Steel Plate Shear Walls Are Not Plate Girders

JEFFREY W. BERMAN and MICHEL BRUNEAU

Steel plate shear wall(s) (SPSW) can be an attractive option for lateral load resisting systems in both new and retrofit construction. Prior to key research performed in the 1980s, the design limit state for SPSW was considered to be out-of-plane buckling of the infill panel. To prevent buckling, engineers designed steel walls with heavily stiffened infill plates that were not economically competitive with reinforced concrete shear walls. However, several experimental and analytical studies using both quasi-static and dynamic loading showed that the post-buckling strength of thin SPSW can be substantial (Thorburn, Kulak, and Montgomery, 1983; Timler and Kulak, 1983; Tromposch and Kulak, 1987; Roberts and Sabouri-Ghomi, 1992; Roberts and Sabouri-Ghomi, 1992; Caccese, Elgaaly, and Chen, 1993; Elgaaly, Caccese, and Du, 1993; Driver, Kulak, Kennedy, and Elwi, 1998; Elgaaly and Liu, 1997; Elgaaly, 1998; Rezai, 1999; Lubell, Prion, Ventura, and Rezai, 2000; Berman and Bruneau, 2003a). Based on some of this research, the Canadian Standards Association steel design standard CAN/CSA S16-01 has implemented design clauses for SPSW allowed to buckle in shear and develop tension field action (CSA, 2001).

In much of the SPSW literature, the analogy that the ultimate behavior of SPSW is similar to a cantilevered vertical plate girder has often been made. However, whether this analogy is purely qualitative or whether it also has quantitative merit, has not been thoroughly discussed. The purpose of this paper is to review the shear strengths of both SPSW and vertical cantilevering plate girders, compare them with results from experimental studies, and show that the tension field inclination angles for SPSW and plate girders are substantially different (due to their different associated boundary conditions) which leads to different tension field strengths. These comparisons also demonstrate that designing SPSW following standard plate girder design requirements, such as those of Appendix G of the American Institute of Steel Construction *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 1999), hereafter referred to as the AISC *LRFD Specification*, can lead to walls with larger-than-expected strengths (over-designed walls), which can negate the purpose of performing capacity design and result in an uneconomical use of steel.

Jeffrey W. Berman is a Ph.D. candidate, department of civil, structural, and environmental engineering, University at Buffalo, Amherst, New York.

Michel Bruneau is director, MCEER, professor, department of civil, structural, and environmental engineering, University at Buffalo, Amherst, New York.

REVIEW OF SPSW BEHAVIOR AND DESIGN

A typical steel plate shear wall (Figure 1) consists of horizontal and vertical boundary elements (that may or may not carry gravity loads), and thin infill plates that buckle in shear and form a diagonal tension field to resist lateral loads. A review of SPSW behavior, and some of the experimental and analytical work that led to this understanding, is presented in Kulak, Kennedy, Driver, and Medhekar (2001). Only a brief review is presented here.

Based on an elastic strain energy formulation, Timler and Kulak (1983) derived the following equation for the inclination angle of the tension field, α, in a SPSW infill plate:

$$\alpha = \tan^{-1}\sqrt[4]{\frac{1+\dfrac{tL}{2A_c}}{1+th\left(\dfrac{1}{A_b}+\dfrac{h^3}{360 I_c L}\right)}} \tag{1}$$

where

- t = thickness of the infill plate
- h = story height
- L = bay width
- I_c = moment of inertia of the vertical boundary element
- A_c = cross-sectional area of the vertical boundary element
- A_b = cross-sectional area of the horizontal boundary element

The flexural stiffness of the horizontal boundary elements was excluded in the derivation because the opposing tension fields that develop above and below these intermediate horizontal members almost cancel out and induce little significant flexure there. Using the inclination angle given by Equation 1, an analytical model, known as a strip model, in which the infill plates are represented by a series of pin-ended, tension only strips, was developed by Thorburn and others (1983), and subsequently refined by Timler and Kulak (1983). A typical strip model representation of a SPSW is shown in Figure 2 and the accuracy of the strip model has been verified through comparisons with experimental results such as in Figure 3, which has been adapted from Driver and others (1998).

Using the strip model as a basis, the ultimate strength of steel plate shear walls can be found using plastic analysis (Berman and Bruneau, 2003b). Assuming that the horizontal boundary elements are simply connected to the vertical boundary elements, the vertical boundary elements are simply connected to the base, and that all the boundary elements are rigid until plastic hinge formation, the collapse mechanism shown in Figure 4 can be used to establish the ultimate strength of a single story SPSW as:

$$V = \frac{1}{2} t L F_y \sin 2\alpha \tag{2}$$

where

V = horizontal shear force applied to the wall

F_y = yield stress of the infill plate and all other parameters have been previously defined

Berman and Bruneau (2003b) also give ultimate strength equations for different SPSW configurations (i.e. multistory SPSW, and SPSW with rigid connections between the horizontal and vertical boundary elements); however, for comparison with the shear strength of plate girders, where no consideration is given to plastic hinging of the flanges, this single story SPSW with simple horizontal elements is appropriate. Verification of the collapse mechanism (Figure 4) and the resulting ultimate strength equation (Equation 2) is given by the comparison with ultimate strengths obtained for SPSW in experimental studies in Table 1, where V_{uexp} is the experimentally obtained ultimate base shear, V_{upred} is the ultimate base shear predicted from Equation 2, and all other parameters have been previously defined. These results show that for SPSW with either simple or semi-rigid horizontal boundary element (HBE) to vertical boundary element (VBE) connections, Equation 2 reasonably predicts the ultimate strength. For SPSW with rigid HBE-to-VBE connections, Equation 2 underestimates the ultimate strength, which is expected since there is no consideration given to plastic hinging of the boundary frame in the derivation.

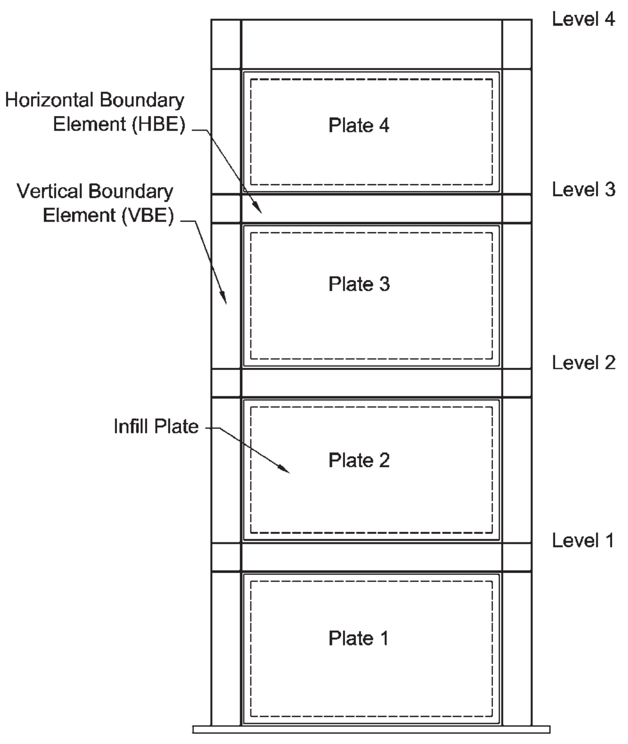

Fig. 1. Typical Steel Plate Shear Wall.

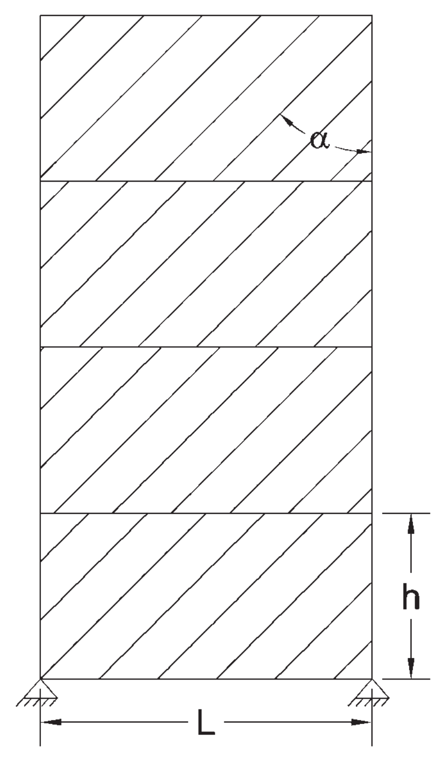

Fig. 2. Typical Strip Model.

Table 1. Comparison of Experimental and Predicted (Equation 2) Ultimate Strengths for SPSW.

Case	Study	Specimen ID	No. Stories	h (mm)	L (mm)	t (mm)	L/t	h/L	F_y (MPa)	α (°)	V_{uexp} (kN)	V_{upred} (kN) Eq. 2	% Error for Eq. 2
(i) Simple (Physical Pin) HBE-to-VBE Connections													
1	Timler and Kulak (1983)	—[c]	1	2500	3750	5	750	0.67	271	42.7	2698	2531	-6.2
2	Roberts and Sabouri - Ghomi (1992)	SW2	1	370	370	0.83	446	1	219	45.0	35.1	33.6	-4.2
3		SW3	1	370	370	1.23	301	1	152	45.0	38.2	34.6	-9.5
4		SW14	1	370	450	0.83	542	0.82	219	45.0	44.5	40.9	-8.1
5		SW15	1	370	450	1.23	366	0.82	152	45.0	45.3	42.1	-7.1
(ii) Semi-Rigid HBE-to-VBE Connections (Web-Angle or Other)													
6	Berman and Bruneau (2003a)	F2	1	1829	3658	0.91	4020	0.50	221	45	364	368	1.05
7	Elgaaly (1998)	SWT11[b]	2	1118	1380	2.28	605	0.81	239	41.5	370	373.1	0.85
8		SWT15	2	1118	1380	2.28	605	0.81	239	41.3	426	372.9	-12
9	Caccese and others (1993)	S22	3	838	1244	0.76	1637	0.67	256	42.2	142	120.4	-15
10		S14	3	838	1244	1.9	655	0.67	332	40.2	356	386.8	8.64
(iii) Rigid HBE-to-VBE Connections													
11	Lubell and others (2000)	SPSW1[a]	1	900	900	1.5	600	1	320	36.9	210	207.3	-1.3[d]
12		SPSW2	1	900	900	1.5	600	1	320	36.9	260	207.3	-20.3[d]
13	Driver and others (1998)	—[c]	4	1927	3050	4.8	635	0.63	355	41.1	3080	2578	-16.3[d]

[a] Testing stopped due to failure of lateral bracing
[b] Testing stopped due to column buckling
[c] Not applicable
[d] Larger error values here due to the presence of rigid boundary frame member connections

It is worthwhile to briefly mention some other design issues for SPSW which are addressed in CAN/CSA S16-01. Horizontal and vertical boundary elements should be designed to elastically resist development of the full expected tensile capacity of the infill plates. This ensures that the infill plate can yield in tension prior to plastic hinging of the boundary elements (providing for substantial energy dissipation in seismic applications). Such capacity design can be achieved by designing the boundary elements for the forces found from pushover analysis of the strip model, or indirectly from the procedure in CAN/CSA S16-01. The connection of the infill plate to the boundary elements should also be designed for the expected tensile capacity of the infill plate and can use either a welded or bolted configuration. Four different connection details were developed, tested, and found to be equivalent by Schumacher, Grondin, and Kulak (1999). Furthermore, the vertical boundary elements should satisfy a minimum stiffness requirement (given in CAN/CSA S16-01) to prevent excessive deformations under the tension field action of the web plate. Finally, stiff horizontal boundary elements should be provided at the top and bottom of a SPSW to anchor the tension field.

THE PLATE GIRDER ANALOGY

Throughout much of the literature regarding SPSW that are allowed to buckle in shear and form a diagonal tension field, there is reference to similarities with cantilevered vertical plate girders. In this analogy, the horizontal boundary elements of the SPSW are similar to stiffeners in a plate girder, the vertical boundary elements are similar to the flanges, and the infill plates are similar to the web of a plate girder. Using this analogy, the story height of a SPSW (*h*) is analogous to the stiffener spacing of a plate girder (*a*), and the bay width of a SPSW (*L*) is analogous to the plate girder depth (*h*). While this analogy is useful in visualizing the behavior of SPSW (especially since many structural engineers are familiar with plate girder behavior) it is misleading to assume, for reasons given in the next sections, that plate girder shear strength equations, such as those of Appendix G of the AISC *LRFD Specification* (AISC, 1999), accurately assess the strength of SPSW.

REVIEW OF PLATE GIRDER SHEAR BEHAVIOR

The shear strength of plate girders (when tension field action is included) is governed by the summation of two components, namely, the buckling strength (V_{cr}) and strength due to tension field action (V_{tf}). Salmon and Johnson (1996) give a thorough review of the shear strength of plate girder webs; what follows is a brief summary of key behavior and modeling relevant to the context of this paper. It should be noted that in the following section the nomenclature used is that consistent with plate girders; *h* is the web depth, t_w is the web thickness, and *a* is the stiffener spacing.

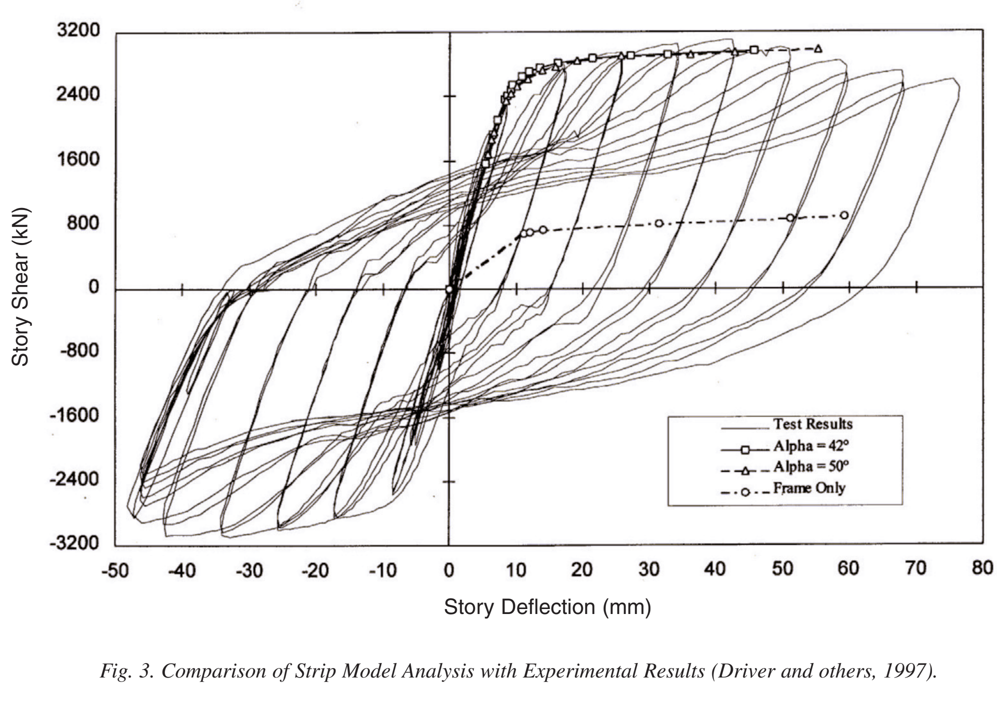

Fig. 3. Comparison of Strip Model Analysis with Experimental Results (Driver and others, 1997).

Plate Girder Shear Buckling Strength

In general, the elastic shear buckling strength of a plate girder web (Timoshenko and Woinowski-Krieger, 1959) is given by:

$$\tau_{cr} = k_v \frac{\pi^2 E}{12\left(1-\nu^2\right)\left(\frac{\textit{short dimension}}{t}\right)^2} \tag{3}$$

where

E = Young's Modulus
ν = Poisson's Ratio
t = web thickness
short dimension = smaller of h (the web depth) and a (the stiffener spacing)
k_v = plate buckling coefficient given by:

$$k_v = 5.34 + 4.0\left(\frac{\textit{short dimension}}{\textit{long dimension}}\right)^2 \tag{4}$$

where

long dimension = longer of the web depth, h, and the stiffener spacing, a

In Appendix G of the AISC *LRFD Specification* the equation for k_v has been modified so that only one equation is used regardless of which dimension (h or a) is larger. This modified equation is:

$$k_v = 5.0 + \frac{5.0}{\left(\frac{a}{h}\right)^2}$$
$$k_v = 5.0 \text{ if } \frac{a}{h} > 3.0 \text{ or } \frac{a}{h} > \frac{260}{\left(h/t_w\right)^2} \tag{5}$$

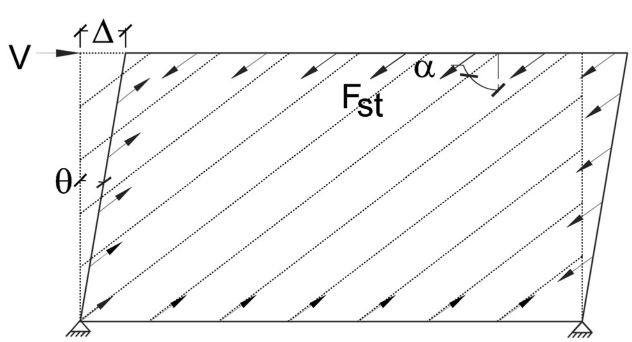

Fig. 4. Single Story Steel Plate Shear Wall Collapse Mechanism.

and, as discussed in Salmon and Johnson (1996), there is a negligible difference between Equation 5 and Equation 4 for typical plate girder and SPSW web slenderness ratios.

The shear buckling strength of plate girder webs, V_{cr}, can then be written as:

$$V_{cr} = 0.6F_{yw}A_wC_v \tag{6}$$

where

F_{yw} = yield strength of the web
A_w = web area (ht_w)
C_v = ratio of buckling stress to shear yield stress and is given by the AISC *LRFD Specification* Appendix G equations:

$$C_v = 1.0 \text{ for } \frac{h}{t_w} \le 1.10\sqrt{\frac{k_vE}{F_{yw}}}$$
$$C_v = \frac{1.10\sqrt{k_vE/F_{yw}}}{h/t_w} \text{ for } 1.10\sqrt{\frac{k_vE}{F_{yw}}} \le \frac{h}{t_w} \le 1.37\sqrt{\frac{k_vE}{F_{yw}}}$$
$$C_v = \frac{1.51k_vE}{\left(h/t_w\right)^2 F_{yw}} \text{ for } \frac{h}{t_w} > 1.37\sqrt{\frac{k_vE}{F_{yw}}} \tag{7}$$

The first C_v expression represents compact webs that can yield in shear prior to buckling, the second expression represents a linear transition from elastic buckling to shear yield, and the third expression represents the elastic buckling curve for slender webs. Figure 5 shows C_v versus the web slenderness, h/t_w. This plot is extended to values of web slenderness outside the range typically used in plate girders

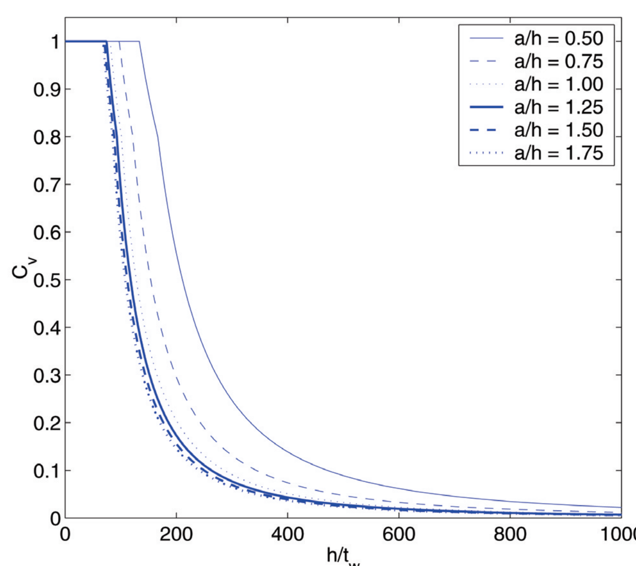

Fig. 5. C_v versus Web Slenderness Ratio (h/t_w).

Table 2. Sensitivity of C_v.

h/t_w [a]	Scenario [b]	C_v [c] (Eq. 7)	k_v [c] (Eq. 5)	C_v [c] (Eq. 7)
144	3658 mm long wall 25.4 mm thick	$k_v/24$	10.0	0.42
576	3658 mm long wall 25.4 mm thick	$k_v/379$	10.0	0.026
864	5486 mm long wall 25.4 mm thick	$k_v/850$	7.2	0.0084

[a] *h*, in this table is the bay width of the SPSW or depth of the plate girder.
[b] a constant height of 3658 mm (12 ft) was assumed.
[c] a steel yield stress of 345 MPa (50 ksi) was assumed in the calculation of these values.

to demonstrate that for thin, unstiffened, SPSW (which typically have slenderness ratios of 300 to 1000) the shear buckling strength is approximately zero. Table 2 shows this trend for some typical wall configurations (recall that the bay width, L, of a SPSW is the web depth, h, of a plate girder). Note that the first row of Table 2 represents a SPSW that is not thin and is not likely to be found in practice unless shear buckling is used as the design limit state. The second and third rows of Table 2 represent more typical and economical SPSW that would be designed to allow shear buckling and the formation of tension field action. Figure 5 and Table 2 demonstrate that the buckling strength of thin, unstiffened SPSW can be neglected.

Plate Girder Tension Field Strength

Plate girder webs can exhibit significant post-buckling strength through the development of a diagonal tension field. The tension field strength, V_{tf}, of plate girder webs (as it appears in the AISC *LRFD Specification*) was derived by Basler (1961). From horizontal and rotational equilibrium, as well as the trigonometric identity sin(γ)cos(γ) = sin(2γ), for the free body diagram of Figure 6, the tension field strength of a single plate girder web panel is:

$$V_{tf} = \sigma_t \frac{ht_w}{2} \sin 2\gamma \tag{8}$$

where

σ_t = membrane tension stress in the web

γ = angle of inclination of the tension field stress measured from the horizontal

and the other parameters have been previously defined. The maximum membrane tension stress can be determined from the energy-of-distortion failure criterion theory described in Salmon and Johnson (1996) and depicted in Figure 7. The principal stress, σ_1, can be expressed as $\tau_{cr} + \sigma_t$ and the maximum strength is reached when σ_1 reaches the tension yield stress of the web, F_{yw}. Therefore, the limiting equation for shear strength from tension field action is:

$$V_{tf} = \left(F_{yw} - \tau_{cr}\right)\frac{ht_w}{2} \sin 2\gamma \tag{9}$$

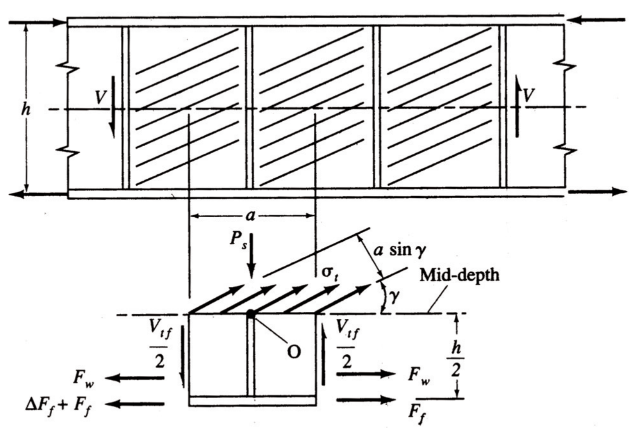

Fig. 6. Free Body Diagram of Stiffener Region (Salmon and Johnson, 1996).

For thin webs, in other words, webs in which the buckling strength is negligible (as in typical SPSW), Equation 9 becomes:

$$V_{tf} = F_{yw} \frac{ht_w}{2} \sin 2\gamma \tag{10}$$

The angle of inclination of the tension field stress in a plate girder web, γ, can be found using the force distribution shown in Figure 8a. Considering only the tension field forces tributary to the stiffener (in other words, in the band of width *s*) the partial shear force, ΔV_{tf}, developed as compression in the stiffener is:

$$\Delta V_{tf} = \sigma_t s t_w \sin 2\gamma \tag{11}$$

From the geometry of Figure 8b, $s = h \cos(\gamma) - a \sin(\gamma)$. Substituting this into Equation 11 and taking the derivative with respect to γ, leads to the equation for γ which produces the maximum partial tension field force:

$$\tan 2\gamma = \frac{1}{a/h} \tag{12}$$

Using the trigonometric relationship of Figure 9, this equation can be expressed as:

$$\sin 2\gamma = \frac{1}{\sqrt{1+(a/h)^2}} \tag{13}$$

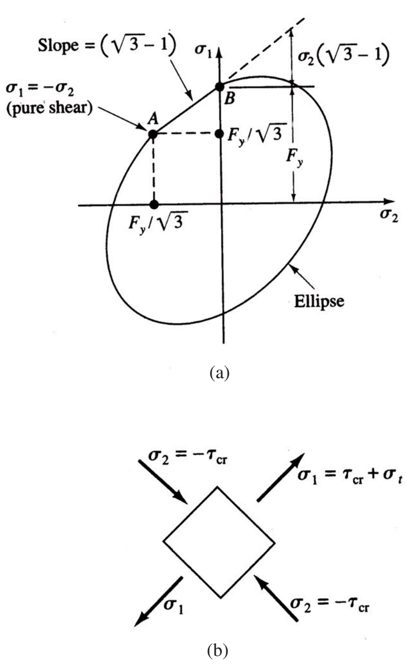

Fig. 7. Energy-of-Distortion Failure Criteria (Salmon and Johnson, 1996).

Equation 13 represents the assumed tension field orientation implicit in the design equations of Appendix G of the AISC *LRFD Specification*. Note that no consideration is given to flexibility of the boundary elements (in other words, the flanges and stiffeners) contrary to what is done in Equation 1 for the tension field orientation angle of SPSW. Although the derivation of Equation 13 is based on a partial tension field, Galambos (1998) noted that Gaylord (1963), and Fujii (1968) have shown that Equation 9 gives the shear strength for a full tension field.

The equations for the strength of plate girders with tension field, V_n, as they appear in Appendix G of the AISC *LRFD Specification* are as follows:

$$V_n = 0.6 F_{yw} A_w \quad \text{for} \quad \frac{h}{t_w} \le 1.10 \sqrt{\frac{k_v E}{F_{yw}}}$$

$$V_n = 0.6 F_{yw} A_w \left(C_v + \frac{1 - C_v}{1.15\sqrt{1+(a/h)^2}} \right) \quad \text{for} \quad \frac{h}{t_w} > 1.10 \sqrt{\frac{k_v E}{F_{yw}}} \tag{14}$$

where all parameters have been previously defined. This is obtained by summation of Equations 6 and 9 (with the shear buckling stress, τ_{cr}, not assumed to be zero), and some algebraic manipulation. V_n from Equation 14 is normalized by the shear yield force ($V_y = 0.6F_{yw}A_w$) and plotted in Figure 10 versus the web slenderness (h/t_w) for different values of panel aspect ratio (a/h) and steel yield stress of 50 ksi. At large values of web slenderness, the shear strength of plate girders is essentially provided by the tension field strength developed at the angle of inclination, γ, which is effectively equivalent to values given by Equation 10. In fact, for values of C_v near zero, Equation 14 reduces to Equation 10 with the approximation of $0.6/1.15 = 0.5$. Note that the limits on the web slenderness and panel aspect ratio have not been included in Figure 10 because they are not applicable to SPSW, as described later.

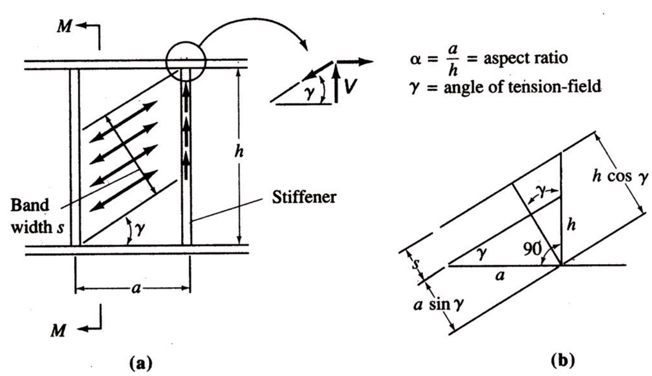

Fig. 8. (a) Force Distribution Near Stiffener (b) Geometry of Tension Field Band (Salmon and Johnson, 1996).

COMPARISON OF SHEAR STRENGTH OF PLATE GIRDERS AND SPSW

Upon examination of Equation 2 and Equation 10 (which is plate girder shear strength for thin webs) it is observed that the only difference between them is that Equation 2 uses the angle of inclination of the tension field (α) as given by Equation 1, while Equation 10 uses the angle (γ) as given by Equation 13. Table 3 gives values of α and γ calculated for the different SPSW test specimens of Table 1, and the difference between them is significant (note that the nomenclature for SPSW is being used now, in other words, *a* in Equation 13 is *h*, and *h* in Equation 13 is *L*). Figure 11 shows how the two different inclination angles change for the same two test specimens as a function of the SPSW story aspect ratio (*h/L*). Note the dependence of α on whether the story height, *h*, or the bay width, *L*, is varied.

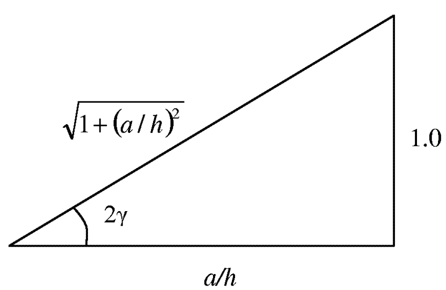

Fig. 9. Triangle Implied by Eq. 12 (Adapted from Salmon and Johnson, 1996).

This dependence is due to the fact that when *h* is increased, the vertical boundary elements get longer and more flexible, while when *L* is increased the horizontal boundary elements are getting longer. Since γ depends only on the aspect ratio, *h/L*, and is not influenced by the stiffness of the boundary members, there is no difference between varying *h* or *L* in Equation 13.

Figure 12 shows the difference between the plate girder shear strength equations from Appendix G of the AISC *LRFD Specification* (Equations 2 and 14), the SPSW ultimate strength equation. The strengths are normalized by the shear yield force ($0.6F_ytL$) and plotted against the infill plate slenderness ratio (L/t) for several values of story aspect ratio (*h/L*). Here the angle of inclination of the tension field used in Equation 2 and found from Equation 1 has been bounded between 35° and 45°, which are reasonable bounds based on the results of testing and finite element analysis. The additional parameters necessary to make use of Equation 1 were assumed as follows: $L = 3048$ mm, $A_c = 18190$ mm^2, $I_c = 347 \times 10^6$ mm^4, and $A_b = 16320$ mm^2. Table 3 gives a comparison between experimentally obtained ultimate strengths, and analytical values calculated using the plate girder equations (Equations 2 and 14). Figure 12 and Table 3 illustrate the poor match of experimental results with values obtained from the plate girder shear strength equations. Underestimating SPSW strength in the perspective of seismic design can be detrimental, as the resulting designs may suffer failure of boundary members (which may or may not carry gravity loads), connections, or foundations prior to yielding of the web plate and hysteretic energy dissipation.

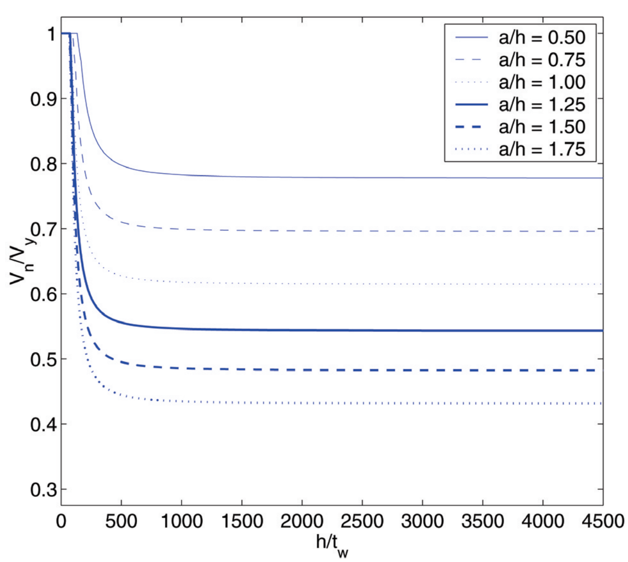

Fig. 10. Plate Girder Shear Strength versus Web Slenderness Ratio for Different Panel Aspect Ratios.

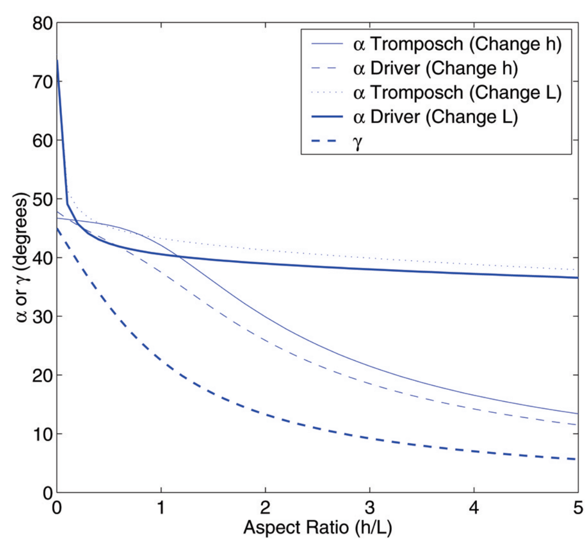

Fig. 11. α and γ versus Steel Plate Shear Wall Story Aspect Ratio.

Table 3. Ultimate Strength Comparison.

Case	Study	Specimen ID	No. Stories	L/t	h/L	α (°)	γ (°)	V_{uexp} (kN)	V_{upred} (kN) Eq. 2	% Error for Eq. 2	V_{upred} (kN) Eq. 14	% Error for Eq. 14
(i) Simple (Physical Pin) HBE-to-VBE Connections												
1	Timler and Kulak (1983)	—[c]	1	750	0.7	42.7	28.2	2698	2531	-6.2	2213	-18.0
2	Roberts and Sabouri-Ghomi (1992)	SW2	1	446	1	45.0	22.5	35.1	33.6	-4.2	25	-27.8
3		SW3	1	301	1	45.0	22.5	38.2	34.6	-9.5	27	-28.6
4		SW14	1	542	0.8	45.0	25.3	44.5	40.9	-8.1	33	-25.1
5		SW15	1	366	0.8	45.0	25.3	45.3	42.1	-7.1	35	-22.4
(ii) Semi-Rigid HBE-to-VBE Connections (Web-Angle or Other)												
6	Berman and Bruneau (2003a)	F2	1	4020	0.5	45	31.7	364	368	1.0	343	-5.7
7	Elgaaly (1998)	SWT11[b]	2	605	0.8	41.5	25.5	370	373.1	0.8	307	-16.9
8		SWT15	2	605	0.8	41.3	25.5	426	372.9	-12.5	307	-27.8
9	Caccese and others (1993)	S22	3	1637	0.7	42.2	28.0	142	120.4	-15.2	105	-26.2
10		S14	3	655	0.7	40.2	28.0	356	386.8	8.6	341	-4.2
(iii) Rigid HBE-to-VBE Connections												
11	Lubell and others (2000)	SPSW1[a]	1	600	1	36.9	22.5	210	207.3	-1.3[d]	161	-23.5
12		SPSW2	1	600	1	36.9	22.5	260	207.3	-20.3[d]	161	-38.2
13	Driver and others (1997)	—[c]	4	635	0.6	41.1	28.9	3080	2578	-16.3[d]	2304	-25.2

[a] Testing stopped due to failure of lateral bracing
[b] Testing stopped due to column buckling
[c] Not applicable
[d] Larger error values here due to the presence of rigid boundary frame member connections

Also shown in Figure 12 are three experimental points (Driver and others, 1998; Roberts and Sabouri-Ghomi, 1992; and Berman and Bruneau, 2003a) for which the ultimate strengths observed during testing are normalized by the shear yield strength, $0.6F_{yw}Lt$, and plotted against the web slenderness ratios for the experiments (only the first story strength and geometry are used in the case of the Driver tests, and the Roberts test refers to specimen SW2). Note that the aspect ratios (*h/L*) for the Driver, Roberts, and Berman tests were 0.67, 1.0, and 0.5 respectively. The experimental results and strengths predicted by Equation 2 agree well (although Equation 2 gives slightly conservative results because it does not include the strength contributions of the boundary frames), while the plate girder equations significantly underestimate the ultimate strength. Note that for capacity protection of foundations, the additional strength provided by the boundary framing (Berman and Bruneau, 2003b) should be considered even when Equation 2 is used to predict the ultimate capacity of a SPSW.

WEB SLENDERNESS

The AISC *LRFD Specification* prescribes the following web slenderness ratio limits for plate girders:

$$\begin{aligned}
\frac{h}{t_w} &\le 260 \quad \text{for} \quad \frac{a}{h} = 0 \\
\frac{h}{t_w} &\le 11.7\sqrt{\frac{E}{F_{yf}}} \quad \text{for} \quad \frac{a}{h} \le 1.5 \\
\frac{h}{t_w} &\le \frac{0.48E}{\sqrt{F_{yf}(F_{yf}+16.5)}} \quad \text{for} \quad \frac{a}{h} > 1.5
\end{aligned} \tag{15}$$

where

F_{yf} = flange yield stress

All other parameters are as previously defined (note that plate girder notation rather than SPSW notation is used in Equation 15).

These limits prevent vertical flange buckling in plate girders due to the forces shown in Figure 13, and their derivation is given in Tall (1974). Furthermore, to ensure safe fabrication, handling, and erection, the AISC *LRFD Specification* limits the web slenderness ratio of plate girders relying on tension field strength to (Salmon and Johnson, 1996):

$$\frac{a}{h} \le \left(\frac{260}{h/t_w}\right)^2 \le 3.0 \tag{16}$$

The above web slenderness limits do not apply to SPSW. In the case of Equation 15, the flanges of SPSW are columns, and buckling is prevented through appropriate column checks. Equation 16 is not necessary because SPSW will be either assembled in place or fabricated in multistory lifts; in either case they are not subject to the same stability concerns during the construction process. For these reasons, and the fact that SPSW with web slenderness ratios of up to 4020 (see Table 3) have been tested and performed satisfactorily, it appears that no web slenderness ratio limits are warranted for SPSW.

CONCLUSIONS

Steel plate shear walls designed to buckle in shear and develop a diagonal tension field are similar to vertical plate girders in a qualitative manner only. The analogy that the

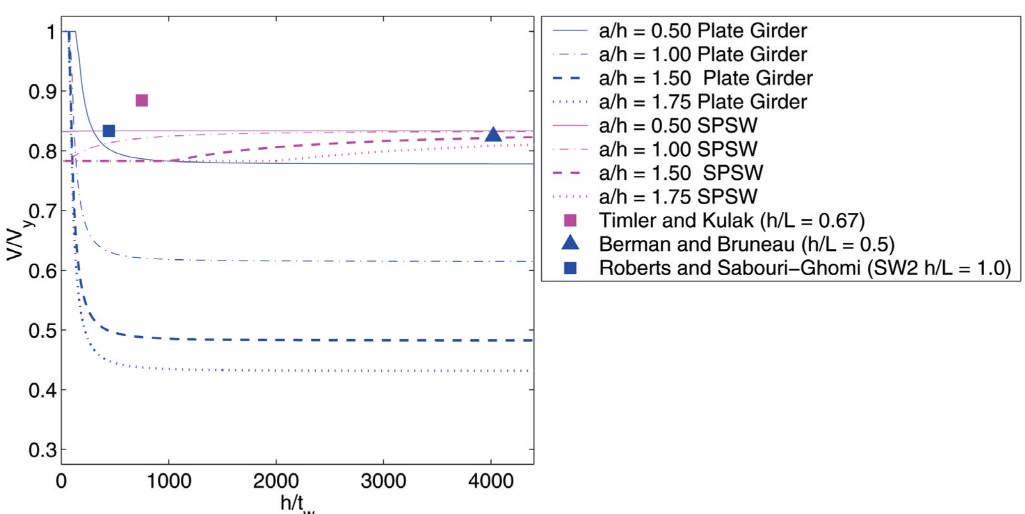

Fig. 12. Comparison of Eq. 2 and Eq. 14 versus Infill plate Slenderness Ratio.

vertical boundary elements of a SPSW are similar to the flanges of a plate girder, the horizontal boundary elements are similar to stiffeners, and the infill plate of a SPSW is similar to the web of a plate girder, is useful in developing a general understanding of SPSW behavior, but it does not fully represent the behavior of this structural system. The underlying difference results from the stiffnesses of the boundary elements. Where plate girder flanges are typically plates with little in-plane bending stiffness, the vertical boundary elements of a SPSW are typically wide flange shapes or hollow structural sections which have a substantial in-plane bending stiffness. The angle of inclination of the diagonal tension field that forms in SPSW depends on the stiffness of these boundary elements; whereas in plate girders, the stiffness of the boundary elements is typically neglected in determining this angle due to their low in-plane stiffnesses (as implicitly assumed in the tension field equations of Appendix G of the AISC *LRFD Specification*). As a result, plate girder shear strength equations considerably underestimate the strength of SPSW. For reasons described in this paper, and particularly in the case of seismic design, it is recommended that the strip model be used to model SPSW, with Equation 1 used to calculate the angle of inclination of the tension field and strips, and Equation 2 used to assess the ultimate strength of SPSW.

ACKNOWLEDGMENTS

This work was supported in part by the Earthquake Engineering Research Centers Program of the National Science Foundation under Award Number ECC-9701471 to the Multidisciplinary Center for Earthquake Engineering Research. However, 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.

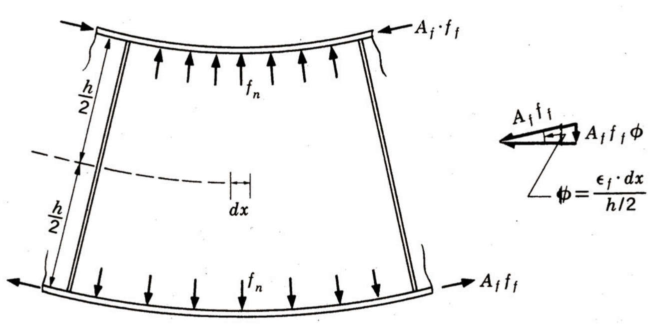

Fig. 13. Compressive Forces on a Girder Web Due to Curvature of Flanges (Adapted from Tall, 1974).

REFERENCES

AISC (1999), *Load and Resistance Factor Design Specification for Structural Steel Buildings*, American Institute of Steel Construction, December 27, Chicago, IL.

Basler, K. (1961), "Strength of Plate Girders in Shear," *Journal of the Structural Division*, ASCE, Vol. 87, No. 7, pp. 150-180.

Berman, J.W., and Bruneau, M. (2003a), "Experimental Investigation of Light-Gauge Steel Plate Shear for the Seismic Retrofit of Buildings," *Technical Report No. MCEER-03-0001*, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY.

Berman, J.W., and Bruneau, M. (2003b), "Plastic Analysis and Design of Steel Plate Shear Walls," *Journal of Structural Engineering*, ASCE, Vol. 129, No. 11, pp 1448-1456.

Caccese, V., Elgaaly, M., and Chen, R. (1993), "Experimental Study of Thin Steel-Plate Shear Walls Under Cyclic Load," *Journal of Structural Engineering*, ASCE, Vol. 119, No. 2, Feb. 1993, pp. 573-587.

CSA (2001), *Limit States Design of Steel Structures*, CAN/CSA S16-01, Canadian Standards Association, Willowdale, Ontario, Canada.

Driver, R.G., Kulak, G.L., Kennedy, D.J.L., and Elwi, A.E. (1998), "Cyclic Test of Four-Story Steel Plate Shear Wall," *Journal of Structural Engineering*, ASCE, Vol. 124, No. 2, Feb. 1998, pp 112-130.

Elgaaly, M. (1998), "Thin Steel Plate Shear Walls Behavior and Analysis," *Thin Walled Structures*, Vol. 32, pp. 151-180.

Elgaaly, M., Caccese, V., and Du, C. (1993), "Postbuckling Behavior of Steel-Plate Shear Walls Under Cyclic Loads," *Journal of Structural Engineering*, ASCE, Vol. 119, No. 2, Feb. 1993, pp. 588-605.

Elgaaly, M., and Lui, Y. (1997), "Analysis of Thin-Steel-Plate Shear Walls," *Journal of Structural Engineering*, ASCE, Vol. 123, No. 11, Nov. 1997, pp. 1487-1496.

Fujii, T. (1968), "On an Improved Theory for Dr. Basler's Theory," *International Association for Bridge and Structural Engineering 8th Congress*, Final Report, New York.

Galambos, T.V. (1998), *Guide to Stability Design Criteria for Metal Structures*, 5th Edition, John Wiley and Sons, Inc., New York.

Gaylord, E.H. (1963), "Discussion of K. Basler 'Strength of Plate Girders in Shear,'" *Transactions of the American Society of Civil Engineers*, ASCE, Vol. 128, Part II, p. 172.

Kulak, G.L., Kennedy, D.J.L., Driver, R.G., and Medhekar, M. (2001), "Steel Plate Shear Walls-An Overview," *Engineering Journal*, AISC, Vol. 38, No. 1, First Quarter 2001, pp 50-62.

Lubell, A.S., Prion, H.G.L., Ventura, C.E., and Rezai, M. (2000), "Unstiffened Steel Plate Shear Wall Performance Under Cyclic Loading," *Journal of Structural Engineering*, ASCE, Vol. 126, No.4, April 2000, pp. 453-460.

Rezai, M. (1999), "Seismic Behavior of Steel Plate Shear Walls by Shake Table Testing," Ph.D. Dissertation, University of British Columbia, Vancouver, British Columbia, Canada.

Roberts, T.M., and Sabouri-Ghomi, S. (1992), "Hysteretic Characteristics of Unstiffened Perforated Steel Plate Shear Walls," *Thin Walled Structures*, Vol. 14, pp. 139-151.

Sabouri-Ghomi, S., and Roberts, T.M. (1992), "Nonlinear Dynamic Analysis of Steel Plate Shear Walls Including Shear and Bending Deformations," *Engineering Structures*, Vol. 14, No. 3, pp. 309-317.

Salmon, C., and Johnson, J. (1996), *Steel Structures—Design and Behavior,* 4th Edition, HarperCollins College Publishers, Inc., New York.

Schumacher, A., Grondin, G.Y., and Kulak, G.L. (1999), "Connection of Infill Panels in Steel Plate Shear Walls," *Canadian Journal of Civil Engineering*, Vol. 26, No. 5, pp. 549-563.

Tall, L. (1974), *Structural Steel Design*, 2nd Edition, The Ronald Press Company, Inc., New York.

Thorburn, L.J., Kulak, G.L., and Montgomery, C.J. (1983), "Analysis of Steel Plate Shear Walls," *Structural Engineering Report No. 107*, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada.

Timler, P.A. and Kulak, G.L. (1983), "Experimental Study of Steel Plate Shear Walls," *Structural Engineering Report No. 114*, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada.

Timoshenko, S., and Woinowsky-Krieger, S. (1959), *Theory of Plates and Shells,* 2nd Edition, McGraw Hill Book Company, Inc., New York.

Tromposch, E.W., and Kulak, G.L. (1987), "Cyclic and Static Behaviour of Thin Panel Steel Plate Shear Walls," *Structural Engineering Report No. 145*, Department of Civil Engineering, University of Alberta, Edmonton, Alberta, Canada.