

A Review of Steel Plate Shear Wall Design Requirements and Research

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A steel plate shear wall (SPSW) (a.k.a. special plate steel wall) is a lateral load resisting system consisting of vertical steel plate infills connected to the surrounding beams and columns and installed in one or more bays along the full height of the structure to form a cantilever wall (Figure 1). SPSWs subjected to cyclic inelastic deformations exhibit high initial stiffness, behave in a very ductile manner, and dissipate significant amounts of energy. These characteristics make them suitable to resist and dissipate seismic loading. SPSWs can be used not only for the design of new buildings but also, as indicated by recent research efforts (Berman and Bruneau, 2003a), for the retrofit of existing constructions. Beam-to-column connections in SPSWs may in principle be either the simple type or moment-resistant; although, as described later, only the latter are allowed by the requirements of the 2005 AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005).

Prior to key research performed in the 1980s, the design limit state for SPSW was considered to be out-of-plane buckling of the infill panel. To prevent buckling, engineers designed steel walls with heavily stiffened infill plates that were not economically competitive with reinforced concrete shear walls. However, several experimental and analytical studies using both quasi-static and dynamic loading showed that the post-buckling strength of thin SPSW can be

substantial (Thorburn, Kulak, and Montgomery, 1983; Timler and Kulak, 1983; Tromposch and Kulak, 1987; Roberts and Sabouri-Ghomi, 1992; Sabouri-Ghomi and Roberts, 1992; Cassese, Elgaaly, and Chen, 1993; Elgaaly, Caccese, and Du, 1993; Driver, Kulak, Kennedy, and Elwi, 1998; Elgaaly and Liu, 1997; Elgaaly 1998; Rezai, 1999; Lubell, Prion, Ventura, and Rezai, 2000; Berman and Bruneau, 2003a; Vian and Bruneau, 2004; Berman and Bruneau, 2004). Based on some of this research, the Canadian Standards Association steel design standard CAN/CSA S16-01 has implemented design clauses for SPSW allowed to buckle in shear and develop tension field action (CSA, 2001).

Additional research on unstiffened steel plate shear walls has investigated the effect of simple versus rigid beam-to-

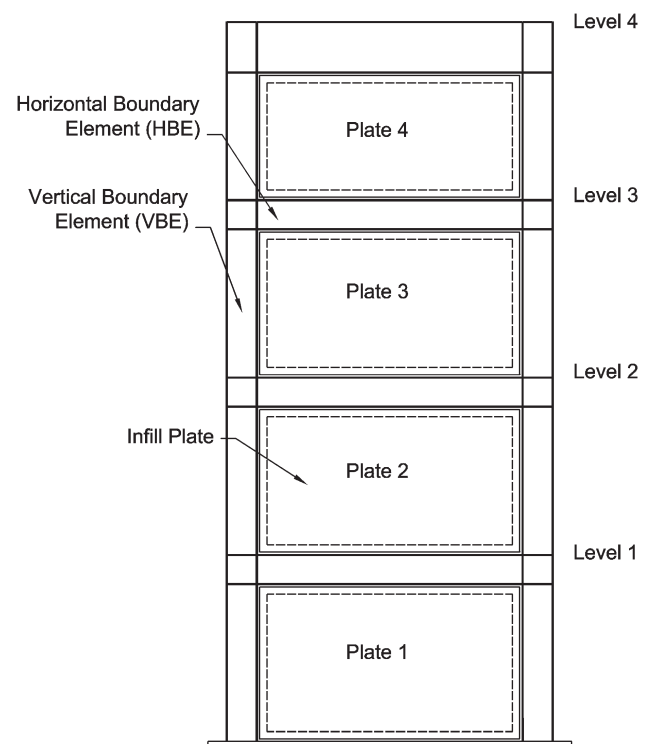


Fig. 1. Typical steel plate shear wall.

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column connections on the overall behavior (Caccese et al., 1993), the dynamic response of steel plate shear walls (Sabouri-Ghomi and Roberts, 1992; Rezai, 1999), the effects of holes in the infill plates (Roberts and Sabouri-Ghomi, 1992; Vian and Bruneau, 2004), the use of low yield point steel or light-gauge steel (Vian and Bruneau, 2004; Berman and Bruneau, 2005), and the effects of bolted versus welded infill connections, as well as other practical considerations, by Elgaaly (1998). Furthermore, finite element modeling of unstiffened steel plate shear walls has been investigated in some of the aforementioned papers as well as by Elgaaly et al. (1993) and Driver et al. (1998).

THE STRIP MODEL REPRESENTATION OF SPSW

A typical SPSW (Figure 1) consists of horizontal and vertical boundary elements (that may or may not carry gravity loads), and thin infill plates that buckle in shear and form a diagonal tension field to resist lateral loads. Based on an elastic strain energy formulation, Timler and Kulak (1983) derived the following equation for the inclination angle of the tension field, α , in a SPSW infill plate:

$$\alpha = \tan^{-1} \sqrt[4]{\frac{1 + \frac{tL}{2A_c}}{1 + th \left(\frac{1}{A_b} + \frac{h^3}{360I_c L} \right)}} \quad (1)$$

where

- t = thickness of the infill plate
- h = story height
- L = bay width
- I_c = moment of inertia of the vertical boundary element
- A_c = cross-sectional area of the vertical boundary element
- A_b = cross-sectional area of the horizontal boundary element

The flexural stiffness of the horizontal boundary elements was excluded in the derivation because the opposing tension fields that develop above and below these intermediate horizontal members almost cancel out and induce little significant flexure there. Using the inclination angle given by Equation 1, an analytical model known as a strip model, in which the infill plates are represented by a series of pin-ended, tension only strips, was developed by Thorburn et al. (1983) and subsequently refined by Timler and Kulak (1983). A typical strip model representation of a SPSW is shown in Figure 2, and the accuracy of the strip model has been verified through comparisons with experimental results such as in Figure 3, which has been adapted from Driver et al. (1998). Note that each strip has a cross-sectional area equal to the strip’s tributary width times the infill thickness.

PLASTIC ANALYSIS OF SPSW

Using the collapse mechanism of a single-story SPSW in a frame with simple connections represented by the strip model, as shown in Figure 4, results in the following equation for base shear (Berman and Bruneau, 2003b)

$$V = \frac{1}{2} F_y t L \sin 2\alpha \quad (2)$$

where

$$F_y = \text{infill yield stress}$$

and other terms are as previously defined.

For a single-story SPSW in a frame with rigid beam-to-column connections plastic analysis can again be used to find the base shear as

$$V = \frac{1}{2} F_y t L \sin 2\alpha + \frac{4M_p}{h} \quad (3)$$

where

$$M_p = \text{smaller of the beam and column plastic moments}$$

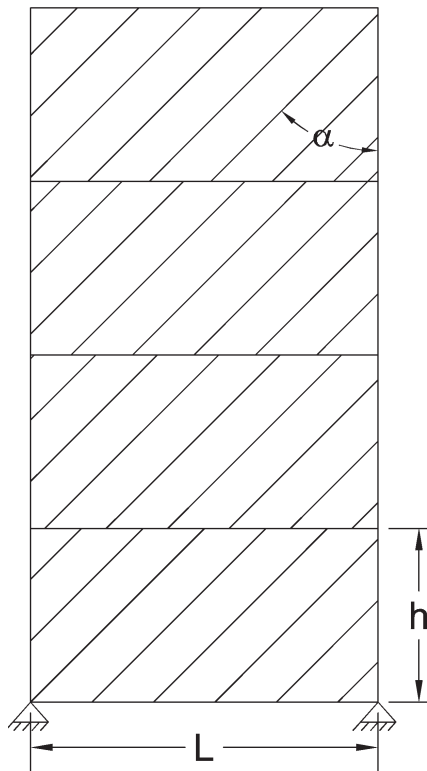


Fig. 2. Strip model representation of a SPSW.

Equations are also derived for various collapse mechanisms in multistory SPSW in Berman and Bruneau (2003b). These equations can be used to determine an infill thickness for use in development of the strip model.

DESIGN OF SPSW

Ideally, a SPSW should be designed in such a way that *all* its steel panels dissipate energy through inelastic deformations when the structure is subjected to the expected seismic actions. Hence, for a given frame geometry (which is often dictated by architectural/functional considerations), the thickness of the steel panel at a given story should be determined as a function of the corresponding story shear demand. A practical approach consists of solving Equation 2 for t , which gives

$$t = t_{wi} = \frac{2V_i}{F_y L \sin(2\alpha_i)} \quad (4)$$

where the subscript i refers to the i -th story.

Equation 4 is slightly conservative for simple beam-to-column connections and somewhat conservative for moment-resistant connections, since the contribution of this type of connection to the lateral resistance of the SPSW is neglected in Equation 2. It must be noted that Equation 4 indicates that steel panels should have different thicknesses at different stories, a condition that is sometimes difficult to achieve in practice due to the availability of steel plates.

As mentioned before, the ultimate strength of a steel panel is fully developed only when the corresponding frame members are sufficiently stiff and strong to “anchor” the tension diagonals. Furthermore, it has been recommended

(Montgomery and Medhekar, 2001) that for vertical boundary elements (VBE) the moment of inertia, I_c , should be such that

$$0.70h \left(\frac{t_w}{2LI_c} \right)^{0.25} \leq 2.5 \quad (5)$$

which leads to

$$I_c \geq \frac{0.00307 t_w h^4}{L} \quad (6)$$

While no practical expressions similar to Equation 6 have been proposed for the horizontal boundary elements (HBEs) at the roof and foundation levels, sections providing the strength necessary to satisfy the corresponding flexural demands are likely to provide an adequate stiffness. It must be remembered that the HBEs at the roof and foundation levels must anchor the pulling action from a yielding steel panel, which generally results in substantial sizes.

It has been argued that, since the behavior of a SPSW is similar to that of vertical cantilever plate girders (VBEs act like “flanges,” HBEs act like “stiffeners,” and steel panels act like the “web”), the former could be designed using well-established procedures suitable for the latter (Astaneh-Asl, 2001). However, while the plate girder analogy is conceptually valid and useful, it is quantitatively inadequate and leads to overly conservative designs. Detailed explanations, including quantitative comparisons, are presented in Berman and Bruneau (2004).

REGULATIONS AND CODE REQUIREMENTS

The 2003 *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (BSSC, 2004) and the 2005 *AISC Seismic Provisions for Structural Steel Buildings* (AISC, 2005) already include minimum design requirements for SPSWs (here, the 2003 NEHRP Provisions will be referred to as FEMA 450). It must be noted that SPSWs are denoted as *special steel plate walls*

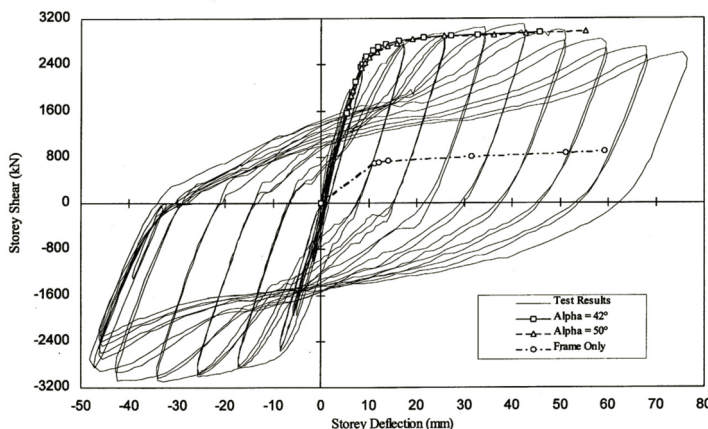


Fig. 3. Comparison of strip model and experimental results (Driver et al., 1998).

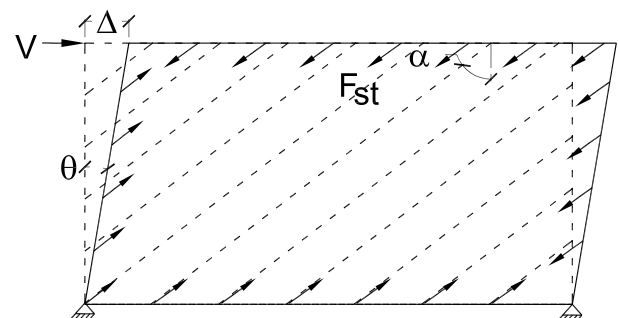


Fig. 4. Single-story collapse mechanism.

in FEMA 450 and as *special plate shear walls* in the 2005 AISC *Seismic Provisions for Structural Steel Buildings*. In both documents, columns are designated as *vertical boundary elements* (VBEs), beams are referred to as *horizontal boundary elements* (HBEs), steel panels are denoted simply as webs, and a web and its surrounding HBEs and VBEs constitutes a panel.

In both BSSC (2004) and AISC (2005), the nominal strength of a web is set equal to

$$V_n = 0.42F_y t_w L_{cf} \sin(2\alpha) \quad (7)$$

where

L_{cf} = clear distance between VBE flanges

In Equation 7, α is to be calculated using Equation 1. It must be noted that Equation 7 is identical to Equation 2 except that L (distance between VBE centerlines) is replaced by L_{cf} and the $\frac{1}{2}$ factor is replaced by 0.42, which is simply 0.50 divided by an overstrength factor equal to 1.2. The design shear strength of a web is given by ϕV_n , where $\phi = 0.90$. According to these regulations, HBEs and VBEs are to remain essentially elastic under forces generated by fully yielded webs, but flexural hinges are allowed at the ends of HBEs. Both documents require that VBEs satisfy Equation 6 and that $0.80 < L/h \leq 2.50$. In addition, several details are specified for HBE-VBE connections. Finally, FEMA 450 imposes a limit on the slenderness of the web, quantitatively expressed by

$$\frac{\min(L, h)}{t_w} \leq 25 \sqrt{\frac{E}{F_y}} \quad (8)$$

where

E = Young's modulus of the web

The Canadian standard CAN/CSA-S16, *Limit States Design of Steel Structures* (CSA, 2001), has included specifications for the design of SPSWs since 1994. In this document, the equivalent truss model (Thorburn et al., 1983) is recommended for preliminary design purposes. The idea consists of modeling the steel panels on each floor as a single diagonal (corner-to-corner) steel truss member designed for the specified lateral loads and to meet drift requirements. The thickness of the steel panel at the i -th story, t_{wi} , is then given by

$$t_{wi} = \frac{2A_i \sin\theta_i \sin 2\theta_i}{L \sin^2 2\alpha_i} \quad (9)$$

where

A_i = area of the equivalent truss member at the i -th story

θ_i = angle of inclination (measured with respect to a vertical axis) of the equivalent truss member at the i -th story

This approach is intended for use as a preliminary design tool because it will generally be unconservative except for nearly square panels (Berman and Bruneau, 2003a). Another interesting feature of the CAN/CSA-S16-01 standard consists of considering limited ductility plate walls (in other words, SPSWs with no special requirements for beam-to-column connections and assigned a force modification factor $R = 2$, which is equivalent to $R = 3.2$ using FEMA 450) in addition to ductile plate walls (in other words, SPSWs with moment-resisting beam-to-column connections and a force modification factor $R = 5$, the largest R value assigned to the most ductile systems in this standard and equivalent to $R = 8$ in FEMA 450).

OTHER DESIGN ISSUES

It is worthwhile to briefly mention some other design issues for SPSW that are addressed in CAN/CSA S16-01 and will likely appear in U.S. design codes as well. Horizontal and vertical boundary elements should be designed to elastically resist development of the full expected tensile capacity of the infill plates. This ensures that the infill plate can yield in tension prior to plastic hinging of the boundary elements (providing for substantial energy dissipation in seismic applications). Such capacity design can be achieved by designing the boundary elements for the forces found from pushover analysis of the strip model, or indirectly from the procedure in CAN/CSA S16-01. The connection of the infill plate to the boundary elements should also be designed for the expected tensile capacity of the infill plate and can use either a welded or bolted configuration. Four different connection details were developed, tested, and found to be equivalent by Schumacher, Grondin, and Kulak (1999). Furthermore, the vertical boundary elements should satisfy a minimum stiffness requirement (given in CAN/CSA S16-01) to prevent excessive deformations under the tension field action of the web plate. Finally, horizontal boundary elements should be provided at the top and bottom of a SPSW to anchor the tension field.

NEW DIRECTIONS FOR SPSW

One difficulty in the selection of SPSW systems is that the available panel material may be stronger or thicker than needed for a given design situation. In the perspective of capacity design, this will increase the necessary sizes of horizontal and vertical boundary members as well as foundation demands. To alleviate this concern, recent work has focused on the use of light-gauge cold-rolled and low yield strength (LYS) steel for the infill panel (Berman and Bruneau, 2005; Vian and Bruneau, 2004), and also by placement of a pattern of perforations to decrease the strength and stiffness of the panel by a desired amount (Vian and Bruneau, 2004). In addition, the use of reduced beam sections at the ends of the horizontal boundary members is being investigated as a

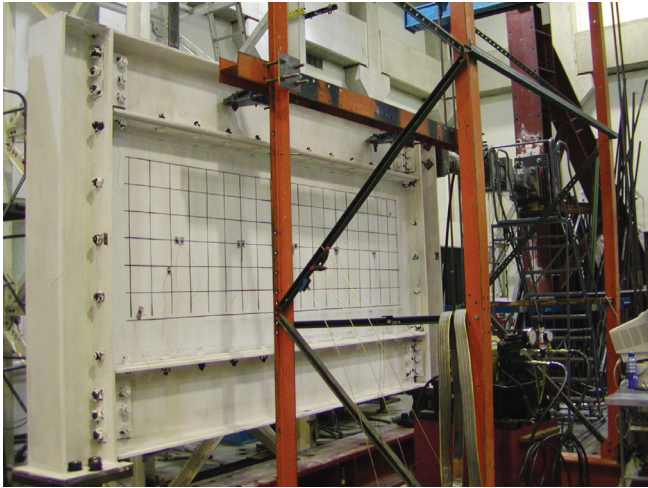


Fig. 5. Light-gauge SPSW prior to testing.

means of reducing the overall system demand on the vertical boundary members (Vian and Bruneau, 2004).

A SPSW test specimen utilizing a light-gauge infill [thickness of 1.0 mm (0.0396 in.)] is shown in Figure 5 (Berman and Bruneau, 2005). The specimen used W310×143 (W12×96) columns and W460×128 (W12×86) beams. This test was performed using quasi-static cyclic loading conforming to the recommended Applied Technology Council (ATC) loading protocol of ATC 24 (ATC, 1992). Hysteretic results are shown in Figure 6 along with the boundary frame contribution. After subtracting the boundary frame contribution, the hysteresis of Figure 7 is obtained. This specimen reached a ductility ratio of 12 and drift of 3.7%, and the infill was found to provide approximately 90% of the initial stiffness of the system. Ultimate failure of the specimen was due to fractures in the infill propagating from the welds which connected it to the boundary frame. Figures 8a and 8b show the

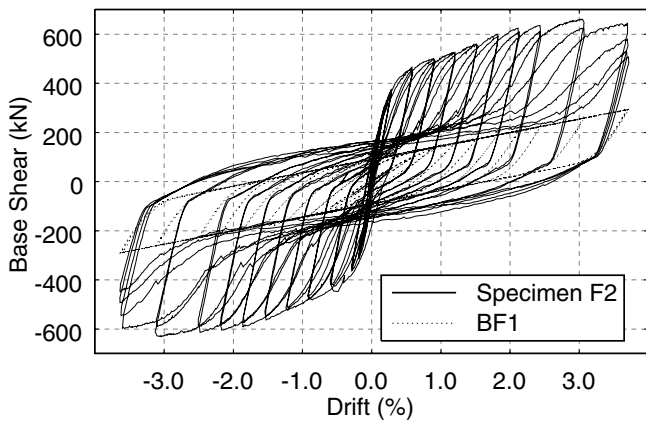


Fig. 6. Light-gauge SPSW and boundary frame hysteresses.

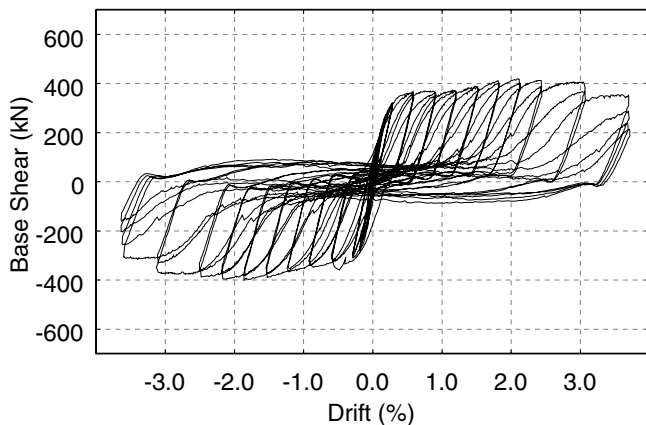
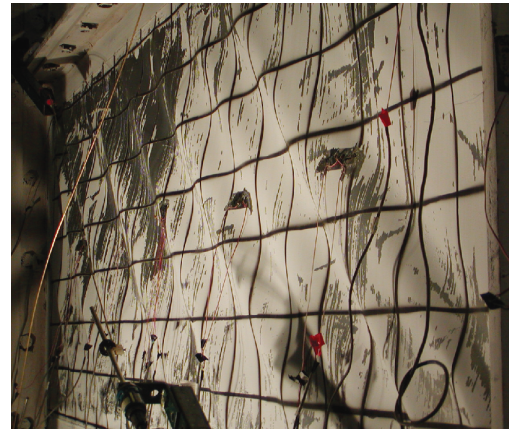


Fig. 7. Light-gauge SPSW hysteresses—infill only.



(a)



(b)

Fig. 8. (a) Buckling of infill at 1.82% drift; (b) Fracture of infill corner at 3.07% drift.

buckling of the infill plate at the peak displacement of cycle 20 (ductility ratio of 6, 1.82% drift) and the fracture at the infill corner during cycle 26 (ductility ratio of 10, 3.07% drift), respectively.

Three SPSW specimens of similar size and dimension, but utilizing LYS infill panels were designed, built, and subjected

to quasi-static cyclic loading (Vian and Bruneau, 2004). The frames consisted of 345 MPa (50 ksi) steel members, while the infill panels were 2.6 mm (0.102 in.) thick, LYS steel plates with an initial yield strength of 165 MPa (24 ksi), and ultimate strength of 300 MPa (44 ksi), important properties that may aid in alleviating overstrength concerns mentioned earlier. All specimens also had beam-to-column connection details that included reduced beam sections (RBS) at each end.

A perforated panel SPSW specimen (Specimen P), consisting of a panel with a total of twenty 200-mm-diameter (7.9 in.) holes, or perforations, is shown in Figure 9 prior to testing. The multiple perforations present in the tested specimen allow for utility access in order to make the SPSW system more acceptable, while also serving as a method of reducing the panel strength and therefore the demand on the surrounding framing. This latter characteristic may prove beneficial in markets that do not have LYS steel readily available for structural applications.

All specimens tested in this experimental program exhibited stable force-displacement behavior, with very little pinching of hysteresis loops until the significant accumulation of damage at large drifts. Specimen P performed well, behaving elastically at small displacements and exhibiting stable hysteretic behavior. The stiffness and strength were both reduced, as anticipated, from the solid panel specimen values, as shown in Figure 10.



Fig. 9. Perforated panel specimen (P) before testing.

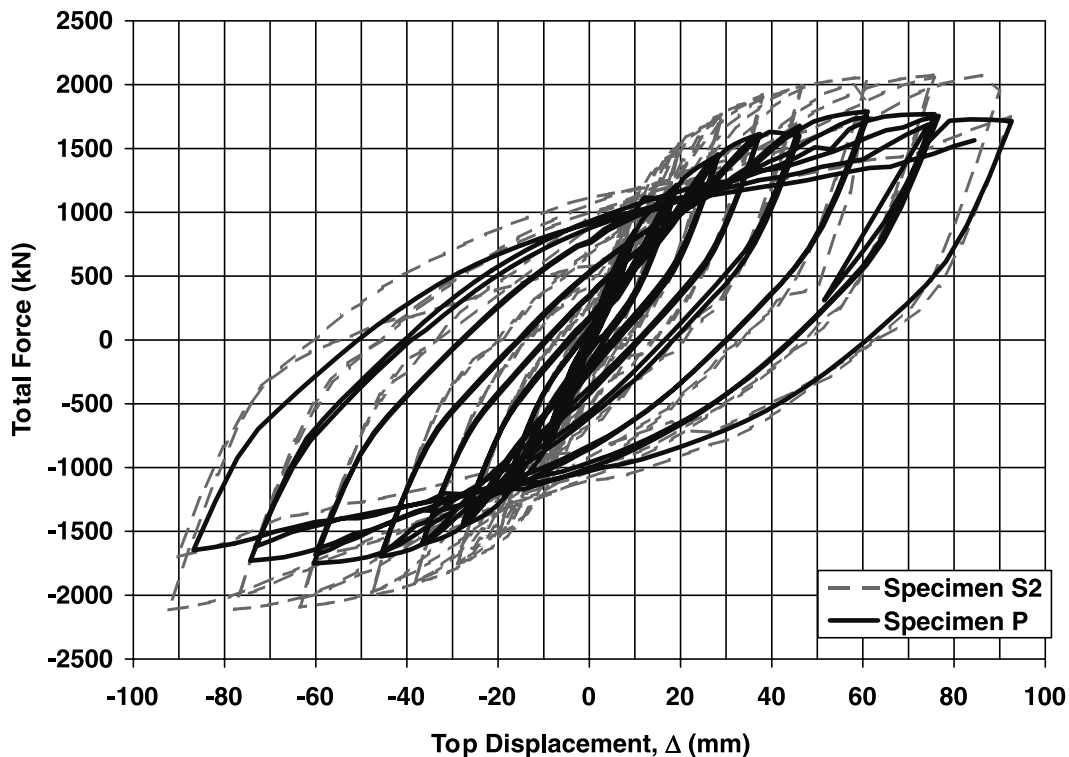


Fig. 10. Solid panel (S2) and perforated panel (P) specimen hysteresis curves.

CONCLUSION

An overview of the design requirements for steel plate shear walls in new and retrofit building construction has been provided, along with an overview of research supporting those design requirements. The strip model, developed by others for the representation of SPSW has been described, and plastic analysis results for that model and their use in design has been discussed. Additional design considerations likely to appear in U.S. design codes have also been presented. Finally, new directions for SPSW research including the use of light-gauge and low yield point steels, strategic placement of perforations in the infill panel, and reduced beam sections have been described in the context of some ongoing research.

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