Economical Steel Plate Shear Walls for Low-Seismic Regions

Hassan Moghimi, M.ASCE[1]; and Robert G. Driver, M.ASCE[2]

Abstract: Previous research on steel plate shear walls (SPSWs) and current design codes have focused principally on achieving highly ductile behavior through stringent detailing requirements. As such, the system is generally considered to be economical only in high-seismic regions. However, lower demands in other regions may permit the use of more economical options. This paper describes a proposed concept for SPSWs that meets the intent of capacity design, while greatly improving competitiveness in seismic regions where maximum ductility is not required. A large-scale, 2-story SPSW specimen was tested to evaluate the associated performance. The wall had standard double-angle beam-to-column shear connections and was tested under vertical gravity load concurrent with reversing lateral loads at each floor level. The specimen survived 25 lateral load cycles, 18 of which were in the inelastic range. The test results indicated that excellent performance can be expected in low-seismic regions, despite significantly reduced costs, compared with traditional designs. The shear wall showed stable performance at large lateral deformation ratios with high levels of ductility and energy dissipation capacity. **DOI: 10.1061/(ASCE)ST.1943-541X.0000662.** *© 2013 American Society of Civil Engineers.*

CE Database subject headings: Cyclic tests; Seismic design; Shear walls; Steel; Plastic hinges.

Author keywords: Cyclic tests; Seismic design; Shear walls; Steel; Plastic hinges; Analysis.

Introduction

The steel plate shear wall (SPSW) has become a viable lateral load resisting system for multistory buildings, with beneficial properties for seismic applications, such as high ductility, robust resistance to cyclic degradation, and resilient redundancy. These properties have been demonstrated mostly through tests on walls that contain moment-resisting beam-to-column connections and, as such, requirements for connections in ductile moment-resisting frames have significantly influenced the evolution of detailing and fabrication practice for SPSWs. The current international research focus is predominantly aimed at improving the performance of SPSWs even further and optimizing their behavior under the overarching capacity design requirements that pervade modern seismic design provisions. As a result, the design and detailing requirements are tending to become more and more onerous, and increasingly the system is being limited economically to high-seismic regions where the cost of maximizing performance can be justified. However, the lower demands on seismic force resisting systems in low- and moderate-seismic regions, which collectively encompass the majority of North America, may permit the use of much more economical SPSW detailing options that would make them competitive with systems that are more commonly used in these regions.

The Canadian steel design standard, CAN/CSA S16–09 [Canadian Standards Association (CSA) 2009], hereafter referred to as S16, has adopted two SPSW performance levels: Type D (ductile) and Type LD (limited ductility). These performance levels are associated with different force modification factors 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_dR_o. The Seismic Provisions for Structural Steel Buildings, ANSI/AISC 341–10 (AISC 2010a), hereafter referred to as AISC 341, adopted only the higher SPSW performance level: special plate shear walls. ASCE/SEI 7-10 (ASCE 2010) defines the associated response modification coefficient, R, which also accounts for both inelastic system response and overstrength. A moderate-ductility (Type MD/intermediate SPSW) option is not presented in either of these standards. Table 1 summarizes the R factors specified by both S16 and ASCE 7 for the SPSW system.

Although the limited-ductility (Type LD) SPSW option exists in S16, there is a general lack of understanding of what constitutes this type of wall, and these provisions are rarely applied in practice. The Type LD category was originally introduced into S16 based primarily on the experimental research of Timler and Kulak (1983) and Tromposch and Kulak (1987), and was intended to permit the use of shear connections between the beams and columns. Although these tests confirmed the SPSW system without moment-resisting beam-to-column connections as a feasible option, they predated the now well-established seismic loading test protocols. 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).

Objectives and Scope

The major objective of this research is the rational development of SPSW concepts that would make them competitive with other systems and materials in low-seismic regions, and to confirm the

[1]Ph.D. Candidate, Dept. of Civil and Environmental Engineering, Univ. of Alberta, Edmonton, AB, Canada T6G 2W2. E-mail: moghimi@ualberta.ca

[2]Professor, Dept. of Civil and Environmental Engineering, Univ. of Alberta, Edmonton, AB, Canada T6G 2W2 (corresponding author). E-mail: rdriver@ualberta.ca

Note. This manuscript was submitted on January 3, 2012; approved on May 25, 2012; published online on February 15, 2013. Discussion period open until August 1, 2013; separate discussions must be submitted for individual papers. This paper is part of the ***Journal of Structural Engineering***, Vol. 139, No. 3, March 1, 2013. ©ASCE, ISSN 0733-9445/2013/3-379–388/$25.00.

Table 1. Seismic Force Modification Factors for SPSWs

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

[a]Dual system with special moment frame capable of resisting at least 25% of prescribed seismic forces.

efficacy of such a system experimentally. Therefore, the need for economical fabrication and erection procedures is considered of fundamental importance. At the early stages of the project, an assessment of major cost implications of the various design and detailing requirements for Type D/Special SPSWs was made with a view to maximizing the economic benefits of switching to the low-seismic concept for regions where such a system would suffice. Whereas most research on seismic force resisting systems aims to develop design and detailing requirements based on anticipated demands, or simply attempts to maximize overall seismic performance through high-performance detailing, this research has taken a fundamentally different developmental approach. In the traditional approach, the cost of the system is largely an uncontrolled outcome of the research. However, it was anticipated that simple and relatively inexpensive detailing could be used in SPSWs and still achieve good seismic behavior because of the nature of the system itself. Consequently, the SPSW test specimen was developed with the main emphasis on minimizing the in-place cost in a real structure, rather than imposing detailing that is known to be highly robust under cyclic loading. In other words, the performance level of the low-seismic concept, rather than the cost of the system needed to maximize performance, was the principal outcome of the research.

A large-scale test was conducted to assess the performance that could be expected from the proposed low-seismic SPSW concept. The test specimen was developed for use with common and economical fabrication methods and simple erection procedures, with input from the steel industry. Selective use of the large body of knowledge available from previous research on highly ductile SPSWs permits the design of a wall that is expected to perform well enough for low-seismic regions, but is much less costly to construct.

Previous Tests on Steel Plate Shear Walls with Simple Connections

Although there are many SPSW tests discussed in the literature, few have incorporated simple beam-to-column connections in the boundary frame. Because the use of conventional shear connections is such a key component of the low-seismic wall being proposed, research on similar walls is reviewed briefly here. Timler and Kulak (1983) tested two single-story SPSWs simultaneously by testing a two-panel arrangement of vertically oriented beams, horizontally oriented columns, and 5-mm-thick infill plates. Two different beam-to-column connections were implemented. At the four extreme corners, pinned joints were used, whereas at the centerline of the test specimen, rigid connections were used. Hence, the specimen is similar to two 1-story SPSWs, each with pinned beam-to-column connections and rigid column bases. Axial loads were not applied to the columns. Three cycles of loading were applied to the allowable serviceability displacement limit (drift ratio of 0.25%, or 6.25 mm), and then a final monotonic loading excursion was applied until failure of the assembly occurred. The test specimen responded elastically during the first three cycles, and the maximum capacity was reached in the final excursion when a weld tear occurred at the infill plate-to-fish plate connection, followed by failure at a pin connection.

Tromposch and Kulak (1987) tested a twinned single-story shear wall arrangement similar to the one tested by Timler and Kulak (1983), but with some modifications. The beam-to-column connections were double-shear tabs welded to the column flange and bolted to the adjacent beam web. To provide better anchorage for the tension field, a thinner infill plate (3.25 mm) and stiffer beams were selected. Simulated gravity loads were applied to each column through full-length prestressing bars. The specimen was tested under fully reversed cyclic lateral load, with amplitudes that were gradually increased up to a drift ratio of 0.78% (17 mm, which corresponded to 67% of the ultimate load). This sequence consisted of 28 cycles. Beyond this point, the test setup was able to apply the load in one direction only and without the column prestressing rods because of the curvature of the columns. The final phase was completed as a monotonic loading sequence to the ultimate capacity of the specimen, which corresponded to a drift ratio of 3.23% (71 mm). The test specimen showed ductile behavior with severely pinched hysteresis curves because of the thin infill plate and flexible boundary frame.

Caccese et al. (1993) conducted a series of tests on one-quarter-scale SPSWs subjected to cyclic loading to study the effect of two main parameters: the beam-to-column connection type and the infill plate thickness. The test specimens were 3 stories high and one bay wide, and had infill plate thicknesses that ranged from 0.76 to 2.66 mm. They reported test results from six specimens, including one moment-resisting frame, three SPSWs with moment-resisting beam-to-column connections and varying infill plate thicknesses, and two SPSWs with shear beam-to-column connections (beam web fillet welded directly to the column flange) and different infill plate thicknesses. Each test specimen underwent 24 cycles of a single in-plane lateral load applied at the roof level, with gradually increasing roof displacements up to a drift ratio of between 1.8 and 2.0%. Subsequently, the specimen was loaded monotonically to the displacement limit of the actuator or failure. The effect of gravity loads was not included in the tests. The authors suggested that the beam-to-column connection type had only a minor effect on the overall performance of the SPSW system.

Berman and Bruneau (2005) tested three single-story SPSW specimens using light-gauge cold-formed steel for the infill plates, with thicknesses ranging from 0.75 to 1.0 mm. The research aimed to study the performance of a prototype designed as a seismic retrofit for a hospital in a zone of high seismicity. Two specimens had a flat infill plate (each with a different method of fastening to the boundary frame), and the third had a corrugated infill plate. The flat infill plates were lap-connected to the stem of a steel T-section by either welds or epoxy, and the T-section flange was bolted to the boundary frame. The corrugated infill plate was connected to an angle on each side by epoxy, and the angles were bolted to the boundary frame. Double-angle beam-to-column connections were used in the boundary frame, which was designed to remain elastic with a safety factor of 2.5. Each test was conducted under cyclic lateral loading, and the effect of gravity loads was excluded. Only the specimen with a flat (and 1.0 mm thick) infill plate that was fastened by welding showed stable and highly ductile performance. The yield displacement of the wall was assumed to be 5.3 mm, at a drift ratio 0.29%, and the wall exhibited its maximum resistance at a ductility ratio of about 10, or a drift ratio of 3.07%. It reached the maximum ductility ratio of about 12 at a drift ratio 3.65%, at which time fractures propagated in all four corners of the infill plate from the endpoints of the welds connecting it to the T-sections.

Chen and Jhang (2011) tested two one-quarter-scale SPSWs with stiffened infill plates, each representing 2 intermediate stories of a multistory wall, under cyclic loading. One had simple (shear tab) beam-to-column frame connections, and the other used rigid connections. A low-yield-point steel (with yield and ultimate stresses of 95 and 279 MPa, respectively) of a 3.5-mm thickness was used for the infill plates, whereas ASTM A572 Grade 50 steel was used for the boundary frame members. The infill plate stiffener arrangement was selected such that the width-to-thickness ratio of each subpanel was 71. A total of 24 cycles of lateral load were applied to the top of each specimen. The yield displacement was assumed to be 0.50%, and both walls showed good performance with a maximum story drift ratio of 6.0%, corresponding to a ductility ratio of 12. The specimen with rigid beam-to-column connections showed a slightly higher energy dissipation capacity.

Dastfan and Driver (2012) tested a 2-story modular SPSW with partially encased composite columns. In this specimen, the 3-mm thick infill plate modules were connected to the columns and beams through bolts to the fish plates that were welded to the surrounding frames in the shop in advance. A double-lap splice at the midheight of each panel was used to connect the infill plate modules together. The beam-to-column connections were a customized double-shear tab connection (on one side of the beam web, a conventional shear tab was used, and on the other side, a continuous fish plate was used, augmented locally to accommodate more bolts). The gravity loads were applied as a constant axial compression to each column, and the specimen was tested under cyclic lateral loading. A total of 27 load cycles were applied. Very good local and global performance was reported. The first story lateral deformation was chosen as the controlling parameter, and the yield displacement of the specimen was assumed to be 8.5 mm, at a drift ratio of 0.48%. The specimen reached its maximum strength at a ductility ratio of 5, or a drift ratio of 2.40%, and achieved a maximum ductility ratio of 8, corresponding to a drift ratio of 3.83%. The test was terminated at this point because the composite columns of the first story were damaged at midheight and at the base, and the tears in the infill plate started to grow rapidly. The beam-to-column connections showed no significant damage. The authors observed that the simple connection rotation appeared to improve the distribution of yielding in the infill plates over the height of the wall, thereby increasing the total amount of energy dissipation in the system compared with a similar wall they tested with rigid beam-to-column connections.

Modular Construction of Steel Plate Shear Walls

Modular construction concepts, with emphasis on repetitiveness of fabrication, ease and speed of erection, and elimination of field welding, can improve the competitiveness of SPSWs in low-seismic regions. Because buildings normally have a constant story height and bay width over their height, implementing simple beam-to-column connections in the SPSW makes them a perfect candidate for the use of modular construction. In this method, modular components with repetitive fabrication processes are produced in the shop and assembled completely by bolting in the field, eliminating the need for field welds. This not only tends to decrease costs, but can also enhance the quality of the finished structure because of the improved control that can be maintained in shop conditions. Although the modular construction method can be cost effective (by reducing the costs of both fabrication and erection) with very rapid assembly time, the resulting shear wall has several distinct differences from most of the walls that have been tested in the laboratory, and the performance of these walls requires verification.

Driver and Moghimi (2011) describe several potential modular options and discuss their advantages and disadvantages. In particular, three concepts for SPSWs are considered that are designed specifically to reduce in-place cost. Several main characteristics distinguish these concepts from conventional SPSWs. Most notably, the beam-to-column connections are simple shear connections, and all concepts constitute modular systems that require no field welding. Also, the infill plates are spliced with single-sided lap plates, and all bolts are intended to be pretensioned to resist slip at design loads, as per conventional practice in bolted seismic force resisting systems.

The first concept [Fig. 1(a)] is intended to maximize the number of similar pieces and minimize the piece sizes that need to be handled on site. The angles for the simple beam-to-column connections are joined to the columns in the shop (by bolting or welding) and bolted to the beam webs in the field. The fish plates are welded to the column and beam centerlines in the shop, and then after assembling the frame on site, the infill plates are bolted to the fish plates. Vertical and horizontal lap splices are used to connect the infill plate modules to each other. Field assembly and erection of numerous infill plate pieces could potentially increase construction time in comparison with other concepts, but it may have applications in small projects or retrofit work.

The second concept [Figs. 1(b and c)] is intended to minimize the number of pieces to be handled during erection by maximizing shop assembly. In this concept, the bay of the wall is divided vertically into two parts, permitting installation in relatively wide bays. Therefore, each module could be up to about 3 stories in height (depending on the maximum practical size for shipping and lifting) and one-half bay wide. The connections (simple or rigid) of the beams to the columns are fabricated in the shop. The fish plates are omitted, and the infill plates are connected directly to the surrounding frames in the shop. Vertical lap splices and beam splices are used to connect the modules together. Although the vertical splice may create erection and plumbing challenges, this method has some potential advantages, including speed of construction and the high out-of-plane stiffness of each module during handling compared with individual infill plates.

The objective of the third concept [Fig. 1(d)] is to provide internal modules of a single story in height (midstory to midstory) with effectively no limit to the bay width imposed by shipping concerns, while not limiting the height of the adjacent column tiers. Therefore, this concept consists of three module types: base, top, and intermediate story modules. As long as the heights of the intermediate stories and the associated beam and column sizes are consistent, all intermediate modules will be identical. In the base and top modules, the infill plates are welded to the base and top beam, respectively, in the shop. In intermediate modules, the infill plates above and below the beam are welded to the beam's flange in the shop, which eliminates the need for horizontal fish plates. Fish plates are welded only to the column flanges in the shop. The fish plates could be continuous or interrupted at the connections, and in either case, the simple connection can be accommodated accordingly, as shown in Fig. 1(d). Shear wall modules (including the beams) are connected to the column fish plates on site by bolting, and then horizontal lap

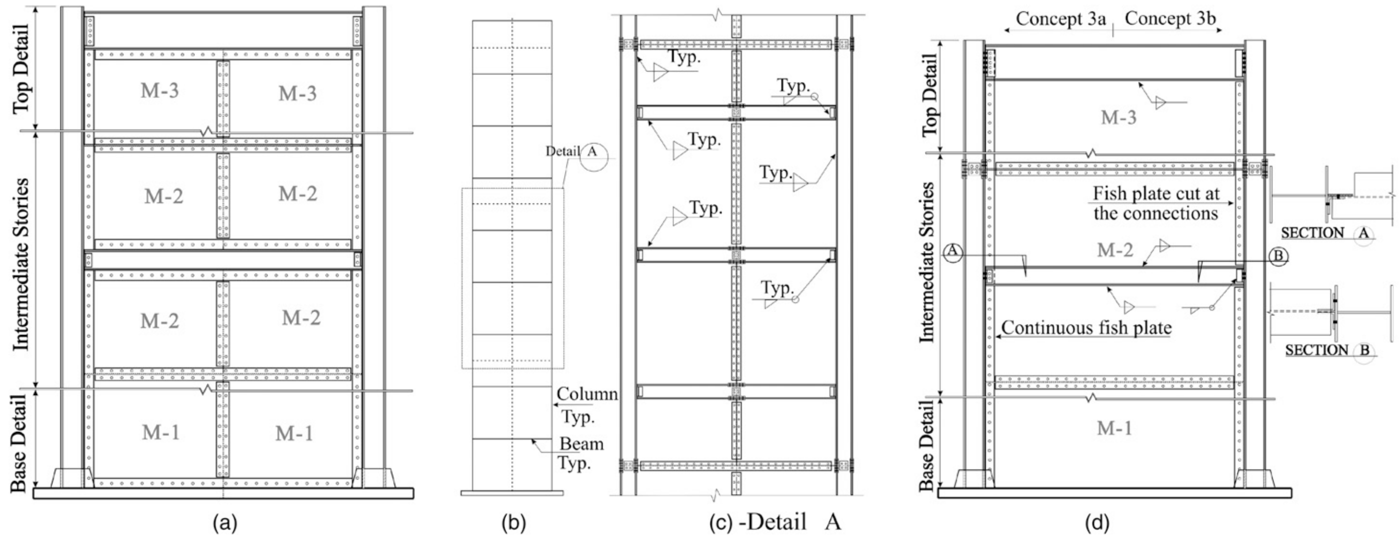

Fig. 1. Modular SPSW concepts: (a) Concept 1; (b) overview of Concept 2; (c) assembled modules of Concept 2; (d) Concept 3 and two potential beam-to-column connections

splices are used to connect the modules to each other. A disadvantage of this system is the lack of out-of-plane stiffness of the modules during handling. Therefore, some consideration is needed regarding erection of the system, such as providing temporary perimeter stiffening to the infill plates until installation of the panel is completed. Further details on these modular concepts and their performance under lateral monotonic forces in comparison with other construction systems are provided by Driver and Moghimi (2011).

Current Steel Plate Shear Wall Design Methods

Standard S16 (CSA 2009) requires using capacity design principles for any structure designed for seismic loads using $R > 1.3$ ($R \leq 1.3$ can be interpreted as essentially elastic design). For Type D SPSWs, it is assumed that significant inelastic deformation can be developed in the system by yielding of the infill plates and the formation of plastic hinges at the ends of the beams, at a short distance from the faces of the columns, and at the bases of the columns. The moment-resisting boundary frame alone must have sufficient capacity to resist at least 25% of the factored story shear at each level, and minimum stiffness requirements are provided for the columns and the top and base beams to ensure the development of reasonably uniform tension fields in the infill plates. In general, the beam-to-column connections must comply with the requirements for Type LD moment-resisting frames and the column joint panel zones with those for Type D frames. All areas that may develop significant yielding—ends of beams, column bases, and infill plates—are designated as protected zones, which prohibits most attachments or discontinuities that may cause stress concentrations.

The AISC 341 (AISC 2010a) provisions stipulate requirements for special plate shear walls that are similar to those for Type D walls in S16 (CSA 2009). They also use the capacity design philosophy and specify that, in general, the beam-to-column connections must comply with the requirements for ordinary moment frames, and the panel zones next to the top and base beams and the boundary member cross-sectional compactness must satisfy the special moment frame requirements. Boundary member minimum stiffness requirements are also specified. Protected zones include the infill plates, boundary frame connections, and potential hinging regions in the beams.

For SPSW column design, S16 (Type D) and AISC 341 both require that the internal forces from frame action (beam hinging at both ends), including the effects of material overstrength and strain hardening, be added to the gravity forces and the distributed forces from the yielded infill plate (including material overstrength). Although S16 explicitly specifies the beam-end moments for which the columns must be designed, AISC 341 instead stipulates that the column–beam moment ratio must comply with the associated requirement for special moment frames. Both requirements serve to ensure strong column–weak beam behavior. Berman and Bruneau (2008) have presented a detailed procedure for capacity design of columns in Type D/special SPSWs.

Design requirements for SPSWs with a lower seismic force modification factor are included in S16 (CSA 2009), but not in AISC 341 (AISC 2010a). Capacity design requirements in S16 for Type LD SPSWs use Type D wall provisions as a starting point, with a few relaxations of the rules as deemed appropriate by the committee. These relaxations include reduced beam compactness requirements and beam-to-column connections other than rigid being permitted. As such, the requirement that the boundary frame be capable of resisting 25% of the factored story shear at each level does not apply. It is also recognized that the column panel zone and connection requirements specified for Type D walls need not be applied if shear connections are used. However, notably, there is no reduction of the column design moment arising from plastic hinging in the beams even for walls with shear connections, although it is stated explicitly that shear forces that develop from these plastic moments need not be considered in this case. In the interest of clarity of intent and technical rigor, it is imperative that Type LD wall provisions be developed within their own context, rather than simply being a modified version of those used to obtain Type D performance, and be based on observations from research specifically attuned to Type LD objectives.

Developmental Philosophy for Low-Seismic Walls

Because the central goal of this research is to develop a SPSW concept suitable for use in low-seismic regions, it is necessary to capitalize on the inherent ductility of the system—which tends to be relatively independent of the frame connection type—so that emphasis can instead be on fabrication economics. Specifically, SPSWs should not require the costly connection detailing and stringent column design requirements that are specified in design provisions for highly ductile walls to perform well in low-seismic regions. Also, because the efficiency of the system tends to result in very small infill plate thicknesses in Type D SPSWs that need to be increased to meet conventional handling and welding requirements, a reduction in the R factor for the proposed low-seismic system may not result in any increase in plate thickness, thus eliminating any increase in demand on the adjacent columns arising from their role in infill plate tension field anchorage under capacity design requirements.

The SPSW system proposed for low-seismic regions has two main differences from the most common SPSWs. First, it uses simple (shear) beam-to-column connections, giving rise to several advantages, both economic and technical. Clearly, it reduces the cost of the system, because simple beam-to-column connections are considerably less costly to fabricate than connections that comply with Type D/special or even Type LD/ordinary moment frame requirements. Also, the moment and shear forces that develop at the beam ends are greatly reduced in this system because of the flexibility at the frame joint, which in turn significantly reduces the moment and axial force demands on the columns from this source.

A drawback of the shear connection application is that it reduces the redundancy of the system as a whole, although the redundancy of the infill plates as a distributed bracing system remains. Using shear connections instead of moment connections also tends to cause more pinching in the hysteresis curves, decreasing the total energy dissipated. However, because the shear connection allows rotation at the beam-to-column interface, the deformed shapes of the beams and columns are less affected by frame action. As a result, a more uniform yielding distribution develops in the infill plates over the height of the wall, potentially increasing the total energy dissipated by the system. This phenomenon has been demonstrated in both experimental and numerical investigations. For example, Dastfan and Driver (2012) compared the energy dissipation capacity of two 2-story SPSWs with partially encased composite columns, one with simple and the other with rigid (and reduced beam section) beam-to-column connections that were identical in all other respects. They found that the total energy dissipated during the test of the system with simple connections was higher. To support the experimental observations of Dastfan and Driver (2012), finite-element pushover analyses of multistory SPSWs conducted as part of the current research project have also shown that changing the beam-to-column connection type from rigid to simple makes the distribution of yielding in the infill plates over the height of the wall more uniform and increases the total energy dissipation capacity of the system. Although there are clearly some advantages of using simple connections in SPSWs, any anticipated improved performance is predicated on the connection behaving in a robust manner under cyclic loading, and this must be demonstrated through physical testing of such a SPSW.

The second main difference from conventional high-ductility SPSWs is the idea that some yielding in the columns can be tolerated in low-seismic regions, as long as it does not cause the formation of a yield mechanism in the system. In the capacity design of Type D/special SPSWs, columns are designed to remain elastic (except that S16 recognizes the formation of plastic hinges at the column bases), whereas the infill plates develop the yielded tension field in each panel and the beams develop a plastic hinge at each end. In addition, allowances are included for potential overstrength of these yielded regions. As a result, and especially in cases where beams or infill plates are oversized, large internal force demands are imposed on the columns of the system. This is particularly severe for the column under the maximum compression, and extremely heavy column sections are frequently needed to satisfy the design criteria.

Allowing partial yielding in the columns in low-seismic regions, where lower ductility systems are typically used, is not without precedent, and this philosophy has been adopted in design provisions (S16 and AISC 341) for other lateral force resisting systems. For instance, columns in Type D/special moment frames are designed for the plastic moments at the ends of each beam, amplified by both material overstrength and strain hardening factors; conversely, in the design of columns in Type LD/ordinary moment frames, the plastic end moments of the beams are not amplified, implicitly allowing some column yielding to occur. This yielding is permitted even though in moment frames the columns do not rely on an integrated direct bracing system for in-plane stability, as is the case in SPSWs and braced frames.

Test Specimen

A laboratory test of a 2-story modular SPSW was conducted to assess the performance of the low-seismic SPSW concept discussed in the preceding sections. Instead of attempting to adapt the current provisions in S16 (CSA 2009) for Type LD walls, which are themselves under scrutiny, the wall was designed using the performance-based methodology described by Moghimi and Driver (2011) and performance criteria specified in ASCE 41 (ASCE 2007). Frame components subjected to deformation-controlled actions use moment-frame acceptance criteria, in the absence of suitable SPSW criteria, and meet the life-safety performance level. Components subjected to force-controlled actions meet the strength design provisions of both S16 (CSA 2009) and AISC 360 (AISC 2010b). Of particular note, the goal was to select a configuration that, according to these performance-based design criteria, barely achieves the ductility level consistent with Type LD walls (i.e., $R_d = 2.0$). Although this method does not result in a system that complies in all respects with the current Type LD SPSW provisions of S16, it was believed that good performance would still be achieved at lower cost. The test results provide evidence of the performance that can be expected from such a system.

Discussions with steel industry personnel have led to the conclusion that Modular Concept 3 (discussed previously) is the most promising in terms of practicality and economics, and it forms the basis of the specimen tested. Double-angle beam-to-column connections were used, which are common in practice and at the same time provide rotational freedom at the joint. The short legs of the angles were welded with 8-mm fillet welds to the beam web in the shop, and the long legs were bolted to the column flanges during module assembly. Connecting the long legs to the columns increases the rotational capacity of the joint and consequently reduces the demand on the columns. The test specimen was constructed using normal industry procedures.

Fig. 2 shows the elevation of the specimen tested. The story height was 1,900 mm, and the center-to-center dimension between columns was 2,440 mm, approximately representative of a half-scale wall for an office building. The story-aspect ratio (story height/ center-to-center distance between columns) was 0.78 for both stories. The columns were continuous W250 × 101 sections (W10 × 68), the intermediate beam was a W250 × 58 section (W10 × 39), the top beam was a W460 × 67 section (W18 × 45), and the double-angle connections were L102 × 76 × 11 sections (L4 × 3 × 7/16) with 170- and 360-mm lengths for the first and second stories, respectively. [It should be noted that the current design procedures in S16 for Type LD SPSWs result in a W310 × 202 column section (W12 × 136)—two times the cross-sectional area and more than three times the moment of inertia of the one selected.] The infill plates were 4.8 mm thick, and the fish plates were 6.35 mm thick and

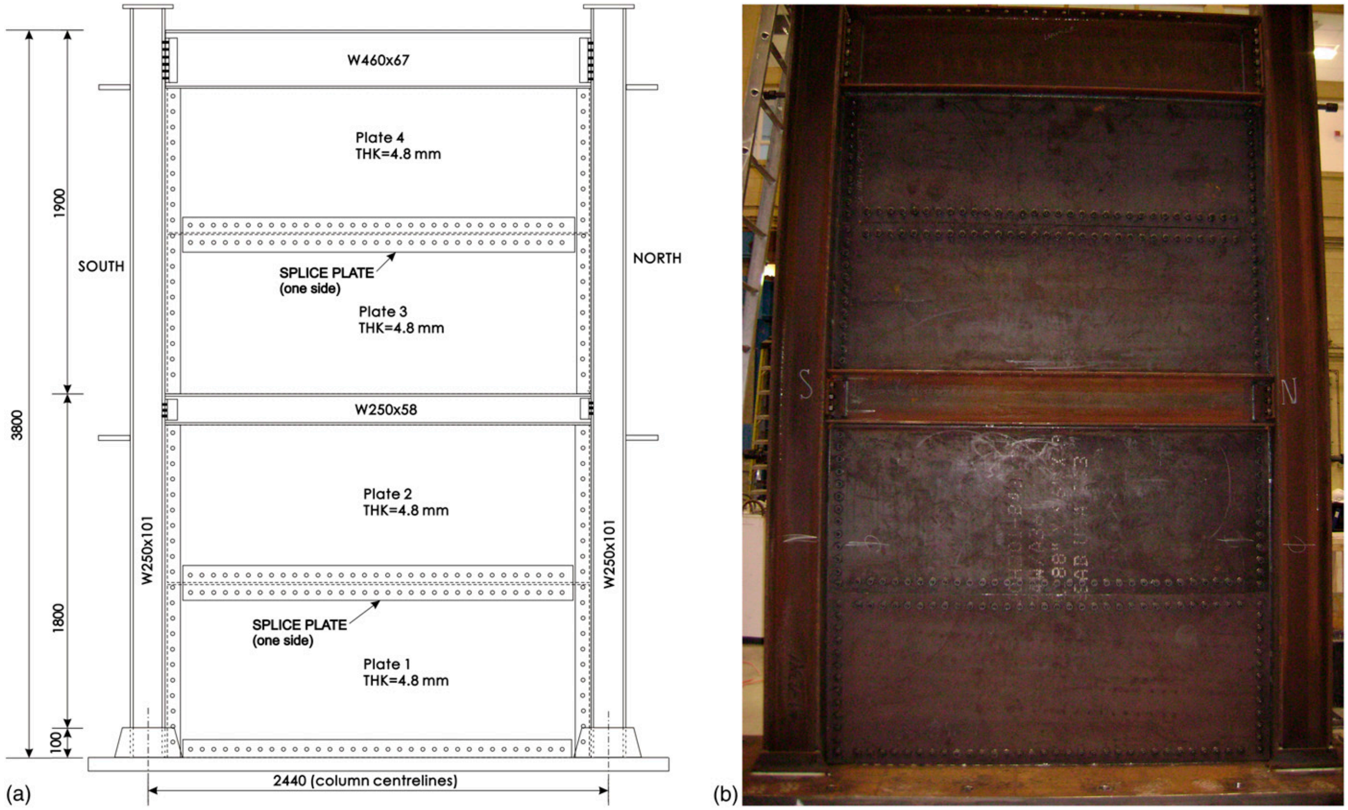

Fig. 2. Test specimen: (a) schematic diagram; (b) east elevation (splice plates installed on far side)

115 mm wide. The infill plates in both stories were spliced horizontally at midheight with a single-sided lap plate of the same thickness as the infill plates. Because plastic hinges were expected to develop at the column bases, stiffener plates at both sides of the columns were provided to increase the ductility in this region.

A 4.8-mm (3/16 in.) infill plate was selected as a readily available thickness, representing a plate that would be easy to work with at full scale. The infill plate connections to the beams and fish plates, as well as the fish plate connections to the columns, were designed to resist the expected tensile yield stress (including material overstrength) in the infill plates. For the design of the infill plate splices, the same expected infill plate yield stress was considered; however, because the splice plates were cut from the same plate as the infill plates themselves for consistency, the expected yield stress was also used in the splice plate resistance equations. When the cutting of splice and infill plates from the same source plate cannot be assured, a thicker splice plate may be needed.

The beams were designed based on the expected tensile yield stress in the infill plate and the fact that the external lateral forces were applied directly to the beams. Because the infill plates in the 2 stories were of the same thickness and the tension field orientations were similar, the intermediate beam was designed mainly for the nonuniform compressive force caused by the inward pull from the infill plates on the columns and the effect of lateral load application to the beam. In addition to this compressive force, the top beam was subjected to significant shear, flexural, and axial loads directly from the tension field in the infill plate below. Therefore, a relatively deep and stiff beam was needed. Because the horizontal component of the tension field offsets the applied lateral load, the top beam was subjected to a fairly uniform compressive force. Both beams were checked against lateral torsional buckling to eliminate the need for intermediate lateral bracing. The double-angle connections were designed for the axial force and shear present at the beam ends.

All frame members were fabricated from Grade 350W steel, and the angles and infill plates were from Grade 300W steel (CSA 2004). The frame members met Class 1 (S16) and Highly Ductile Member (AISC 341) compactness requirements. ASTM A325 bolts were used, and all bolted connections in shear were designed to be of bearing type. Nevertheless, the bolts were pretensioned to meet the requirement (S16 and AISC 341) that all bolts resisting cyclic loading be pretensioned high-strength bolts to avoid slip at design loads. The infill plates were connected to the fish plates and splice plates by 19.1-mm (3/4 in.)-diameter A325 bolts with a 60-mm spacing between the centerlines of the fasteners. The same bolt size and spacing were used to fasten the connection angles to the column flanges. The infill plates adjacent to the beams and fish plates were connected to the surrounding frame by 5- (infill plates) or 6-mm (fish plates) fillet welds on both sides. The electrode classification was E70XX.

Loading Scheme

The distribution of inertial loads on a seismic force resisting system depends on the earthquake ground motion characteristics and severity, as well as the properties of the system itself, including geometry, distribution of mass, stiffness, strength, and damping. These properties influence the relative magnitudes of the deformations and internal forces within the structure, which can vary significantly during an earthquake as the stiffness distribution changes because of progressive yielding. Moreover, changes in the seismic acceleration history and frequency content excite different mode shapes of the system, causing changes in the force distribution. Hence, the distribution of inertial forces has been an issue of debate in recent decades, and the use of more than one lateral load pattern has been recommended for nonlinear static design to bound the range of design actions that may occur during a seismic event (FEMA 1997). However, ASCE 41 (ASCE 2007) suggests the use of a single lateral load pattern based on the first mode shape, because recent research has shown that using multiple patterns is not particularly effective in improving the accuracy of a nonlinear static analysis. The first mode load pattern is most appropriate for taller structures and it emphasizes a gradual increase in inertial force from the lower to the upper stories, which underscores the influence of story overturning moment over shear force in comparison with a uniform load pattern.

Based on the discussion in the preceding paragraph, the first mode load distribution was selected for the test. From eigenvalue analyses of the wall using several assumptions of story masses, the first mode shape has normalized lateral deformations of 1 and approximately 0.55 at the roof and top of the first story, respectively. However, the significant difference between these deformations comes from the fact that the specimen is 2 stories tall; for a taller structure, the difference between two adjacent floors would be smaller. As a result, a hypothetical first mode shape arising from normalized loads of 1 and 2/3 for the second and first story, respectively, was selected for use throughout the test, and it is believed to represent a range of intermediate-height structures adequately. To simulate the location of inertial forces induced by floor masses, the lateral loads were applied through two sets of twin actuators (supported by a reaction wall) positioned in line with the top flanges of the intermediate and top beams. The SPSW was loaded through each beam top flange to simulate the delivery of load to the wall through a horizontal diaphragm.

To study the P-Δ effect on the overall behavior of the SPSW system subjected to cyclic lateral loading, reasonable unfactored gravity loads must be applied. As such, a constant gravity load of 600 kN was applied to the top of each column by two sets of independent hydraulic jacks connected to a cross-shaped distributing beam supported at the top of the specimen by the columns. Four gravity load simulators, designed so that the gravity loads remained in a vertical orientation throughout the cyclic lateral deformation, were employed in conjunction with these jacks. An articulated bracing system that prevented out-of-plane deformations, but provided no restraint to lateral and vertical deformations, was affixed to each column at each floor level.

The loading history for the test specimen was selected based on the methodology outlined by the Applied Technology Council (ATC 1992). The 2 stories of the test specimen had the same infill plate and column, but the lateral shear force and overturning moment resisted by the first story were 67 and 167%, respectively, larger than the corresponding values in the second story. Therefore, the majority of deformation, yielding, and energy dissipation was expected to take place in the first story. As such, the lateral deformation of the first story was selected as the deformation control parameter (δ), and the base shear was selected as the force quantity (Q)—or the force corresponding to this deformation—and these two parameters constituted the test control parameters. The point of significant yield (δ_y, Q_y), which is essential information needed for controlling the test, was first estimated by finite-element analysis and then adjusted during the early stage of the test based on the observed behavior. It was found that the yield displacements of the first story in the push (north) and pull (south) directions were equal to 11 and 13 mm, respectively. Hence, the first story lateral yield displacement was selected as $\delta_y = 12$ mm, the average value of the two directions. The yield displacement at the top of the specimen was found to be equal to $\delta_{r,y} = 22$ mm for both the push and pull directions.

A large number of data collection devices, including load cells, LVDTs, cable transducers, clinometers, strain gauges, and rosettes, were used to control the test and monitor and record the important structural responses. The first story and top lateral displacements were measured at heights of 1,845 and 3,755 mm, respectively, from the top surface of the base plate.

Test Results

Table 2 shows the loading/displacement regime throughout the test in the push direction. (The corresponding values for the pull direction differ only slightly from those in Table 2.) The hysteresis curves based on the test control parameters (first story) are shown in Fig. 3. From Cycles 1 to 7, the test was conducted in force control to measure the elastic and initial inelastic behavior of the wall. From Cycle 8 forward, the test was carried out in displacement control. Cycles 8 to 10 were completed with the yield displacement of $\delta_y = 22$ mm. For reference, the nominal shear capacity of the specimen (according to S16 and AISC 341, with 0.5 used as the coefficient to represent the fully yielded strength of the infill plate) is also shown in Fig. 3, and it is appreciably less than the base shear of 1,920 kN resisted during Cycles 8 to 10. The hysteresis curves show that the first story absorbed significant energy during and after Cycle 8.

During the first half of Cycle 11 (push direction) with a target lateral displacement of $2\delta_y$ (24 mm), a loud sound caused by buckling of the infill plates was heard at a displacement of approximately 22 mm ($1.8\delta_y$), and the first story load cell began giving erroneous readings because of a break in the wiring. For safety reasons, the test specimen was unloaded without reaching the full target displacement, and the load cell wiring was repaired. For the part of Cycle 11 where load data are unavailable, the estimated curve is shown as a dashed line in Fig. 3. The second half of Cycle 11 (pull direction) was done with the same lateral deformation level as the push half-cycle (22 mm) for symmetry. Cycle 12 was then carried out with a lateral deformation of 26 mm ($2.2\delta_y$) to compensate for the smaller lateral deformation in the previous cycle. The last cycle of the $2\delta_y$ lateral deformation level (Cycle 13) was done with the targeted displacement of 24 mm. The hysteresis curve for the second story (not shown) indicates that this story started to absorb a considerable amount of energy from Cycle 11 forward. Cycles 17 and 18 consisted of a lateral deformation of $4\delta_y$ (48 mm), and in the first half of cycle 18 (push direction), the first tear in the system was observed at the top-north corner of the lower infill plate.

Cycle 19 was completed with a lateral deformation of $5\delta_y$ (60 mm), and the maximum base shear in the push direction of 2,625 kN occurred in this cycle. Toward the end of the push loading in Cycle 20, at a lateral deformation of 55 mm, the welds connecting the column base stiffeners to the south flange of the south column ruptured, followed by the initiation of fracture at the adjacent flange tips. This caused a reduction (about 11%) in the load-carrying capacity of the system. However, no such fracture occurred while loading in the pull direction. Cycles 21 and 22 were completed with a lateral deformation of $6\delta_y$ (72 mm). In the push direction of Cycle 21, although the fracture in the south column propagated across the whole south flange and almost through the web, the wall system maintained good shear capacity and ductility. The north flange of the south column remained intact and acted similar to a pin connection (up to the end of the test), and the column tensile load transferred to the foundation through the infill plate and the north flange. In the pull direction, the fracture in the south column closed and the wall reached its maximum base shear of 2,660 kN. In this cycle, the double-angle connections of the intermediate beam showed some minor, but visible, permanent deformation. Local buckling occurred during Cycle 22 (push direction) in the north-east flange of the north column, right above the column base stiffeners. In the pull direction, at a lateral deformation of approximately 40 mm ($3.3\delta_y$), the north

Table 2. Cyclic Base Shear and Displacement History

Cycle number	Loading type	Base shear, Q (kN)	First story lateral displacement			Top lateral displacement			Second story drift ratio (%)
			δ (mm)	$\mu = \delta/\delta_y$	Drift ratio (%)	δ_r (mm)	$\mu_r = \delta_r/\delta_{r,y}$	Drift ratio (%)	
1	F	200	0.5	0.04	0.03	1.5	0.07	0.04	0.05
2	F	400	1.3	0.11	0.07	3.2	0.15	0.09	0.10
3	F	600	2.3	0.19	0.12	5.2	0.24	0.14	0.15
4	F	800	3.0	0.25	0.16	6.7	0.30	0.18	0.19
5	F	1,000	4.4	0.37	0.24	9.2	0.42	0.25	0.25
6	F	1,000	4.4	0.37	0.24	9.2	0.42	0.25	0.25
7	F	1,000	4.4	0.37	0.24	9.2	0.42	0.25	0.25
8	D	1,920	12	1	0.65	22	1	0.59	0.52
9	D	1,920	12	1	0.65	22	1	0.59	0.52
10	D	1,920	12	1	0.65	22	1	0.59	0.52
11	D	2,320	22	1.8	1.19	36	1.6	0.96	0.73
12	D	2,320	26	2.2	1.41	41	1.9	1.09	0.79
13	D	2,150	24	2	1.30	38	1.7	1.01	0.73
14	D	2,400	36	3	1.95	56	2.5	1.49	1.05
15	D	2,350	36	3	1.95	56	2.5	1.49	1.05
16	D	2,320	36	3	1.95	56	2.5	1.49	1.05
17	D	2,500	48	4	2.60	74	3.4	1.97	1.36
18	D	2,450	48	4	2.60	74	3.4	1.97	1.36
19	D	2,625	60	5	3.25	97	4.4	2.58	1.94
20	D	2,350	60	5	3.25	96	4.3	2.54	1.86
21	D	2,200	72	6	3.90	112	5.1	2.98	2.09
22	D	2,050	72	6	3.90	111	5.0	2.96	2.04
23	D	2,060	84	7	4.55	126	5.7	3.36	2.20
24	D	1,900	84	7	4.55	125	5.7	3.33	2.15
25	D	1,830	96	8	5.20	137	6.2	3.65	2.15

Note: D = displacement control; F = force control.

flange of the north column and half the web fractured, causing a decrease in the shear capacity of the system of about 20%, after which the capacity of the system increased considerably (because of the same phenomenon as mentioned for the south column), and the half-cycle was completed to the target displacement. From this point forward, the columns both behaved as though they were pinned at the base when in tension. In Cycles 23 and 24, a lateral displacement of $7\delta_y$ (84 mm) was applied. In Cycle 23, the wall response produced a stable and relatively wide hysteresis curve with good lateral strength (in excess of 2,000 kN in both directions), whereas in Cycle 24, the wall showed a similar response, but with about 10% less shear capacity. Cycle 25 was completed with a lateral deformation of $8\delta_y$ (96 mm), and the specimen again showed a stable and relatively wide hysteresis curve with an average base shear for the two directions of 1,900 kN, which was still greater than the nominal shear capacity indicated in Fig. 3.

Despite the fact that the wall could still resist considerable shear force (more than 70% of the maximum base shear achieved), the test was terminated after 25 cycles of load because the tears in the first story infill plate began to grow more rapidly. Although the second story was subjected to substantial nonlinear behavior, no tears occurred in the second story infill plate. Comparing the drift ratios in Table 2 shows that the contribution of the second story to the lateral deformation ductility of the system decreased in the last cycle because of the growth of the tears in the first story infill plate. However, hysteresis curves for the second story show that it absorbed a considerable amount of energy, even in the last cycle. At the end of the test, the second story had dissipated 21% of the cumulative energy dissipated by the first story.

For a qualitative comparison with the response of the modular wall tested, the result of another SPSW test by Driver et al. (1997), commonly referenced by researchers as evidence of the excellent cyclic behavior that can be achieved using the traditional SPSW configuration with moment-resisting connections, is also shown in Fig. 3. It was a 4-story SPSW with a total height of 7,420 mm and a distance between column centerlines of 3,050 mm. The columns were W310 × 118 sections (W12 × 79), and the infill plates in the bottom two and the top 2 stories were 4.8 and 3.4 mm thick, respectively, with no splices. This wall reached its maximum base shear at a ductility ratio of 5, and attained a maximum ductility ratio of 9 during the test. Although the modular test specimen has a smaller elastic stiffness, lower yield strength, and larger yield displacement that arise mainly because of the effects of the simple beam-to-column connections, bolted infill plates, and geometrical differences, in terms of overall ductility and robustness, the walls demonstrated remarkably similar behavior.

Discussion

One interesting result of this test is the shear connection performance and its influence on the overall system behavior. Fig. 4 shows the relative (column-minus-beam) rotations between the beam ends and the adjacent column at the north-side connections. (No readings were obtained during Cycle 19 because of an instrument malfunction.) A positive rotation is clockwise when looking from east toward west, so a positive relative rotation represents the closing of the joint in the story below the connection. Fig. 4 indicates that the double-angle connections provided very good rotational freedom at the beam-to-column joints during the inelastic cycles, especially at the intermediate beam where the beam depth and angle length were smaller. The connection angles in the first story showed the first sign of slight yielding (via the whitewashed surfaces on one side of the wall) at the bolt line connecting the long legs to the column flanges during cycle 17 ($4\delta_y$); toward the end of the test, the yielding had spread to the short legs connected to the beam web. At the end of the test, limited

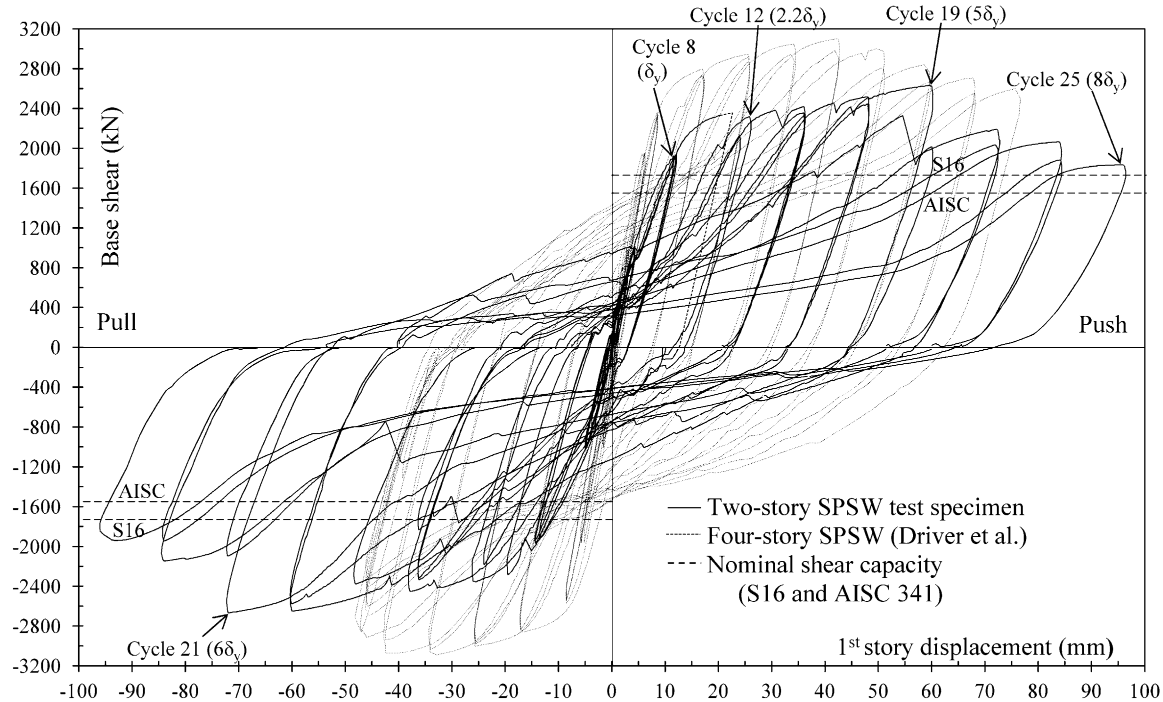

Fig. 3. Hysteresis curves for the first story lateral displacement versus base shear

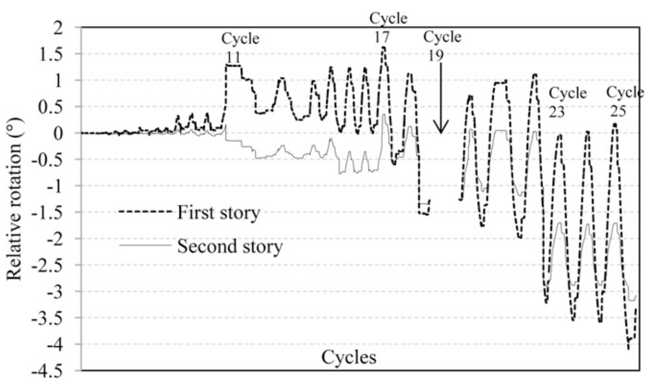

Fig. 4. Relative rotations between north end of beams and adjacent column

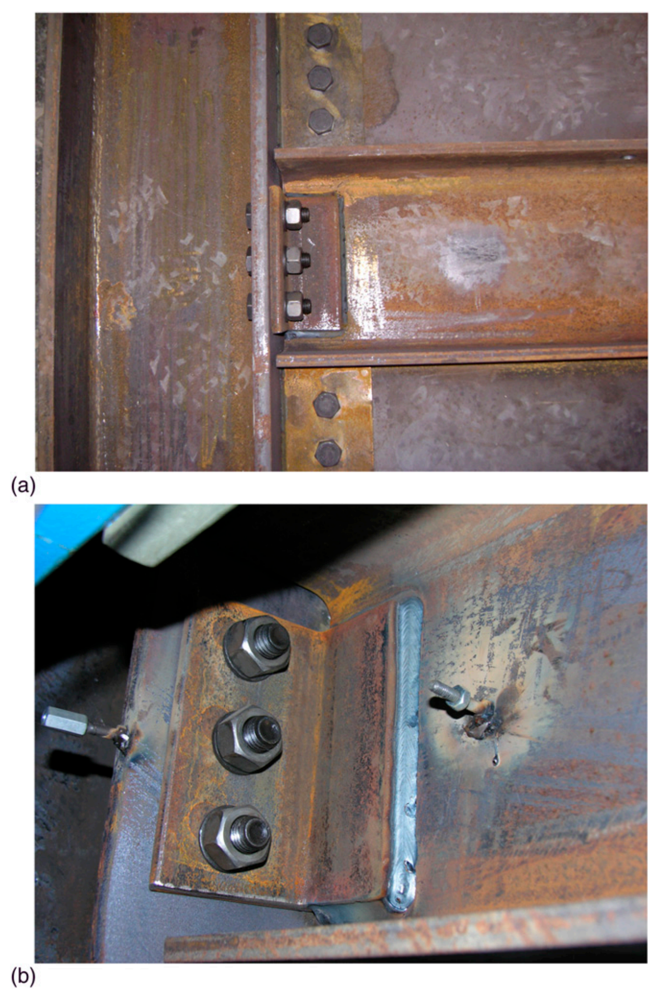

Fig. 5. Shear connection: (a) as fabricated; (b) at end of test

permanent deformation existed in the connections in the first story, whereas the connections in the second story exhibited signs of only minor yielding. Fig. 5(a) shows the first story connection as fabricated, and Fig. 5(b) shows one of these connections at the end of the test, indicating that it underwent little inelastic response and did not deteriorate even when the specimen was loaded to its greatest deformations. Because of the importance of the shear connection to the low-seismic SPSW concept, this result supports its use when moment-resisting frame connections are not needed to meet strength or stiffness criteria. The application of simple beam-to-column connections in the test specimen resulted in robust connection performance, limited the demand on the columns, and enhanced the total energy dissipation capacity of the system by pushing the second story far into nonlinear response. Although their use may not be appropriate when extremely high ductility and maximum redundancy are needed, they appear to be well-suited for applications in low-, or even moderate-, seismic regions.

As predicted by a finite-element analysis of the test specimen, the column strain readings showed that partial yielding occurred in the first story columns right below the beam-to-column connections. The yielding was concentrated in the column webs and extended downward a distance of approximately 250 mm from the intermediate beam's lower flange. Minor yielding also occurred in the inner column flanges that extended downward a distance of approximately 120 mm, but no yielding occurred in the outer column flanges. The strains in the yielded areas remained well below the strain-hardening strain, although significant shear deformations were evident in the columns at this location by Cycle 21 ($6\delta_y$). No collapse mechanism developed in the system, despite the large localized deformations that were present in the columns by the end of the test.

Although the overall SPSW system performed well throughout the test, clearly rupture of a portion of the column cross section at the base is not an outcome that is tolerable in practice. Nevertheless, detailing of the column bases to enhance ductility can only improve the observed SPSW system behavior. The columns performed as intended up to a first story displacement of $5\delta_y$, when the first column tear initiated. Thereafter, they behaved as if pinned at the base and permitted investigation of the behavior of the modular SPSW system at very large displacement ductilities.

Summary and Conclusions

A SPSW concept consisting of a modular construction technique with simple fabrication details and shear connections at the frame joints is proposed for adoption in low-seismic regions where extremely high levels of ductility and redundancy are not required. Based on the proposed scheme, a large-scale, 2-story SPSW was designed using performance-based criteria that resulted in columns considerably smaller than those that would have been required based on current seismic design provisions. The wall was tested under gravity and cyclic lateral loading and demonstrated very good performance and energy dissipation capacity under 25 cycles of loading to a story displacement of eight times the yield displacement.

The conventional double-angle shear connections showed remarkably good performance with no significant damage, even at the end of the test after many nonlinear cycles. They provided rotational freedom at the beam-to-column joints, which reduced the demand on the columns compared with the use of moment-resisting connections. The rotation also tended to improve the distribution of yielding in the infill plates, potentially increasing the total energy dissipated by the system. Neither the one-sided infill plate lap splices nor the bolted nature of the system contributed to any deterioration, and they permitted the full development of the infill plate capacity in the critical story. The wall reached its maximum shear capacity at a lateral drift ratio of 3.9%, which is well beyond the displacement ductilities expected from limited- and moderate-ductility seismic systems. Therefore, the SPSW concept discussed herein that aims primarily to achieve low in-place cost appears to be well-suited for use in low-seismic regions. The development of appropriate capacity design provisions for these SPSWs is ongoing, and they will be proposed in a future paper.

Acknowledgments

The test specimen was fabricated and donated by Supreme Steel Edmonton. Funding for this research program was provided by the Natural Sciences and Engineering Research Council, Steel Structures Education Foundation, Alberta Heritage Foundation, Canadian Society for Civil Engineering, and the University of Alberta. All support is gratefully acknowledged.

References

AISC. (2010a). "Seismic provisions for structural steel buildings." *ANSI/AISC 341-10*, AISC, Chicago.

AISC. (2010b). "Specification for structural steel buildings." *ANSI/AISC 360-10*, AISC, Chicago.

Applied Technology Council (ATC). (1992). "Guidelines for cyclic seismic testing of components of steel structures." *Rep. No. 24*, Applied Technology Council, Redwood City, CA.

ASCE. (2007). "Seismic rehabilitation of existing buildings." *ASCE/SEI 41-06*, ASCE, New York.

ASCE. (2010). "Minimum design loads for buildings and other structures." *ASCE/SEI 7-10*, ASCE, Reston, VA.

Berman, J. W., and Bruneau, M. (2005). "Experimental investigation of light-gauge steel plate shear walls." *J. Struct. Eng.*, 131(2), 259–267.

Berman, J. W., and Bruneau, M. (2008). "Capacity design of vertical boundary elements in steel plate shear walls." *Eng. J.* 45(1), 57–71.

Caccese, V., Elgaaly, M., and Chen, R. (1993). "Experimental study of thin steel-plate shear walls under cyclic load." *J. Struct. Eng.*, 119(2), 573–587.

Canadian Standards Association (CSA). (2004). "General requirements for rolled or welded structural quality steel/structural quality steel." *CSA G40.20-04/G40.21-04 (R2009)*, Canadian Standards Association, Mississauga, ON, Canada.

Canadian Standards Association (CSA). (2009). "Design of steel structures." *CAN/CSA S16-09*, Canadian Standards Association, Mississauga, ON, Canada.

Chen, S.-J., and Jhang, C. (2011). "Experimental study of low-yield-point steel plate shear wall under in-plane load." *J. Construct. Steel Res.*, 67(6), 977–985.

Dastfan, M., and Driver, R. G. (2012). "Ductile steel plate shear walls with PEC columns." *Structural Engineering Rep. 298*, Dept. of Civil and Environmental Engineering, Univ. of Alberta, Edmonton, AB, Canada.

Driver, R. G., Kulak, G. L., Kennedy, D. J. L., and Elwi, A. E. (1997). "Seismic behaviour of steel plate shear walls." *Structural Engineering Rep. 215*, Dept. of Civil and Environmental Engineering, Univ. of Alberta, Edmonton, AB., Canada.

Driver, R. G., and Moghimi, H. (2011). "Modular construction of steel plate shear walls for low and moderate seismic regions." *Proc., Structures Congress*, Structural Engineering Institute, ASCE, Reston, VA, 758–769.

FEMA. (1997). "NEHRP commentary on the guidelines for the seismic rehabilitation of buildings." *FEMA 274*, Washington, D.C.

Moghimi, H., and Driver, R. G. (2011). "Effect of regular perforation patterns on steel plate shear wall column demands." *Proc., Structures Congress*, Structural Engineering Institute, ASCE, Reston, VA, 2917–2928.

Timler, P. A., and Kulak, G. L. (1983). "Experimental study of steel plate shear walls." *Structural Engineering Rep., No. 114*, Dept. of Civil Engineering, Univ. of Alberta, Edmonton, AB, Canada.

Tromposch, E. W., and Kulak, G. L. (1987). "Cyclic and static behaviour of thin panel steel plate shear walls." *Structural Engineering Rep., No. 145*, Dept. of Civil Engineering, Univ. of Alberta, Edmonton, AB, Canada.