Design of Steel Plate Shear Walls Considering Boundary Frame Moment Resisting Action

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Abstract: Conventional design of steel plate shear walls (SPSWs) assumes that 100% of the story shear is resisted by each infill panel. Following this approach, strength provided by the boundary frame moment resisting action, which provides the SPSW with overstrength, is neglected. While this design assumption has a positive impact on seismic performance of SPSWs, no analytical work has been done to quantify the magnitude of this overstrength in general terms. Such preliminary work is conducted in this paper. Based on plastic analysis of SPSWs, this paper investigates the relative and respective contributions of boundary frame moment resisting action and infill panel tension field action to the overall plastic strength of SPSWs, followed by a proposed procedure to make use of the strength provided by the boundary frame moment resisting action. Procedures for design of SPSWs having weak infill panels are also developed in this paper. Then, results from a series of time history analyses using validated models are presented to compare the seismic performances of SPSWs designed using different design assumptions. Future work needed to provide greater insight on SPSW designs is also identified.

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Introduction

A typical steel plate shear wall (SPSW) consists of infill steel panels surrounded by columns, called vertical boundary elements (VBEs), on each side, and beams, called horizontal boundary elements (HBEs), above and below. Previous tests and analytical studies on single-story and multistory SPSWs (Berman and Bruneau 2003; Berman and Bruneau 2005; Driver et al. 1997, to name a few) recognized that a SPSW's ultimate strength combines the contributions of both the moment resisting boundary frame and the infill panels. However, the strength of the wall provided by the moment resisting action of boundary frame is not explicitly taken into account in the design of SPSWs by codes [American Institute of Steel Construction 2005; Canadian Standards Association (CSA) 2000], typically resulting in a conservative but possibly more expensive SPSW design.

To investigate the relative contribution of boundary frames to the overall strength of SPSWs and possibly achieve an optimum design of SPSWs accounting for that contribution, this paper reviews knowledge on the plastic strength of SPSWs, and summarizes some design assumptions in current codes. Then, design procedures considering boundary frame moment resisting actions are derived followed by a case study to compare the performances of SPSWs designed using various assumptions on the relative strength and design of boundary frames. A final section discusses the future work needed to further investigate the effectiveness of the proposed models.

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Plastic Strength of Steel Plate Shear Walls

Plastic collapse mechanisms for SPSWs subjected to lateral loads have been investigated by Berman and Bruneau (2003). From that work, equations for the ultimate strength of SPSWs have been shown to agree well with the results obtained from tests. For the desired SPSW plastic mechanism (i.e., the uniform collapse mechanism shown in Fig. 1), by equating the internal and external work, Berman and Bruneau (2003) derived the following general equation for the overall plastic strength of a SPSW with moment resisting HBE-to-VBE connections

$$\sum_{i=1}^{n_s} F_i h_i = \underbrace{\sum_{i=1}^{n_s} (M_{\mathrm{pl}_i} + M_{\mathrm{pr}_i})}_{\text{Contribution of boundary frame}} + \underbrace{\sum_{i=1}^{n_s} \frac{1}{2} R_{\mathrm{yp}} f_{\mathrm{yp}} L h_i (t_{wi} - t_{wi+1}) \sin(2\alpha_i)}_{\text{Contribution of infill panels}} \quad (1)$$

where F_i=lateral force applied on the wall to develop the desired mechanism; h_i=ith story elevation; M_{pl_i} and M_{pr_i}=expected plastic moments at the left and right ends of the ith HBE, respectively; t_{wi}=thickness of the infill panel at the ith story; R_{yp}=ratio of expected to nominal yield strength of infill panels; f_{yp}=nominal yield strength of infill panels; L=SPSW bay width; n_s=total number of stories; and α_i=tension field inclination angle at the ith story. Note that it is assumed here that VBEs of the wall are pinned to the ground and HBE hinges will form instead of VBE hinges at the roof level.

While Eq. (1) provides the analytical expression that separates the contribution of boundary frame and the contribution of infill

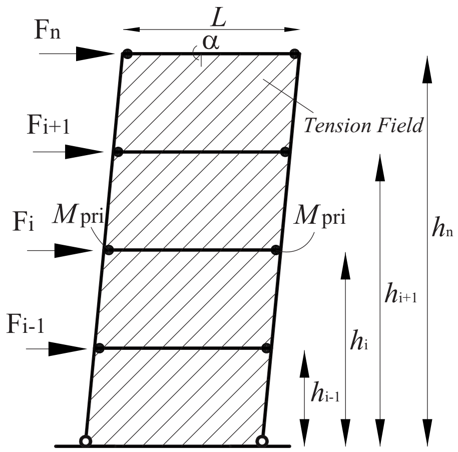

Fig. 1. Desired SPSW plastic mechanism

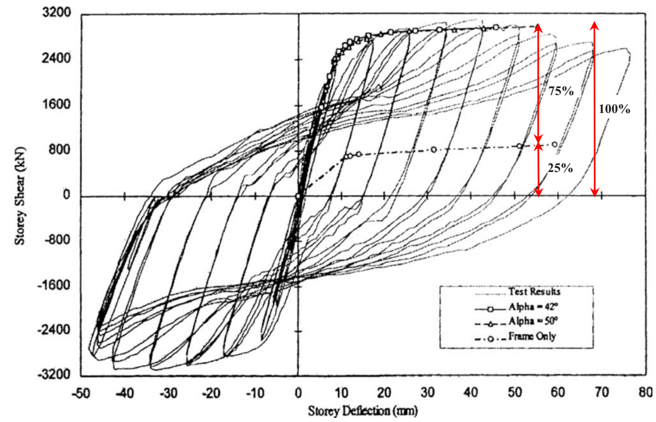

Fig. 3. Test results of a multistory SPSW [adapted from Driver et al. (1997)]

panels to the total lateral strength of the SPSW, further experimental results in the literature explicitly quantify the contribution of each system. For example, Fig. 2(a) shows the hysteric curves obtained from tests of a single-story SPSW by Berman and Bruneau (2005). Note that the web-angle HBE-to-VBE connections in that specimen had a nonnegligible moment resisting capacity. Berman and Bruneau subtracted the hysteretic behavior of boundary frame numerically obtained following the procedure proposed by Chen et al. (1996) from the total hysteretic response of the SPSW, and obtained the results of infill panel only as shown in Fig. 2(b). Comparing the curves shown in Figs. 2(a and b), it is observed that the overall strength of the wall reduces from 645 kN to about 400 kN due to the absence of the contribution of boundary frame.

In another example of multistory SPSW with moment resisting HBE-to-VBE connections tested by Driver et al. (1997), similar observation was obtained. Fig. 3 presents the hysteretic curves and pushover curves of the specimen. As shown, the strength due to the boundary frame moment resisting action contributes to about 25% of the global plastic strength of the wall.

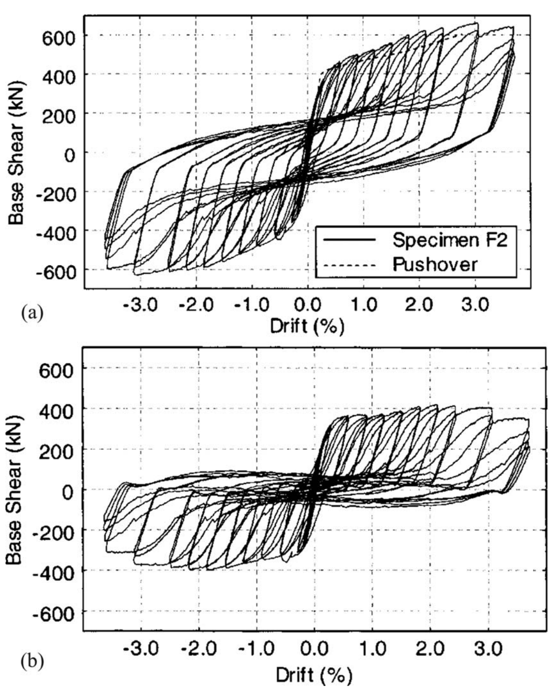

Fig. 2. Hystereses of a single-story SPSW: (a) overall; (b) infill only [from Berman and Bruneau (2005)]

Current Design Requirements

As shown in Fig. 2(b), without the contribution of its boundary frame, the hysteresis loops exhibit severely pinching behavior, and, correspondingly, less energy dissipation would exist in a SPSW with simple HBE-to-VBE connections. Such a system would need to progressively drift to larger drifts to continue to dissipate substantial hysteretic energy. In addition, a SPSW with simple HBE-to-VBE connections will not have any additional lateral force resistance beyond that provided by the infill tension field actions, resulting in a system with less redundancy. Hence, the *American Institute of Steel Construction (AISC) Seismic Provisions* (American Institute of Steel Construction 2005) requires the HBE-to-VBE connections to be rigid for the system. The CSA S16 Standard [Canadian Standards Association (CSA) 2000] requires similarly for SPSW designs with the largest R factor and allows SPSWs with simple HBE-to-VBE connections to be also implemented in seismic regions albeit with a significantly lower R factor ($R=2.0$ versus 5.0, where 5.0 is the largest permitted for any systems by the CSA S16 Standard). However, neither design code takes into account the SPSW strength provided by the boundary frame moment resisting action as described next to resist the prescribed seismic loads.

Based on Eq. (1) presented in the previous section for the SPSW plastic strength, one can obtain the following equation for calculating the shear strength of a single infill panel:

$$V_i = \frac{1}{2} R_{yp} f_{yp} L t_{wi} \sin(2\alpha_i) \tag{2}$$

where V_i=expected strength of the considered infill panel and the other terms have been defined previously.

Dividing the infill panel strength determined from Eq. (2) by an overstrength factor, as defined by FEMA 369 (FEMA 2001), and taken as 1.2 in this case (Berman and Bruneau 2003), and also excluding the R_y factor used for calculating the expected plate strength, one can obtain the following infill panel nominal shear strength

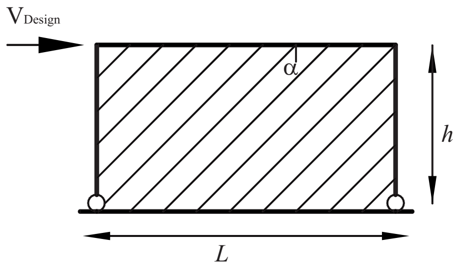

Fig. 4. Single-story SPSW example

$$V_{ni} = 0.42 f_{yp} L t_{wi} \sin(2\alpha_i) \quad (3)$$

where V_{ni}=nominal strength of the considered infill panel.

Eq. (3) is implemented in the AISC Seismic Provisions and the CSA S16 Standard, and is used for sizing the thickness of infill panels of SPSWs. Thus, by neglecting the contribution from the boundary frame moment resisting action on the SPSW strength, and solving for t_{wi} from Eq. (3), one can obtain the following design equation to select thickness of the infill panel:

$$t_{wi} = \frac{V_{ni}}{0.42 f_{yp} L \sin(2\alpha_i)} \quad (4)$$

Then, the AISC Seismic Provisions and the CSA S16 Standard require that the boundary frame members be designed using capacity design procedures to ensure that the boundary frame members can anchor the infill panel yield forces developed by the infill panel thickness determined from Eq. (4). As such, following this approach, strength provided by the boundary frame moment resisting action provides the SPSW with an overstrength (which has a positive impact on seismic performance). At the time of this writing, no analytical work has been done to quantify the magnitude of this overstrength in general terms and to investigate how to make use of it to achieve an optimum design of SPSWs. Such work is conducted in the following section.

Steel Plate Shear Wall Overstrength and Balanced Design

In order to best understand the SPSW overstrength resulting from the aforementioned design approach inferred by current design codes, which assumes that all the lateral forces applied on the wall are resisted by the infill panel tension field actions alone, this section investigates the overstrength of SPSWs designed considering that various percentages of the lateral design forces are resisted by the infill panels. Both single-story and multistory SPSWs are considered.

Single-Story SPSW

A single-story SPSW is first studied here because this simple case provides some of the building blocks necessary to understand the more complex scenario (i.e., multistory SPSWs) presented later. Consider the single-story SPSW shown in Fig. 4 and assume that its VBEs are pinned to the ground. This assumption is done for two reasons, namely (1) the strengths of the plastic hinges at the column bases add very little to the lateral load resistance of a multistory SPSW, which is to be discussed later as an extension of the study of single-story SPSW and (2) VBE design requires consideration of the loads caused by HBE flexural actions, which is different from HBEs that can be sized only considering the loads due to the fill panel tension field actions (as shown in the following derivations). As a result, taking into consideration of plastic hinges at the column bases will increase the complexity of the equations derived next.

Assuming that the percentage of the total lateral design force assigned to the infill panel is κ, the required infill panel thickness is determined by solving for t_w from the following equation:

$$\kappa V_{\text{design}} = \frac{1}{2} R_{yp} f_{yp} L t_w \sin(2\alpha) \quad (5)$$

where V_{design}=lateral design force applied on the wall. Note that Eq. (2) is consistent with Eq. (5) when $\kappa=1$ (i.e., when 100% of the lateral force is assumed to be resisted by the infill panel).

As described in Berman and Bruneau (2008) and Sabelli and Bruneau (2007), when the wall is fully yielded, the vertical and horizontal components of the distributed loads to be applied along HBEs (ω_{xb} and ω_{yb}) from infill panel yielding can be determined as

$$\omega_{yb} = R_{yp} f_{yp} t_w (\cos\alpha)^2 \quad (6)$$

$$\omega_{xb} = \frac{1}{2} R_{yp} f_{yp} t_w \sin 2\alpha \quad (7)$$

Substituting Eq. (7) into Eq. (5), one can obtain the following relationship between lateral design force and horizontal tension field component along the HBE:

$$\kappa V_{\text{design}} = \omega_{xb} L \quad (8)$$

Assuming that HBE plastic hinges (as normally the case rather than VBE plastic hinges) will form at the roof level, and following the plastic analysis procedure presented in Berman and Bruneau (2003), one can obtain the following equation for ultimate strength of the wall (equating the internal and external work):

$$V_p h = \omega_{xb} L h + 2M_{pb} \quad (9)$$

where V_p=plastic strength of the wall; L and h=wall width and height, respectively; and M_{pb}=plastic strength of the HBE-to-VBE connections.

Assuming that the top HBE is proportioned using the previously proposed design procedure (Vian and Bruneau 2005; Vian et al. 2009), which ensures that the beam could anchor the infill panel yield forces without developing in-span plastic hinges that could compromise the overall system strength, its resulting plastic section modulus, Z_b, is given as

$$Z_b = \frac{\omega_{yb} L^2}{4 f_y} \times \frac{1}{1 + \sqrt{1 - \eta^2}} \quad (10)$$

where f_y=nominal yield strength of boundary frame and η =plastic section modulus reduction ratio accounting for the possible presence of reduced beam section (RBS) connections. Note that η may vary from unity (when there are no RBS connections in the top HBE) to the minimum value of RBS flange reduction permitted by the design specifications and guidelines such as FEMA 350 [FEMA (2000)].

Accordingly, the plastic strength of the top HBE-to-VBE connections is

$$M_{pb} = f_y \eta Z_b \quad (11)$$

Substituting Eq. (10) into Eq. (11) into, the plastic strength of the top HBE-to-VBE connections can be further expressed as

$$M_{pb} = \frac{\omega_{yb}L^2}{4} \times \frac{\eta}{1+\sqrt{1-\eta^2}} \quad (12)$$

Substituting Eq. (12) into Eq. (9) and solving for V_p

$$V_p = \omega_{xb}L + \frac{\omega_{yb}L^2}{2h} \times \frac{\eta}{1+\sqrt{1-\eta^2}} \quad (13)$$

Based on Eqs. (6) and (7), one can obtain the following relationship between ω_{xb} and ω_{yb}:

$$\omega_{yb} = \tan^{-1}(\alpha)\omega_{xb} \quad (14)$$

Substituting Eq. (14) into Eq. (13), the plastic strength of the wall becomes

$$V_p = \omega_{xb}L\left[1 + \frac{1}{2}\tan^{-1}(\alpha)\left(\frac{L}{h}\right) \times \frac{\eta}{1+\sqrt{1-\eta^2}}\right] \quad (15)$$

Substituting Eq. (8) into Eq. (15), one can obtain a relationship between the SPSW plastic strength, V_p, and the lateral design force, V_{design}, namely

$$V_p = \kappa V_{\text{design}}\left[1 + \frac{1}{2}\tan^{-1}(\alpha)\left(\frac{L}{h}\right) \times \frac{\eta}{1+\sqrt{1-\eta^2}}\right] \quad (16)$$

Here, the ratio of V_p to V_{design}, which is denoted as Ω_κ, is used to describe the overstrength of the SPSWs designed using different values of κ. Dividing by V_{design} on both sides of Eq. (16), one can determine Ω_κ as

$$\Omega_\kappa = \kappa\left[1 + \frac{1}{2}\tan^{-1}(\alpha)\left(\frac{L}{h}\right) \cdot \frac{\eta}{1+\sqrt{1-\eta^2}}\right] \quad (17)$$

Explicitly shown in Eq. (17), the factor, Ω_κ, depends on a series of variables including, κ, α, L/h, and η. The effects of those terms can be investigated based on Eq. (17). Here, a parametric study is conducted to discuss the impact of κ on Ω_κ for the given values of other terms.

Note that, for simplicity, the inclination angle of the tension field action is assumed to be 45° and η is assumed to be unity (i.e., no RBS connections are used in the HBEs). Note that results are not expected to vary substantially for other values of α. In addition, in the parametric study, the infill panel aspect ratio (i.e., L/h) was chosen to vary between 0.8 and 2.5, which are the limits allowed by the AISC Seismic Provisions (American Institute of Steel Construction 2005).

The corresponding results are illustrated in Fig. 5. As shown, a higher percentage of the lateral design forces assigned to the infill panel (i.e., greater value of κ) results in a greater overstrength of the wall (i.e., greater value of Ω_κ). Under the design assumption presented in the AISC Seismic Provisions and the CSA S16 Standard (i.e., when $\kappa=1.0$), the wall has a significant overstrength varying from 1.4 to 2.25 over the code-compliant range of infill panel aspect ratios of $0.8 \le L/h \le 2.5$. Note that the example wall assumes that the VBEs are pinned to the ground. It is recognized that, for the SPSWs that with VBEs fixed to the ground, the overstrength would be even greater.

Also observed from Fig. 5, when κ is reduced to certain level, showed by the circles on that figure, the lateral force resisted by the boundary frame of the SPSW is exactly equal to that which will be required if that frame is designed to resist the infill panel yield forces per capacity design principles. Therefore, at that particular point, the boundary frame does not provide any overstrength for the system as the division of the lateral load resistance, and the overstrength (Ω_κ) is therefore equal to unity.

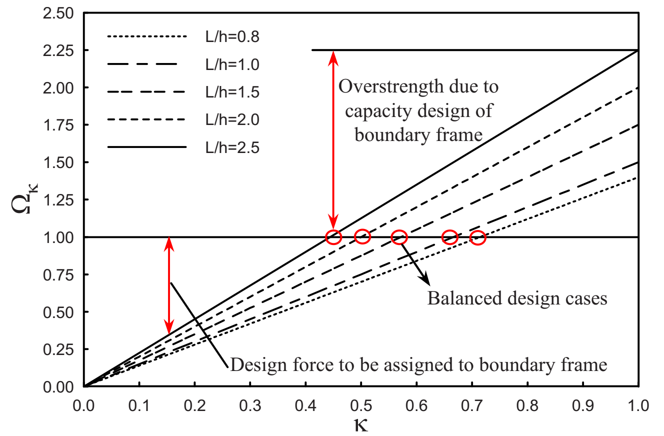

Fig. 5. Relationship between Ω_κ and κ (assuming $\alpha=45°$ and $\eta=1.0$)

Such a design case is termed "balanced" design case in this paper. For this case, the value of κ can be determined by setting the constraint $\Omega_\kappa=1.0$ into Eq. (17) and solving for κ. The resulting value of κ for the balanced case, designated as κ_{balanced}, is therefore

$$\kappa_{\text{balanced}} = \left[1 + \frac{1}{2}\tan^{-1}(\alpha)\left(\frac{L}{h}\right) \times \frac{\eta}{1+\sqrt{1-\eta^2}}\right]^{-1} \quad (18)$$

Figs. 6 and 7 respectively plot, based on Eq. (18), the relationships between κ_{balanced} and η for various values of L/h, and κ_{balanced} and L/h for various values of α. As shown in Fig. 6, the value of κ_{balanced} increases when η reduces. This observation is reasonable because the reserved strength of the wall due to the moment resisting action of the boundary frame decreases when RBS connections are introduced in the HBE (i.e., when $\eta \le 1.0$), which means that, in this case, a higher percentage of lateral design force should be resisted by the infill panel tension field action. Note that, when η reduces to zero, which physically corresponds to simple HBE-to-VBE connections, κ_{balanced} becomes unity, indicating that 100% of the lateral force is resisted by the infill panel.

Fig. 7 illustrates the trends in κ_{balanced} for the code-compliant range of infill panel aspect ratios and the typical range of tension field inclination angles. As shown, the value of κ_{balanced} decreases when the aspect ratio increases. This is also reasonable since a

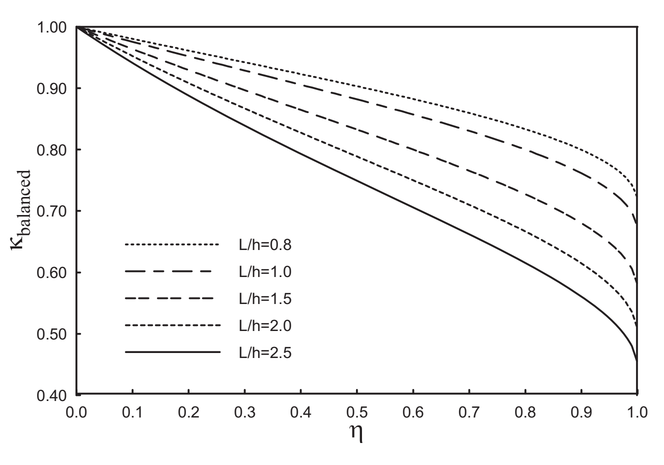

Fig. 6. Relationship between κ_{balanced} and η (assuming $\alpha=45°$)

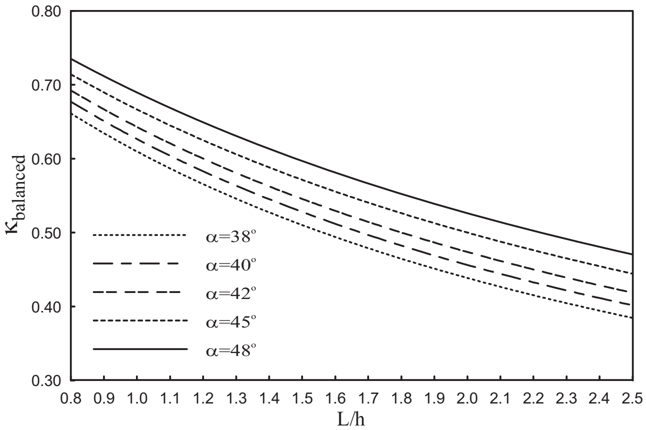

Fig. 7. Relationship between aspect ratio (L/h) and $\kappa_{balanced}$ (assuming $\eta = 1.0$)

bigger HBE member has to be used to anchor the tension field action in a "squat" wall (which has a greater aspect ratio) in comparison with a slender wall (which has a smaller aspect ratio), resulting in a higher strength of the wall provided by the moment resisting action of the boundary frame.

As shown in Fig. 5, when reducing κ to a value below $\kappa_{balanced}$, the SPSW plastic strength is not sufficient to resist the lateral design force. In other words, the boundary frame designed only to resist the infill panel yield forces, per capacity design principles, has to be strengthened to fill the gap between the available strength of the wall and the expected lateral design demand. Incidentally, detailed information about the design of such SPSWs will be presented later.

Multistory Steel Plate Shear Wall

The derivations presented for single-story SPSWs can be extended to the case of multistory SPSWs. The related procedures are briefly described next. Note that an index, i, is assigned to the variables associated with the ith floor level.

Consider a multistory SPSW with rigid HBE-to-VBE connections and VBEs pinned to the ground (for the same reasons as before) shown in Fig. 8. Here, Fig. 8(a) shows the lateral design forces applied on the SPSW; Fig. 8(b) shows the modified lateral design force to size the infill panels, and Fig. 8(c) shows the lateral force needed to develop the desired SPSW plastic mechanism. Based on those figures, the corresponding derivations are presented next.

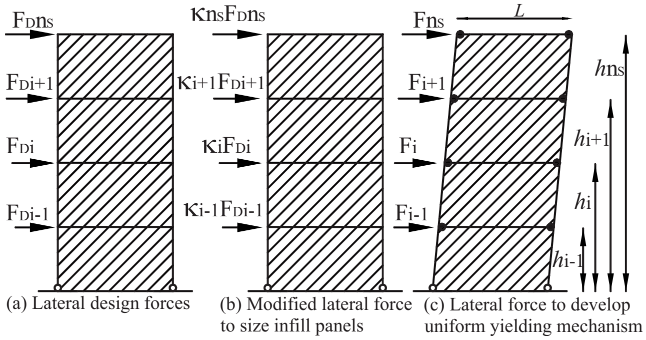

Fig. 8. Schematic of a typical multistory SPSW

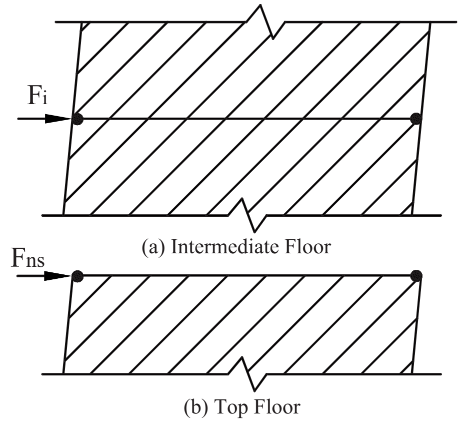

Fig. 9. Segments of a uniformly yielded multistory SPSW

Similar to the procedure for single-story SPSWs, assigning part of the lateral design forces to the infill panel system, as shown in Fig. 8(b) and equating the infill panel shear strength and the corresponding design story shear at the ith and $i+1$th story, respectively, one can have

$$\omega_{xbi}L = \sum_{k=1}^{n_s} \kappa_k F_{Dk} \tag{19}$$

$$\omega_{xbi+1}L = \sum_{k=i+1}^{n_s} \kappa_k F_{Dk} \tag{20}$$

Subtracting Eq. (20) from Eq. (19)

$$(\omega_{xbi} - \omega_{xbi+1})L = \kappa_i F_{Di} \tag{21}$$

It is assumed that the SPSW is able to develop the anticipated uniform plastic mechanism shown in Fig. 8(c). Consider an intermediate floor along the height of the wall as shown in Fig. 9(a). Note that the derivations presented next are also valid for the top floor shown in Fig. 9(b), simply by setting the magnitude of the tension field components of the upper story equal to zero.

Using the principle of virtual work (Neal 1977), equating the external and internal work tributary to the considered intermediate floor, one can obtain the following equation:

$$F_i h_i = (\omega_{xbi} - \omega_{xbi+1})Lh_i + 2M_{pbi} \tag{22}$$

Assuming again that each intermediate HBE is proportioned using the design procedure proposed by Vian and Bruneau (2005), the resulting plastic section modulus is given as

$$Z_{bi} = \frac{(\omega_{ybi} - \omega_{ybi+1})L^2}{4f_y} \times \frac{1}{1 + \sqrt{1 - \eta_i^2}} \tag{23}$$

Note that the procedure by Vian and Bruneau (2005) was originally developed for anchor HBEs. However, that procedure can be alternatively used for intermediate HBEs by considering the resulting vertical component of the tension field actions along the beam. Accordingly, the plastic moment of any HBE is determined as

$$M_{pbi} = \frac{(\omega_{ybi} - \omega_{ybi+1})L^2}{4} \times \frac{\eta_i}{1 + \sqrt{1 - \eta_i^2}} \tag{24}$$

Substituting Eqs. (24) and (21) into Eq. (22), also considering Eq. (14) and solving for F_i, one can determine the force applied at each floor level to develop the expected mechanism, which is

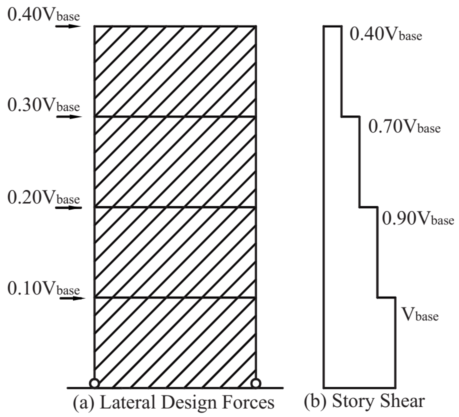

Fig. 10. Description of example four-story SPSW

$$F_i = \kappa_i F_{Di}\left[1 + \frac{1}{2}\tan^{-1}(\alpha_i)\left(\frac{L}{h_i}\right) \times \frac{\eta_i}{1 + \sqrt{1 - \eta_i^2}}\right] \tag{25}$$

For the balanced design case

$$F_i = F_{Di} \tag{26}$$

which physically means the lateral design force applied at each level of the SPSW is equal to that needed to develop the desirable mechanism. Substituting Eq. (26) into Eq. (25), one can solve for κ_i for the balanced case

$$\kappa_{\text{balanced } i} = \left[1 + \frac{1}{2}\tan^{-1}(\alpha_i)\left(\frac{L}{h_i}\right) \times \frac{\eta_i}{1 + \sqrt{1 - \eta_i^2}}\right]^{-1} \tag{27}$$

To have a better understanding of the lateral forces respectively resisted by the infill panels and the boundary frame in the balanced design case for a multistory SPSW, consider a four-story SPSW without RBS connections as an example. Assume that the lateral design forces linearly distribute along the height of the wall as shown in Fig. 10 and the story heights and infill tension field inclination angles (45°) are constant in all stories. Fig. 11 illustrates the percentage of the story shear resisted by the infill panel at each level (i.e., to be considered to size the infill panels at each story). For comparison purpose, the results for the case when

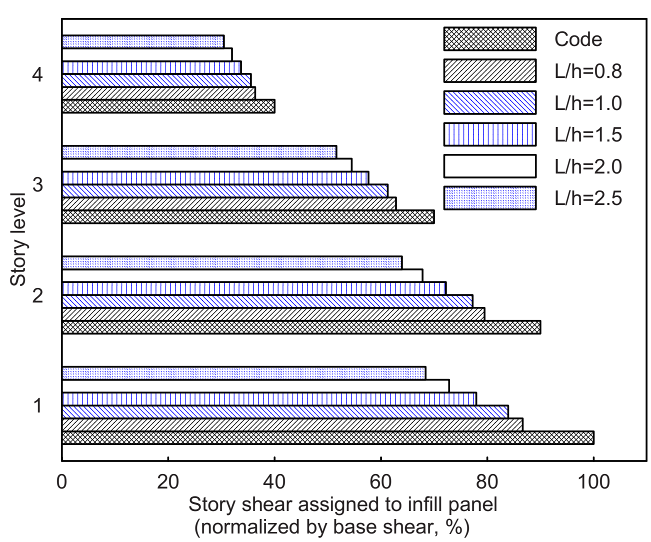

Fig. 11. Modified design forces of an example four-story SPSW

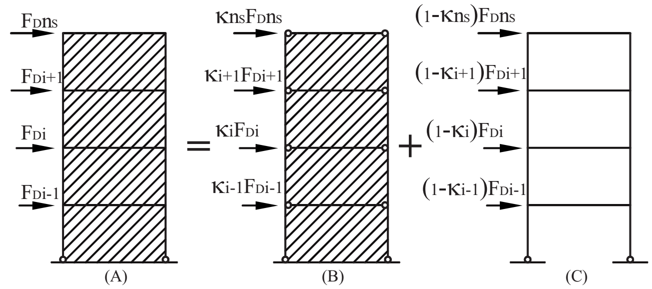

Fig. 12. Decompositions of lateral forces and SPSW system

100% of the story shear is resisted by each infill panel (i.e., the design case implied by the AISC Seismic Provisions) are also provided. As shown, the story shears assigned to the infill panels are reduced in the balanced design case. For example, when the infill panel has an aspect ratio of 1.5, 78% of the base shear is resisted by the first-story infill panel when the wall develops the desired plastic mechanism.

Boundary Frame Design of SPSWs Having Weak Infill Panels

When the infill panel thickness is smaller than that corresponding to the balanced design case, the SPSW will not have a sufficient strength to resist the lateral design force if the boundary frame is only proportioned using capacity design procedures (i.e., designed only to resist the infill panel yield forces). Here, such walls will be termed SPSWs having weak infill panels.

As a first step, the principle of SPSWs having weak infill panels is studied from a theoretical perspective to understand the implications of that concept. There are no explicit benefits in using an infill panel thickness less than the balanced case. However, without the knowledge of the equations presented earlier, an engineer may decide to apportion the percentages of the strength provided by the boundary frame and the infill panels arbitrarily to any numbers (e.g., 50–50%). When weak infill panels are used, the boundary frame resulting from capacity design procedures needs to be strengthened to ensure that the overall SPSW (including both the infill panels and boundary frame) is able to resist the specified lateral design forces, i.e., in that case, capacity design considerations do not drive the design of the boundary frame.

To address design procedures of the boundary frame of SPSWs having weak infill panels, consider the multistory SPSW shown in Fig. 12(a). Assume the VBEs are pinned to the ground. To better understand the lateral design demand resisted by the boundary frame, the SPSW is decomposed into two lateral force resisting systems as shown in Fig. 12, namely: (1) Frame B consisting of infill panels, which resists the lateral loads entirely through infill tension field actions together with a boundary frame without moment resisting connections; and (2) Frame C as a frame without infill panels, which resists lateral loads only through moment frame actions up to the development of plastic moments at the HBE-to-VBE connections. Note that the HBE end fixities are removed in Frame B, since their contribution to lateral force resistance is taken into account in Frame C. The summation of lateral force resistances of the aforementioned two systems (i.e., Frames B and C) is equal to the SPSW lateral strength. Accordingly, the lateral force applied at each floor level of Frame C is $(1-\kappa_i)F_{Di}$.

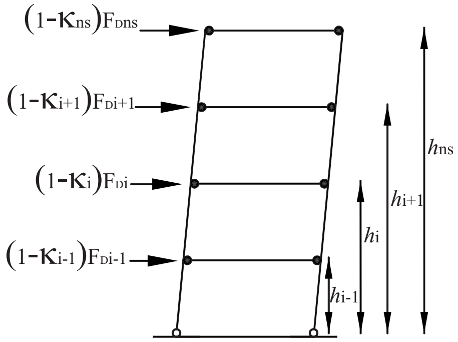

Fig. 13. Plastic mechanism of boundary frame

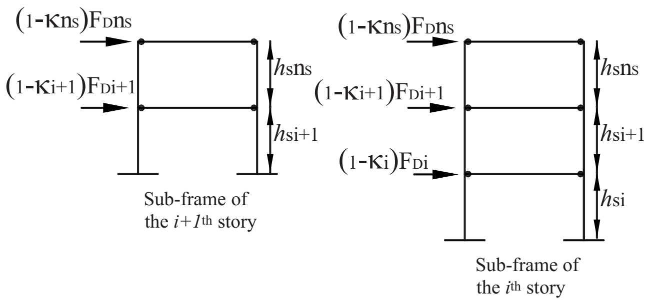

Fig. 14. SPSW subframes

Design of Frame C can be achieved by following plastic analysis procedure and ensuring sufficient frame plastic strength. Three methods, which lead to different designs but same global plastic strength of the boundary frame, are presented next. What conceptually differs in the three methods considered next is that each case assumes a different distribution of HBE strength along the height of the SPSW for which the global plastic strength is satisfied but the local story-by-story strength is not necessarily satisfied. Note that the following derivations are based on plastic analysis and the yielding sequence of members and infill panels of the SPSW under the lateral forces is out of the scope of the work presented here.

For all methods used here to design the boundary frame, the resulting HBEs are checked to comply with the result determined from Eq. (23), i.e., capacity design is checked to ensure that all members are able to anchor the infill panel yield forces. In addition, the resulting designs are checked to satisfy the strong column and weak beam requirement to avoid undesirable behavior of the wall (e.g., soft story mechanism).

Design Method I

The first method to design the boundary frame assumes constant HBE cross sections along the height of the wall. When the boundary frame as part of the wall develops the plastic mechanism shown in Fig. 13, equating the external and internal work of the boundary frame, one can derive the following equation:

$$\underbrace{\sum_{i=1}^{n_s}(1-\kappa_i)F_{Di}h_i}_{\text{External work}}=\underbrace{\sum_{i=1}^{n_s}2\cdot\eta_i\cdot f_y\cdot Z_b}_{\text{Internal work}} \tag{28}$$

Solving for Z_b gives

$$Z_b=\frac{\sum_{i=1}^{n_s}(1-\kappa_i)F_{Di}h_i}{2f_y\sum_{i=1}^{n_s}\eta_i} \tag{29}$$

Design Method II

The second method to design the boundary frame determines HBEs from the virtual work equation of each story. For the plastic mechanism shown in Fig. 13, equating the external and internal work tributary to the ith story, one can derive the following equation:

$$\underbrace{(1-\kappa_i)F_{Di}h_i}_{\text{External work}}=\underbrace{2\cdot\eta_i\cdot f_y\cdot Z_{bi}}_{\text{Internal work}} \tag{30}$$

Solving for Z_{bi} gives

$$Z_{bi}=\frac{(1-\kappa_i)F_{Di}h_i}{2f_y\eta_i} \tag{31}$$

It is recognized that Design Method II is actually an equilibrium equation considering the work at a typical floor. From a perspective of plastic analysis, such a method will not provide either lower bound or upper bound solutions of the entire frame. However, solutions from Design Method II can mathematically satisfy the system equation obtained by using the Principle of Virtual Work [i.e., Eq. (28) when replacing Z_b by Z_{bi}]. Therefore, for comparison purpose, although not rigorously a correct approach, it was decided to also investigate the seismic behavior of the SPSWs designed using Design Method II. As such, out of curiosity and with the preceding caveat, this method is also considered here.

Design Method III

The third method to design the boundary frame determines HBEs from the virtual work equation of the subframe from the ith story to the top story. For the plastic mechanism shown in Fig. 13, equating the internal and external work of the $i+1$th subframe as shown in Fig. 14, one can derive the following equation:

$$\underbrace{\sum_{j=i+1}^{n_s}\sum_{k=i+1}^{j}(1-\kappa_j)F_{Dj}h_{sk}}_{\text{External work}}=\underbrace{2f_y\cdot\left[\sum_{j=i+1}^{n_s}\eta_j Z_{bj}\right]}_{\text{Internal work}} \tag{32}$$

where h_{si}=ith story height, different from h_i used in the previous derivations (i.e., the ith story elevation).

Similarly, the equation for the ith subframe shown in Fig. 14 is

$$\underbrace{\sum_{j=i}^{n_s}\sum_{k=i}^{j}(1-\kappa_j)F_{Dj}h_{sk}}_{\text{External work}}=\underbrace{2f_y\cdot\left[\sum_{j=i}^{n_s}\eta_j Z_{bj}\right]}_{\text{Internal work}} \tag{33}$$

Subtracting Eq. (32) from Eq. (33) and solving for Z_{bi}

$$Z_{bi}=\frac{\sum_{j=i}^{n_s}(1-\kappa_j)F_{Dj}h_{si}}{2f_y\eta_i} \tag{34}$$

Table 1. Summary of Design Story Shears and Infill Panel Thicknesses

Story level	Elevation (ft)	Lateral force (kip)	Modified story shear (kip)			Infill panel thickness (in.)		
			AISC	Balanced design	Weak infills	AISC	Balanced design	Weak infills
8	80	127.1	127.1	113.0	50.8	0.033	0.029	0.013
7	70	137.9	264.9	233.6	106.0	0.069	0.060	0.027
6	60	117.9	382.8	335.5	153.1	0.099	0.087	0.040
5	50	97.9	480.7	422.9	192.3	0.124	0.109	0.050
4	40	78.0	558.7	492.0	223.5	0.144	0.127	0.058
3	30	58.2	616.9	542.8	246.8	0.160	0.140	0.064
2	20	38.5	655.5	575.8	262.2	0.170	0.149	0.068
1	10	19.0	674.5	590.1	269.8	0.174	0.153	0.070

Case Study

Prior sections presented different approaches for SPSW design. To be able to determine the relative merits of any of those designs, which will provide the same overall lateral load resistance, it is important to conduct nonlinear time history analyses to compare the seismic performances of the walls respectively designed assuming various distributions of the lateral loads between the boundary frame and infill panels as well as various distributions of HBE strength that satisfy the total plastic strength of the structure. Such a study is conducted in this section. The following briefly describes assumptions, design results, and the performances of those differently designed SPSWs in the nonlinear time history analyses.

Assumption and Design Summary

An eight-story single-bay SPSW was used as the prototype structure in this study. The VBEs were assumed to be pinned to the ground and the first-story infill panel was assumed to be anchored to the ground rather than to an anchor HBE at that level. The bay width and constant story height were assumed to be 18 ft and 10 ft, respectively, resulting in an infill panel aspect ratio of 1.8, which is within the range allowed in the 2005 AISC Seismic Provisions.

The structure was assumed to be located on Class B soil in Northridge, Calif. Its weight was assumed to be 5092.5 kip (22,653 kN) distributed as 652.5 kip (2,093 kN) at all levels except at the roof where it was 525 kip (2,335 kN). Seismic design loads were calculated using FEMA 450 (FEMA 2003) and the associated spectral acceleration maps. Design short and 1-s spectral ordinates, S_{DS} and S_{D1}, were respectively calculated to be 1.43 g and 0.50 g. The period of the structure was estimated (using the FEMA procedures) to be 0.54 s, and using a response modification factor, R, of 7, and an importance factor I, of 1, the base shear was found to be 674.5 kip (3,000 kN).

Corresponding lateral loads up the height of the structure are presented in Table 1. As shown, the AISC design procedure, the balanced design procedure, and the design procedure assuming that 40% of the story shear is resisted by the infill panel at each story (i.e., the procedure for the SPSWs having weak infill panels) are considered to select the SPSW infill panel thicknesses. Note that it is assumed that the calculated infill panel thicknesses are available in all cases.

For the design using weak infill panels, all three HBE design methods were considered for design of the boundary frame. As a result, a total of five SPSW designs were obtained, i.e., one from the AISC design procedure, one from the balanced design procedure, and three from the procedure for SPSWs having weak infill panels (respectively using Methods I, II, and III). The boundary

Table 2. Design Summary of Boundary Frame Members

Frame member[a]	AISC	Balanced design	Weak infills Method I	Weak infills Method II	Weak infills Method III
HBE-8	W18×76	W18×71	W18×158	W18×311	W18×50
HBE-7	W18×86	W18×71	W18×158	W18×311	W18×97
HBE-6	W18×65	W18×60	W18×158	W18×234	W18×130
HBE-5	W18×65	W18×60	W18×158	W18×158	W18×158
HBE-4	W18×60	W18×55	W18×158	W18×106	W18×192
HBE-3	W18×55	W18×50	W18×158	W18×65	W18×211
HBE-2	W18×50	W18×40	W18×158	W18×40	W18×211
HBE-1	W18×50	W18×40	W18×158	W18×40	W18×211
VBE-8	W30×116	W30×108	W40×167	W40×235	W40×149
VBE-7	W30×116	W30×108	W40×167	W40×235	W40×149
VBE-6	W40×183	W40×167	W40×235	W40×235	W40×235
VBE-5	W40×183	W40×167	W40×235	W40×235	W40×235
VBE-4	W40×277	W40×264	W40×264	W40×264	W40×264
VBE-3	W40×277	W40×264	W40×264	W40×264	W40×264
VBE-2	W40×431	W40×362	W40×362	W40×362	W40×362
VBE-1	W40×431	W40×362	W40×362	W40×362	W40×362

[a]HBE and VBE at the ith story are represented by HBE-i and VBE-i, respectively.

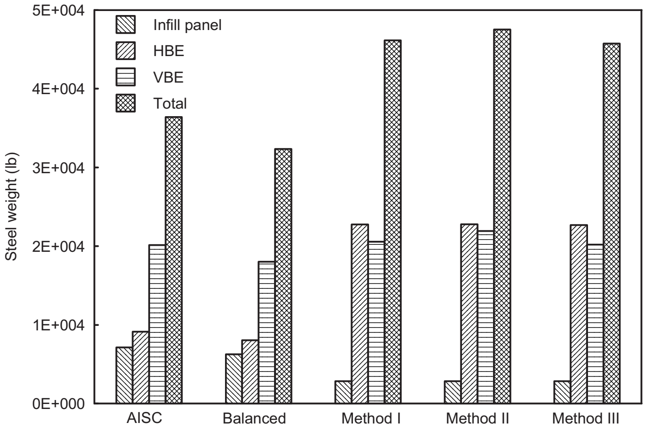

Fig. 15. Comparison of steel weight

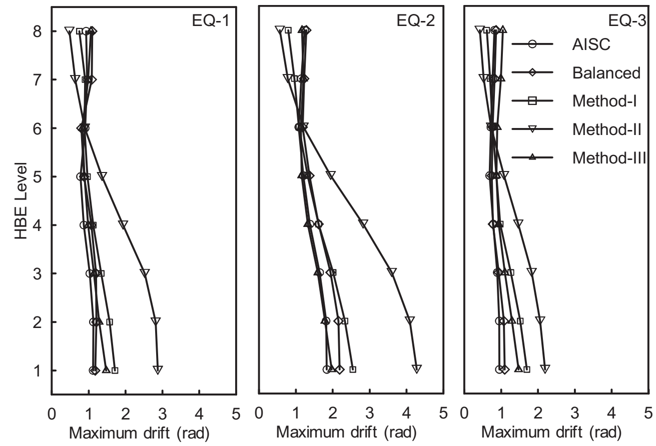

Fig. 17. Results from time history analyses

frame members from those five designs are listed in Table 2. Note that the three designs of the SPSW with weak infill panels had the same infill panels but different boundary frame members.

As shown in Table 2, the boundary frame members from the balanced design are smaller than those from the AISC code design because the former considers the lateral force resistance provided by the boundary frame moment resisting action. However, the boundary frame members are getting stronger at some stories of the designs using weak infill panels. This is because each weak infill panel was sized to resist only 40% of the story shear and the boundary frame members proportioned using capacity design principles (i.e., designed only to resist the infill panel yield forces) are not sufficient to resist the remaining 60% of the story shear and therefore those boundary frame members had to be strengthened. Also shown in Table 2, the HBE size distribution along the height of the SPSW, as expected, varies in the three designs using weak infill panels.

For comparison purposes, Fig. 15 illustrates the resulting steel weight for each design (also broken down in terms of each component). As shown, the balanced design is the most optimum in terms of the total weight of steel. Also shown in Fig. 15, for designs using weak infill panels, although the steel weight of infill panels decreases, the steel weight of boundary frame members increases for the reason presented earlier, resulting in an even higher value of the total steel weight in those designs.

Analytical Model and Artificial Ground Motions

To quantify the seismic performance of those SPSWs designed using different procedures, nonlinear time history analyses were conducted on models constructed following the dual strip procedure described and validated against cyclic test results (Qu et al. 2008). Note that in these models the infill steel plates were represented by two series of inclined tension members (i.e., strips) to replicate the behavior of SPSWs under cyclic loads. Three realizations of the target design spectra compatible ground motions were obtained using the computer program, TARSCTHS, by Papageorgiou et al. (1999) and were used as excitations for the nonlinear time history analyses. The dual strip model, ground motion realizations and acceleration spectra are shown in Fig. 16.

Result Comparison

The maximum drift of each story was obtained from the nonlinear time history analyses to compare the seismic performances of the five different SPSWs. Fig. 17 presents the corresponding results along the height of the walls for each considered earthquake.

As shown, the wall from the balanced design exhibits similar performance to that of the wall designed using the AISC Seismic Provisions. For example, the average of the maximum first-story drifts of the wall designed using the balanced design procedure for the three earthquake realizations is 1.48%, which is close to

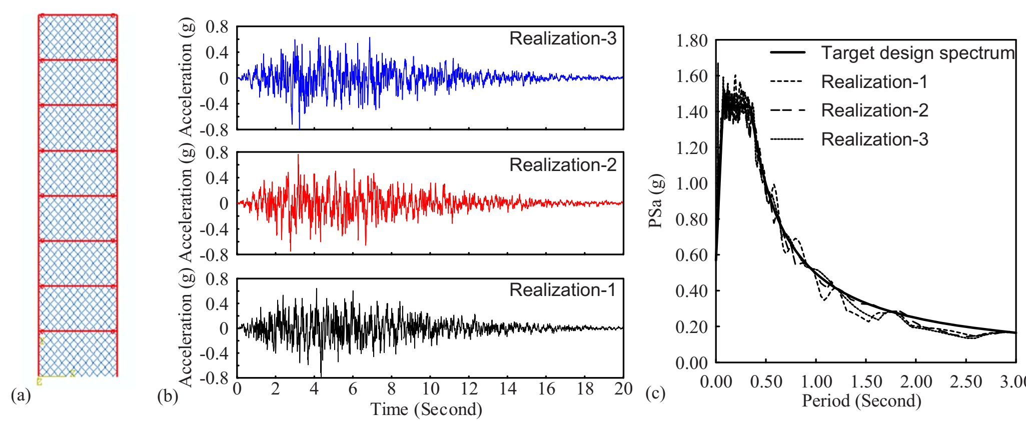

Fig. 16. Dual strip model and ground motion information: (a) dual strip model; (b) ground motion histories; and (c) pseudoacceleration spectra

the corresponding result of 1.30% from the wall designed using the AISC Seismic Provisions. For comparison purposes, the corresponding averages of the maximum first-story drifts the SPSWs having weak infill panels and designed using Methods I, II and III are calculated to be 1.98%, 3.12%, and 1.65%, respectively.

Another observation from Fig. 17 is that the maximum story drifts are distributed in a uniform pattern along the heights of all SPSWs except for the wall having weak infill panels and designed using Method II. As shown, significant deformations concentrated in the lower stories of that SPSW although smaller responses are observed in its upper stories.

Further Consideration

In SPSW design, the lateral loads are determined by code procedures, which reduce the maximum elastic demands by the response modification factor (typically expressed by the parameter R). Traditionally, the R factor has been implicitly tied to the hysteretic energy dissipation capabilities of the structural systems. As stated in FEMA 450, "…Structural systems with larger energy dissipation capacity have larger R_d values, and hence are assigned higher R values, resulting in design for lower forces, than systems with relatively limited energy dissipation capacity…" In other words, systems being more ductile are afforded larger R values while those with highly pinched hysteretic behaviors have been penalized by lower R values.

As discussed earlier, what is unique in SPSWs is the substantial overstrength of the boundary frame (HBEs and VBEs) introduced as a consequence of capacity design principles, combined to the fact that the infill plate can only yield once in tension. The overall behavior of SPSWs combines the behaviors of the moment resisting boundary frame and the infill panel system. At the extreme, if the boundary frame was noncontributing to lateral load resistance (as if simple connections were used), behavior would approach that of a tension-only system, a structural system not permitted in severe seismic regions in the United States and only permitted with a low R-factor in Canada. Hence, the SPSWs designed using different assumptions on the relative and respective strengths of the infills and boundary frames, which exhibit different inelastic responses, may result in different R values. While the previously presented case study is informative, it remains incomplete as it assumes that all SPSWs are designed for the same R value as typically done. Opportunity exists for future research to investigate what value of R should be used for those walls using different design assumptions and to provide greater insight as well as more definitive conclusions on the observations from the case study.

Given that some of the systems designed for the same lateral loads experienced greater story drifts, it is conceivable that it might be desirable to reduce the R factor of the system that exhibits much greater inelastic response at specific stories. Therefore, if the case study was repeated with those systems designed for lower R values as a consequence of their decreased capability to dissipate hysteretic energy, it is possible that the structural steel weights shown in Fig. 15 may not exhibit the same trends and that the apparent savings might be even eliminated.

In the past, R values have been assigned based on the judgment of code committee members on the respective and relative ability to dissipate energy of various structural systems as compared to each other. An effort is underway to develop a systematic and rigorous technical procedure to develop R factors, as documented in the ATC-63 90% Draft that can be downloaded from the website of the Applied Technology Council (FEMA 2008). The recommended methodology for reliably quantifying R values as described in the ATC-63 90% Draft is based on a review of relevant research on nonlinear response and collapse simulation, benchmarking studies of selected structural system, feedback from an expanded group of experts and potential users, and evaluations of additional structural systems to verify the technical soundness and applicability of the approach.

At this point, it would be a major undertaking beyond the scope of this work to conduct such a derivation of appropriate R factors. However, the work undertaken in this paper illustrates the technical issues that need to be resolved to establish to which degree the seismic loads applied to a SPSW could partly be resisted by the boundary frame surrounding the infill panels. While awaiting further results on such studies on R values, it is recommended for conservatism to continue designing SPSWs according to the AISC Seismic Provisions, i.e., to design the infill panels for 100% of the story shears calculated using the current system performance factors (specified per ASCE 7-05 or AISC 341-05, as applicable). Although the balanced design procedure derived in this paper could lead to a SPSW that exhibits satisfactory responses according to the limited analytical data provided here, such a procedure should not be used without further confirmation from experimental studies on the system performance of SPSWs.

Conclusions

This paper investigated the lateral load resistance of SPSWs respectively provided by the boundary frame moment resisting action and the infill panel tension field action. A design procedure considering the contributions of these two actions to the overall SPSW strength (i.e., a procedure to achieve a balanced design) was developed. The design of SPSWs having weak infill panels was also studied and three different methods that assume different HBE strength distributions along the SPSW height were developed.

A series of nonlinear time history analyses were conducted to evaluate the seismic performance of SPSWs designed using these different procedures (i.e., respectively designed using the AISC Seismic Provisions, using the developed balanced design procedure, and using the three methods for the SPSWs having weak infill panels). It was shown that the SPSW designed using the balanced design procedure exhibits similar performance to that designed using the AISC Seismic Provisions. Additionally, future work needed to provide greater insight on SPSW designs is outlined.

Acknowledgments

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Notation

The following symbols are used in this paper:

F_{Di} = lateral design force applied on the wall;

F_i = lateral force applied on the wall to develop the desired mechanism;

f_y = nominal yield strength of boundary frame;

f_{yp} = nominal yield strength of infill panels;
h = height of a single-story SPSW;
h_i = elevation of the ith story;
h_{si} = ith story height;
L = bay width of the wall;
M_{pl_i} = plastic moment at the left end of the ith HBE;
M_{pr_i} = plastic moments at the right end of the ith HBE;
n_s = number of stories;
R = response modification factor;
R_{yp} = ratio of expected to nominal yield strength of infill panels;
S_{D1} = design 1-s spectral ordinates;
S_{DS} = design short spectral ordinates;
t_w = thickness of the infill panel in a single-story SPSW;
t_{wi} = thickness of the infill panel at the ith story;
V_{design} = lateral design force applied on the wall;
V_i = expected strength of the infill panel at the ith story;
V_{ni} = nominal strength of the infill panel at the ith story;
V_p = plastic strength of the wall;
Z_{bi} = plastic section modulus of the ith HBE;
α = inclination angle of the tension field in a single-story SPSW;
α_i = inclination angle of the tension field at the ith story;
η = plastic section modulus reduction ratio accounting for the possible presence of RBS;
κ = percentage of the total lateral design force assigned to the infill panel;
$\kappa_{balanced}$ = value of κ for the balanced case;
Ω_κ = overstrength factor of the SPSWs;
ω_{xbi} = horizontal component of the infill tension fields along HBEs at the ith story; and
ω_{ybi} = vertical component of the infill tension fields along HBEs at the ith story.

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