Steel Plate Shear Walls in the Upcoming 2010 AISC Seismic Provisions and 2009 Canadian Standard S16

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INTRODUCTION

Steel plate shear walls were introduced into US codes as "Special Plate Shear Walls" (SPSW) in the 2005 edition of the AISC Seismic Provisions for Structural Steel Buildings (AISC 2005), also referred to as AISC 341; the provisions were derived from those in Canadian Standards Association (CSA) standard CAN/CSA-S16-01 (CSA 2001), Limit States Design of Steel Structures. Steel plate shear wall provisions originally appeared in the 1994 edition of Canadian standard S16 as a non-mandatory appendix.

Since this introduction, the system's use in the US and elsewhere, as well as design studies, has led to a refinement of several aspects of the design requirements, which are being considered for the 2010 AISC editions. Simultaneously, AISC 341 is being revised to clarify the desired behavior and basis of design for each system. These developments have resulted in the following changes for SPSW: explicit definition of the capacity-design mechanism; identification of expected regions of inelastic strain; simplification of the calculation of web-tension angle; clarification of strong-column/weak-beam design procedures; and inclusion of methodologies for perforated web plates and walls with reinforced corner cut-outs. Additionally, changes to AISC 341 that would permit design of columns considering expected forces below those corresponding to the sum of all member capacities are being considered. This paper presents some of the anticipated changes to the SPSW provisions in the context of the broader changes to AISC 341.

Since their original introduction into CSA standard S16, steel plate shear wall design provisions have evolved as new research into their behavior has become available and field experience has been gained. In the upcoming 2009 edition of the standard, changes to the provisions are expected to be largely consistent with those being considered for incorporation into the new AISC seismic provisions, including such issues as improved capacity design guidance and the introduction of a means of design for steel plate shear walls with perforated infill plates. Some of the anticipated changes to S16 are also discussed below.

UPCOMING CHANGES TO AISC 341

OVERARCHING CHANGES

With the wider use of the SPSW system, several aspects of the design methodology have been identified as being in need of clarification, and corresponding changes are under review. Additionally, research into a number of variations on the SPSW system has provided useful tools for designers; these variations are being discussed for inclusion in AISC 341. Most noticeably, AISC is considering a radical reorganization of the seismic provisions to more clearly define the expected behavior of each system and each of the system components.

It should be noted that all changes discussed below are proposals under review by the Task Committee on the Seismic Provisions; they have not been approved and cannot be considered planned changes. The process for updating the provisions also requires both public review and approval by the AISC Committee on Specifications.

With the proposed change in organization, requirements for each system are organized as follows:

1. Scope
2. Basis of Design
3. Analysis
4. Ductile Elements
5. System requirements
6. Members
7. Connections

SECTION F5.1 TO F5.7

The reorganized requirements for SPSW are discussed below, following the section format described above. Note that, per the proposed revised structure of AISC 341, the design of SPSW would be addressed in Section F5.

1. Section F5.1 – Scope: The scope provision simply defines that the Section applies to a specific seismic force resisting system. Here the proposed F5.1 states that Section F5 applies to the design of SPSW.
2. Section F5.2 – Basis of Design: This section identifies any special testing or analysis required for the seismic force resisting system type, or if neither is required. It also defines the expected inelastic behavior of the system. Here the proposed F5.2 states that analysis per Section F5.3 is required, and that webs are expected to yield in tension and that beams are expected to form plastic hinges at each end. This description of expected behavior is essentially the same as in AISC 341-05.
3. Section F5.3 – Analysis: This section defines special analyses required for the seismic force resisting system type, typically a plastic mechanism analysis to establish forces at the limit of the lateral resistance. Here the proposed F5.3

requires that beam and column axial and flexural forces are to be determined from a plastic mechanism analysis in which all webs are at their expected tension strength at the diagonal angle established from frame properties, and the beams have plastic hinges formed at each end. The requirement to consider web-plate yielding was included in AISC 341-05, but was not presented as a plastic-mechanism analysis.

4. Section F5.4 – Ductile Elements: This section identifies areas of members and welded joints where inelastic strain is expected. In such areas, special provisions apply: limitations on attachments, notch toughness, etc. Here the proposed F5.4 identifies the webs and the beams as the ductile elements, with the area of the beam near the column as the expected plastic hinge location. The concept of ductile elements was implicit in AISC 341-05, and protected zones within such members were defined; however, no such zones were identified for SPSW.
5. Section F5.5 – System requirements: This section identifies any system limitations and requirements, such as out-of-plane bracing of connections or members. Here the proposed F5.5 limits the panel aspect ratio, provides for a required column flexural stiffness, prescribes lateral bracing of beams, and presents requirements for openings in webs. Each of these requirements appeared in AISC 341-05. In addition, the proposed F5.5 provides a complementary beam stiffness requirement, which would be new to AISC 341. The strong-column-weak-beam proportioning rule from AISC 341-05 would also appear here; however, this requirement might be superseded by the analysis requirements of F5.3.
6. Section F5.6 – Members: This section includes various member requirements: limits on element width-to-thickness ratios, required strength of members, etc. Here the proposed F5.6 includes both of those, identifying both beams and columns as "highly ductile members," for which low limits on width-to-thickness ratios apply so that significant inelastic rotation can be achieved. Additionally, the required strength of beams and columns would also appear here; however, this requirement might be superseded by the analysis requirements of F5.3. Finally, the proposed section also defines the nominal strength of the web member, as AISC 360, the main Specification for Structural Steel Buildings, does not include such a member.
7. Section F5.7 – Connections: This section includes requirements for the design of connections. In the proposed F5.7, the required strength of web connections is defined based on its expected yield. Additionally, the requirement that the beam-to-column connection comply with Ordinary Moment Frame requirements is included. Both of these requirements appeared in AISC 341-05.

In addition to the seven subsections that follow the typical pattern, there is a section F5.8 in the proposed draft, which includes two variations on the SPSW system. F5.8a defines a SPSW with a regular array of circular openings. F5.8b addresses the design of quarter-circle corner cut-outs in the SPSW.

SECTION F5.8a

The yield stress for hot-rolled steel material typically available in North America results in panel thicknesses that might be less than the minimum panel thickness available from steel producers. In such cases, use of the minimum available thickness may result in large panel force over-strength. Attempts at alleviating this problem were recently addressed by the use of light-gauge, cold-formed steel panels, in a new application by Berman and Bruneau (2003a, 2005). Another approach consists of reducing the strength of the web by adding a regular grid of perforations across the web (Vian and Bruneau 2005). This solution simultaneously helps address the practical concern of utility placement across special plate shear walls, which has been in some cases an impediment to more widespread acceptance of the SPSW structural system. In a regular SPSW, the infill panel which occupies an entire frame bay between adjacent Horizontal Boundary Elements (HBEs) and Vertical Boundary Elements (VBEs) is a protected element, and utilities that may have otherwise passed through at that location must either be diverted to another bay, or pass through an opening surrounded by HBEs and VBEs. This either results in additional materials (for the extra stiffening) or in labor (for the re-location of ductwork in a retrofit, for example).

Special Perforated Plate Shear Walls are a special case of SPSWs in which a special panel perforations layout is used to allow utilities to pass-through and which may be used to reduce the strength and stiffness of a solid panel wall to levels required in a design when a thinner plate is unavailable. This concept has been analytically and experimentally proven to be effective and to remain ductile up to the drift demands corresponding to severe earthquakes (Vian and Bruneau 2005, Purba and Bruneau 2006). A typical hole layout for this system is shown in Fig. 1.

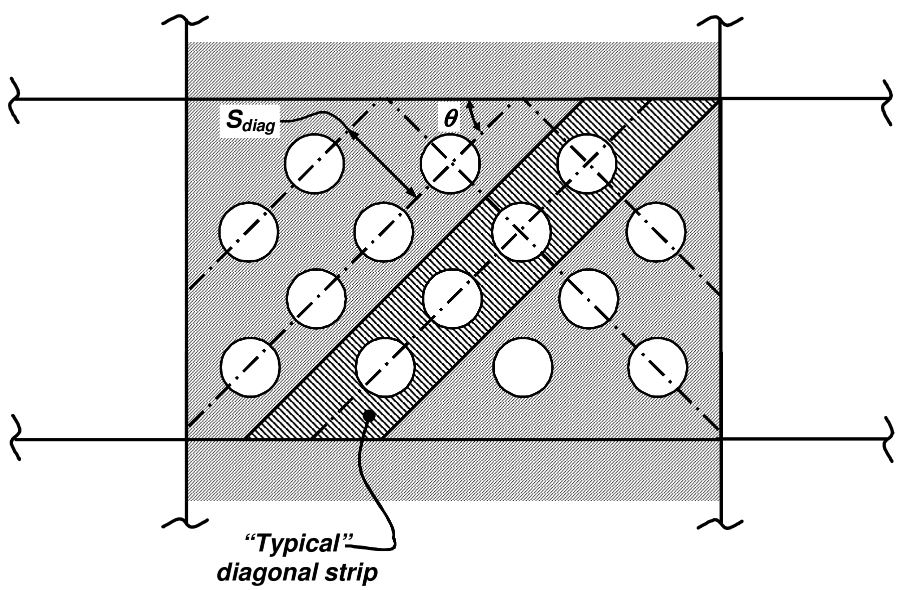

Figure 1: Schematic Detail of Special Perforated Plate Shear Wall

The introduction of an array of circular holes offers numerous design and construction advantages:

- The angle alpha defining the angle of tension stress can be known directly, controlled, and fixed so that the typical design iterations do not necessitate recalculation of the angle nor adjustments to the model;
- A thicker web plate may be employed, permitting larger welds in the attachment to the frame, as well as easier fabrication and placement; and
- Conduit and small-to-medium pipes can cross the SPSW plane without onerous local boundary elements.

Coefficients for calculating the effective thickness to use in strength and stiffness calculations are presented, based on the work of Bruneau and Vian (2005) and Bruneau and Purba (2006).

SECTION F5.8b

It is also possible to allow utility passage through a reinforced cutout designed to transmit the web forces to the boundary frame. While providing utility access, this proposed system provides strength and stiffness similar to a solid panel SPSW system. The openings are located immediately adjacent to the column in each of the top corners of the panel, a location where large utilities are often located. Cut-out radii as large as 500 mm for a ½ scale specimen having a 2000 mm c/c distance between HBEs has been successfully verified experimentally and analytically by Vian and Bruneau (2005) and Purba and Bruneau (2006).

Forces acting in the reinforcing arch are a combination of effects due to arching action under tension forces due to web yielding, and thrusting action due to change of angle at the corner of the SPSW (Figs. 2 and 3). The latter is used to calculate the required maximum thickness of the "opening" corner arch (top left side of Fig. 2, with no web stresses assumed to be acting on it). The arch plate width is not a parameter that enters the solution of the interaction equation in that calculation, and it is instead conservatively obtained by considering the strength required to resist the axial component of force in the arch due to the panel forces at the closing corner (top right side of Fig. 2). Since the components of arch forces due to panel forces are opposing those due to frame corner opening (Fig. 3), the actual forces acting in the arch plate will be smaller than calculated by considering the components individually as done above for design.

Note that when the fish plate is added to the reinforcement arch to facilitate infill panel attachment to the arch, it results in a stiffer arch section that could (due to compatibility of frame corner deformation) partly yield at large drifts. However, Vian and Bruneau (2005) and Purba and Bruneau (2006) showed that the thickness of the flat plate selected per the above procedure is robust enough to withstand the loads alone, and that the presence of the stiffer and stronger T-section is not detrimental to the system performance.

Nonlinear static pushover analysis is a tool that can be used to confirm that the selected reinforcement section will not produce an undesirable "knee-brace effect" or precipitate column yielding or beam yielding outside of the RBS region.

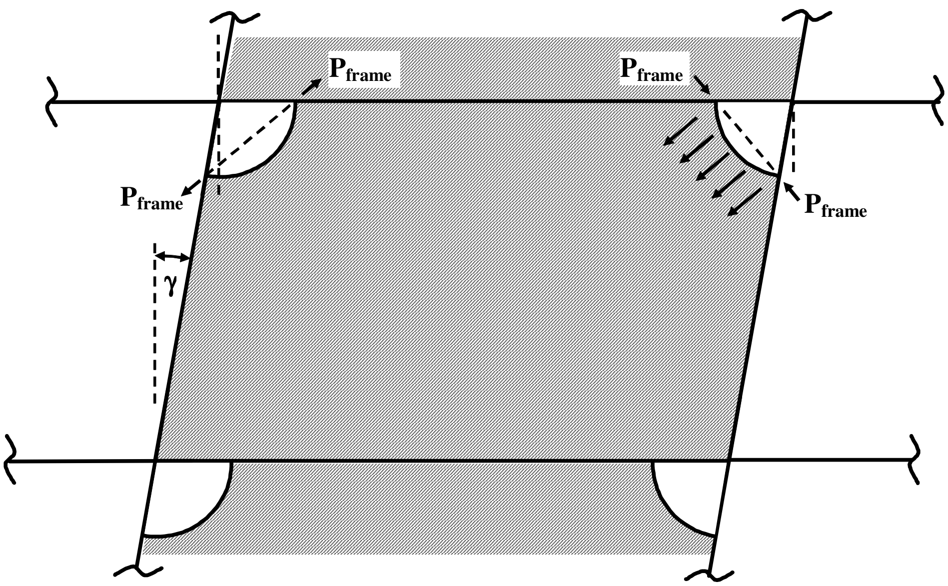

FIGURE 2: Arch End Reactions Due to Frame Deformations, and Infill Panel Forces on Arches Due to Tension Field Action on Reinforced Cut-Out Corner

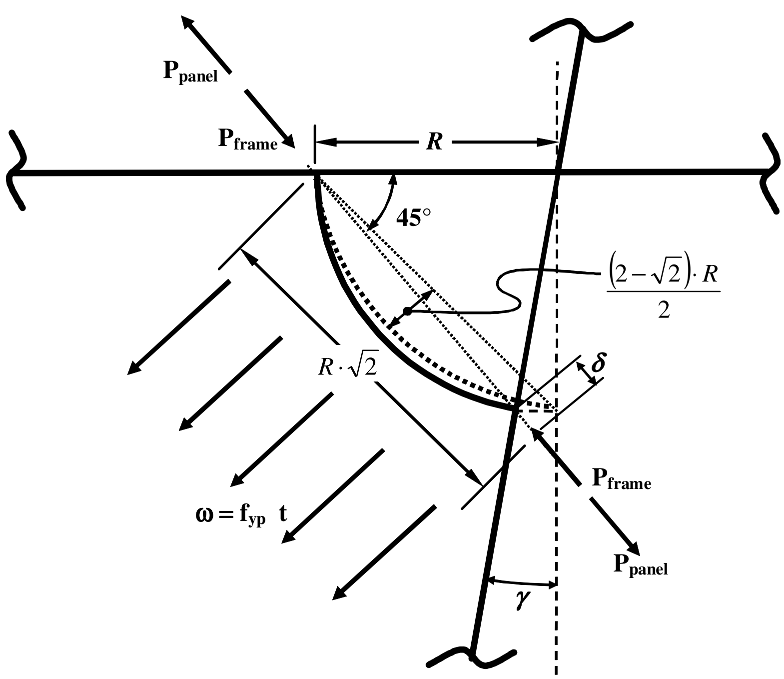

FIGURE 3: Deformed Configurations and Forces Acting on Right Arch

Improved Capacity Design Guidance for VBEs

The AISC 341 commentary states that, "per capacity design principles, all edge boundary elements (HBE and VBE) shall be designed to resist the maximum forces developed by the tension field action of the webs fully yielding." A number of simplified approaches are proposed in the existing commentary to obtain the axial forces, shears, and moments that develop in the boundary elements of the SPSW as a result of the response of the system to the overall overturning and shear, and tension field action in the webs. However, recent work by Berman and Bruneau (2008) demonstrated that some of these methods give incorrect results.

A proposed revised commentary will not include the *Combined linear elastic computer programs and capacity design* method. While this method is capable of producing VBE axial forces that are reasonable relative to pushover results depending on the value of overstrength factor used, it does not provide accurate local bending moments in the VBEs resulting from both web plate yielding and frame action corresponding to the externally applied loads that caused the web plates to yield. The proposed revised commentary will retain the *Indirect capacity design approach* (first introduced in CSA-S16-01 (CSA, 2001)), but will clarify that the method. While being capable of producing reasonable results for approximating VBE capacity design loads, it can be unconservative in some circumstances – guidance will be provided to account for the possibility of significant strength of the surrounding frame and consideration of the full collapse mechanism (Berman and Bruneau 2003b) when determining the amplification factor.

A new *Combined Plastic and Linear Analysis* is proposed. This procedure has been shown to give accurate VBE results compared to push-over analysis (Berman and Bruneau 2008). In this approach, the required capacity of VBEs is found from free body diagrams that account for web plate yielding, moments from plastic hinging of HBEs, axial forces from HBEs, applied lateral seismic loads found from consideration of the plastic collapse mechanism, and base reactions for those lateral seismic loads.

Upcoming Changes to CSA Standard S16

As with the AISC Provisions, the changes described in this paper are proposals that are under discussion by the committee and must yet undergo a full public review before they can be adopted.

The structure and organization of CSA standard S16 will remain the same in the next edition, but will include much of the clarification presented above for the AISC seismic design of SPSW. In particular, there will be improved transparency of and adherence to capacity design principles and an explicit establishment of protected zones. Design provisions for shear walls with plate perforations (including those with corner cut-outs) will also be consistent with the proposed AISC Provisions described above. In addition, the moment-resisting frame, acting without the infill plate, will be required to resist a minimum of 25% of the design shear force at each story to ensure that the implied benefits of the redundancy of the system are realized.

The strip model (Thorburn et al., 1983) is a well-established method of analysis for steel plate shear walls wherein the tension field in the infill plate is idealized as a series of tension strips. However, one its drawbacks is that the angle of the tension strips needs to be recalculated, and the model revised, as the design of the frame members evolves, making the method somewhat cumbersome for larger structures. Shishkin et al. (2005) have established through a detailed analytical investigation that the overall behavior of steel plate shear walls is insensitive to the angle of inclination assumed in the model. As such, for aspect ratios within the range $0.6<L/h<2.5$, where L is the center-to-center column spacing and h is the story height, a constant strip angle from the vertical of 40 degrees may be used. Shishkin et al. (2005) have shown that this simplification gives an accurate ultimate capacity in a pushover analysis and a slightly conservative elastic stiffness.

In order to provide effective anchorage to the infill plate tension field, the current edition of S16 requires that the column flexibility parameter, ω_h, be limited to a maximum value of 2.5. This parameter is defined as follows:

$$\omega_h = 0.7h \sqrt[4]{\frac{w}{2LI_c}} \tag{1}$$

where w is the infill plate thickness and I_c is the moment of inertia of the columns. The limit on ω_h indirectly imposes a minimum column moment of inertia of:

$$I_c = 0.00307 \frac{wh^4}{L} \tag{2}$$

Although a stiffness limit for the top beam exists in the current edition of S16 with the same intent, it requires an iterative solution and in many cases simply cannot be met. Dastfan and Driver (2008) have developed an equation to address the stiffness requirements of the top panel (and the panel at the base of the wall if the tension field is anchored by a girder), where the assumptions behind ω_h do not apply, in a manner consistent with the method used for columns. Due to the combined effects of the top beam and the adjacent column behaviors on the plate tension field uniformity in the top panel, the moment of inertia of both members must be considered. For consistency, the requirement is formulated in terms of a boundary member flexibility parameter, ω_L:

$$\omega_L = 0.7 \sqrt[4]{\left(\frac{h^4}{I_c} + \frac{L^4}{I_b}\right)\frac{w}{4L}} \tag{3}$$

where I_b is the moment of inertia of the beam anchoring the extreme panel and all other symbols have been defined previously. The upper limit on ω_L is 2.5 in the top panel and 2.0 in the panel at the base of the wall.

The existing provisions for steel plate shear walls with “limited ductility”—with a ductility force reduction factor, R_d, of 2.0 (as compared to 5.0 for the “ductile” wall)—will remain in the standard, but with nominal capacity design requirements added. This system is limited to structures not exceeding 60 m in height.

CONCLUSIONS

This paper has presented a number of proposed changes for the AISC and CSA seismic design of steel plate shear walls. These enhancements are expected to increase the range of applicability of this structural system.

ACKNOWLEDGEMENT

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The AISC Seismic Provisions are the work of AISC Task Committee 9, as well as the AISC Committee on Specifications. The Canadian provisions are the work of Canadian Standards Association Technical Committee on Limit States Design for Steel Structures (S16). The work of these committees in producing, reviewing, and refining the provisions on steel plate shear walls has merely been described by the authors of this paper.

Any opinions, findings, conclusions, and recommendations presented in this paper are those of the authors and do not necessarily reflect the views of the sponsors.

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