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SEISMIC PERFORMANCE OF STEEL PLATE SHEAR WALLS CONSIDERING VARIOUS DESIGN APPROACHES

Ronny Purba[1] and Michel Bruneau[2]

ABSTRACT

Research was conducted to investigate the seismic performance of steel plate shear walls (SPSWs) designed by different philosophies. First, analytical study was conducted to investigate impact of formation of in-span plastic hinges on horizontal boundary elements (HBEs) on the seismic behavior of SPSWs. The development of in-span plastic hinges has significant consequences on the behavior of SPSWs, namely: lower lateral strength due to partial yielding of the infill plates and significant plastic incremental deformations on the HBEs than can reach total HBE rotations greatly exceeding 0.03 radians when the structure was pushed cyclically up to a maximum lateral drift of 3%. Second, collapse assessment of steel plate shear walls with various structural configurations (e.g., panel aspect ratio, seismic weight intensity, and number of story) was conducted to investigate impact of sharing of story shear forces between the boundary frames and infill plates on the performance of SPSWs. The FEMA P695 methodology was used for this purpose. SPSWs designed with the current seismic performance factors specified in the ASCE 7-10 (i.e., R, Ω_o, and C_d factors are 7, 2, and 6, respectively) and neglecting the contribution of their boundary moment resisting frames to resist story shear forces met the FEMA P695 performance criterion while that was not the case for SPSWs designed considering the sharing of story shear forces between the boundary frame and infill plates. Adjusted seismic performance factors (i.e., R, Ω_o, and C_d factors are 5, 1, and 5, respectively) were required for the latter SPSWs to rigorously meet the FEMA P695 performance criteria.

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Research was conducted to investigate the seismic performance of steel plate shear walls (SPSWs) designed by different philosophies. First, analytical study was conducted to investigate impact of formation of in-span plastic hinges on horizontal boundary elements (HBEs) on the seismic behavior of SPSWs. The development of in-span plastic hinges has significant consequences on the behavior of SPSWs, namely: lower lateral strength due to partial yielding of the infill plates and significant plastic incremental deformations on the HBEs than can reach total HBE rotations greatly exceeding 0.03 radians when the structure was pushed cyclically up to a maximum lateral drift of 3%. Second, collapse assessment of steel plate shear walls with various structural configurations (e.g., panel aspect ratio, seismic weight intensity, and number of story) was conducted to investigate impact of sharing of story shear forces between the boundary frames and infill plates on the performance of SPSWs. The FEMA P695 methodology was used for this purpose. SPSWs designed with the current seismic performance factors specified in the ASCE 7-10 (i.e., R, Ω_o, and C_d factors are 7, 2, and 6, respectively) and neglecting the contribution of their boundary moment resisting frames to resist story shear forces met the FEMA P695 performance criterion while that was not the case for SPSWs designed considering the sharing of story shear forces between the boundary frame and infill plates. Adjusted seismic performance factors (i.e., R, Ω_o, and C_d factors are 5, 1, and 5, respectively) were required for the latter SPSWs to rigorously meet the FEMA P695 performance criteria.

Introduction

In seismic design applications, the primary energy dissipating elements of steel plate shear walls (SPSWs) resisting lateral loads are their unstiffened infill plates (webs), which buckle in shear and form a series of diagonal tension field action (TFA). In a capacity design perspective, the tension force from the infill plates must be resisted by the surrounding horizontal and vertical boundary elements (HBEs and VBEs). The AISC Seismic Provisions for Structural Steel Buildings (AISC 2010) requires that HBEs and VBEs shall be designed to remain essentially elastic under the maximum tension forces from the yielded infill plates, with the exception of plastic hinging at the ends of HBEs. Implicitly, this indicates that in-span plastic hinges should be avoided.

However, the provision does not specify an analysis procedure to guarantee that this intent is met (although the commentary provides some guidance that could be used for this purpose). As a result, structural engineers might not anticipate that their designs may lead to in-

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span HBE plastic hinges (unless these analyses are complemented by the use of nonlinear analysis programs to predict the plastic mechanism of structures). In parallel, some structural engineers fully recognize the potential for in-span hinging to develop, but question the merit of limiting the location of plastic hinges to only occur at the ends of HBEs because, in general, this design requirement results in a relatively substantial size of boundary elements. Thus, to achieve more economical designs, structural engineers may try to minimize overstrength by allowing plastic hinges to occur along HBE span as this leads to relatively smaller boundary elements. Whether or not in-span hinging is acceptable has been a contentious issue, particularly in the absence of factual data to support either position.

When rigid connections are specified between HBEs and VBEs, as well as between VBEs and the ground (as specified in many applications of SPSWs), SPSWs also benefit from the boundary frame's moment resisting action to resist the applied lateral loads. Nonetheless, it is specified in the current Canadian Standard (i.e., CSA 2009) for the design of steel structures that infill plates of SPSWs must be designed to resist the entire lateral loads, without considering the possible contribution from the surrounding boundary moment resisting frame. Such a statement is not explicitly included in the American Seismic Provisions (i.e., AISC 2010), but one possible interpretation of the AISC design specifications could lead to the same design approach.

As reported in past experiments, this overstrength in conventional SPSWs can be quite significant. For example, Driver *et al*. (1997) reported that boundary frame moment resisting action contributed about 25% of the global plastic strength of their four-story SPSW specimen. Qu and Bruneau (2009) demonstrated that boundary frame moment resisting action can contribute up to 50% of the total strength of a SPSW with aspect ratio of 2.0 when its boundary elements are designed per capacity design principles. This provides a significant incentive to reduce overstrength by explicitly considering boundary frame moment resisting action as contributing to the SPSW overall lateral strength. However, the consequences of reducing this overstrength are unknown, and opinions vary as to whether this should be permitted.

This paper presents the investigation on the seismic performance of SPSWs considering various design approaches to address the above two concerns, namely: in-span HBE plastic hinging, and sharing of lateral loads between the boundary frame and infill plates. To investigate the first concern, one must first determine whether in-span HBE hinging, when it happens, can impact in any way to the seismic performance of SPSWs – irrespectively of whether it develops in a SPSW intentionally or as a result of unintended design consequences. To address the second concern, this paper investigates the seismic performance of SPSWs having infill plates designed per two different philosophies, to sustain different percentages of the applied lateral loads.

Impact of HBE Design on Seismic Behavior of Steel Plate Shear Walls

As a case study to investigate the possible significance of in-span HBE plastic hinges, a three-story single-bay SPSW was selected. Bay width and typical story height were arbitrarily chosen equal to 20 and 10 ft, respectively. It was also assumed that the structure is located on Class D soil in downtown San Francisco, California and designed for an office building. Total weight of the structure W_t was 1085 kips and the total base shear V resisted by the structure was 176 kips. Two design procedures were applied to design the boundary elements: (1) the Indirect Capacity Design approach (AISC 2010) and (2) the capacity design approach which combines the procedure proposed by Vian and Bruneau (2005) for HBEs and that proposed by Berman and Bruneau (2008) for VBEs. The resulting SPSWs obtained by the two different design procedures are denoted as SPSW-ID and SPSW-CD, respectively. Detail of member sizes and strip models in SAP2000 used for this study can be found in Purba and Bruneau (2010).

Nonlinear Static Analysis (Pushover Analysis)

Obtained from the monotonic pushover results at 4% drift, the base shears were 311 and 477 kips for SPSW-ID and SPSW-CD, respectively. For comparison, their respective theoretical values were 351 and 488 kips, obtained using the plastic analysis equations for uniform plastic sway mechanism (Berman and Bruneau 2003). While the theoretical and calculated base shears for SPSW-CD were less than 3% differences, those for SPSW-ID were about 13% differences. This significant discrepancy on SPSW-ID was attributed to the fact that in-span plastic hinges developed on its HBEs thus SPSW-ID did not follow the assumed uniform plastic sway mechanism (also known as 'panel mechanism') but rather consists of a 'sway' and 'beam' combined mechanism. An equation to calculate the theoretical base shear strength of SPSW having in-span plastic hinges considering their actual plastic mechanism was derived in Purba and Bruneau (2010). Using that equation, a theoretical base shear for SPSW-ID was 304 kips, which agreed within 2.2% with the aforementioned 311 kips result from the SAP2000 analysis.

To investigate whether plastic hinging along an HBE span could lead to progressively increasing deformations in the HBEs of both SPSW-ID and SPSW-CD, and whether it may affect structural performance, cyclic pushover analysis was conducted with a progressively increasing cyclic displacement history of up to 3% drift (in increment of 0.5%). Fig. 1a shows the plastic hinge and strip yielding distributions on SPSW-ID. When the structure experienced +1% and –1% lateral drift, respectively, a total of four and five plastic hinges occurred at the HBE ends. In addition, three strips (the right- or left-leaning strips for the positive or negative direction, respectively) on the second and the third floor remained elastic and only two strips on the first floor had yielded. Though more strips yielded as the pushover displacement increased, some strips remained elastic. Beyond the plastic hinges that occurred at the HBE ends, three locations of in-span plastic hinges were also observed on HBE2 and HBE3 at the end of 2% drift cycle; and the yielding condition occurred along the span of HBE0 and HBE1. At the end of the 3% drift cyclic, in-span plastic hinges on the HBEs occurred at 4 locations for both positive and negative drift excursions. In contrast with SPSW-CD (presented in Fig. 1b), most of the strips had yielded at the end of the 1% drift cycle and only four right-leaning strips and five left-leaning strips in total had remained elastic. All strips have completely yielded at the end of the 3% drift cycle. In addition, all plastic hinges have developed at its HBE ends and no in-span plastic hinge developed.

A most significant phenomenon observed is the HBE vertical downward deformation of SPSW-ID, progressively increasing and of significant magnitude as the lateral drift increased, as one example is shown in Fig. 2a. This figure compares vertical displacement history at the mid-span of the top HBE for both SPSWs. The HBE vertical downward displacement for SPSW-ID increases faster than that for SPSW-CD. In other words, this accumulative plastic incremental deformation due to cyclic pushover displacement detrimentally affects the structural performance of SPSW-ID. For example at +3% drift, the HBE3 vertical displacement of SPSW-ID was 2.3 in.; about 2.6 times larger than that of SPSW-CD, which was 0.9 in.

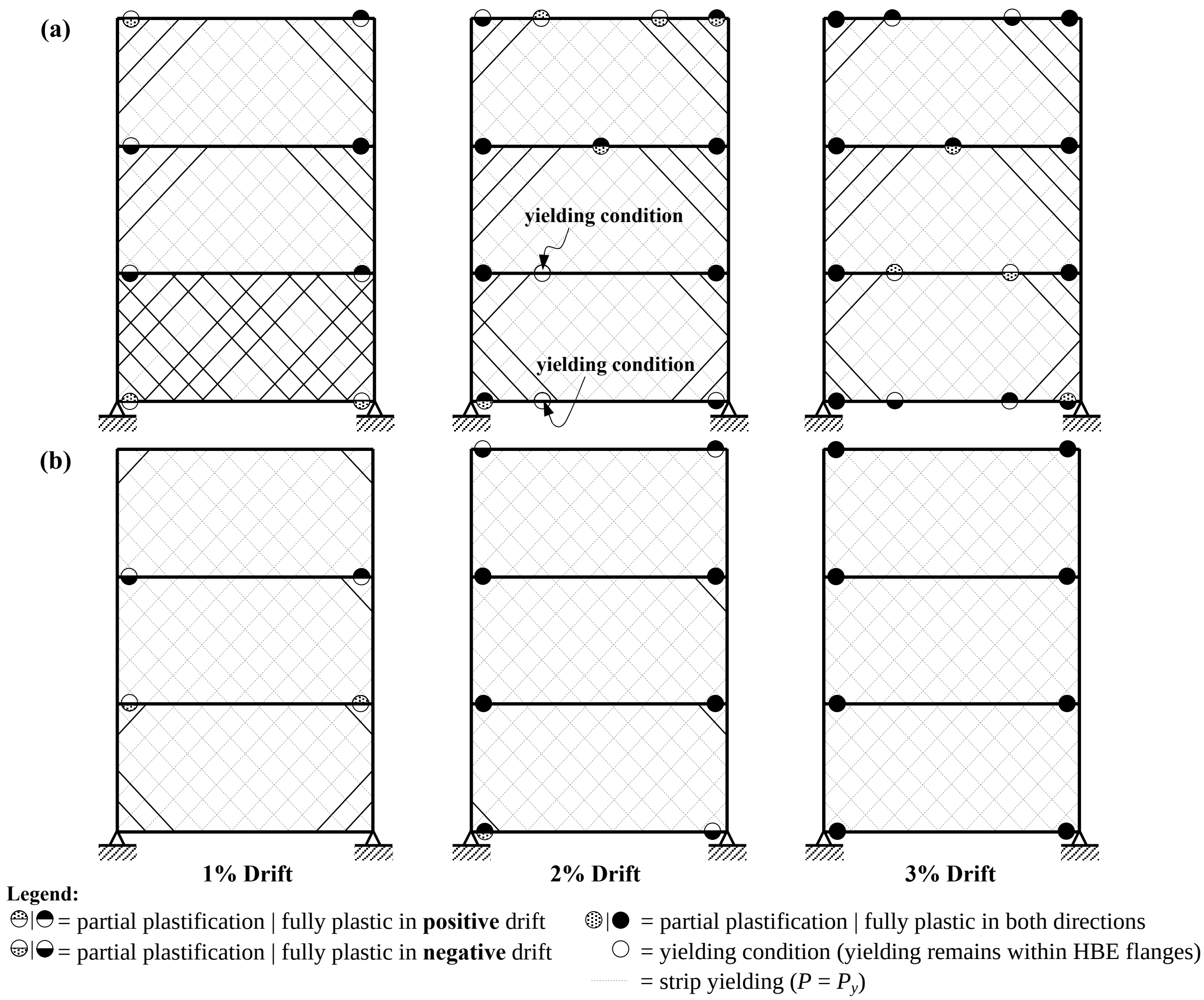

Figure 1. Plastic Hinge and Strip Yielding Distributions on (a) SPSW-ID; (b) SPSW-CD

Another approach that can be used to examine the behavior of the two SPSWs is by comparing the moment-rotation hysteresis of their HBEs, as one example shown in Fig. 3. Unlike the general hysteresis curve for special moment resisting frames, which is typically symmetric with respect to positive and negative rotations developed under a symmetric cyclic pushover displacement history, the hysteresis curves of both SPSWs considered here are not symmetric but looping with a bias toward one direction. The tension forces from the infill plates contribute to this behavior by always pulling the HBE in the direction of the tension forces. Interestingly, except for the bottom HBE, all the moment-resisting ends of the HBEs of SPSW-ID developed a cross-section rotation (i.e., cross-section curvature multiplied by plastic hinge length) greater than 0.03 radians after the structure was pushed cyclically up to a maximum lateral drift of 3%. In one case (i.e., HBE2), the total rotations even reached 0.062 radians. Such a significantly high cyclic rotation demand would be difficult to achieve using the type of moment resisting connections used in SPSW (the AISC 2010 Seismic Specifications only require that Ordinary-type connections be used in SPSW). In fact, it might also be difficult to

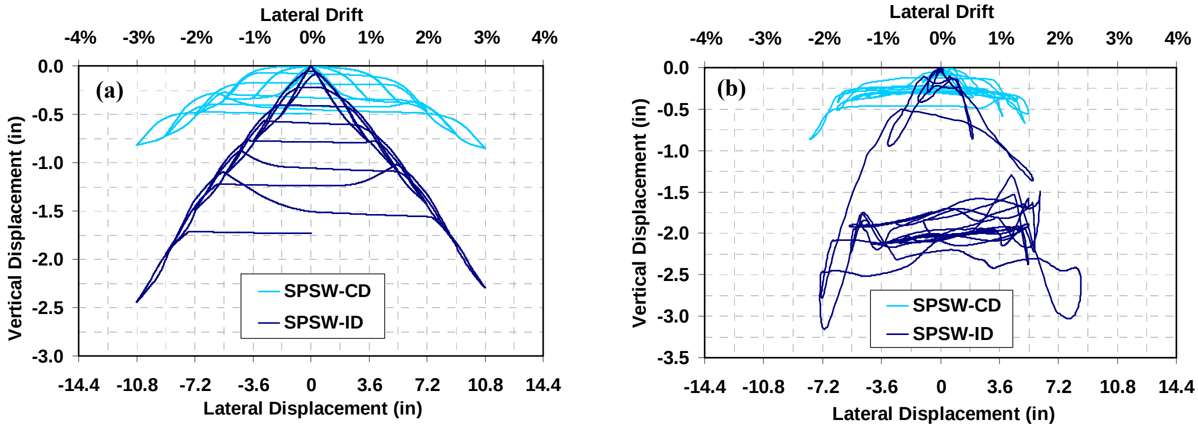

Figure 2. History of HBE3 Vertical Displacement (a) Cyclic Pushover; (b) Time History

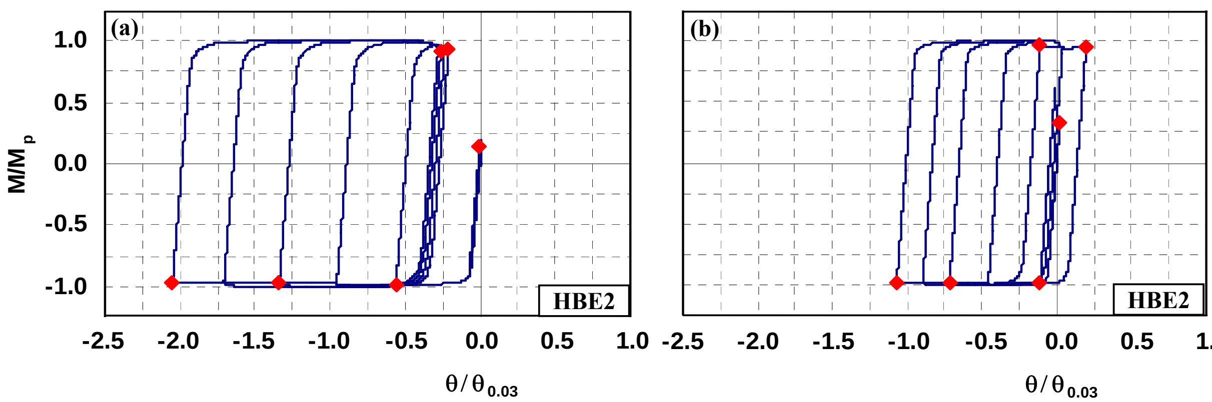

Figure 3. Normalized Moment Rotation Hysteresis (a) SPSW-ID; (b) SPSW-CD

achieve with special moment resisting frame (SMRF) beam-to-column connections approved by AISC 2010, which are experimentally verified to perform well up to +/–0.04 radians total rotations, or +/–0.03 radians plastic rotations. By comparison for SPSW-CD, all HBE total rotations obtained were less than or equal to 0.03 radians under the same cyclic pushover displacements up to 3% drift.

In an overall perspective, although failures of HBE to VBE connections have been few in SPSW tested at the time of this writing, these results might also suggest that large drift may translate into large plastic rotations even for SPSW-CD. However, before mandating the use of SMRF connection for HBEs to VBEs, it is important to recognize that the plastic rotations demands observed here were not symmetric, by contrast with moment frame behavior. More research is desirable in this regard.

Nonlinear Time History Analysis

Nonlinear time history analysis was conducted to investigate whether those previous results would be replicated during earthquake excitations and whether additional seismic behaviors for the aforementioned SPSW systems would emerge as a consequence of the random nature of earthquake records. Three synthetic time histories of ground acceleration were generated for this purpose, which their spectra matched the design basis earthquake (DBE) spectra.

The accumulative plastic incremental deformation is still observed (Fig. 2b), with maximum and residual vertical deformations more apparent on SPSW-ID than on SPSW-CD. For example, when SPSW-CD reached a lateral drift of 1% for the first time, the largest HBE3 vertical displacement at the same drift for SPSW-ID was 2.25 larger. This implies that the HBE3 vertical downward displacement for SPSW-ID increased faster than that for SPSW-CD as the lateral drift increased.

The nonlinear time history analyses were then extended to investigate the performance of both SPSWs under the more severe maximum considered earthquake (MCE). It was observed that as the severity of the synthetic ground motions increased for the MCE case (consequently generating higher lateral drifts on both SPSWs), HBE vertical deformations of SPSW-ID especially at the top two floors significantly increased compared to the corresponding magnitudes in the DBE case. For example, HBE3 maximum vertical deformation increased from 3.2 inches in the DBE case to 5.1 inches in the MCE case. By comparison for SPSW-CD, only minor changes of HBE vertical deformations occurred. Hence, when formation of in-span plastic hinges on HBEs is possible, such as in the case of SPSW-ID, the more severe the ground excitations, the more accumulation of plastic incremental deformation observed.

Impact of Infill Plate Design on Seismic Behavior of Steel Plate Shear Walls

Seismic performance of SPSWs having infill plates designed per two different philosophies was investigated. Using the FEMA P695 methodology (FEMA 2009), which defines the performance in terms of collapse potential under MCE ground motions, the assessment was first conducted on SPSWs designed neglecting the contribution of their boundary moment resisting frames to resist story shear forces (a.k.a. conventional design). Then, this assessment of collapse potential was repeated for SPSWs designed considering the sharing of story shear forces between the boundary frames and infill plates such that the sum of the strength of the two SPSW components was exactly equal to the required strength to resist the designed lateral loads (a.k.a. balanced design).

Twelve SPSW archetypes with various structural configurations (i.e., panel aspect ratio, seismic weight intensity, and number of story) were prepared. Their loading information, floor plans, and elevations were taken as similar to the SAC model building. Each SPSW archetype was designed to have one bay width, 13 ft story height, and low to moderate aspect ratio (i.e., aspect ratio of either 1.0 or 2.0). All SPSWs had moment resisting HBE-to-VBE connections. Detail of those archetypes can be found in Purba and Bruneau (2013). Fig. 4 shows an example two-dimensional nonlinear model for collapse simulation of 3-story SPSW archetypes developed in OpenSees with the deterioration material models for SPSW components (i.e., strips and boundary elements) and the gravity leaning column elements to capture the P-Δ effects.

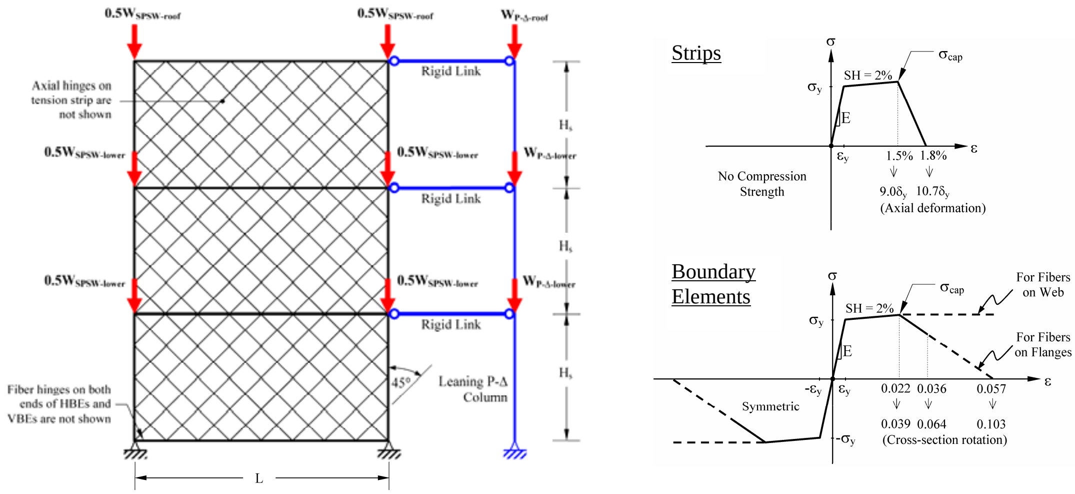

Figure 4. Nonlinear Model for Collapse Simulation

Assessment of Collapse Potential

The assessment started by conducting nonlinear pushover analysis and incremental dynamic analysis (IDA) on each SPSW archetype. The former was performed to estimate system overstrength (Ω_o) and period-based ductility (μ_T) factors. The latter was performed to obtain collapse margin ratio (CMR) of which each archetype was subjected to 44 "Far-Field" ground motions. Fig. 5 presents an example of IDA results for conventional and balanced three story SPSW archetypes. The CMR values obtained from the IDA were then adjusted to consider frequency content of the selected ground motion records (i.e., the effect of spectral shape). Spectral shape factor (SSF) values used to modify the CMR to the adjusted collapse margin ratio (ACMR) are a function of the archetype fundamental period (T) and μ_T factor obtained from the pushover analysis. The resulting ACMR was compared to the acceptable ACMR for 10% collapse probability under MCE ground motions (i.e., $ACMR_{10\%}$) of 2.16 for a total system collapse uncertainty (β_{TOT}) of 0.6.

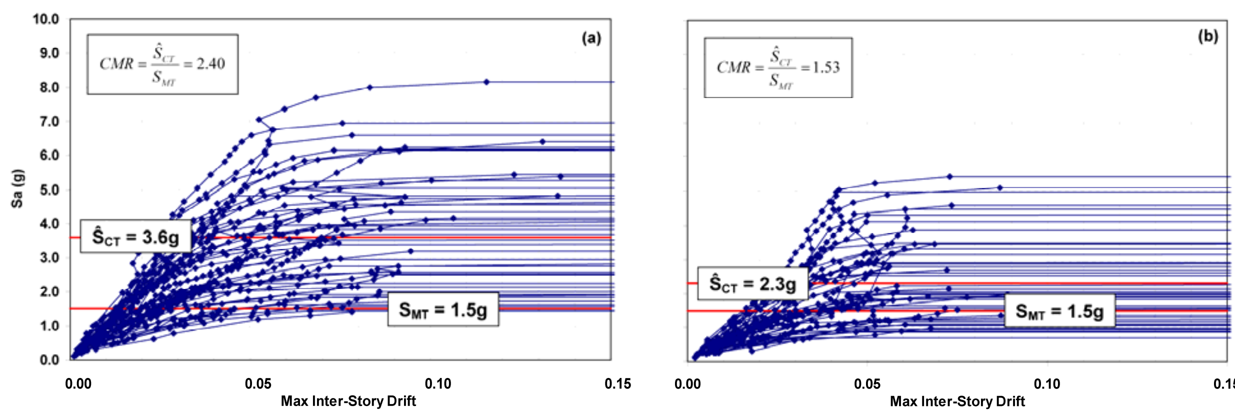

Figure 5. Incremental Dynamic Analysis (IDA) Results: (a) SW320; (b) SW320K

Table 1 presents performance evaluation of each SPSW archetypes per the FEMA P695 methodology. Detail information can be found in Purba and Bruneau (2013). All conventional archetypes passed the performance criterion. The computed ACMR for each archetype was larger than the acceptable $ACMR_{10\%}$ of 2.16. These results indicate that each archetype has a reasonable safety margin against collapse (i.e., a lower probability of collapse) as a result of the overstrength reserve provided by the boundary frame. For this type of SPSW, results indicate that the *R* factor of 7 used in design is adequate (i.e., satisfied the ACMR requirement). The Ω_o factor for the archetypes considered (based on the pushover analysis results) varied from 2.3 to 3.1. Considering the limited numbers of SPSW archetypes designed in this research, the Ω_o factor of 2.0 can be considered adequate for conventional SPSW. Assuming the inherent damping available in SPSW to be 5% of critical damping, a C_d factor of 7 can be considered for conventional SPSWs. Note that the resulting seismic performance factors for conventional SPSW obtained in this case are somewhat similar to those specified in the ASCE 7-10 (i.e., R, Ω_o, and C_d factors are 7, 2, and 6, respectively).

Table 1. Performance Evaluation for SPSW Archetypes with Various Structural Configurations

Archetype ID	**Pushover Results**				**IDA Results**		**Performance Evaluation**		
	V_d (kips)	**V_{max} (kips)**	**$\Omega_0 = V_d/V_{max}$**	**μ_T**	**$\hat{S}_{CT}$ (g)**	**CMR**	**SSF[1]**	**ACMR[2]**	**Pass/ Fail[3]**
SW310	155	401	2.6	5.5	3.14	2.10	1.26	2.64	**Pass**
SW320	176	495	2.8	4.9	3.60	2.40	1.25	3.00	**Pass**
SW320G	465	1440	3.1	5.5	4.08	2.72	1.26	3.43	**Pass**
SW520	255	578	2.3	4.2	3.40	2.42	1.25	3.03	**Pass**
SW520G	766	1924	2.5	4.8	4.26	3.03	1.27	3.85	**Pass**
SW1020	681	1975	2.9	5.2	3.40	4.08	1.25	5.09	**Pass**
SW310K	155	236	1.5	5.0	2.28	1.52	1.25	1.90	**Fail**
SW320K	176	226	1.3	4.8	2.29	1.53	1.24	1.90	**Fail**
SW320GK	465	618	1.3	5.1	2.32	1.55	1.25	1.93	**Fail**
SW520K	255	254	1.0	4.3	2.10	1.50	1.25	1.80	**Fail**
SW520GK	766	837	1.1	4.7	2.64	1.88	1.27	2.39	**Pass**
SW1020K	681	953	1.4	5.2	1.92	2.30	1.25	2.88	**Pass**
SW320KR6	205	270	1.3	5.0	2.47	1.65	1.25	2.06	**Fail**
SW320KR5	246	334	1.4	5.1	2.87	1.91	1.25	2.39	**Pass**

Note: 1) SSF obtained from FEMA P695 table for a given *T* and μ_T
2) ACMR = SSF (T, μ_T) × CMR
3) Acceptance criteria: $ACMR_{10\%}$ for β_{TOT} of 0.6 = **2.16**. Pass if ACMR ≥ $ACMR_{10\%}$.

For the balanced archetypes, except for the 10-story archetype and 5-story archetype design with high seismic weight (i.e., SW1020K and SW520GK), all other archetypes did not meet the performance criterion because their computed ACMR was smaller than $ACMR_{10\%}$. These results indicate that the *R* factor of 7 used in the initial step to design the balanced SPSW would not lead to an adequate design (i.e., the resulting did not satisfy the ACMR requirement). Hence, another 3-story balanced archetype with *R* factor of 6 was designed (i.e., denoted as SW320KR6). As shown in Table 1, compared with the results for SW320K, SW320KR6 show a slight increase in the calculated ACMR. The calculated ACMR of 2.06 is approximately 5% below the acceptable $ACMR_{10\%}$ of 2.16. Although some could consider that difference acceptable, to be rigorous, another design iteration was performed using an R factor of 5 (i.e., denoted as SW320KR5). As hoped, SW320KR5 satisfied the performance criteria. Here, the calculated ACMR of 2.39 is 11% higher than the threshold $ACMR_{10\%}$ (Table 1).

Based on the above results, seismic performance factors for SPSW designed with $\kappa_{balanced}$ are recommended to be smaller compared to that for conventional SPSW (i.e., the 100% design case, $\kappa = 1.0$). Results above indicate that an R factor of 5 should be used for the design of balanced SPSWs. No system overstrength factor is available in balanced SPSWs (i.e., $\Omega_o = 1$). Like for conventional SPSWs, the C_d factor for balanced SPSWs should be taken as similar to the assigned R factor (i.e., $C_d = 5.0$).

Interstory Drift as Damage Measure

It is also meaningful to interpret the IDA results in terms of drift demands. At the MCE level (i.e., $S_{MT} = 1.5$g), there is approximately a 50% probability that drifts will exceed 2% and 3.5% interstory drifts for SW320 and SW320K, respectively. More significantly at a 20% probability of exceedance, the respective archetypes will exceed 3% and 7% interstory drifts. The results indicate that SW320K has higher probability to suffer significant larger interstory drift, which can be associated with larger structural and non-structural damages. The same results were also obtained when comparing SW1020 and SW1020K. For the conventional archetype, half of the ground motions resulted in approximately 2% interstory drift, while that for the balance archetype resulted in 3% interstory drift. At a 20% probability of exceedance, the respective 10 story archetypes will exceed 2.5% and 4.5% interstory drifts

It should be emphasized that even though the 10-story balanced archetype (i.e., SW1020K) had a calculated ACMR that met the acceptable ACMR limit (Table 1), its probability to undergo significantly large interstory drift (i.e., ≥ 3%) can be as high as 50% under MCE ground motions. While this SPSW designed with balanced case and R factor of 7 have sufficient margin to collapse, its ability to prevent damage to the structure and to drift-sensitive non-structural components is significantly less than for its counterpart archetype (i.e., SW1020). Hence, the need to design balanced archetypes with smaller R factor is deemed necessary.

Conclusions

Using the nonlinear static analysis and nonlinear time history analyses, significant consequences to having in-span plastic hinges were identified. It was demonstrated that plastification along HBE spans can induce significant accumulation of plastic incremental deformations on the HBEs, themselves leading to partial yielding of the infill plates and correspondingly lower global plastic strength compared to the values predicted by code equations (i.e., AISC 2010).

All conventional SPSW archetypes met the FEMA P695 performance criterion for the R factor of 7 used in their design. By contrast, the balanced archetypes designed with an R factor of 7 did not meet the FEMA P695 performance criteria. Adjusted seismic performance factors for the balanced archetypes were obtained by design iterations with a lower value of R factor. Most importantly, the balanced archetypes were found to have a higher probability to suffer significantly larger interstory drift than the conventional archetypes. Savings in steel when designed balanced SPSWs with a lower *R* factor came at the cost of the SPSWs developing larger interstory drifts compared to the conventional SPSWs under MCE ground motions.

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