Seismic Design and Analysis of Self-Centering Steel Plate Shear Walls

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ABSTRACT:

An innovative Self-Centering Steel Plate Shear Wall (SC-SPSW) system is proposed. It relies on post-tensioned (PT) beam-to-column connections that allow beams to rock about their flanges and provide system re-centering capabilities. A design procedure for the SC-SPSW system, developed based on a performance based design (PBD) approach, is presented, followed by analytical results for a prototype SC-SPSW building designed using this PBD approach, and subjected to a suite of ground motions simulating three different seismic hazard levels. The results of the nonlinear response history analyses show the proposed SC-SPSW design procedure to adequately achieve the desired enhanced performance objectives. Concepts of capacity design principle are integrated in the above approach, to prevent in-span plastic hinges of the beam considering reduced moment capacity due to the presence of axial and shear forces and to ensure that PT reinforcement remain elastic, among other things. To facilitate understanding of the behavior and design of an SC-SPSW system, the moment, shear and axial force distribution along the length of a boundary beam are established based on first principles. Closed form formulations describing the moment, shear and axial force beam diagrams are developed based on component capacity design approach and are used in the performance-based system design approach.

Keywords: Self-centering; Rocking connection; Steel plate; Shear wall; Earthquake engineering

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INTRODUCTION

Steel plate shear wall (SPSW) systems are frames having steel plates (a.k.a. webs) connected between their beams and columns (Sabelli and Bruneau, 2007). During severe earthquakes, the unstiffened plates of ductile SPSWs buckle in shear and yield by developing a diagonal tension field, together with plastic hinging of the beams at their ends. While SPSW systems are desirable for their significant stiffness, strength, and energy dissipation, strategies to eliminate residual drifts and to localize structural damage only in easily replaceable structural elements are desirable in SPSWs (as in other systems). In moment resisting frames, use of post-tension (PT) rocking moment connections was analytically and experimentally investigated to provide frame self-centering capability and to limit hysteretic damage to replaceable energy dissipating elements during earthquakes (e.g., Ricles *et al.* 2001, Christopoulos *et al.* 2002; Garlock *et al.* 2005; Rojas *et al.* 2005; to name a few). Building on this idea, this paper investigates the potential of achieving Self-Centering Steel Plate Shear Walls (SC-SPSW) by using similar post-tensioned rocking beam connections. In this proposed system, the SC-SPSW web plate is the replaceable energy dissipation element, and beam-plastic hinging is eliminated. The system combines the advantages of high lateral stiffness, a substantial energy dissipation capacity, and self-centering capability, at the expense of additional challenges to understanding the flow of forces within the structure compared to conventional SPSW (themselves, more complex than moment frames).

The objective of this paper is to provide insights on SC-SPSW's beam and system fundamental behavior, through free-body-diagrams of individual beams and non-linear response history dynamic analyses of multistory frames. This is done by developing equations for the moment, shear and axial force diagrams along the HBE from a capacity design approach based on yielding of the SPSW web plate, showing the respective contribution of each factor to the total demand. These closed-form solutions are then integrated into a design procedure proposed to aid in the selection of PT reinforcement area and HBE sizing to prevent in-span plastic hinging of the beam, to keep PT reinforcement elastic at a specified target drift, and to select an adequate initial PT force. Lastly, a performance-based design procedure for the SC-SPSW system is proposed that incorporates this fundamental behavior knowledge and component capacity design method with response-based performance objectives. The seismic response of a prototype SC-SPSW designed using the proposed design procedure was evaluated using non-linear response history dynamic analyses.

BASIC PRINCIPLES OF SELF-CENTERING SPSW SYSTEMS

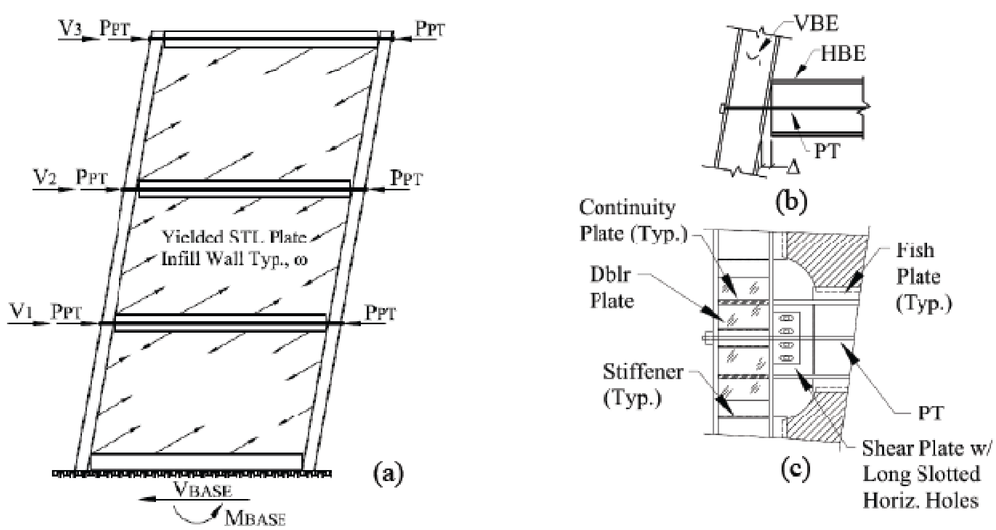

Fig. 1. SC-SPSW: (a) Yield Mechanism; (b) rocking joint; (c) rocking detail

Fig. 1a shows the free-body-diagram (FBD) of a SC-SPSW frame, where HBE is the horizontal boundary element, VBE is the vertical boundary element, P_{PT} is the post-tension (PT) axial compression force applied to the HBE, V_i is the externally applied lateral forces at story i due to applied seismic forces, and ω is the diagonal tension yield force per unit length developed by the steel web plates (Sabelli and Bruneau 2007, Berman and Bruneau 2008).

A SC-SPSW differs from a conventional SPSW in that HBE-to-VBE rigid moment connections in a conventional SPSW are replaced by PT rocking moment connections. This allows a joint gap opening to form between the VBE and HBE interface about a rocking point (i.e. connection decompression), leading to a PT elongation, this being the self-centering mechanism (shown schematically in Fig. 1b). One possible rocking detail configuration is shown in Fig. 1c. The PT boundary frame is designed to essentially remain elastic and hysteretic energy dissipation is intended to be provided by the web plate only. The total hysteretic response of a SC-SPSW is provided by the combined elastic response of the PT boundary frame and the inelastic energy dissipation of the web plate.

FREE-BODY-FORCE DIAGARAM

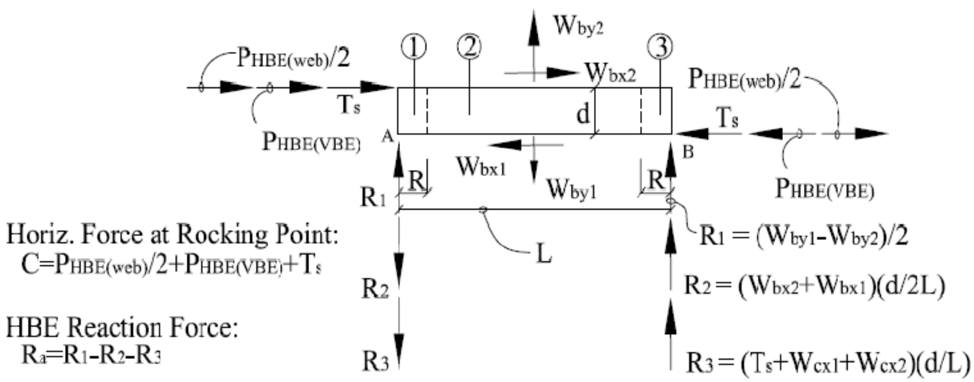

Fig. 2. Force Resultant FBD of HBE

Fig. 2 shows the FBD of an HBE and VBE element located at an intermediate floor level of a SC-SPSW frame once the web plate has fully yielded and the connection has decompressed for a rightward drift, where W_{bx1}, W_{bx2} and W_{by1}, W_{by2} are respectively the horizontal and vertical force resultants along the length of the HBE, W_{cx1},W_{cx2} and W_{cy1}, W_{cy2} are respectively the horizontal and vertical force resultants along the height of the VBE, subscripts 1 and 2 respectively denote the level below and above the HBE, L is the HBE span length, d is the HBE depth and R is the length of the web plate corner cut-out at each end of the HBE provided to accommodate the HBE-to-VBE joint rocking connection detailing and to reduce the potential for corner tear out of the web plate due to high localized web plate strain effects during opening of the rocking joint connection.

The horizontal forces at the rocking point $P_{HBE(web)}$, $P_{HBE(VBE)}$ and T_s shown on Fig. 2 are the reactions due to the story shear contribution due to the web plate, horizontal component of web plate yield forces acting along the VBE and PT tension force calculated as follows:

$$P_{HBE(web)} = W_{bx1} - W_{bx2} = \frac{1}{2}(t_1 - t_2)F_{yp}(L - 2R)\sin(2\alpha) \quad (1)$$

$$P_{HBE(VBE)} = W_{cx1} + W_{cx2} = (\omega_{cx1} + \omega_{cx2})\left(\frac{h}{2} - \frac{d}{2} - R\right) \quad (2)$$

$$T_s = T_o + \frac{A_{PT} E_{PT}}{L_{PT}} \left(\Delta_{drift} - \Delta_{loss} \right) \tag{3}$$

where t_1 and t_2 are the web plate thicknesses below and above the HBE respectively, α is the angle of inclination of the diagonal tension field from the vertical axis, F_{yp} is the web plate yield stress, h is the story height, T_o is the initial PT force applied at the time of construction of the SPSW system, A_{PT}, E_{PT} and L_{PT} are the sectional area, modulus of elasticity and length of PT provided. Additionally, Δ_{drift} is the total drift induced elongation of the PT when the SC-SPSW joint connections open due to rocking action at the HBE-to-VBE joints during building drift and Δ_{loss} is the PT relaxation due to HBE axial shortening calculated as:

$$\Delta_{drift} = 2\left(\phi_{drift} \frac{d}{2} \right) = \phi_{drift} d \tag{4}$$

$$\Delta_{loss} = \frac{P_{HBE(VBE)}}{k_b + k_{PT}} + \left(\frac{k_{PT}}{k_b + k_{PT}} \right) \Delta_{drift} \tag{5}$$

where ϕ_{drift} is the relative joint rotation, k_b and k_{PT} are the axial stiffness (i.e. AE/L) of the HBE and PT elements respectively. Note that for use with multistory frames, the additional lateral story shear force at each HBE level due to multi-story PT frame stiffness (i.e. VBE's fighting beam growth) would have to be considered for preciseness in calculating the HBE demands (Kim and Christopoulos 2009).

Development of HBE Moments

The moment distribution to be used in the design of an HBE of a SC-SPSW can be determined from the FBD of Fig. 2 by taking moment equilibrium at three locations identified where x is any distance from point A leading to:

$$M_1 = \frac{Cd}{2} + R_a x \tag{6}$$

$$M_2 = R_a x + C\frac{d}{2} + \left(W_{by2} - W_{by1}\right)\left(\frac{x-R}{2}\right) + \left(W_{bx2} + W_{bx2}\right)\frac{d}{2} \tag{7}$$

$$M_3 = R_a x + C\frac{d}{2} + \left(W_{by2} - W_{by1}\right)\left(x - \frac{L}{2}\right) + \left(W_{bx2} + W_{bx1}\right)\frac{d}{2} \tag{8}$$

It is instructive to plot the moment distribution along the length of an HBE for a SC-SPSW frame. For illustrative purposes only, to avoid abstract complexities in keeping the problem parametric, all results here are presented in terms of an example. This example considers a W18x HBE with a clear span of 5.74 m (226 inches) and a story height of 3.89 m (153 inches) of a single bay frame. The SPSW web plates consists of 14 Gauge (1.83 mm) and 18 Gauge (1.77 mm) thicknesses below and above the HBE respectively with a corner cut-out radius of 254 mm (10 inches). Area of PT steel was arbitrarily chosen to produce a maximum moment of 60% of the full HBE plastic moment capacity at the end span of the HBE for an arbitrary 2% lateral drift, with a T_o of 30% of the assumed yield strength of the PT. A yield stress of 207 MPa (30 ksi) was assumed for the web plates, an ultimate yield stress of 1034 MPa (150 ksi) was assumed for the PT reinforcement and ASTM A572 (F_y = 345 MPa) steel was used for the boundary frame. After providing substitution of all applicable defined parameters into Eqns. (6) to (8), the resulting moment equations will each yield five distinct components which are separated and plotted on Fig. 3a with the composite moment diagram plotted in Fig. 3b.

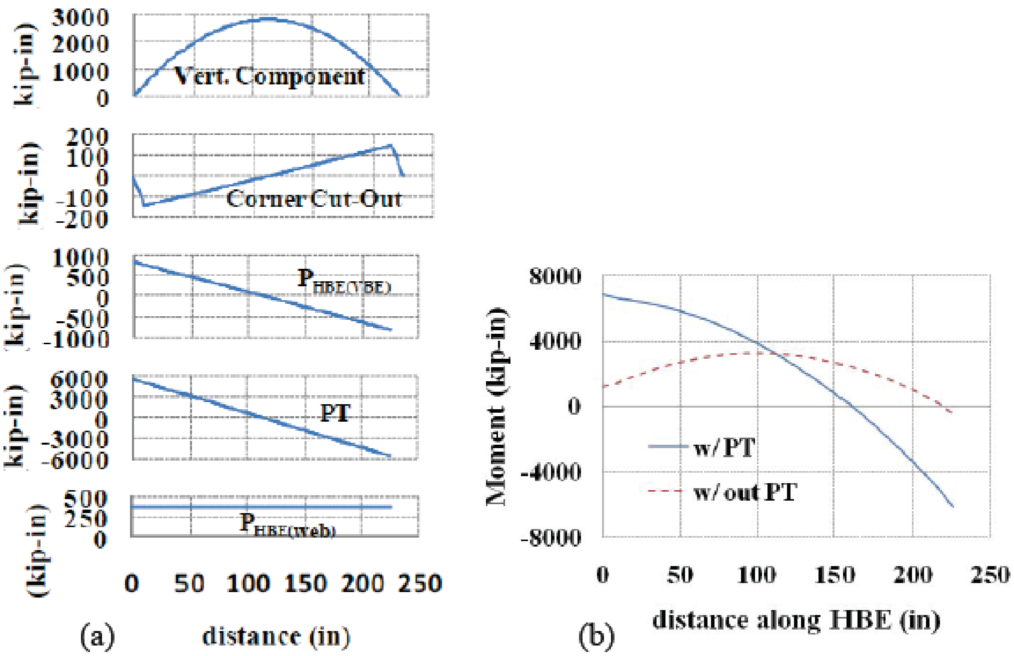

Fig. 3. (a) HBE Moment Components (b) Composite Moment Diagram

It is observed from the moment diagrams that the horizontal reactions at the rocking point will always generate double curvature moments at the ends of the HBE. Additionally, the moment diagrams are of identical shape, only the moment magnitudes are different. The contribution to the HBE moment from the VBE horizontal reaction at the rocking point, will always be additive to the HBE moment produced by the PT component. Without the PT, it can be seen that the maximum moment occurs close to the HBE mid-span, which is not desirable for reasons described later. Therefore, in addition to acting as a self-centering mechanism, the PT forces can be designed to shift the point of maximum moment towards the ends of the HBE (although at the cost of increased HBE moment demand).

Development of HBE Shears and Axial Forces

The shear distribution to be used in the design of an HBE incorporating self-centering components can be determined using the same free-body-diagrams developed earlier for the HBE moments. As a result, for the same 3 zones defined previously in terms of ω defined earlier follows:

$$V_1 = R_a = \frac{(\omega_{by1} - \omega_{by2})(L - 2R)}{2} - \frac{(\omega_{bx2} + \omega_{bx1})(L - 2R)}{2L} d - (\omega_{cx1} + \omega_{cx2})\left(\frac{h}{2} + \frac{d}{2} - R\right)\left(\frac{d}{L}\right) - T_s\left(\frac{d}{L}\right) \quad (9)$$

$$V_2 = R_a - (\omega_{by1} - \omega_{by2})(x - R) \quad (10)$$

$$V_3 = R_a - (\omega_{by1} - \omega_{by2})(L - 2R) \quad (11)$$

Similarly, for axial forces over each of the three zones:

$$P_1 = C = \frac{(\omega_{bx1} - \omega_{bx2})(L - 2R)}{2} + (\omega_{cx1} + \omega_{cx1})\left(\frac{h}{2} - \frac{d}{2} - R\right) + T_s \quad (12)$$

$$P_2 = C - (\omega_{bx1} - \omega_{bx2})(x - R) \quad (13)$$

$$P_3 = C - (\omega_{bx1} - \omega_{bx2})(L - 2R) \quad (14)$$

DESIGN CONSIDERATION OF SELF-CENTERING CONNECTION

The above understanding of behavior and closed-form solutions also serve to aid in the selection of PT reinforcement area and HBE sizing to force the maximum HBE moment to be at its ends, to prevent in-span plastic hinging and account for moment-axial-shear interaction in the HBE, to keep PT reinforcement elastic, to select an adequate initial PT force, and to account for PT losses due to HBE axial shortening.

Knowing that PT force influences HBE moment distribution (Fig. 3), it is proposed here to design for the smallest PT force needed to shift the point of maximum moment to occur at the HBE ends. One should recognize that the moment distribution including a PT force contribution will vary depending on the magnitude of drift. For SC-SPSW systems to have a comparable performance with conventional SPSWs (e.g.,Park *et al.* 2007; Qu and Bruneau, 2009), it is proposed here that the PT reinforcement be designed to remain elastic at least up to 4% drift to maintain its self-centering capability. In addition to the above requirement, it is proposed that SC-SPSW be designed such that the total PT force that shifts the point of maximum moment to the HBE ends is reached at the target drift. This is proposed to avoid the formation of plastic hinges along the HBE span which can detrimentally impact the behavior of SPSW (Purba and Bruneau 2010).

Building on the system behavior and performance objectives presented above, the following design procedure for SC-SPSW HBE-to-VBE connection is proposed.

1. Select initial boundary elements sizes and web plate thickness (of many possible approaches, this could be done by designing a conventional SPSW, although other approaches are acceptable too).
2. Design the self-centering connection with the least PT forces that would result in the maximum moment occurring at the HBE ends at the target drift.
3. Select post-tension to ensure that the PT reinforcement remain elastic at least up to 4% drift.
4. Select the initial PT force applied to the self-centering connection.
5. Select the least cross sectional areas of PT reinforcement that satisfy the previous conditions.
6. Consider the effect of PT on reducing the HBE plastic moment as well as the effect of PT losses due to axial shortening to assess the adequacy of the HBE.
7. Iterate as needed to reduce the HBE size, ensuring that the HBE moment capacity reduced due to axial and shear forces remains adequate.

PERFORMANCE-BASED DESIGN OF SYSTEM

The fundamental system behavior and HBE-to-VBE design procedure previously described can be incorporated into a design procedure for the overall SC-SPSW system. For design of the system, a performance-based design approach has been developed to achieve the following structural performance objectives:

1. *No connection decompression under wind or gravity loading.* The decompression moment, M_d, of the PT connection (M_d=T_o $d/2$, where d and T_o were defined previously) should be designed to be greater than the HBE end-moments induced under wind or gravity loading.
2. *System recenters and no repair required under frequent earthquake demands* (i.e. the demand representing the 50% probability of exceedence in 50 year earthquake [50/50]). Recentering is assessed using a residual drift limit of 0.2%, corresponding to out-of-plumb limits in construction. The no repair limit state is assessed using a peak story drift limit of 0.5%, the median drift at which web plate repair is required in conventional SPSW experimental studies (Baldvins et al. 2010).

3. *System recenters and only web plate repair required under design earthquake demands* (i.e. approximated here as the demand representing the 10% probability of exceedence in 50 year earthquake [10/50]). The web plate may have significant yielding; however, the boundary frame and PT elements should remain elastic and the system should recenter. The damaged web plate can be replaced relatively quickly and simply, resulting in a more rapid return to occupancy following an earthquake (Qu et al. 2008). The story drift at this hazard level should be less than the 2% code-based drift limit for design level earthquakes.
4. *Collapse prevention for the maximum credible earthquake* (i.e. the demand representing the 2% probability of exceedence in 50 year earthquake [2/50]). Residual drifts and minor frame yielding may occur; however, soft story mechanisms and significant PT and frame yielding should be avoided. A target drift limit of this performance objective was assumed to be 4%, based on engineering judgment and the drift at which significant strength loss was observed in conventional SPSW experimental studies (Baldvins et al. 2010).

The following performance-based design procedure, as shown in Fig. 4, was developed to meet the previously described performance objectives.

1. Make initial assumptions about SC-SPSW geometry and number of walls in the building.
2. Determine the seismic loads and lateral load distribution. Here a modified load distribution as developed by Chao et al. (2007) was used to account for higher-mode effects, as SC-SPSWs and other self-centering PT systems (Garlock 2003) designed using typical code-based load distributions have exhibited larger drift demands and damage in the upper stories due to these higher-mode effects.
3. Design the web plates to remain elastic under 50% in 50 year loads.
4. Design the PT connection to have sufficient stiffness to ensure recentering under the influence of P-Delta effects.
5. Capacity design the PT and HBE to remain elastic and prevent in-span hinging at 4% drift according to the procedure previously described.
6. Capacity design the VBE to remain elastic up to 5% drift (Berman and Bruneau 2008).
7. Check that design meets code-based drift limits and that the required amount of PT can fit reasonably within the depth of the HBE.
8. Iterate as needed until a final design is converged upon.

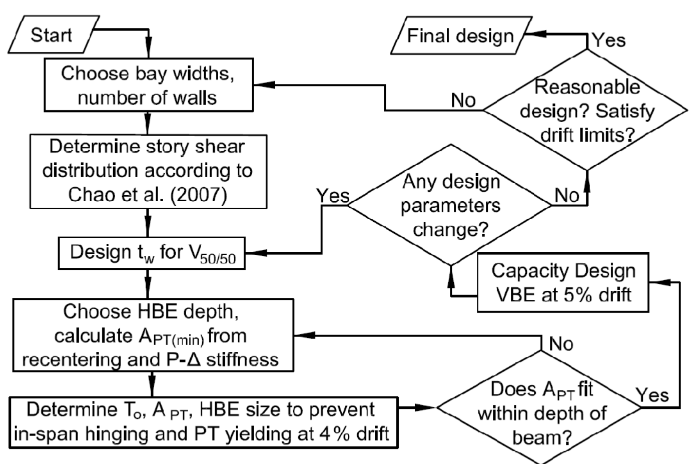

Fig. 4: SC-SPSW performance-based design procedure

A prototype 3-story SC-SPSW was designed according the proposed system design procedure. The 3-story building has story heights of 3.96m (13ft) with plan dimensions, loading, and seismic masses equivalent to those of the 3-story SAC building. The building was assumed to be on soil of Site Class D with spectral response parameters compatible with the ground motions developed for the Los Angeles, California location in the SAC project (Gupta and Krawinkler 1999). The prototype building was designed to have six 4.57m (15ft) wide bays of SC-SPSW in each direction of the building. The web plates were designed using available thicknesses of ASTM A36 steel, and the PT was designed using ½" diameter, Grade 270, seven-wire strand with a cross sectional area of 98.7mm^2 (0.153in^2) and a yield stress of 1689MPa (245ksi). Table 1shows the resulting prototype SC-SPSW design.

Table 1: 3-story prototype SC-SPSW design parameters

Story	Web Plate Thickness (mm)	HBE size	VBE size	# of PT strands	Initial PT force, T_o (kN)
1	7.94	W24x229	W14x605	16	231
2	6.35	W30x211	W14x605	12	503
3	4.18	W14x302	W14x605	32	467

NONLINEAR RESPONSE HISTORY ANALYSES

To assess the adequacy of the proposed performance-based design procedure, a numerical model of the prototype SC-SPSW was subjected to a suite of ground motions representing the seismic hazard levels considered in the design, and the response of the structure was evaluated. The numerical model was developed in OpenSees (Mazzoni et al. 2006). The steel web plates were modeled using the tension strip method consisting of a series of tension-only truss elements oriented in the direction of the diagonal tension-field. The plate material had a yield stress of R_yF_y, where F_y was 248MPa (36ksi) and R_y was 1.3 to account for the ratio of expected to nominal yield strength of A36 plate steel (AISC 2005a).

A PT connection model was developed to allow the HBE to rock about its flanges and cause PT elongation and to allow for shear load transfer without inhibiting the rocking action. The PT connection model (Berman et al. 2010) consisted of (1) stiff horizontal compression-only springs located at the HBE flanges to allow for decompression of the flanges at gap opening, (2) compression-only diagonal springs to transfer shear forces while allowing for gap opening and rotation, and (3) rigid link elements connecting the end of the HBE to the connection springs to ensure that the end of the HBE in contact with the VBE remains planar and is orthogonal to the HBE's longitudinal axis at that point. The tension only truss elements used for the PT elements were connected to the VBEs and were placed within the depth of the HBE.

The boundary frame elements were modeled using nonlinear beam-column elements with fiber cross-sections. The fibers were modeled using the Giuffre-Menegotto-Pinto material model, with an effective yield stress of 345MPa (50ksi), a rounded yield surface to help with numerical convergence, and 2% strain hardening. P-Delta columns were placed symmetrically on each side of the wall to simulate the gravity loads that would contribute to P-Delta effects on the wall. Seismic masses were lumped at the HBE-to-VBE connections, and damping was modeled using Rayleigh damping of 2% in the first and third modes. The earthquake ground motion records used in the nonlinear dynamic analyses were those developed for the SAC project for the Los Angeles site, with 20 ground motions for each seismic hazard level considered in the design procedure (50/50, 10/50, and 2/50).

The response parameters that were evaluated during the dynamic analyses were the peak story drift, $\theta_{s,max}$; the peak residual story drift, $\theta_{resid,max}$; and the HBE and VBE damage value, D_{HBE} and D_{VBE}, respectively, as evaluated by the interaction equation per Eqn. H1-1 (AISC 2005b), where a damage value of one indicates combined axial-flexural hinging. The median and 84th percentile responses for these parameters are shown for each seismic hazard level in **Table** 2.

Table 2: Response parameter values for prototype SC-SPSW

Response Parameter	50% in 50 yr		10% in 50 yr		2% in 50 yr	
	Median	84th percentile	Median	84th percentile	Median	84th percentile
$\theta_{s,max}$(%)	0.40	0.61	0.54	0.84	1.16	1.86
$\theta_{resid,max}$(%)	0.002	0.002	0.002	0.002	0.002	0.002
D_{HBE}	0.52	0.63	0.59	0.71	0.74	0.84
D_{VBE}	0.37	0.53	0.43	0.56	0.66	0.74

The results in **Table** 2 show that the SC-SPSW was able to recenter ($\theta_{resid,max}$ < 0.02%) at the 50% and 10% in 50 year hazard levels (Performance Obj. 2 and 3), and had no frame yielding (D_{HBE} and D_{VBE} < 1.0) at the 2% in 50 year hazard level (Performance Obj. 4). The prototype SC-SPSW had peak story drifts less than the 2% code-based drift limits at the 10% in 50 year hazard level (Performance Obj. 3), and the model had story drifts less than the 0.5% no web plate repair drift limit under the 50% in 50 year earthquake at the median level (Performance Obj. 2). Also, as the peak story drifts are well below the 4% target drift assumed in the PT design at the 2% in 50 year hazard level, there were no instances of PT yielding in any of the models at this hazard level (Performance Obj. 4). From all of these results, it is shown that the proposed performance-based system design procedure is capable of producing SC-SPSWs that can meet the intended performance objectives.

CONCLUSIONS

Fundamental behavior of a SC-SPSW was presented, in terms of closed form equations for moment, shear, and axial forces along the HBE. These formulations and development of the free-body-diagrams presented, not only provide insight on the behavior of a SC-SPSW system, but also provide a means to inform design. On the basis of that knowledge, a proposed HBE and PT connection design procedure based on a capacity design approach was formulated. Additionally, a SC-SPSW system design procedure was presented based on proposed structural performance targets and capacity design of the HBE and PT connection components using the previously described methods. The results of a series of nonlinear response history analyses on a prototype SC-SPSW designed according to the proposed performance-based method indicate that the system is capable of achieving the proposed performance objectives at three different seismic hazard levels. The findings presented indicate that SC-SPSW systems could be a viable alternative to traditional lateral force resisting systems. Future research is needed to further validate this system, including experimental work to investigate its behavior and self-centering characteristics.

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