Performance-Based Capacity Design of Steel Plate Shear Walls. I: Development Principles

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Abstract: This is Part I of two companion papers on performance-based capacity design of steel plate shear walls. Most previous research has been conducted with the primary aim of maximizing ductility and robustness under severe cyclic loading, without any explicit consideration of the costs of achieving this behavior. This has resulted in onerous capacity design rules in current codes and standards for achieving highly ductile systems, and has effectively discouraged their use in low and moderate seismic regions. These companion papers aim to provide a holistic and sound basis for capacity design to any of three explicit performance levels. In this paper, Part I, two target yield mechanisms associated with the two extreme performance levels (ductile and limited-ductility) are identified and justified, and the capacity design principles applicable to these performance levels are discussed. The limited-ductility mechanism departs from conventional treatment and is established based on finite element simulations and experimental observations. Two complementary new concepts for designing moderately ductile walls are also proposed and verified. Because design is an iterative process, modeling efficiencies for use with the performance-based approach are suggested and validated. Inconsistencies between current capacity design methods for evaluating the demands imposed by the infill plates on the boundary elements and the true infill plate behavior are identified and discussed. **DOI: 10.1061/(ASCE)ST.1943-541X.0001023.**

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Introduction

Steel plate shear walls (SPSWs) are a lateral load resisting system that has been developed to an advanced stage primarily based on research that focuses on its expected performance under severe earthquake loading. Although the system is undeniably well-suited for high seismic regions, its potential applications in zones of low and moderate seismicity, encompassing the majority of the North American continent, have largely been neglected. Research on SPSWs for high-seismic applications is focused on maximizing the system ductility and overall cyclic robustness by incorporating high-performance detailing, and the relatively high cost of the system is a direct outcome. However, by focusing instead on lower-cost details and construction economy, SPSWs suitable for low-seismic applications can be developed and their performance verified under the lower demands associated with these regions using a combination of physical tests and numerical simulations.

Comparing SPSW systems with the treatment of moment resisting frames (MRFs) in current design standards gives a perspective on where the former system stands in the evolution of its design provisions in North America. The Canadian Standards Association (CSA) steel design standard, S16-09 *Design of Steel Structures* (CSA 2009), hereafter referred to as S16, has adopted three performance levels for MRFs: Type D (ductile), Type MD (moderately ductile), and Type LD (limited-ductility). In the case of SPSWs, only two performance levels are recognized: Type D and Type LD. Each performance level is associated with a force modification factor used to reduce the seismic load effects to account for both the capability of the structure to dissipate seismic energy through stable inelastic response and the dependable overstrength. As such, this factor is defined as the product of two separate coefficients, R_d (ductility-related force modification factor) and R_o (overstrength-related force modification factor). Although not used explicitly in S16, for convenience in this paper the product of these two factors is denoted simply as R, rather than $R_d R_o$. ANSI/AISC 341-10 *Seismic Provisions for Structural Steel Buildings* (AISC 2010b), hereafter referred to as AISC 341, also provides for three different MRF performance levels: Special, Intermediate, and Ordinary. Conversely, only one performance level was adopted for SPSWs: Special. ASCE/SEI 7-10 *Minimum Design Loads for Buildings and Other Structures* (ASCE 2010), hereafter referred to as ASCE 7, defines the associated response modification factors, R, which also account for both inelastic system response and overstrength. Table 1 summarizes the R-factors specified currently by S16 and ASCE 7 for both the MRF and SPSW systems. (Note that the "conventional construction" category in S16 and seismic design categories B and C in ASCE 7 permit SPSWs to be designed to resist earthquake loading without rigorous adherence to capacity design requirements. In general, these systems can only ensure "very limited" ductility and are beyond the scope of the current research.)

The current S16 provisions (CSA 2009) for achieving $R_d = 2.0$ (Type LD) state that the requirements for $R_d = 5.0$ (Type D) must be met, with certain relaxations; i.e., these provisions were not developed independently for the $R_d = 2.0$ case. Therefore, new requirements—developed from the ground up to optimize designs for low-seismic regions—are needed for limited-ductility SPSWs that comply with the intent of the capacity design principles stated in S16 and AISC 341. With ductile and limited-ductility design provisions both available, a moderately ductile option can then be rationalized to achieve performance levels between the two

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Table 1. Current Seismic Response Modification Factors for SPSWs and MRFs

System	Performance level	ASCE 7-10		CSA S16-09			
		Designation	R	Designation	R_d	R_o	R
MRF	High	Special	8	Type D	5	1.5	7.5
	Medium	Intermediate	4.5	Type MD	3.5	1.5	5.25
	Low	Ordinary	3.5	Type LD	2	1.3	2.6
SPSW	High	Special	7 (8[a])	Type D	5	1.6	8
	Medium	Intermediate	—	Type MD	—	—	—
	Low	Ordinary	—	Type LD	2	1.5	3

[a]Dual system with Special MRF capable of resisting at least 25% of the prescribed seismic forces.

extremes. Under the resulting three-tier system (analogous to the current three-tier system for MRFs), several technical and economic benefits will accrue. Most importantly, it will give designers additional options in low and moderate seismic regions (versus braced frames, MRFs, concrete shear walls, etc.) and, in the same way that for economic reasons highly ductile MRFs are unlikely to be selected in low seismic zones, the lower ductility demands in these regions would make limited-ductility SPSWs economically superior to those designed to be highly ductile, while still exhibiting the required performance. Lower-ductility options also increase opportunities for utilizing SPSWs for seismic upgrades where parts of the existing structure do not themselves possess high ductility.

Scope, Objectives, and Roadmap to the Companion Papers

To achieve compliance with current seismic design provisions, the fabrication of SPSWs tends to be expensive due to the necessity of high-ductility and cyclically-robust connection detailing combined with requirements intended to meet capacity design objectives that include the prevention of any yielding in the columns above the base, thus making SPSWs generally uneconomical for low and moderate seismic regions. This research aims to develop reliable and economical performance-based capacity design methods to achieve different design goals—including for limited-ductility and moderately ductile walls that are suitable principally in low and moderate seismic regions—and to set them within the context of a three-tier framework that is capable of addressing SPSWs with a range of performance objectives. An additional objective is to meld the proposed performance-based design methods with simplified analysis techniques to create an efficient, but sufficiently accurate, design process.

The research is presented as two companion papers. The first (this paper) provides the background and theory necessary to establish reliable and economical performance-based design methods for SPSWs, the details of which are then developed from these concepts in the second paper. Because capacity design is a force method, and therefore does not explicitly provide deformation demands for individual system components, the three seismic performance levels considered are defined in terms of combinations of system ductility and system redundancy. The redundancy level is distinguished mainly by the beam-to-column connection type, while the ductility of a SPSW design is defined by the yield mechanism that eventually develops. As a result, a major part of the first paper is dedicated to establishing yield mechanisms for different performance levels and presenting the principal concepts necessary for the judicious application of capacity design tenets. A target yield mechanism concept for limited-ductility walls is proposed that departs from the usual capacity design treatment, and two new classifications of SPSWs designed for achieving moderate ductility are introduced.

The second paper (Moghimi and Driver 2014b) develops specific capacity design provisions for limited-ductility SPSWs based on observations from research specifically attuned to limited-ductility objectives. Having the limited-ductility wall provisions, and with the accumulated extensive knowledge about ductile walls from the literature, design provisions for moderately ductile walls are rationalized as an additional option for designers between the two extremes. The proposed design provisions for limited-ductility and moderately ductile walls are then applied to design examples and substantiated against experimental results. Finally, the paper characterizes and discusses implications of the proposed design and modeling approaches on the accuracy of the resulting boundary member design forces.

Literature Review

Capacity Design of SPSWs

Standard S16 (CSA 2009) stipulates that capacity design principles must be implemented in the seismic design of any structure with $R > 1.3$ (utilizing $R \leq 1.3$ is thereby interpreted as being essentially elastic design). It is assumed that the Type D SPSW system can develop significant inelastic deformation in its protected zones—infill plates, ends of beams, and column bases—where most attachments or discontinuities that can cause stress concentrations are prohibited. To ensure the development of reasonably uniform tension fields in the infill plates, minimum flexural stiffness requirements are provided for the columns, as well as the top and base (if present) beams. Type D SPSWs are assumed to constitute a dual system and the beam-to-column connections and the column joint panel zones must comply with the requirements for Type LD and Type D MRFs, respectively. The AISC 341 (AISC 2010b) provisions are also based upon capacity design philosophy and they stipulate requirements for special plate shear walls that are similar to those for Type D walls in S16. In general, the beam-to-column connections must comply with the ordinary moment frame requirements, and both the panel zones next to the top and base beams and the boundary member cross-sectional compactness must satisfy the special moment frame requirements. For Type D/Special SPSWs, S16 and AISC 341 both require that strong column–weak beam behavior be ensured in the boundary frame.

Design requirements for SPSWs with lower ductility are included in S16, but not in AISC 341. S16 introduced the Type LD provisions by adopting the capacity design requirements of Type D walls as a starting point, with a few relaxations as deemed appropriate by the committee. These relaxations include a reduction in the beam compactness requirements, permission to use other than rigid beam-to-column connections, and elimination of the dual-system requirement. However, the resulting provisions were not based on achieving any specific performance criteria.

Berman and Bruneau (2003) evaluated plastic collapse loads for SPSWs using the concepts of the strip model and plastic analysis. Two types of mechanism were considered for multi-story walls: soft story and uniform yielding of all infill plates and beam ends simultaneously. Based on the more desirable latter mechanism, Berman and Bruneau (2008) presented a detailed procedure for capacity design of columns in ductile SPSWs. The collapse lateral load is calculated based on a uniform mechanism, and a linear-elastic column model resting on linear springs (representing the beams) was proposed for evaluating the beam axial compressive loads due to the inward pull on the columns of the yielding infill plates. Each column is then designed for the actions caused by applying the lateral mechanism loads and internal forces to the column free body diagram.

Qu and Bruneau (2010) discussed the capacity design of intermediate beams of SPSW systems with reduced beam sections and moment connections. Various sources of axial force in the beams were identified, and for the portion arising from the column reaction (i.e., the reaction to the inward forces on the columns caused by the infill plate tension field) the method presented by Berman and Bruneau (2008) was used. Sources of shear force demand in the intermediate beams and moment demand at the faces of the columns were also discussed. A capacity design procedure for intermediate beams was presented.

Experimental Studies on SPSWs

Due to the extensive body of literature available on SPSWs, only research used directly in these companion papers is summarized. Previous research on SPSW systems has been reviewed comprehensively by Driver et al. (1997) and Sabelli and Bruneau (2006). Also, tests on SPSWs with simple beam-to-column connections are reviewed by Moghimi and Driver (2013).

Driver et al. (1998) tested a four-story SPSW with rigid beam-to-column connections under concurrent vertical column loads of 720 kN and cyclic lateral loads distributed equally to the four floor levels. The test specimen elevation and its normalized hysteresis curves are shown in Figs. 7(b) and 8, respectively, of the companion paper. The system was tested under increasing cyclic lateral displacement, and a total of 30 cycles—with 20 in the inelastic range—were applied. The first story yield displacement occurred during cycle 11 at the lateral deflection of $\delta_y = 8.5$ mm (corresponding to a drift ratio of 0.44%). The system achieved its maximum base shear of 3,080 kN in cycle 22, corresponding to a first-story displacement ductility ratio of 5 (drift ratio of 2.21%), and its lateral resistance then declined gradually to about 85% of the maximum value at a displacement ductility ratio of 9 (drift ratio of 3.97%) in cycle 30. The specimen showed a high initial stiffness, large energy dissipation capacity, and excellent ductility and redundancy.

Qu et al. (2008) performed a two-phase experiment on a two-story SPSW with composite floors and beams with reduced beam sections and rigid connections to the columns. Each story was 4.0 m high and the columns were spaced at 4.0 m center-to-center. The infill plate thicknesses for the first and second stories were 3.0 and 2.0 mm, respectively, in phase 1 of the experiment, and 3.2 and 2.3 mm in phase 2. In the first phase, the specimen was subjected to three pseudo-dynamic load histories. The specimen survived all the simulated ground motions with only moderate damage. In the second phase, the infill plates were replaced and the specimen first was subjected to another pseudo-dynamic loading sequence and then to quasi-static cyclic loading to failure. The specimen reached its maximum base shear in cycle 5 at the first-story drift ratio of 3.0%. In cycle 9, at the drift ratio of 3.3%, the bottom flange of the intermediate beam fractured at the face of a column and the connection of the infill plate to the adjacent boundary frame unzipped throughout the remainder of the test.

Moghimi and Driver (2013) tested a two-story SPSW with simple beam-to-column connections and a modular construction scheme under cyclic displacement concurrent with gravity column loads of 600 kN. The test specimen elevation and its normalized hysteresis curves are shown in Figs. 7(a) and 8, respectively, of the companion paper. Standard double-angle beam-to-column shear connections were used to provide rotational flexibility at the joint. As a key component of the modular concept, the infill plates in both stories were spliced horizontally at mid-height with a bolted single-sided lap plate of the same thickness as the infill plates. A lateral load ratio for the two levels was selected for the cyclic test to represent the first mode in two central stories of a multi-story building. In cycle 8, the specimen reached the first-story yield displacement of $\delta_y = 12$ mm (corresponding to a drift ratio of 0.65%). The specimen reached its maximum base shear of 2,625 kN in cycle 19, corresponding to a displacement ductility ratio of 5 (drift ratio of 3.25%). The specimen demonstrated very good performance and energy dissipation capacity under 25 cycles of loading. The shear connections showed only nominal plastic deformation at the end of the test and their flexibility reduced the moment demand on the columns compared to rigid connections.

Yield Mechanisms

Lateral loads on conventional SPSWs are resisted by a combination of tension field action in the infill plates and frame action of the boundary members. The infill plates are the primary elements for dissipating seismic energy; however, the surrounding frame also undergoes inelastic behavior and must support the gravity loads throughout the seismic event for any performance level. As such, the yield mechanism of a SPSW develops in part by tension yielding of the infill plates, but it is mostly the inelastic behavior of the boundary frame that determines the seismic performance level of the system. Tests on different multi-story SPSW systems [e.g., Driver et al. (1998); Moghimi and Driver (2013); Qu et al. (2008)] have shown that distinctly different yield mechanisms are conceivable for ductile and limited-ductility SPSWs, while still providing satisfactory performance for the design objectives.

The performance level of a seismic system is often defined by the deformation limits under the deformation-controlled actions on its components, while the strength capacities under force–controlled actions are treated essentially the same for all performance levels. Because capacity design is in effect a force method, and therefore does not directly provide the deformation demands for the individual components of the system, the seismic performance level is instead defined in terms of the system ductility and redundancy. The ductility of a SPSW design is influenced by the yield mechanism that develops, and the redundancy level is distinguished mainly by the beam-to-column connection type. Using this approach, the "ductile" performance level is assigned to SPSWs that possess both high system ductility and the redundancy enabled through rigidly connected ductile frame joints. The "limited-ductility" performance level is assigned to SPSWs that are permitted to develop a less ductile yield mechanism and exhibit reduced redundancy through the use of ductile shear connections at the frame joints. The "moderately ductile" performance level is assigned to hybrids of these two extremes, where the SPSW is designed for either the higher-ductility yield mechanism and the lower redundancy, or the lower-ductility yield mechanism and the higher redundancy. Table 2 summarizes the distinguishing features among the three performance levels, which are discussed subsequently and in the companion paper.

Ductile SPSW System

In ductile walls, the columns are designed according to capacity design procedures to remain elastic above the base under any potential seismic loading, and the resulting yield mechanism of the system is ideally similar to that shown in Fig. 1(a) for a typical four-story wall. The figure shows a uniform mechanism wherein all infill plates are fully yielded, along with the formation of plastic hinges at the column bases and all beam ends. With minor modifications to the assumptions of Berman and Bruneau (2003), the

Table 2. Comparison of Proposed SPSW Performance Levels

Criterion	Limited-ductility	Moderately ductile		Ductile
Redundancy				
Frame joints	Simple	Simple	Rigid	Rigid
Ductility				
Yield mechanism	Partial	Uniform	Partial	Uniform
Infill plate yield stress	$R_y F_{yw}/1.1$	$R_y F_{yw}$	$R_y F_{yw}/1.1$	$R_y F_{yw}$
Partial yield in column	Allowed	Not allowed	Allowed	Not allowed

system yield mechanism load of a SPSW can be calculated by equating external to internal virtual work, as follows:

$$\sum_{i=1}^{n} F_i H_i = \sum_{i=1}^{n} 0.5(w_i - w_{i+1}) R_y F_{ywi} L_c H_i \sin(2\alpha_i) + \beta \sum_{i=1}^{n} 1.1 R_y (M^*_{pbLi} + M^*_{pbRi}) + 1.1 R_y (M^*_{pcL} + M^*_{pcR}) \qquad (1)$$

where the subscript i represents the ith story; subscripts L and R indicate left and right, respectively; n is the total number of stories; F_i is the system yield mechanism force at each level; w and F_{yw} are the infill plate thickness and nominal yield stress, respectively; H_i is the height of the beam of each story from the base; L_c is the clear distance between columns; and α_i is the tension field angle from vertical. M^*_{pb} and M^*_{pc} are the nominal plastic moment capacities of the beam ends and column bases, respectively, where the superscript * indicates a capacity reduction to account for the effect of axial force in the member. R_y is the ratio of expected-to-nominal yield stress in the associated element, and the coefficient 1.1 represents the effect of material strain hardening at the point when the complete mechanism forms. For any given lateral load distribution over the height of the wall, Eq. (1) renders the system yield mechanism force at each story, F_i. The coefficient β, which is a positive variable less than or equal to unity, accounts for the fact that it is unlikely that all beam ends in the system develop plastic hinges under the design event, as discussed below for moderately ductile SPSWs. Taking β equal to unity results in upper-bound values for the yield mechanism forces and seems appropriate for use with ductile walls. Because the infill plates are not expected to undergo significant strain hardening, even at large wall displacements, the strain hardening factor is not applied to the infill plate yield force. Also, Eq. (1) assumes that all infill plates along the height of the wall are yielded, and for tall walls this tends to be a conservative assumption in this application because higher mode effects make it unlikely that all the infill plates are yielded simultaneously. However, more research is required if any reduction factor (similar to the factor β for beams) is to be applied to the infill plate forces.

Limited-Ductility SPSW System

Limited-ductility walls are expected to provide lower levels of inelastic deformation capacity compared with ductile walls. This reduced ductility, along with reduced redundancy due to the use of simple connections at the frame joints, results in larger seismic design forces because of the lower associated R-factor (primarily due to the component R_d). Therefore, a thicker infill plate—which is the main source of lateral force capacity of the SPSW system—is required compared to that in a ductile wall in a similar seismic region. The relatively thick infill plates in limited-ductility walls with simple frame connections tend to disrupt the yield mechanism of the system from the ideal pattern of Fig. 1(a) toward that of Fig. 1(b), if minor yielding is permitted in specific regions of the compression column and it is assumed that only partial tensile yielding develops in some of the infill plates. This latter phenomenon is particularly relevant when the same infill plate thickness is used in multiple stories, as would commonly be the case for economy. Nevertheless, in principle this modified pattern should be acceptable due to the greatly reduced ductility demands compared with those of a ductile wall design. While the beam-to-column joints are flexible in limited-ductility SPSWs, hinges are not explicitly shown in Fig. 1(b) to reflect the fact that rotations at these locations tend to be small.

Two scenarios contribute to the modification of the yield pattern for the limited-ductility case toward that represented by Fig. 1(b), which was observed in both the numerical analyses presented herein for the limited-ductility cases and in the associated physical test specimen (Moghimi and Driver 2013). First, the relatively thick infill plates impose internal force demands on the surrounding frame as the yield mechanism condition is approached and they also tend to restrain the free rotation of the simple frame connections. These behaviors are liable to change the yield pattern of the system so that limited yielding takes place locally where the maximum shear demand occurs in the frame, as depicted in Fig. 2(a). The existence of a large shear force simultaneously with the axial force in the compression column (which is larger than the axial force in the tension column in the same story), and in the beam

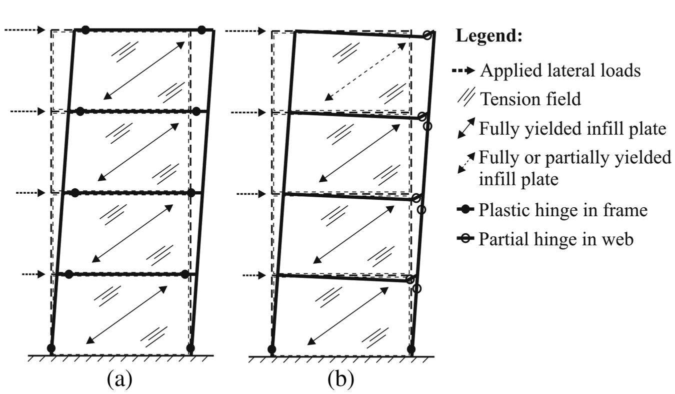

Fig. 1. Yield mechanisms: (a) uniform yield pattern for ductile wall; (b) partial yield pattern for limited-ductility wall

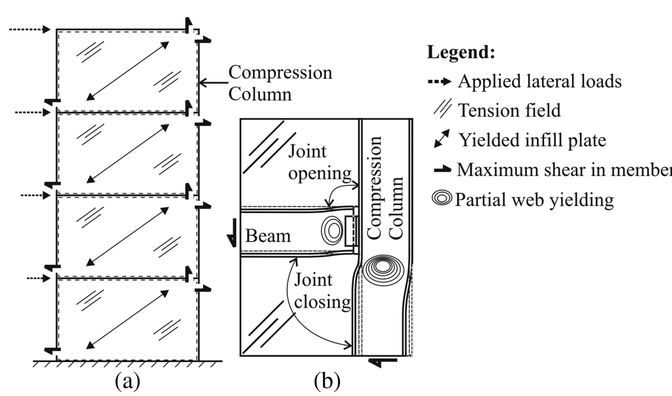

Fig. 2. (a) Locations of maximum shear forces in frame; (b) beam-to-compression-column joint yielding for limited-ductility wall

end adjacent to this column, can cause partial web yielding in the corresponding frame member, as shown schematically in Fig. 2(b). Depending on the relative sizes of the frame members and the infill plate thickness, either column or beam partial web yielding could occur. As such, a yield pattern similar to that shown in Fig. 1(b) forms, with only small rotations occurring at the beam-to-column connections themselves.

The second, and perhaps more significant, scenario that contributes to the formation of the modified yield pattern of Fig. 1(b) occurs when an infill plate at a given story experiences only partial yielding, while the infill plate in the story below is fully yielded. This would be common in multi-story walls when either the column flexural stiffness is insufficient to ensure a uniform tension field in the upper infill plate or the infill plate thickness distribution over the wall height is such that the story shear distribution cannot yield the plates at every story. To illustrate this concept, Fig. 3(a) shows a typical two-story wall with the same infill plate thickness in each story. While the base shear is large enough to yield the infill plate relatively uniformly in the first story, the second story shear is only sufficient to yield parts of the associated infill plate. Therefore, the plastic deformation of the second story is much smaller than that of the first story, as shown in Fig. 3(b), and the second story infill plate in effect restrains the lateral deflection of the compression column. The dashed column outline in Fig. 3(b) represents the position the compression column would take if the second story infill plate were yielded fully. To account for the restraint afforded by the partially elastic infill plate, a reduction in lateral deflection at the top of the second story, Δ^*, is experienced. This action bends the compression column against the first-story beam, causing partial yielding in the column immediately below the frame joint (both the shear and axial forces are higher below the joint than above), and also in the beam web adjacent to the connection where the reactive force opposing this action is concentrated. This scenario contributes to the localized yield regions shown in Fig. 2(b) and in the compression column it consists primarily of web yielding and minor internal-flange flexural yielding. The formation of this type of partial yielding pattern in the column was observed during a physical test on a two-story SPSW specimen (Moghimi and Driver 2013) designed specifically for limited-ductility applications. The test demonstrated that the partial yielding in the frame members poses no threat to the reliability of the SPSW system, even at critical (first) story drift ratios up to 5.2% (corresponding to a roof drift ratio of 3.7%), which represents a story displacement ductility ratio of 8 (roof ductility ratio of 6.2), as the wall maintained a capacity of 70% of the maximum base shear. The wall achieved its maximum base shear capacity at a story displacement ductility ratio of 5 (roof displacement ductility ratio of 4.4), which is well beyond what is normally required of limited-ductility—and even moderately ductile—seismic systems.

While a yield mechanism similar to the one depicted in Fig. 1(a) could be assumed by again enforcing elastic column behavior during design, the significantly reduced ductility demands placed on limited-ductility SPSWs permit the spirit of capacity design to be upheld, while accounting explicitly for the somewhat lower mean infill plate stresses consistent with the yield mechanism shown in Fig. 1(b). If this philosophy is taken, the yield mechanism lateral forces for a limited-ductility SPSW with simple connections are reasonably estimated as follows:

$$\sum_{i=1}^{n} F_i H_i = \frac{1}{1.1}\sum_{i=1}^{n} 0.5(w_i - w_{i+1}) R_y F_{ywi} L_c H_i \sin(2\alpha_i) + 1.1 R_y (M^*_{pcL} + M^*_{pcR}) \tag{2}$$

It is noted that in Eq. (2), about 90% of the expected infill plate yield stress is used (via the coefficient 1/1.1), which may appear contrary to conventional capacity design philosophy. The reasons for and implications of this are discussed later in the paper.

Moderately Ductile SPSW System

Although the moderately ductile SPSW system is a new concept, clearly these walls need a combination of ductility and redundancy that is lower than in ductile walls and higher than in limited-ductility walls. Based on the ductile and limited-ductility systems discussed above, two different moderately ductile system concepts are envisioned.

The first moderately ductile SPSW concept has a redundancy consistent with limited-ductility walls and relies on the high-ductility design philosophy for the columns. By utilizing simple beam-to-column connections and designing for the uniform yield mechanism [Fig. 1(a)], a moderately ductile wall can be achieved. In effect, real hinges exist at both ends of each beam, and when the mechanism loads are applied, plastic hinges form only at the bases of columns and the infill plates are fully yielded.

The second moderately ductile SPSW concept incorporates greater redundancy than the first, but permits limited yielding in the columns above the base. That is, rigid beam-to-column connections are incorporated, but the partial yield mechanism [Fig. 1(b)] is permitted. Under this mechanism, the connections experience relatively small rotations and, as a result, the strain hardening factor need not be applied when calculating the beam plastic moment. As such, the coefficient 1.1 is omitted from the evaluation of

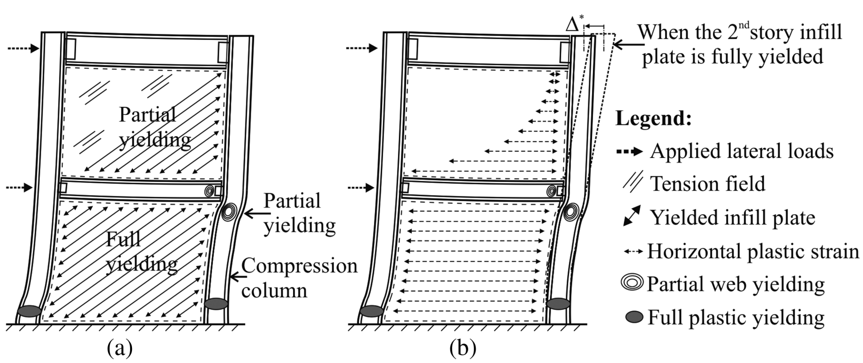

Fig. 3. Non-uniform infill plate yield pattern in a two-story limited-ductility wall: (a) yielded tension field distribution; (b) horizontal component of plastic strain distribution

the beam plastic moments in Eq. (1). Moreover, as Fig. 1(b) shows, the partial yield mechanism causes the beam to develop a plastic hinge at the compression-column end, while the end connected to the tension column tends to undergo far less yielding than when the system is designed for the uniform mechanism. This response was observed in many finite element pushover analyses of a variety of walls conducted as part of this research. The numerical model geometries include half-scale wall (Driver et al. 1997) and full-scale wall proportions (design example in the companion paper) with different boundary frame cross-section sizes and infill plate thicknesses, and with rigid beam-to-column connections. The material models are described by Moghimi and Driver (2014a). As such, in Eq. (1) only the right connection plastic moment needs to be considered. Alternatively, both plastic moments can be included, but the factor β can reasonably be selected as 0.5. This latter method facilitates the design calculations, while providing a good approximation of the actual behavior.

Performance-Based Capacity Design Approach for SPSWs

The effects of gravity load are treated the same for all performance levels, and hereafter are not mentioned unless required to underscore specific points. These effects can be superimposed onto the effects of the seismic loads described below. The terms "tension column" and "compression column", based on seismic lateral loading effects, are used throughout these companion papers for expediency, although with the addition of gravity loads both columns could ultimately be in compression. Moreover, these terms are used to describe specific behaviours, even though they belie the fact that each column will act as the tension column and the compression column at different points in time during an earthquake.

To ensure that the SPSW system is able to develop the full tensile yield capacity of the infill plates, the seismic design loads are replaced with yield mechanism forces, F_i, as demonstrated in Fig. 4(a) for an n-story building. The figure also highlights the yielded infill plate diagonal tension fields in stories i and $i+1$, below and above beam i, which can readily be decomposed into their component vertical and horizontal uniformly distributed forces on the adjacent columns and beam, as shown in Figs. 4(b and c), respectively. The inclination angle, α_i, can be estimated based on the provisions of standard S16 or AISC 341.

To evaluate the yield mechanism forces, F_i, the lateral load distribution over the wall height, which is defined by the coefficients μ_i in Fig. 4(a), is assumed. In this regard, a lateral load pattern similar to that of the seismic design loads from the appropriate building code [e.g., ASCE 2010; National Research Council of Canada (NRCC) 2010] or the first mode distribution (ASCE 2007) can be selected. By setting $F_i = \mu_i F_s$, Eq. (1) or (2), as appropriate, then returns the total base shear, F_s. The yield mechanism force at each story, F_i, is then distributed to each side of the wall as in Figs. 4(b and c) such that $F_i = F_{iL} + F_{iR}$. (The selection of appropriate force components, F_{iL} and F_{iR}, is discussed below.) For any performance level, every non-fuse element of the system is designed to resist the expected tensile yield stress in the infill plates and, if rigid frame connections are present, the expected plastic moment capacity of the beams at their anticipated hinge locations and the resulting shears, while the system is subjected to the yield mechanism forces.

The boundary frame flexibility limits, as stated in the relevant design standard, should be satisfied regardless of the performance level to prevent excessive pull-in of the boundary members as the mechanism load is approached. Other parts of the SPSW are designed differently based on the target seismic performance level, as explained in the companion paper.

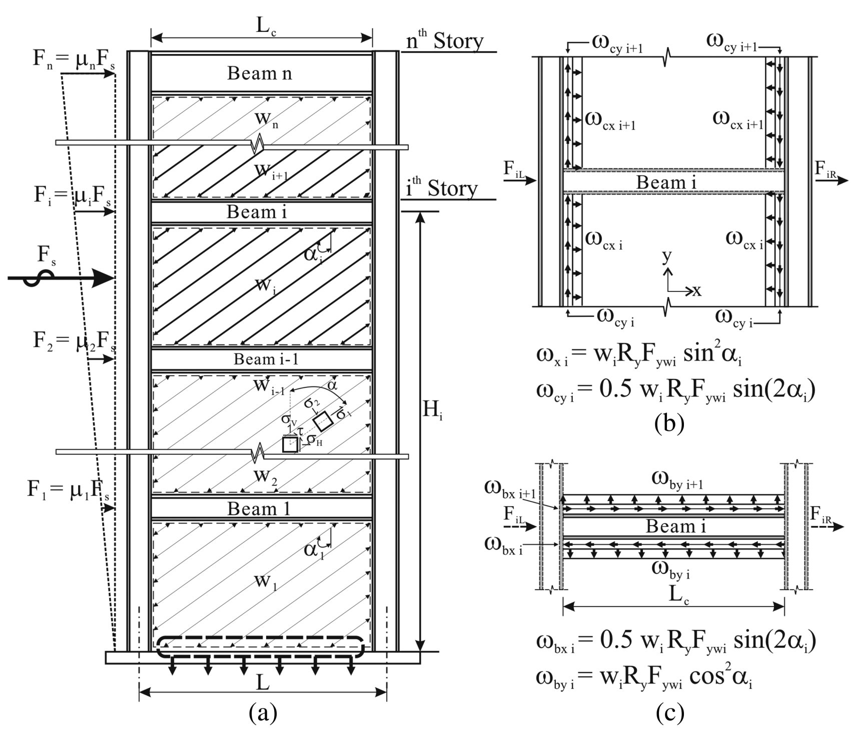

Fig. 4. (a) System yield mechanism loads of a multi-story SPSW with simple or rigid frame joints; (b) full tensile yield forces on columns in stories i and $i+1$; (c) full tensile yield forces on ith story beam

Applied Forces from Infill Plates on Boundary Frames

The stress state in the infill plates of SPSWs at the ultimate capacity of the system is considerably more complex than is typically assumed in design. It is influenced by the stiffness of the boundary elements, rigidity of the frame connections, panel aspect ratio, continuity of the columns from story to story, distribution of the lateral loads over the height of the wall, demand-to-capacity ratio of the panel—as well as the panels above and below—and thickness of the infill plate itself. Not only does the stress field vary in character over the infill plate surface and through its thickness, the principal stress orientation also varies along the lengths of the boundary frame elements. A typical stress field taken at the mid-surface of the infill plate of a SPSW under the lateral mechanism load is depicted in Fig. 5, showing the variation of the principal stress vectors in terms of both magnitude and orientation. While the variations over the entire panel area may appear dramatic, fortunately several key aspects can be identified to assist in interpreting the consequences of using conventional analytical idealizations.

In the analysis and design of SPSW systems, the infill plate is often replaced by a series of parallel tension strips oriented in the direction of the major principal stress in the plate [see Fig. 4(a), where σ_1 and σ_2 are the major and minor principal stresses, respectively]. The discretized "tension strip analogy" implies that the minor principal stress, $\sigma_2 = 0$. As a result, any yield criterion—such as the von Mises criterion—is satisfied only when σ_1 reaches the uniaxial yield stress. This assumption is reasonable for the central region of the infill plate, which is susceptible to buckling under a small compressive force that typically does not exceed about 5% of the yield stress. As such, it tends to provide a good estimation of the lateral shear capacity of the overall SPSW system. However, adjacent to the boundary frame members, the analogy breaks down because of the creation of a stiffened band around the periphery of the panel that is far less susceptible to instability. In fact, as supported by Table 6 of the companion paper, in these regions considerable compressive principal stresses develop such that when the mechanism load is reached the mean principal stress ratio along each boundary member, $\psi = |\sigma_2/\sigma_1|$, tends to be on the order of 0.2–0.3 (usually close to 0.3) for beams and 0.3–0.5 (usually close to 0.4) for columns, depending the wall configuration, applied load over the height of the wall, etc.

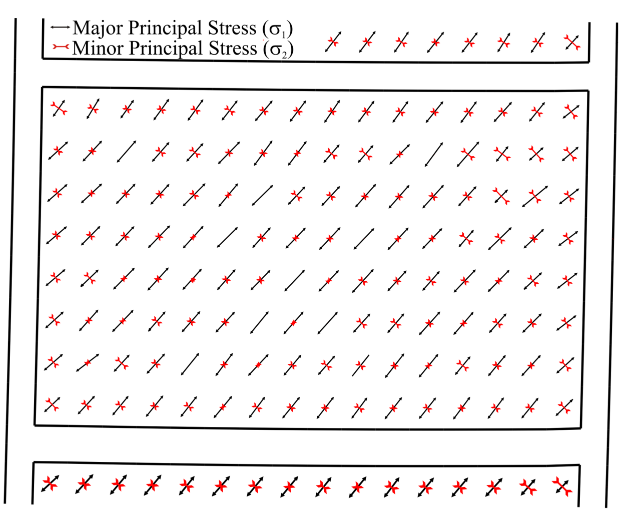

Fig. 5. Major and minor principal stress distribution in a typical infill plate

The phenomenon described above has three major implications for determining accurate applied forces (stresses) on the boundary frame members that are consistent with capacity design principles. First, the non-zero value of ψ results in earlier yielding of the infill plate around the boundary frame than is predicted by the tension strip analogy. For instance, by applying the von Mises yield criterion the acceleration of yielding can be described as follows:

$$\sigma_1/\sigma_y = (1 + \psi + \psi^2)^{-1/2} \tag{3}$$

where σ_y is the uniaxial (tension strip) yield stress. Local yielding therefore occurs when σ_1 reaches only $0.85\sigma_y$ or $0.8\sigma_y$ for ψ ratios equal to 0.3 (beams) or 0.4 (columns), respectively. Second, the presence of σ_2 increases the stresses applied parallel to the boundary member and decreases those perpendicular (for both beams and columns). Third, the angle from vertical, α of the major principal stress deviates from the codified value (S16 and AISC 341), which provides a good approximation of the average value only for the middle region of the infill plate. Table 6 of the companion paper (and other numerical studies by the authors to be reported in a future publication) indicates that for common SPSW configurations designed based on capacity design principles, the mean value of the angle α at the ultimate capacity of the wall tends to be close to 39° and 51° adjacent to the beam and compression column, respectively (considering analogous stress states adjacent to the beam and column, the two values of α constitute complementary angles).

By and large, the tension strip analogy provides conservative capacity design forces for both the beams and columns, as explained subsequently, but the degree of conservatism varies and the simplifications of the method rely on certain compensating factors. The codified value of α tends to be fairly close to the actual mean value adjacent to the beams at the system mechanism load; however, because σ_2 is neglected the axial stresses applied to the beam are slightly underestimated and the transverse stresses overestimated. The underestimation of the axial stresses is mitigated by the slight acceleration of yielding (causing σ_1 to be smaller than the uniaxial yield stress) due to the presence of σ_2, as defined by Eq. (3), although this same phenomenon increases the conservatism of the transverse design stresses on the beam. In general, the tension strip analogy provides acceptable and conservative results for the beam, especially considering the fact that the net demands on the beam are derived from the differences in stresses in the infill plates above and below. However, the method can be quite conservative for the top beam, where there is no infill plate above to alleviate the overestimates of shear and moment. The inability of the system to yield the top infill plate under the mechanism loads, as would likely occur if the plate in this story is thicker than required to resist its story shear, increases this conservatism further under capacity design methods. However, it should be noted that proper performance of the top beam is a fundamental parameter for the effective development of tension field action and achieving the required dynamic performance of the overall SPSW system. As such, a higher degree of conservatism is acceptable here.

For the compression column, the same phenomena as those described above for the beams cause the axial force to be underestimated and the shear force and bending moment to be overestimated at each story. However, the axial design force in the compression column in general would be conservative because, as shown in the companion paper, a considerable portion of column axial compression comes from the shear reaction of the beam, which is itself overestimated by the tension strip analogy, as discussed in the previous paragraph. In total, the axial compression in the column tends to be moderately conservative (depending on the wall aspect ratio and number of stories), while the design shear and bending moment at each story are highly conservative due mainly to the σ_2 effect that both accelerates yielding of the material and reduces the transverse stresses on the column. This conservatism is only slightly mitigated by the underestimate of the angle α adjacent to the column. The composite effects of these phenomena are demonstrated numerically in the design examples in the companion paper.

Design of Beams and their Connections

The beams in a SPSW are designed to resist forces due to tensile yielding of the infill plates and the external seismic loads. When rigid connections are used, the shear and moments from frame action also contribute to the design forces. As such, three design actions—axial force, shear, and moment—are applied to the beam. Because the moment demand is easy to evaluate from the shear force distribution, only the effects of axial and shear forces are discussed below.

Evaluation of Axial Forces

The beam axial design force can be evaluated by independently considering three constituents: the change in the axial force between the beam ends, and the magnitudes of the axial forces at each end. Each of these constituents itself can be induced from different sources. When the infill plate thicknesses above and below an intermediate beam differ, there is a distributed unbalanced force on the beam. The horizontal component of the unbalanced forces [the difference between ω_{bxi} and ω_{bxi+1} in Fig. 4(c)] causes a change in axial force along the beam's length. Because the lower-story infill plate would not normally be thinner than that in the upper story, this effect imposes tension at the compression-column end and compression at the tension-column end of the beam. In a case where the infill plates above and below an intermediate beam have the same thickness (and material grade), this unbalanced force would be negligible. The yielded infill plates also apply distributed inward forces on both columns [ω_{cxi} and ω_{cxi+1} in Fig. 4(b)], inducing a uniform compressive force in the beam. In other words, in a laterally loaded SPSW the columns are pulled toward each other by the internal forces in the infill plates, and the beams act as struts that keep the columns apart.

The seismic design load distributions in the floor and roof diaphragms, and the means of transferring these loads to the SPSW, also influence the axial force distribution in the beams, primarily by determining the proportions of the forces F_{iL} and F_{iR} in Fig. 4(c). In a case where the seismic design load at a floor is distributed equally to each side of the wall, the beam can be designed conservatively for the compressive force at the tension-column side. However, depending on the layout of the building plan and the means of tying the diaphragm to the SPSW, the seismic design loads can be transmitted into the SPSW in different ways, as shown schematically in Fig. 6. This figure shows two-story SPSWs with four possible diaphragm load transfer mechanisms, and the corresponding axial force distributions in both beams. In order to highlight the effect of the diaphragm force transfer mechanism, the same infill plate thickness is assumed for both panels. As such, the horizontal component of the unbalanced infill plate force on the intermediate beam is zero, while it is a significant contributor to the axial force in the top beam. Considering the fact that the inward reactive forces from the columns induce a uniform compression in the beams, the differences in the axial force distributions of the intermediate beams in Fig. 6 are associated exclusively with the means of load transfer from the diaphragms.

In a real design case, the SPSW would be subjected to some combination of the lateral load distributions in Figs. 6(a–d). For instance, in the case of a SPSW perpendicular and adjacent to the edge of a building, a combination of distributions [(a) and (d)] or [(b) and (d)] would likely occur, while for a system at the middle of the building plan, a combination of distributions (c) and (d) would exist. As such, a designer must consider possible combinations, and design the beam for the envelope of potential axial force demands, as any combination could lead to the maximum tension or compression in the beam. It is important to note that although the seismic load transfer pattern from the diaphragms to the SPSW has a considerable effect on the axial force demands in the beams, its effects on other forces and deformations—such as shear and moment in the beams, internal forces in the columns, deformed shape and yielding pattern, and pushover response of the entire wall—are typically small. As such, it generally only needs to be considered in the design of the beams and their connections (in addition to the diaphragm-force tributary members and their connections, as needed).

Based on extensive numerical studies, Moghimi and Driver (2014a) proposed a method to evaluate the design axial forces in beams of SPSWs with simple beam-to-column connections, and verified the method against experimental results. This method builds on the observation that the shear and moment distributions in the compression column vary with SPSW geometry far less than those in the tension column. It also makes use of the foregoing considerations regarding the mechanism load distribution over the height of the wall and the lateral load transfer mechanism from the diaphragms. The axial forces applied to the intermediate beams by the compression column (i.e., the sum of the shear forces in the column above and below each beam-to-column joint) were found to be 70 ~ 100% (100 ~ 125% for two-story walls) of the horizontal component of the tributary infill plate yield forces (considering a half-story tributary width above and below the beam for simplicity) and the force applied to the top beam was 50 ~ 90% of the horizontal component of the tributary infill plate yield force applied to the top-story column. The variations in the forces transferred to the beam occur mainly because of differences in the relative lateral

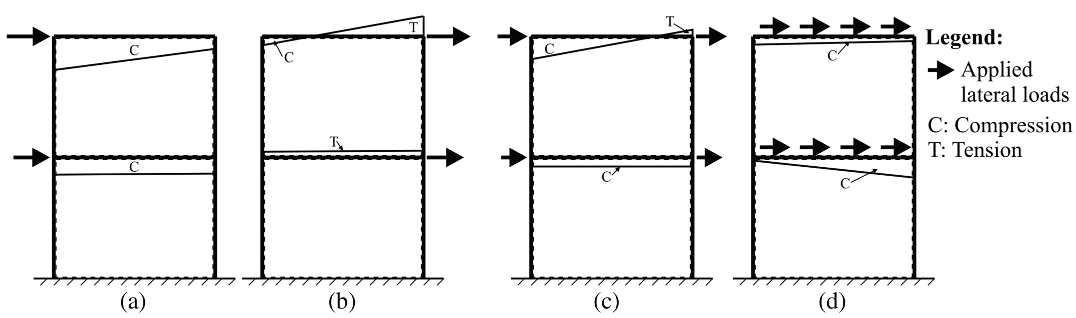

Fig. 6. Effect of lateral load transfer pattern from floor and roof diaphragms on beam axial forces

story deflections and the occurrence of incomplete yielding in some panels, which are a function of infill plate thickness and lateral load distribution over the wall height, as discussed in relation to Fig. 3. Regardless of the diaphragm load transfer mechanism, the maximum compression in each beam occurs when the force transferred from the compression column is maximum [i.e., using the 100% (125% for two-story walls) and 90% factors for the intermediate and top beams, respectively] and the maximum tension happens when the force is minimum [i.e., using the 70% (100% for two-story walls) and 50% factors, respectively]. Although it is recommended that both the maximum tension and compression cases be checked, in most instances the compression case governs the beam design because a portion of the tensile force tends to be transmitted to the surrounding frame through the infill plates. Due to the large beam normally required at the top of the wall to provide adequate flexural stiffness for anchoring the infill plate tension field below and the relatively low internal axial force compared to the those in the intermediate beams, the axial force may not have a significant influence on the top beam selected. Having evaluated both the net axial force applied to the beam by the compression column and the collapse mechanism force at each beam-to-compression-column joint, the axial force at the beam end (and in the adjacent connection) is calculated from the free body diagram of the joint by subtracting the force induced by the column from the mechanism force (refer to Moghimi and Driver 2014b, Fig. 1). The axial force demand at the other end of the beam (adjacent to the tension column) can then be evaluated by subtracting the horizontal component of the unbalanced infill plate tensile yield force from the beam axial force demand at the compression-column end. While the system is actually highly indeterminate, this simple method provides reasonable axial forces for designing the beams of SPSWs with simple frame connections.

For cases with rigid beam-to-column connections, the commentary to AISC 341 recommends the method "combined plastic and linear analysis", originally developed primarily to evaluate the design actions on the columns (Berman and Bruneau 2008). The method does not consider the lateral load transfer mechanism from the diaphragm to the wall, and as a result may need to be modified accordingly under certain circumstances (Moghimi and Driver 2014a). In the case where the lateral loads are distributed equally to the left and right sides of the wall [i.e., the case of Fig. 6(c)], the results obtained for the axial force in the beam would be similar to those from the proposed method above. In such a case, the compressive force in each beam from the column reaction is approximately equal to the infill plate yield forces on the columns above and below the beam based on tributary widths (i.e., using a factor of 100% for both the intermediate and top beams, as described in the method for SPSWs with simple frame connections). If this (compressive) force is taken as positive, the total beam axial forces at the tension-column and compression-column ends are evaluated, respectively, by adding to or subtracting from this force one-half of the horizontal component of the unbalanced infill plate tensile yield force. As mentioned earlier, the lateral load transfer pattern from the diaphragms mainly affects the axial force demand in the beams. As a result, regardless of the real diaphragm force transfer mechanism, the "combined plastic and linear analysis" method, which assumes the lateral loads to be distributed equally between the left and right sides of the SPSW, is expected to provide reasonable estimates of the column design forces.

Evaluation of Shear Forces

The shear forces in SPSW beams are induced by the tension field action in the infill plates combined with frame action, if present. Fig. 7(a) shows the net actions at the centerline of a beam with simple beam-to-column connections. The associated shear and moment diagrams are also shown, where the shear reaction forces at the faces of the left and right columns are V_{bL} and V_{bR}, respectively. The net actions shown in Fig. 7(a) can be separated into their constituents shown in Figs. 7(b and c). First, as shown in Fig. 7(b), the vertical component of the unbalanced infill plate tensile yield forces on the beam ($\Delta\omega_{by}$) causes internal shear and moment along the beam's length, resulting in a shear reaction of $V_{Ib} = \Delta\omega_{by}L_c/2$ at the face of each column (the effect of the horizontal component, $\Delta\omega_{bx}$, was discussed in the previous section). Second, as discussed by Qu and Bruneau (2010), the horizontal components of the tension field forces above and below each beam apply a distributed moment, $m_b = d_b(\omega_{bxi+1} + \omega_{bxi})/2$, to the beam, where d_b is the depth of the beam. In simply supported beams, this results in a constant shear force in the beam ($V_{Mb} = m_b$), but no internal moment, as indicated in Fig. 7(c). The net beam shear distribution in Fig. 7(a) is determined by superimposing the distributions in Figs. 7(b and c). When the infill plates above and below an intermediate beam differ, the beam shear is non-uniform with its maximum magnitude at the compression-column side, as shown in Fig. 7(a). The same force components and equations are applicable to the top beam, considering the fact that the force above the beam is zero.

In a case where the infill plates above and below an intermediate beam have the same thickness (and material grade), the unbalanced distributed forces in Fig. 7(b) would theoretically be negligible according to capacity design procedures, and only the constant shear force induced by the distributed moments [Fig. 7(c)] would exist. However, depending on the design seismic load distribution over the height of the wall, in practice the upper infill plate may not have yielded fully because the shear resistance of both stories is similar, while the shear demand on the upper story is generally smaller. In such a case, similar to the case where the infill plate thicknesses differ, there is an extra shear and moment demand on the beam because of the vertical component of the resulting unbalanced infill plate forces. As a result, when the infill plates above and below an intermediate beam have similar thicknesses, it is recommended that when determining the shear and moment demands in the beam, the upper infill plate force be taken as 80% of its nominal capacity

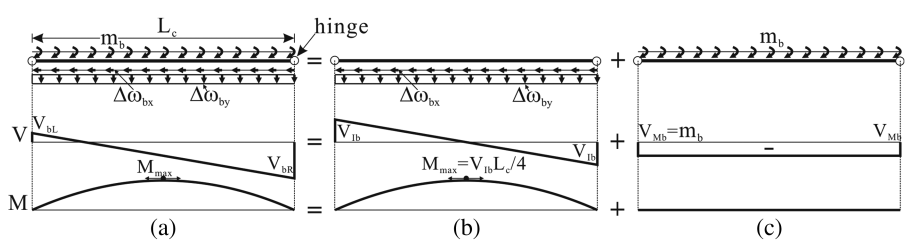

Fig. 7. Internal actions for beams with simple connections due to full tensile infill plate yielding: (a) net actions; (b) orthogonal components of unbalanced force; (c) distributed moment

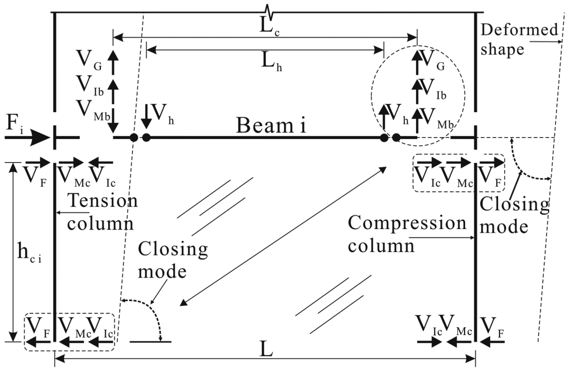

Fig. 8. Shear forces on beams and columns of SPSW system

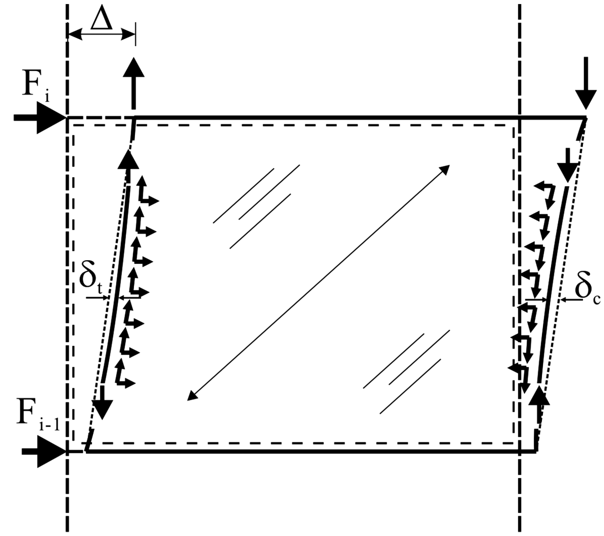

Fig. 9. P-δ effect on columns of SPSW system

design value to account for the possibility of incomplete yielding (Moghimi and Driver 2014a). (Note that this value was substantiated for walls up to four stories in height, with a variety of lateral load transfer mechanisms from the floor and roof diaphragms, different lateral load and infill plate thickness distributions over the height of the wall, panel bay width to story height ratios from 1.2 to 2.2, and story height to infill plate thickness ratios from 208 to 1,440.)

When rigid frame connections are used in a SPSW system, the shear induced in the beam by the plastic moments due to frame action are also considered in the design and are additive to the shears discussed above for SPSWs with simple connections. The shear is applied at the plastic hinge locations at each end of the beam, located at the distance S_h from the faces of the columns (AISC 2010a). As such, the distance between the plastic hinges is $L_h = L_c - 2S_h$, and the induced shear, V_h, is equal to the sum of the plastic moments at the ends divided by L_h. The induced shear forces from the plastic hinges near the beam ends are discussed further in the companion paper.

By adding the shear forces due to frame action (V_h) and gravity loads (V_G) to those shown in Fig. 7 (V_{Ib} and V_{Mb}), the resultant shear forces at each beam end can be found from Fig. 8 for the general case. The beam shear at the compression-column side is maximum, as indicated in Fig. 2(a) and highlighted by the dashed circle in Fig. 8.

Design of Columns

As for the beams, the columns of SPSWs are designed to resist forces due to tensile yielding of the infill plates and the external seismic design loads. When rigid frame connections are used, the shear and moments from frame action are added. In general, the compression column is critical for design, although special conditions such as uplift can impose critical design requirements on the tension column as well. Also, special attention needs to be paid to the compression-column base, where in a ductile SPSW inelastic demand is expected to be extensive in a design earthquake.

Critical Column (Compression Column)

The vertical component of the infill plate tension field applies tension or compression on the left and right column, respectively, as shown in Fig. 4(b). However, at any given level in the wall the amount of axial force in the compression column is larger than that in the tension column for the following reasons. First, some of the tension from the overturning moment is transmitted directly through the infill plate, diminishing the force in the tension column, while the compression component is applied essentially entirely through the compression column, as illustrated schematically at the foundation in Fig. 4(a). Second, the gravity forces are additive to the compressive column force, while they reduce the tensile force. These combined effects may cause a substantial difference in the magnitudes of the tensile and compressive column loads at the same elevation.

The P-δ effect in the compression column of a SPSW potentially has a greater detrimental effect than in columns of other systems. Fig. 9 shows the ith story of a SPSW under story shear forces, and the resulting tension field force components are shown on the columns. It can be seen that the horizontal component of the tension field can exacerbate the P-δ effect by applying an inward force to the compression column in the same direction as its deflection between floors. While this destabilizing condition must be evaluated during design, the minimum column flexural stiffness criteria in most design standards ameliorate this situation considerably.

Common practice is to provide a fixed-support condition where the columns meet the foundation, and under capacity design procedures plastic hinges are expected to develop at the column bases. However, the column cross-section need not be designed explicitly at the base for the plastic moment (M^*_{pc}) as long as the stability and ductility of the region is assured. The actual plastic column-base moment at the yield mechanism load can converge to a small value (compared to the plastic moment capacity of the cross–section, M_{pc}) for the critical (compression) column in certain cases. Figs. 10(a and b) show moment distributions in the compression columns of SPSWs with simple and rigid beam-to-column connections, respectively, under the yield mechanism loading. The moments are normalized by the corresponding plastic moment capacity of the column cross-section. For the wall with simple connections, the moment distribution resembles that of a continuous beam under distributed loading, but for the wall with rigid connections the plastic moments from the beam ends are added at the joint locations. For each type of wall depicted in Fig. 10, the moment distributions corresponding to two performance levels are illustrated based on the three-tier performance-based design concepts proposed in this paper. The higher and lower performance levels are associated, respectively, with moderately ductile and limited-ductility designs for walls with simple connections [Fig. 10(a)] and ductile and moderately ductile designs for walls with rigid connections [Fig. 10(b)].

For higher performance levels, where strong columns are needed, the cross-section at the base of the compression column

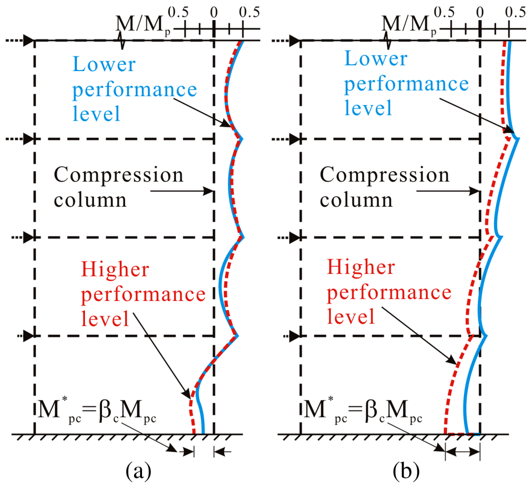

Fig. 10. Normalized moment distributions in compression column: (a) SPSW with simple frame connections; (b) SPSW with rigid frame connections

can develop a significant percentage of its plastic moment, and the dashed lines in Fig. 10 show the moment diagrams for such a case. For lower performance levels, for which the moment diagrams are depicted in the figure by solid lines, a smaller column section can be justified for a given infill plate thickness. The reduction in cross-sectional area causes an increase in the axial compressive stresses in the column at the mechanism condition. Therefore, the compressive force uses up a greater proportion of the normal stress capacity of the cross-section and the remaining moment capacity of the column at the base is reduced considerably. The coefficient β_c in Fig. 10 accounts for the reduction in the plastic moment capacity of the column at the base due to the presence of axial force. This coefficient is less than unity and its value depends on the axial force in the column under the yield mechanism loading. For higher performance levels, β_c tends to be in the range of 0.4–0.6, but for lower performance levels the factor converges to a small value, as shown in Fig. 10. This moment capacity reduction at the base affects mainly the moment distribution in the first story of SPSWs with simple frame connections and the first two stories of SPSWs with rigid frame connections. The differences in the moment distributions in the upper stories between the two performance levels is mainly caused by the differences in the cross-sectional dimensions of the columns.

Due to the presence of the large axial column force that develops in the first story, the columns can be analyzed and designed conservatively for a pinned-base condition when elastic materials are assumed for the boundary members. The moment distributions shown in Fig. 10 suggest that the pinned-base assumption for the compression column in lower performance walls is fairly consistent with the real response, despite the fixed column-base detailing. Also, for higher performance walls, although some level of rotational fixity is likely to exist, the pin assumption is often conservative and acceptable for pushover analysis providing appropriate cross-sectional compactness requirements are met and seismic bracing is provided.

Evaluation of Shear Forces

Similar to the beams of SPSWs, the columns have two main sources of shear force: tensile yielding of the infill plates and frame action. The horizontal component of the infill plate force applies a distributed load that causes a shear (V_{Ic}) in the column at the top and bottom of the story, as shown in Fig. 8. The vertical component of the infill plate force applies a distributed moment about the column centerline, which causes additional shear reactions (V_{Mc}). When rigid beam-to-column connections are used, the frame action induces further shear (V_F). The three shear reactions—V_{Ic}, V_{Mc}, and V_F—correspond, respectively, to shear reactions V_{Ib}, V_{Mb}, and V_h in the beams. The maximum net shear force in each column at each story occurs where the beam-to-column connection is in the closing mode, as shown in the deformed shape of the story depicted in Fig. 8 (locations distinguished by dashed rectangles) and also in Fig. 2(a).

Design of Infill Plate Splices

Although the infill plates are among the protected zones specified in S16 and AISC 341, a previous test (Moghimi and Driver 2013) has shown that infill plates with a single-sided lap splice exhibit excellent performance up to a lateral drift ratio of 4.6%, which is far greater than the ductility required of a limited-ductility SPSW system. Similar results were obtained for double-sided infill plate splices used in a test by Dastfan and Driver (2010). The design of these splices must follow capacity design principles; the expected yield stress factor (R_y) is applied to the nominal yield stress of an infill plate to determine the forces on the splice plate if the splice material is different from that used for the infill plate. However, this factor may be omitted if it can be assured that the splice plates will be cut from the same source plate as the infill plates themselves. In the latter case, the splice plate capacity also need not be decreased by the resistance factor, as the material that produces the demand (infill plates) is the same as that of the designed element (splice plate). When the cutting of splice and infill plates from the same source plate cannot be assured, a thicker splice plate will be needed to meet capacity design objectives.

Summary and Conclusions

Previous research on SPSWs has focused on maximizing system ductility and seismic performance, effectively making them economically competitive only in high seismic regions. A new three-tier, performance-based capacity design framework for SPSWs has been proposed that will accrue several technical and economic benefits and increase the competitiveness of this system in low and moderate seismic regions. Besides the ductile SPSW format, a limited-ductility and two moderately ductile SPSW concepts have been defined and developed. In a SPSW system, it is primarily the behavior of the boundary frame—beams, columns, and beam-to-column connections—that defines the performance level, which is evaluated in this paper in terms of both redundancy and ductility of the overall system. The type of beam-to-column connection describes the redundancy of the wall, with simple and rigid connections delineating the system redundancy limits, and the ductility of the wall is classified in terms of the yield mechanism that develops. In this context, a system yield mechanism that is less ductile than the one assumed for highly ductile walls has been defined, based on observations about SPSW behavior from both physical tests and numerical simulations, that targets performance considered adequate for limited-ductility applications.

Two main aspects of behavior that could result in limited-ductility yield patterns forming have been identified. The high level of compressive force in the critical column of a SPSW along with coexisting moments and shear forces can cause a yield mechanism in the system that exhibits somewhat lower ductility. Also, when the infill plate thickness distribution over the wall height is not proportional to the shear demands imposed by the seismic loads, a lower-ductility yield mechanism can occur due to incomplete yielding in some infill plates and non-uniform lateral deformation of the compression column. By selecting somewhat smaller columns, the performance of the wall may deviate from that targeted by the ductile uniform yield mechanism pattern, yet still be quite acceptable for walls under lower ductility demands.

Methods for determining the design forces in the components of ductile, moderately ductile, and limited-ductility SPSWs have been presented and discussed. It was found that although the conventional tension strip analogy, commonly used in design development, provides generally conservative design forces in the boundary members, this outcome arises due to a variety of compensating factors. These include the accelerated yielding caused by the two-dimensional stress state in the infill plate that is neglected by the strip analogy, the contribution of the minor principal stresses to the design actions on the frame, and the variability of the principal stress directions in the vicinity of the frame members. Even though the combination of these effects cause the axial stresses applied to the column from the infill plate to be underestimated, the net axial design forces in the columns tend to be reasonable because the same phenomena cause the shear reactions from the beams to be overestimated. Conversely, the design moments in the columns of SPSWs obtained using the tension strip analogy tend to be highly conservative.

Acknowledgments

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