Steel Structures Research Update

Ring-Shaped Steel Plate Shear Walls for Improved Seismic Performance of Buildings

JUDY LIU

INTRODUCTION

Recently completed work on the innovative ring-shaped steel plate shear wall (RS-SPSW) is highlighted. The research was led by Dr. Matthew Eatherton, Associate Professor at Virginia Tech and an AISC Milek Fellow. Dr. Eatherton was also awarded an NSF Faculty Early Career Development (CAREER) grant in 2015 for "Innovative Structural Systems for Multi-hazard Resistance Using Steel Plate with Cutouts."

Abhilasha Maurya, an engineer at Walter P. Moore in Florida, conducted some of the original work on RS-SPSWs for her M.S. thesis. Small-scale tests on the RS-SPSW were completed by Natalia Egorova, who earned her M.S. degree and is now an engineer for AECOM in Moscow. Adam Phillips, whose Ph.D. dissertation focused on cyclic testing and development of RS-SPSWs, is an Assistant Professor at Washington State University.

The work has inspired research internationally on steel plates with cutouts, including a computational parametric study on the behavior of steel plate shear walls with constrained ring holes (Parvathy and Manoharan, 2015). Contemporary, related work has been done by Alavi and Nateghi (2012), who proposed a combination of diagonal stiffeners and a central, circular perforation in a steel plate shear wall with the objective of improving ductility as compared to solid steel plate shear walls. Featured here is the research conducted at Virginia Tech, with a brief discussion of the RS-SPSW concept and behavior; a summary of the large-scale experiments; and selected nonlinear response history analysis results comparing conventional SPSW and RS-SPSW prototype buildings.

RS-SPSW CONCEPT AND BEHAVIOR

The ring-shaped steel plate shear wall (RS-SPSW) seeks to improve upon the conventional steel plate shear wall (SPSW) with improved stiffness, energy dissipation, and seismic performance. Conventional SPSWs (Figure 1a) have thin steel web plates that buckle at relatively low shear values and that impose large post-buckling demands on the boundary members as they yield in tension field action. SPSWs, therefore, require large boundary members (e.g., W36×800 vertical boundary elements for 3⁄16-in. web plates in a 14-story building). Moment-resisting beam-column connections are also necessary for lateral resistance during load reversals because the SPSW has a pinched hysteretic behavior (Maurya et al., 2013). In contrast, the RS-SPSW utilizes a unique pattern of cutouts to improve upon the hysteretic response while reducing out-of-plane buckling and eliminating the need for moment connections (see Figure 1b). The ring shape was inspired by research demonstrating significant energy dissipation of devices capitalizing on the plastic hinging mechanism of a ring (Tyler, 1985; Ciampi et al., 1993; Rogers and Morrison, 2011). Development of the ring-shape concept, with computational and experimental validation, confirmed the RS-SPSW as a viable alternative to the conventional SPSW (Maurya et al., 2013; Egorova et al., 2014).

RS-SPSW Concept

The basic RS-SPSW concept, as shown in Figure 2, relies upon elongation of the rings as the full wall deforms in shear. For each ring, an elongation in tension, δ_1, is accompanied by a transverse deformation, δ_2, as shown in Figure 2a. Figure 2c illustrates that, at small deformations, the ratio of δ_2 to δ_1 is 1 for the ring. As a result, there is no "slack" or material that would buckle in compression as the full wall elongates by Δ along the tension diagonal and is shortened by Δ along the compression diagonal (see Figure 2d). The deformation of the solid SPSW web plate in diagonal tension is shown in Figures 2b and 2c. A Poisson's ratio of 0.3 for the steel corresponds to a lower ratio of δ_2 to δ_1 and material that buckles in the transverse direction as the wall deforms (Maurya et al., 2013).

RS-SPSW Behavior

The RS-SPSW can be tuned to achieve desired strength, stiffness and ductility. Ring size, width of the rings, width of the links between rings, and plate thickness influence the

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behavior of the RS-SPSW. Eight RS-SPSW web plates were detailed, fabricated, and subjected to cyclic shear deformations to study the influence of these parameters. Specific objectives were to "investigate the cyclic hysteretic behavior of these panels, evaluate the effectiveness of a derived strength equation, and study the potential buckling modes associated with these plate configurations. The tests were performed on plates that were approximately one meter square which was large enough to test rings that might be used in full-scale structures" (Egorova et al., 2014). A solid web panel, representative of a conventional SPSW, was included with the test specimens. A companion finite element (FE) parametric study was used to further validate and extend the experimental results. The test setup, shown in Figure 3, was used to apply cycles of increasing shear deformations to the RS-SPSW specimens. Additional details can be found in Egorova et al. (2014).

The cyclic behavior of the RS-SPSW specimens compared well to that of the solid plate specimen, which saw shear buckling at approximately half of its shear capacity, followed by pinched behavior and near-zero stiffness during load reversal. The RS-SPSW developed plastic hinging of the rings, and shear buckling was delayed for specimens with lower plate slenderness. The relatively full hysteretic response of the RS-SPSW can be seen in Figure 4a as compared to Figure 4b. Figure 4 also illustrates good agreement between the experimental and computational results, using stress-strain constitutive models based on tension coupon tests for the plate steel. Ring slenderness, as defined by outside ring radius divided by plate thickness, as well as ring width divided by plate thickness, correlated to lateral torsional buckling in the RS-SPSWs. Fracture of the RS-SPSW specimens did not occur until shear angles of 8% or more. The data were used to further develop the strength equation and design procedures for ductile RS-SPSWs.

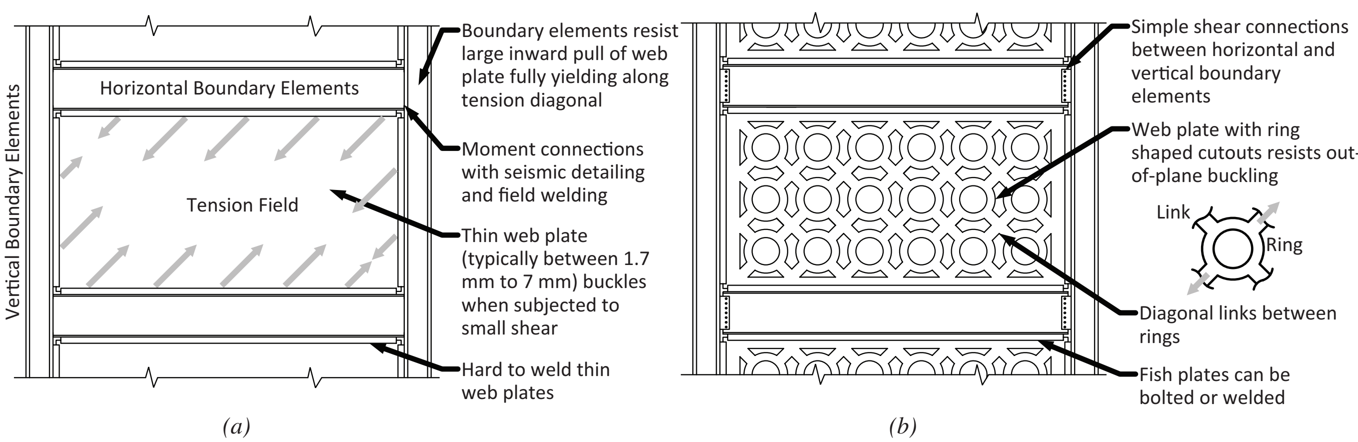


Fig. 1. (a) Conventional steel plate shear wall (SPSW); (b) ring-shaped steel plate shear wall (RS-SPSW).

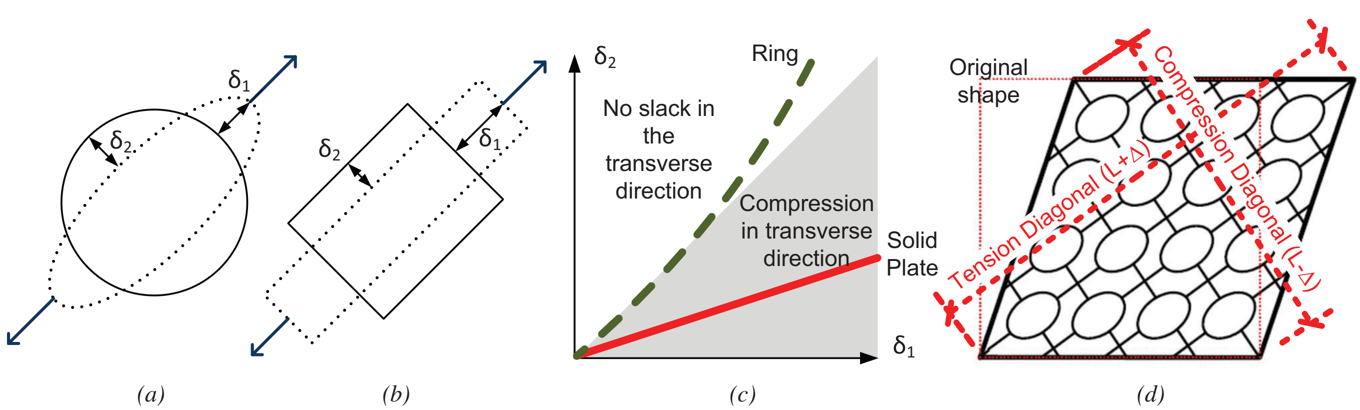


Fig. 2. RS-SPSW concept: (a) ring; (b) solid plate; (c) relationship between δ_1 and δ_2; (d) ring shapes in a shear panel.

LARGE-SCALE EXPERIMENTS ON RING-SHAPED STEEL PLATE SHEAR WALLS

Four large-scale specimens have been tested as further validation of the RS-SPSW system. The specimens were based on the design of a six-story prototype building in San Francisco, California (IBC/SEAOC, 2012; Phillips and Eatherton, 2016). The testing program was used to evaluate the cyclic response, energy dissipation, and deformation capacity of RS-SPSWs.

Large-Scale Specimens and Test Setup

The four RS-SPSW specimens were tested along with one solid plate specimen for comparison. The specimens were approximately two-thirds scale of the first-story walls (three RS-SPSW specimens) and fifth-story walls (fourth RS-SPSW specimen) in the prototype building. The plates were connected with bolted double angles to W12×170 and W21×111 boundary elements. In order to isolate the response of the infill panel, the column base connections and connections of the vertical boundary elements (VBEs) to the horizontal boundary elements (HBEs) used 3-in.-diameter hardened steel pins (Figure 5). Recall that the RS-SPSWs do not require moment connections for supplemental energy dissipation; HBE-to-VBE connections can be simple connections. A modified ATC-24 loading protocol was used; the cyclic displacement protocol increased from 0.25% drift to 5.0% drift and then returned to 4.0% drift until failure. "This modification was made to ensure that the specimens were cycled well past an initial fracture and to ultimate

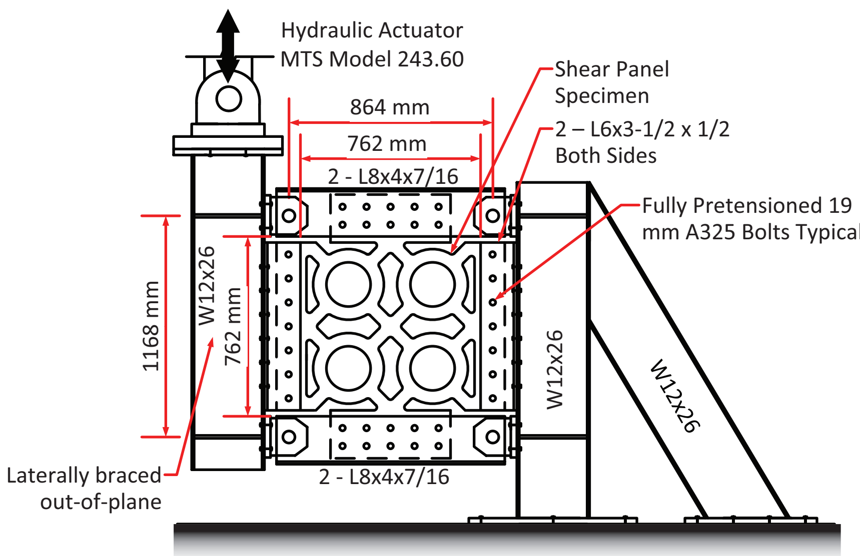


Fig. 3. Experimental setup for validation of RS-SPSW panels.

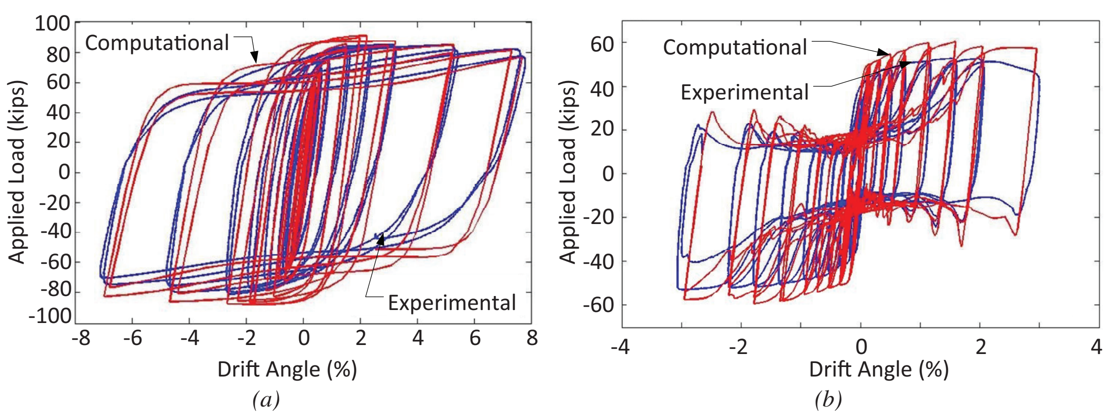


Fig. 4. Comparison of computational and experimental results: (a) RS-SPSW specimen; (b) SPSW specimen.

strength. Through a longer loading protocol an accumulated inelastic displacement at ultimate failure can be calculated which provides additional data on the fatigue failure limit state of the RS-SPSW panels" (Phillips and Eatherton, 2016).

Figure 6 shows the fifth-story RS-SPSW specimen (Figure 6a) and one of the first-story RS-SPSW specimens (Figure 6b). The fifth-story RS-SPSW specimen used 0.375-in. plate and had 12-in.-diameter, 1.75-in.-wide rings; 2.5-in.-wide links; and a vertical stiffener at the middle. This first-story RS-SPSW specimen used 0.375-in. plate as well but had 8.8-in.-diameter, 1.55-in.-wide rings; 2.2-in.-wide links; and two vertical stiffeners at approximately third points. The other two first-story RS-SPSW specimens used 12- and 8.8-in. ring diameters, 2.20- and 1.8-in. ring widths, 0.375- and 0.25-in. plate thicknesses, 3.0- and 2.2-in. link widths, and one and two vertical stiffeners, respectively. Among the four RS-SPSW specimens, the number of ring columns varied between four and six. Parameters were chosen to study RS-SPSWs designed for small loads, for the highest level of energy dissipation, with some pinching expected from global-buckling but high-energy dissipation, and with lateral torsional buckling of rings. The solid plate specimen was 0.063 in. thick and provided data on performance of an

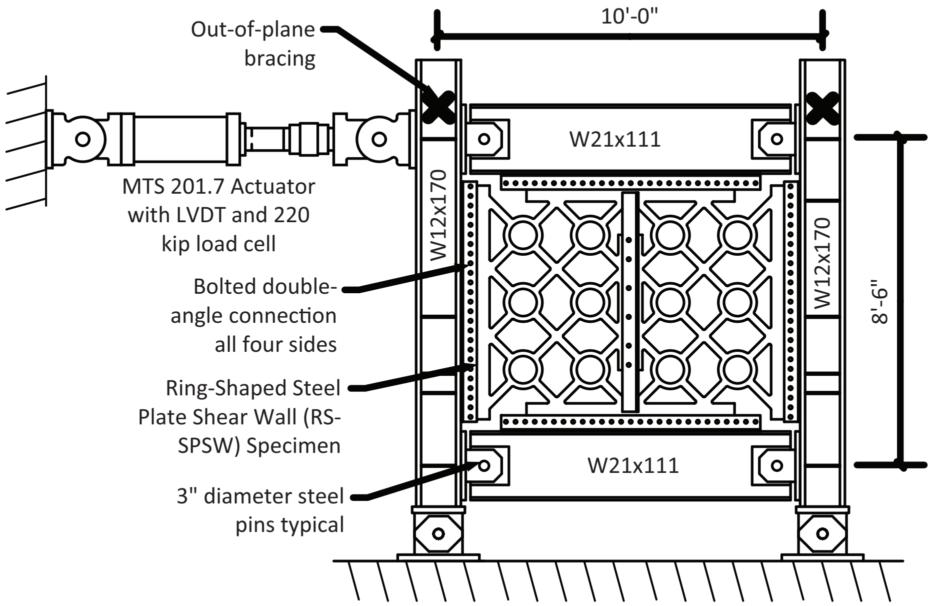
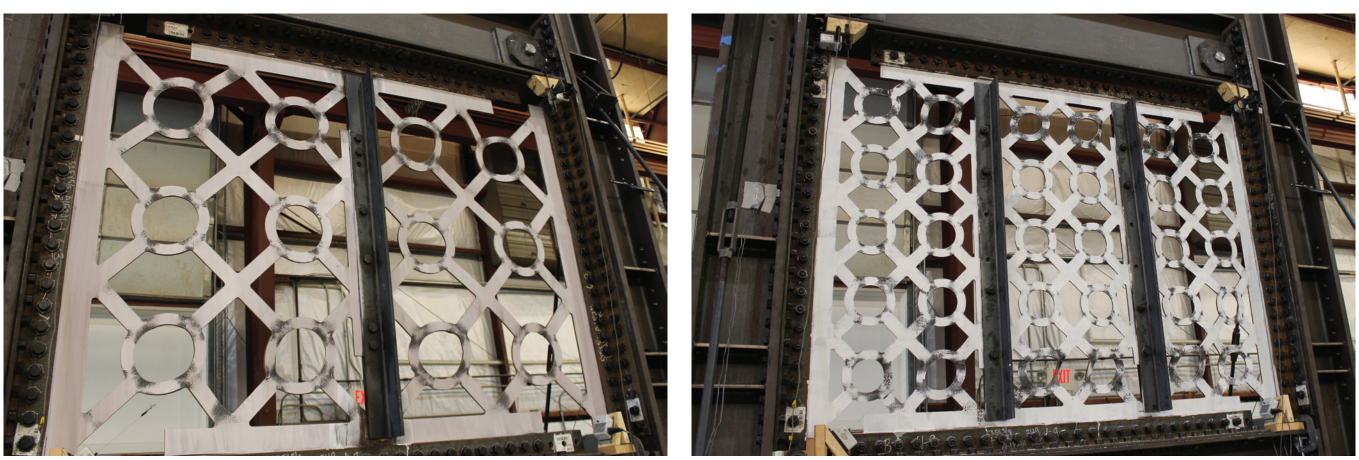


Fig. 5. Large-scale experimental test setup.



(a) (b)

Fig. 6. (a) A fifth-story RS-SPSW specimen; (b) a first-story RS-SPSW specimen.

SPSW infill plate with similar strength as the RS-SPSW, isolated from a moment frame. Additional details can be found in Phillips and Eatherton (2016) and Phillips (2016).

Experimental Results

The fifth-story specimen was used to demonstrate good energy dissipation from an RS-SPSW that is "tuned" for small-story shear demands. Figure 7 shows the story force–drift response for this specimen, which was tested through the 5% story-drift cycles and three additional cycles at 4%. Yielding and some global shear buckling, but no pinching in the hysteretic response, was detected by the 1% story-drift cycles. The global shear buckling was more apparent after the 2% story-drift cycles, as shown by the pinching in Figure 7 and visual observation of the panel (Figure 8). Fracture in rings began in the first 4% story-drift cycle.

Other RS-SPSW specimens exhibited similar responses initially, with yielding in 1% story-drift cycles, followed by global shear buckling, typically visibly evident by 1.5 to 2% story drift. Buckling caused controlled strength and stiffness degradation. The story drift at onset of buckling and the degree of strength degradation were correlated to panel properties such as ring geometry. In specimens with two vertical stiffeners, the out-of-plane displacements concentrated in the corner rings. Fractures in the rings commenced in the 2.5 or 3% story-drift cycles, leading to further drops in strength and stiffness. Specimens with fewer rings experienced more significant loss of shear strength with ring fractures.

Overall behavior correlated well with predicted limit states for the test specimens. For example, the intended purpose of the 0.25-in. plate RS-SPSW specimen was to isolate the lateral torsional buckling mode. Lateral torsional buckling appeared in this specimen by 1% story drift and was the likely reason this specimen did not reach its plastic strength. Meanwhile, all but the 0.25-in. specimen were able to resist the design story shear demand through at least 2% story drift. The fifth-story RS-SPSW specimen sustained its story shear capacity with slight degradation by the end of the test at 5% story drift. Others dropped below the design story shear demand after 2% story drift.

ANALYSIS AND DESIGN OF RING-SHAPED STEEL PLATE SHEAR WALLS

Design of RS-SPSWs

The experimental results, detailed finite element models, and the strength and stiffness equations from Maurya (2012) and Egorova (2013) were synthesized into design recommendations for RS-SPSWs. These recommendations were applied to the design of prototype buildings. Nonlinear response history analyses of the prototype buildings confirmed the viability of RS-SPSWs for seismic resistance of steel frame buildings.

Design Recommendations

The primary design objectives for RS-SPSW are (1) to provide sufficient story shear strength, (2) to prevent lateral torsional buckling of the rings, (3) to delay global shear buckling at least until plastic shear strength of the panel is reached, and (4) to prevent fracture of the rings during

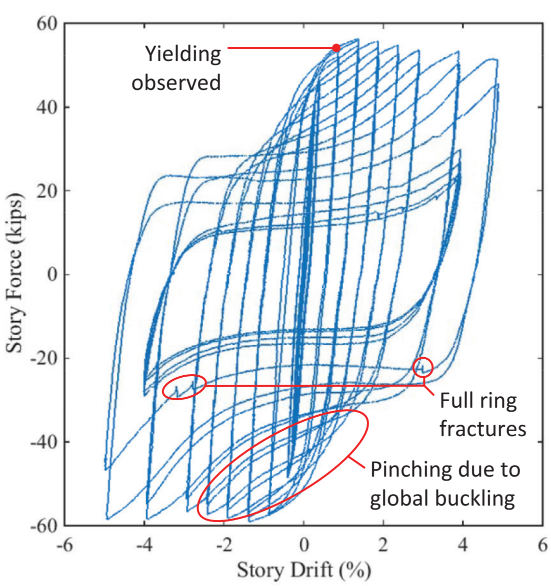


Fig. 7. Story force–drift response for fifth-story RS-SPSW specimen.

Fig. 8. Global shear buckling in fifth-story RS-SPSW specimen.

large earthquakes. Lateral torsional buckling was shown to reduce the peak shear strength, cause severe pinching in the hysteresis, and increase plastic strain demand, leading to fracture. Global shear buckling causes pinching in the hysteresis but does not reduce peak shear strength. Preventing global shear buckling at lower levels of drift will ensure greater energy dissipation, while global shear buckling at higher drift levels "may help prevent the system from having high over-strength due to strain hardening" (Phillips, 2016). Ductile design of RS-SPSWs relies on web plate yielding. Design recommendations were developed to ensure ductile behavior and included guidelines for panel layout, calculation of target shear strength, limits on slenderness ratios, and placement and detailing of intermediate stiffeners.

Prototype Buildings

Two three-story and two nine-story prototype buildings were designed for a location in Seattle, Washington. One building in each pair was designed with conventional SPSWs and the other with RS-SPSWs. Figures 9a and 9b show plan and elevation drawings of the three-story prototype buildings, which had a total of eight SPSWs or RS-SPSWs on the perimeter. Figure 9c shows the RS-SPSW used in the three-story building. An intermediate stiffener was used at the 1.125-in.-thick first-story infill panel. The second- and third-story RS-SPSW panels were 1.125 in. and 0.875 in. thick, respectively. The conventional SPSWs used 0.123-in.-thick web plates at the first and second stories and 0.063-in. web plates in the third story. The nine-story prototype building had a total of 12 SPSWs or RS-SPSWs on the perimeter. Intermediate stiffeners were used at all stories for RS-SPSW panels ranging from 0.75 in. to 1.25 in. thick. The conventional SPSW web plates ranged from 0.063 in. to 0.187 in. thick. Adapted from the SAC Steel Project buildings (Gupta and Krawinkler, 1999), the prototype buildings' 30-ft bays were shortened to 15 ft at shear wall locations. Additional details—including RS-SPSW web plate geometry, boundary element section sizes, and a cost comparison—can be found in Phillips (2016).

Nonlinear Response History Analysis

Reduced-Order Models

Computationally efficient reduced-order models were developed for nonlinear static and nonlinear response history analyses of the RS-SPSWs. The rings of the RS-SPSW panel were modeled as square with Hughes-Liu, fully integrated beam elements and a side length twice the centerline ring radius (Figure 10a). In order to capture the plastic shear strength of the ring with the equivalent square shape, a formulation was developed for the beam element width. Links were also modeled with Hughes-Liu, fully integrated beam elements. Through comparison with experimental results, it was determined that a beam element thickness equal to two-thirds the actual plate thickness for the ring elements and link elements provided the best representation of RS-SPSW behavior (Figure 10b). Meanwhile, the commonly used tension-only strip model, with strips in each direction for cyclic loading, was used for the SPSWs. Additional details of the reduced-order models can be found in Phillips (2016).

Nonlinear Response History Results

The prototype buildings were subjected to 22 randomly selected ground motions from the FEMA P-695 far field set (FEMA, 2009). The ground motions were scaled to 2% probability and 10% probability of exceedance in 50 years (i.e., maximum considered earthquake and design basis earthquake). The scaling was "such that the median

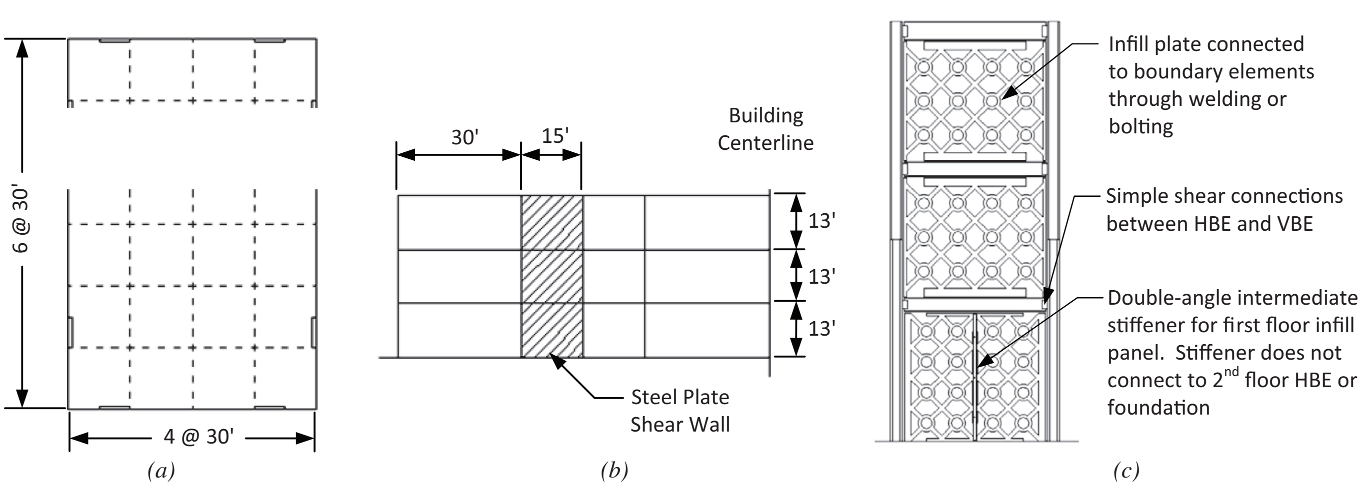


Fig. 9. Three-story prototype building: (a) plan; (b) elevation; (c) RS-SPSW.

spectrum matched the design spectrum at 0.5 seconds for the three-story prototype buildings and 1.8 seconds for the nine-story prototype buildings" (Phillips, 2016). The analyses utilized stiffness proportional damping of approximately 2% critical.

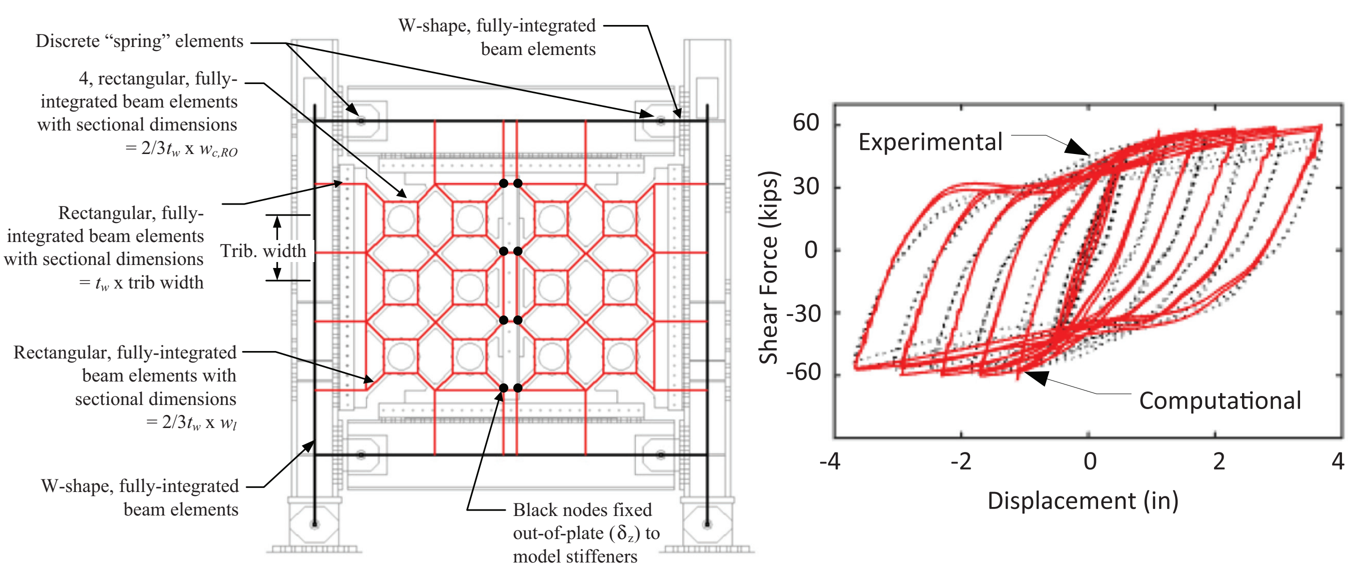
Observations from the nonlinear response history analyses included lower interstory drift values for the three-story RS-SPSW buildings as compared to the conventional SPSW buildings and more comparable roof drifts but less concentration of interstory drift demands for the nine-story RS-SPSW buildings. For example, for one Northridge ground motion, use of RS-SPSWs instead of conventional SPSWs in the three-story prototype resulted in a 43% decrease in peak roof drift and a 35% decrease in peak interstory drift for the 2% in 50-year hazard level. For the same ground motion but the 10% in 50-year hazard level, the use of RS-SPSWs resulted in a 57% decrease in peak roof drift and a 55% decrease in peak interstory drift. For the 2% in 50-year hazard level, replacing SPSWs in the nine-story prototype building with RS-SPSWs resulted in a 20% increase in peak roof drift but a 33% decrease in peak interstory drift. "The RS-SPSW building did a better job at spreading the total building drift out over multiple floors. Whereas, the conventional SPSW building concentrated story drift in the upper floors where the infill panel was thinner. This resulted in both prototype buildings having similar roof drift, but the RS-SPSW building having smaller interstory drifts" (Phillips, 2016). For the 10% in 50-year hazard, the use of RS-SPSWs in the nine-story prototype building resulted in a 13% decrease in peak roof drift and a 36% decrease in peak interstory drift. Overall, the greater energy dissipation observed for the RS-SPSW buildings was credited for the lower interstory drift values.

Residual drift values were similar for the SPSW and RS-SPSW prototype buildings for both hazard levels. Peak floor accelerations and local deformation demands in the RS-SPSW panels were also investigated. Additional details, including a statistical analysis of the nonlinear response history results, can be found in Phillips (2016).

SUMMARY

A new, ring-shaped steel plate shear wall (RS-SPSW) system has been developed and validated through large-scale experimental investigations, detailed finite element analyses, design of prototype buildings, and performance evaluation of those buildings. The RS-SPSW does not require a supplementary moment-resisting frame, exhibits good energy dissipation, and can result in lower interstory drift values than a comparable, conventional SPSW system. Design provisions for the RS-SPSW ensure a stable plastic mechanism in the rings without lateral torsional buckling or fracture and delay of shear global buckling until the plastic strength of the panel has been achieved.

In future work, different cutout geometries and possible topology optimization will be considered. Additional large-scale testing will be conducted on steel plates with cutouts, with an expansion of potential applications to those for any metallic panel subjected to shear deformations. One research objective is to use cutouts in steel plates to create local yielding mechanisms that can dissipate energy from extreme loads.



(a) (b)

Fig. 10. (a) Reduced-order model for RS-SPSW with intermediate stiffener; (b) comparison of computational and experimental results.

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