Grade 50 whereas the HBEs and fishplates were ASTM A572 Grade 50. Due to thickness constraints of the infill plate a hot-rolled commercial sheet stock (ASTM A1011) was specified with a measured yield strength of 330 MPa and an elongation at break of 32%. The VBEs of each SPSW were cast directly into a foundation beam and supported at the center line of the top HBE by a steel reaction frame. The boundary frame dimensions were 1830 mm and 1575 mm from center-to-center of the VBEs and HBEs, respectively.

The experimental program consisted of a series of tests performed on the infill plates and VBEs, however, for brevity only the primary infill plate tests are described here. Table 1 presents summary information for the primary plate tests consisting of two explosive detonations. For security purposes exact values of the charge weights and standoff distances have been omitted and are instead presented a multiple of a parameter for the smallest charge and distance. Test number 1 (SPSW 1) is representative of a hand placed explosive with a charge weight of W (equivalent weight of TNT) at a standoff distance of X at the center of the infill plate and a height of 0.9 m (mid-height of the infill plate). Test number 2 (SPSW 2) is representative of a vehicle bomb with a charge weight of 3W at a standoff distance of 2.4X at the center of the infill plate and a height of 0.9 m. Photographs of SPSW 1 and SPSW 2 taken prior to testing are presented in Figures 6a and 7a, respectively.

**Table 1. Summary information for blast testing**

| Test No. | Specimen | Charge Weight, w | Standoff Distance, x | Charge Height, z |
|---|---|---|---|---|
| | | | | (m) |
| 1 | SPSW 1 | W | X | 0.9 |
| 2 | SPSW 2 | 3W | 2.4X | 0.9 |

Figure 6b presents a photograph of the backside of SPSW 1 following test number 1 and illustrates the residual out-of-plane deformation of the plate. The maximum residual out-of-plane deformation occurred at the center of the plate and was measured to be 133 mm. The length of the plate in the horizontal and vertical direction were measured to calculate the residual plastic elongation according to

$$\varepsilon_p = \frac{L - L_o}{L_o} \tag{1}$$

where $L$o=initial length of plate in a particular direction and L=measured length of plate along a particular grid line. Using Eq. (1) the residual plastic elongation at the center of the plate in the horizontal and vertical directions was calculated to be 1.85% and 1.25%, respectively. Although not illustrated here, substantial inelastic deformations were observed in the fishplate and HBE elements. Additionally, partial length fractures were observed in the full penetration groove welds connecting the HBE flanges to the VBEs. Figure 7b presents a photograph of the front of SPSW 2 following test number 2. Failure occurred at the weld connecting the infill plate to the fishplate around three quarters of the perimeter of the plate. The measured length of the plate in the horizontal direction was approximately equal to the original length suggesting the welds failed prior to yielding of the plate. Although no inelastic deformation was observed in the plate, substantial inelastic deformations were observed in the fishplate and

HBE elements, especially the bottom HBE. Again, partial length fractures were observed in the full penetration groove welds connecting the HBE flanges to the VBEs.

*a. before* *b. after*

*Figure 6 Photographs of the backside of SPSW 1 taken before and after Test 1*

*a. before* *b. after*

*Figure 7 Photographs of the front SPSW 2 taken before and after Test 2*

**SPSW WITH INFILL PLATES AND BOUNDARY FRAME DESIGNED TO SHARE VARIOUS PERCENTAGE OF LATERAL FORCES**

Plastic mechanisms of SPSWs subject to lateral loads have been studied by Berman and Bruneau (2003), including the desirable expected plastic mechanism for a ductile multi-story SPSW shown in Figure 8.

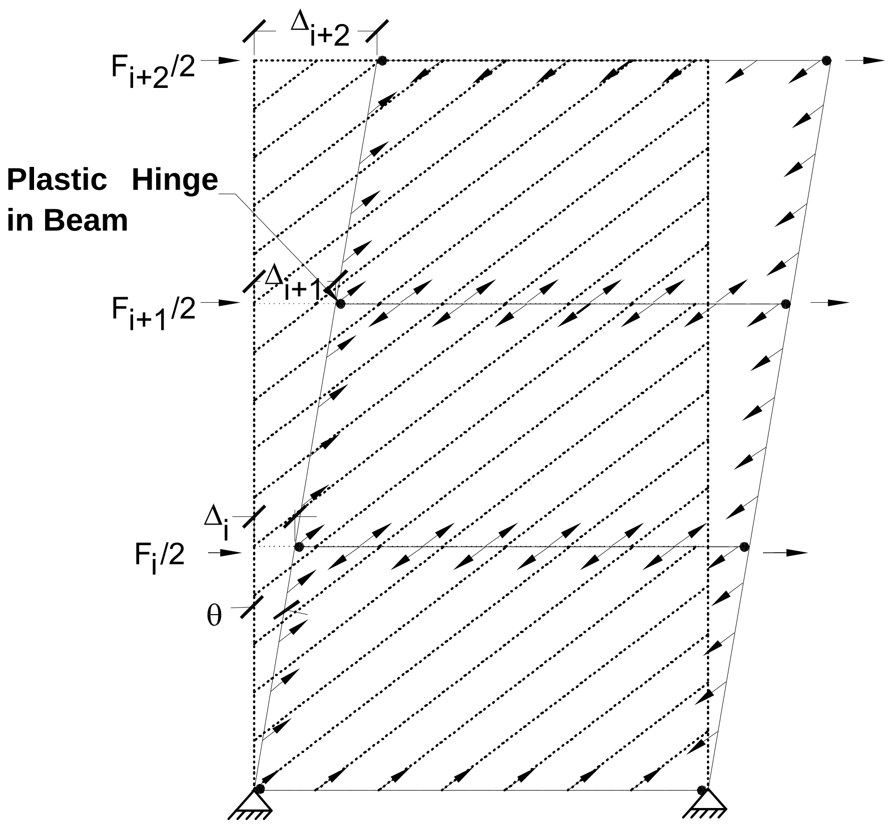



*Figure 8 Expected plastic mechanism of a multi-story SPSW (Berman and Bruneau 2003)*

Using the kinematic method of plastic analysis, the ultimate strength of a SPSW having rigid beam-to-column connections capable of developing the expected plastic moment, can be calculated from the following equation

$$\sum_{i=1}^{n} F_i h_i = \underbrace{\sum_{i=0}^{n}\left(M_{pl_i} + M_{pr_i}\right)}_{\text{Contribution of boundary frame}} + \underbrace{\sum_{i=1}^{n}\frac{1}{2}\left(t_{wi} - t_{wi+1}\right) f_{yp}\, L h_i \sin(2\ \alpha_i)}_{\text{Contribution of infill panels}} \tag{2}$$

where $F_i$ is the lateral force applied at the $i^{th}$ story; $h_i$ is the $i^{th}$ story elevation; $M_{pli}$ and $M_{pri}$ are the plastic moments at the left and right ends of the $i^{th}$ beam respectively; $t_{wi}$ is the infill thickness at the $i^{th}$ story; $f_{yp}$ is the infill plate yield stress, $L$ is the distance between column centerlines; and $\alpha_i$ is the tension field inclination angle at the $i^{th}$ story.

Explicitly shown in Eq (2), internal work of the SPSW includes two parts, namely, (i) the contribution of boundary frame, and (ii) the contribution of infill steel plates, indicating that the lateral force resistance of the wall combines the strengths of boundary frame and infill panels.

Neglecting the contribution of boundary frame, the 2005 *Seismic Provisions for Structural Steel Buildings* (AISC 2005) assumes 100% of story shear is resisted by infill plates. In the perspective of capacity design, this assumption will increase the demands on boundary frame members as well as foundations, resulting in uneconomical designs.

To alleviate this concern, current work is focusing on the investigations into the SPSWs for which the combined resistance of infill plate and boundary frames are designed to share various percentage of the specified lateral load.

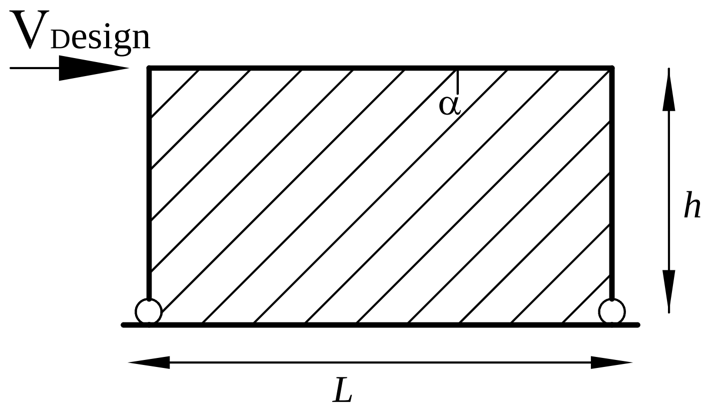



*Figure 9 Single-story SPSW with simple column-to-ground and fixed beam-to-column connections*

Consider a single-story SPSW with simple column-to-ground and fixed beam-to-column connections as shown in Figure 9. Assigning part of the story shear is assigned to infill panels, results in the flowing equation

$$\kappa \cdot V_{design} = \frac{1}{2} f_{yp} t_w L h \sin(2\alpha) \tag{3}$$

where $V_{\text{design}}$ is the specified base shear and $\kappa$ is the percentage of base shear resisted by infill plates which may vary between zero and unity (equivalent to AISC design assumption).

Per the capacity design procedure proposed by Vian and Bruneau (2005), plastic section modulus of the top beam, $Z_{\text{b}}$, can be determined

$$Z_b = \frac{L^2 \cdot t_w \cdot \cos^2 \alpha}{4} \cdot \frac{f_{yp}}{f_{yb}} \cdot \frac{1}{1+\sqrt{1-\beta^2}} \tag{4}$$

where $f_{\text{yb}}$ is the yield stress of top beam, and β is the RBS plastic section modulus reduction ratio which may vary between a theoretical value of zero (equivalent to a simple support) and one (fully fixed support and without flange reduction).

Assuming the beam hinges will form instead of column hinges at the roof levels, results in the following equation for plastic strength of the wall

$$V_{plastic} = \frac{1}{2} f_{yp} t_w L \sin(2\alpha) + \frac{2 f_{yb} Z_b}{h} \tag{5}$$

Solving for $V_{\text{design}}$ from Eq. (3), and Substituting Eq. (4) into (5) and solving for $V_{\text{plastic}}$, one can obtain the following overstrength ratio of the wall

$$\frac{V_{plastic}}{V_{design}} = \kappa \cdot \left[ 1 + \frac{L}{2h} \cdot \tan^{-1}(\alpha) \cdot \frac{\beta}{1+\sqrt{1-\beta^2}} \right] \tag{6}$$

Based on Eq. (6), results from a parametric study are shown in Figure 9 for the given values of aspect ratio (i.e. L/h), inclination angle, and RBS plastic section modulus reduction ratio. As shown, assuming that 100% of the story shear (i.e. κ=1) is resisted by infill plates, may result in a wall with significant overstrength (i.e. $V_{\text{plastic}}/V_{\text{design}}$ is greater than 1). Assigning a certain percentage (i.e. $\kappa_{\text{balance}}$ shown in figure 8) of story shear to the infill plates, it is possible to obtain a wall with plastic strength equal to the specified design force. However, if the percentage of story shear is smaller than $\kappa_{balance}$, boundary frame members should be amplified to ensure that plastic strength of the wall is not smaller than design force. While all conceptual designs shown in Figure 8 are theoretically possible, the relative seismic performance of such different designs is unknown at this time. The resulting designs are expected to have quite different hysteretic properties and seismic behavior, which may bear a relationship to the seismic reduction factor (R) that should be considered for each case. The authors are currently conducting research to assess these effects and provide design recommendations. Until then, the infills of SPSW should be designed for 100% of the seismic force, as implied by the AISC 2005 seismic provisions.

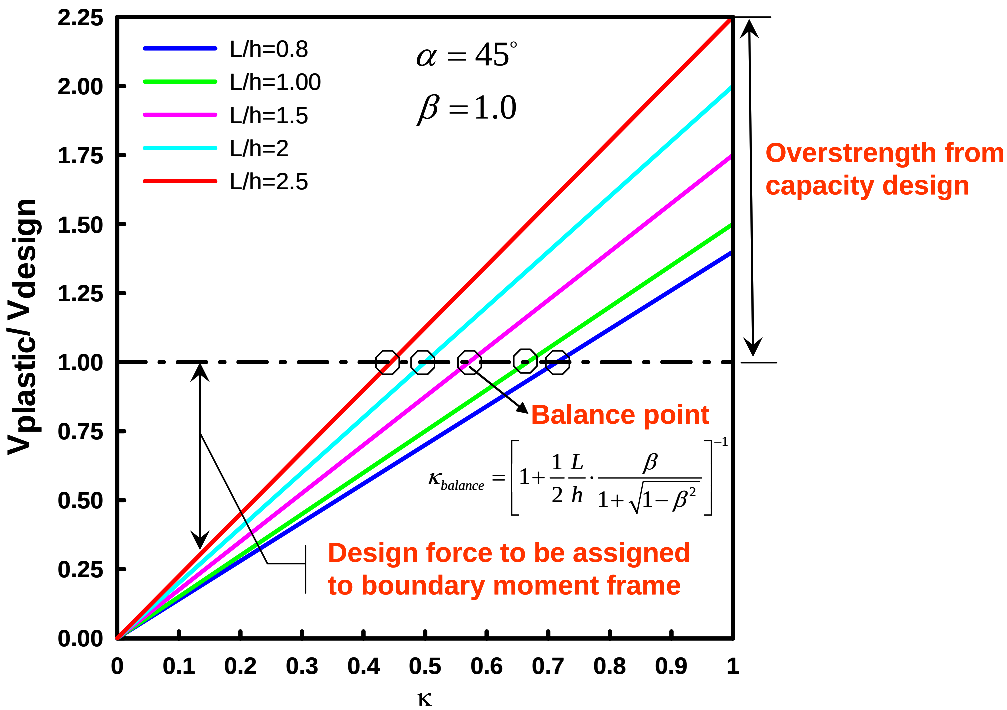



*Figure 8 Behaviors of SPSWs with infill plate designed using various percentage of the specified lateral load.*

## CONCLUSION

This paper provided an overview of recent development on SPSW. It was shown that: (i) replace buckled steel plates without major impact on seismic performance, thus allowing repair of SPSWs following an earthquakes, (ii) SPSW can be design to have some level of blast resistance; and (iii) it is possible to obtain a more economical design accounting for the contribution of boundary frame to lateral force resistance while there remain uncertainties regarding the behavior of the resulting wall.


## ACKNOWLEDGEMENTS

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