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PLASTIC DESIGN AND TESTING OF LIGHT-GAUGE STEEL PLATE SHEAR WALLS

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SUMMARY

In this paper, the strip model developed by others and implemented in a Canadian Standard to model steel plate shear walls (SPSW) is used to develop, investigate, and quantify, through plastic analysis, the various possible collapse mechanisms of SPSW. Comparisons of experimentally obtained ultimate strengths of steel plate shear walls and those predicted by plastic analysis are given and reasonable agreement is observed. Modifications are proposed to a section of an existing codified procedure for the design of steel plate walls which is shown could lead to designs with less-than-expected ultimate strength. In addition to the above, this paper also describes the results of an experimental study to determine the feasibility of light-gauge SPSW for use in the seismic retrofit of buildings. Three specimens were constructed and tested under quasi-static loading, one using a corrugated infill and epoxy connection to the surrounding frame, one using a flat infill with an epoxy connection to the surrounding frame, and one using a flat infill with a welded connection to the frame. The flat infill with the welded connection reached a ductility ratio of 12 and had substantially superior behavior when compared to the other two specimens.

INTRODUCTION

Steel plate shear walls (SPSW) have sometimes been used as the lateral load resisting system in buildings. Until the 1980's, the design limit state for SPSW in North America was out-of-plane buckling of the infill plates. This led engineers to design heavily stiffened plates that offered little economic advantage over reinforced concrete shear walls. However, as Basler [1] demonstrated for plate girder webs, the post-buckling tension field action of steel plate shear walls can provide substantial strength, stiffness, and ductility. The idea of utilizing the post-buckling strength of steel plate shear walls was first formulated by Thorburn [2] and verified experimentally by Timler and Kulak [3]. Studies performed to evaluate the strength, ductility, and hysteretic behavior of such SPSW designed with unstiffened infill plates demonstrated their significant energy dissipation capabilities [3] and substantial economic advantages [4].

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At the time of this writing, there are no U.S. specifications or codes addressing the design of steel plate shear walls. The 2001 Canadian standard, CAN/CSA S16-01 [5], now incorporates mandatory clauses on the design of steel plate shear walls; these are reviewed briefly in the next section. One of the models recommended to represent steel plate shear walls, which was developed by Thorburn [2] and named the strip model, is recognized for providing reliable assessments of their ultimate strength. In this paper, using this strip model as a basis, the use of plastic analysis as an alternative for the design of steel plate shear walls is investigated. Fundamental plastic collapse mechanisms are described for single story and multistory SPSW with either simple or rigid beam-to-column connections. Ultimate strengths predicted from these collapse mechanisms are compared with experimental results by others, and used to assess the CAN/CSA S16-01 design procedure.

Since SPSW can have substantial strength and stiffness, they sometimes may require reinforcing of boundary columns in retrofit scenarios, which can drastically increase retrofit cost. Therefore, light-gauge steel plate shear walls, which yield at lower force values, may provide reasonable alternatives for retrofit. As such, the design and quasi-static testing of three light-gauge steel plate shear wall specimens are briefly discussed. The purpose of the testing is to investigate the feasibility of these systems for seismic retrofit applications.

ANALYSIS AND DESIGN OF STEEL PLATE SHEAR WALLS - CAN/CSA S16-01

The CAN/CSA S16-01 seismic design process for steel plate shear walls follows the selection of a lateral load resisting system (i.e., shear walls with rigid or flexible beam-to-column connections), calculation of the appropriate design base shear, and distribution of that base shear along the building height by the usual methods described in building codes. Preliminary sizing of members is done using a model that treats the plate at each story as a single pin-ended brace (known as the equivalent story brace model) that runs along the diagonal of the bay (Fig. 1a). From the area of the story brace, A, determined from that analysis, an equivalent plate thickness can be calculated using the following equation based on an elastic strain energy formulation [2]:

$$t = \frac{2A \sin\theta \sin 2\theta}{L \sin^2 2\alpha} \tag{1}$$

where θ is the angle between the vertical axis and the equivalent diagonal brace, L is the bay width, and α is the angle of inclination of the principal tensile stresses in the infill plate measured from vertical, which is given by:

$$\tan^4\alpha = \frac{1 + \dfrac{tL}{2A_c}}{1 + t h_s\left(\dfrac{1}{A_b} + \dfrac{h_s^3}{360 I_c L}\right)} \tag{2}$$

where t is the thickness of the plate, A_c and I_c are respectively the cross-sectional area and moment of inertia of the bounding column, h_s is the story height, and A_b is the beam cross-sectional area [3]. CAN/CSA S16-01 also provides the following equation to ensure that a satisfactory minimum moment of inertia is used for columns in steel plate shear walls to prevent excessive deformation leading to premature buckling under the pulling action of the plates [6]:

$$I_c \geq \frac{0.00307\, t h_s^4}{L} \tag{3}$$

Once the above requirements have been satisfied, a more refined model, known as the strip or multi-strip model, that represents the plates as a series of inclined tension members or strips (Fig. 1b) is required for the analysis of steel plate shear walls (with α as calculated by Eq. (2)). Through comparison with experimental results, the adequacy of the strip model to predict the ultimate capacity of SPSW has been verified in several studies such as Driver [7].

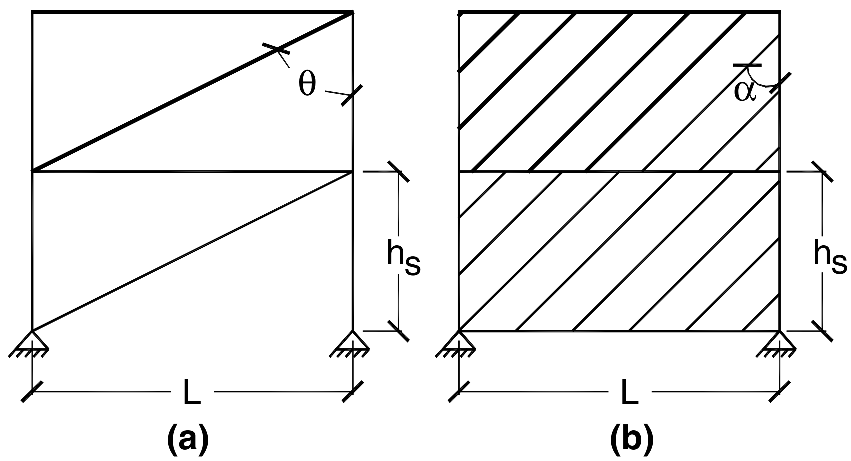

Figure 1 (a) Equivalent Story Brace Model
(b) The Strip Model

A minimum of ten strips is required at each story to adequately model the wall. Each strip is assigned an area equal to the plate thickness times the tributary width of the strip. Drifts obtained from the elastic analysis of the multi-strip model are then amplified by factors prescribed by the applicable building code to account for inelastic action and then checked against allowable drift limits. For SPSW having rigid beam-to-column connections, CAN/CSA S16-01 also requires that a capacity design be conducted to prevent damage to the bounding columns of the wall. Due to practical considerations, infill thicknesses may be larger than necessary to resist the seismic loads, therefore, capacity design is required to insure a ductile failure mode (i.e. infill yielding prior to column buckling). To achieve this, the moments and axial forces (obtained from an elastic analysis) in these columns are magnified by a factor B, defined as the ratio of the probable shear resistance at the base of the wall for the supplied plate thickness, to the factored lateral force at the base of the wall obtained from the calculated seismic load. The probable resistance of the wall (V_{re}) is given by:

$$V_{re} = 0.5 R_y F_y t L \sin 2\alpha \quad \textbf{(4)}$$

where R_y is the ratio of the expected (mean) steel yield stress to the design yield stress (specified as 1.1 for A572 Gr. 50 steel), F_y is the design yield stress of the plate, and all other parameters have been defined previously. Note that B need not be greater than the ratio of the ultimate elastic base shear to the yield base shear, which is the ductility factor, R , specified as 5.0 by CAN/CSA S16-01. Column axial loads are found from the overturning moment BM_f, where M_f is the factored overturning moment at the bottom of the wall. Local column moments from tension field action of the plates, as determined from the elastic analysis, are also amplified by B. If a nonlinear pushover analysis is carried out, these corrections need not be done and more accurate values for the column axial forces and moments can be obtained. Since pushover capabilities are becoming more common in structural analysis software, this is also a viable option.

PLASTIC ANALYSIS OF STEEL PLATE SHEAR WALLS

In this section, plastic analysis of the strip model is used to develop equations for the ultimate capacity of different types of steel plate shear walls. In cases where general equations depend on actual member sizes and strengths, procedures are presented to determine the necessary equations. In the following section the results of these analyses are used to develop a simple, consistent method for determining the preliminary plate sizes for steel plate shear walls.

Single Story Frames - Simple Beam-to-Column Connections

Consider the frame with inclined strips shown in Fig. 2a and assume that the beam-to-column connections are simple. When the shear force, V, displaces the top beam by a value Δ sufficient to yield all the strips, the external work done is equal to $V\Delta$ (see Fig. 2b). If the beams and columns are assumed to remain elastic, their contribution to the internal work may be neglected when compared to the internal work done by the strips, hence, the internal work is $(n_b A_{st} F_y \sin\alpha)\Delta$, where n_b is the number of strips anchored to the top beam. This result can be obtained by the product of the yield force times the yield displacement of the strips, but for simplicity it can also be found using the horizontal and vertical components of these values. Note that the horizontal components of the yield forces of the strips on the columns cancel (the forces on the left column do negative internal work and the forces on the right column do positive internal work) and the vertical components of all the yield forces do no internal work because there is no vertical deflection. Therefore, the only internal work done is by the horizontal components of the strip yield forces anchored to the top beam. Equating the external and internal work gives:

$$V = n_b F_{st} \sin\alpha \tag{5}$$

Using the geometry shown in Fig. 2a, $n_b = (L\cos\alpha)/s$ and the strip force F_{st} is again $F_y ts$. Substituting these into Eq. (5) and knowing $(1/2)\sin 2\alpha = \cos\alpha \sin\alpha$, the resulting base shear relationship is:

$$V = \frac{1}{2} F_y t L \sin 2\alpha \tag{6}$$

Note that this equation is identical to the one used to calculate the probable shear resistance of a SPSW in the CAN/CSA S16-01 procedure, Eq. (4), without the material factor R_y.

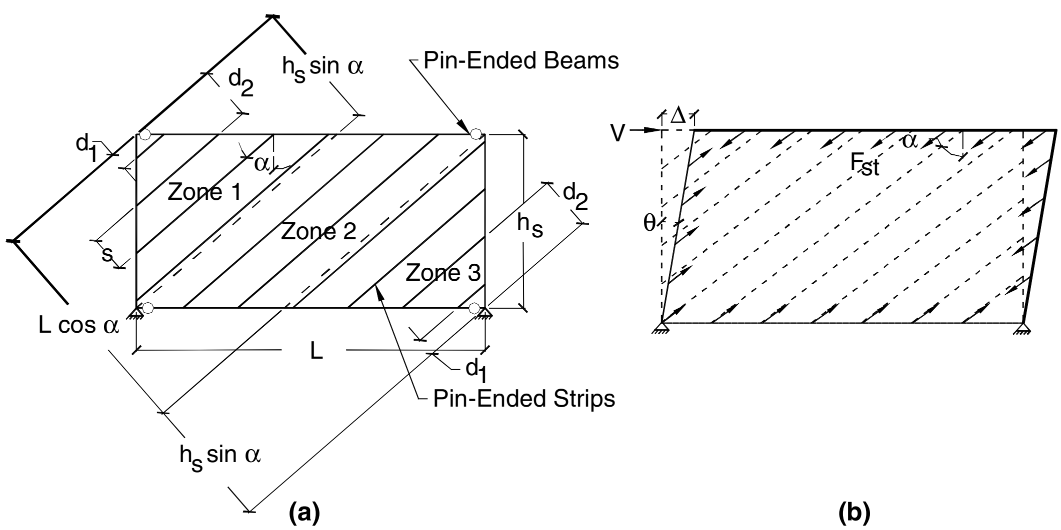

Figure 2 (a) Single Story Strip Model (b) Single Story Collapse Mechanism

Single Story Frames - Rigid Beam-to-Column Connections

In single story steel plate shear walls having rigid beam-to-column connections (as opposed to simple connections), plastic hinges also need to form in the boundary frame to produce a collapse mechanism. The corresponding additional internal work is $4M_p\theta$, where $\theta = \Delta/h_s$, is the story displacement over the story height, and M_p is the smaller of the plastic moment capacity of the beams M_{pb}, or columns M_{pc} (For most single-story frames that are wider than tall, if the beams have sufficient strength and stiffness to anchor the tension field, plastic hinges will typically form at the top and bottom of the columns and not in the beams). The ultimate strength of a single-story steel plate shear wall in a moment frame with plastic hinges in the columns becomes:

$$V = \frac{1}{2} F_y t L \sin 2\alpha + \frac{4M_{pc}}{h_s} \quad \textbf{(7)}$$

In a design process, failure to account for the additional strength provided by the beams or columns results in larger plate thicknesses than necessary, this would translate into lower ductility demands in the walls and frame members, and could therefore be considered to be a conservative approach. However, capacity design of the beams and columns must still be performed to insure that a ductile failure mode will be achieved (i.e. plate yielding prior to columns or beams developing plastic hinges).

Multistory Frames - Rigid Beam-to-Column Connections

For multistory SPSWs with pin-ended beams, plastic analysis can also be used to predict the ultimate capacity. The purpose here is not to present closed-form solutions for all possible failure mechanisms, but to identify some key plastic mechanisms that should be considered in estimating the ultimate capacity of a steel plate shear wall. These could be used to define a desirable failure mode in a capacity design perspective, or to prevent an undesirable failure mode, as well as complement traditional design approaches.

In soft-story plastic mechanisms (Fig. 3a), the plastic hinges that would form in the columns at the mechanism level could be included in the plastic analysis. Calculating and equating the internal and external work, the following general expression could be used for soft-story i, in which all flexural hinges develop in columns:

$$\sum_{j=i}^{n_s} V_j = \frac{1}{2} F_y t_i L \sin 2\alpha + \frac{4M_{pci}}{h_{si}} \quad \textbf{(8)}$$

where V_j are the applied lateral forces above the soft-story i, t_i is the plate thickness at the soft-story, M_{pci} is the plastic moment capacity of the columns at the soft-story, h_{si} is the height of the soft-story, and n_s is the total number of stories. Note that only the applied lateral forces above the soft-story do external work and they all move the same distance (Δ). The internal work is done only by the strips on the soft-story itself and by column hinges forming at the top and bottom of the soft-story. Using the above equation, the possibility of a soft-story mechanism should be checked at every story in which there is a significant change in plate thickness or column size. Additionally, the soft-story mechanism is independent of the beam connection type (simple or rigid) because hinges must form in the columns, not the beams.

A second (and more desirable) possible collapse mechanism involves uniform yielding of the plates over every story (Fig. 3b). For this mechanism, each applied lateral force, V_i, moves a distance $\Delta_i = \theta h_i$, and does external work equal to $V_i \theta h_i$, where h_i is the elevation of the i^{th} story. The internal work is done by the strips of each story yielding. It is important to note that the strip forces acting on the bottom of a story beam do positive internal work and the strip forces acting on top of the same beam do negative internal work. Therefore, the internal work at any story i is equal to the work done by strip yield forces along the bottom of the story beam minus the work done by strip yield forces on the top of the same beam. This indicates that in order for every plate at every story to contribute to the internal work, the plate thicknesses would have to vary at each story in direct proportion to the demands from the applied lateral forces. Even with this in mind, this mechanism provides insight into the capacity and failure mechanism of the wall. The general equation for the ultimate strength of a multistory SPSW with simple beam-to-column connections and this plastic mechanism (equating the internal and external work) is:

$$\sum_{i=1}^{n_s} V_i h_i = \sum_{i=1}^{n_s} \frac{1}{2} F_y (t_i - t_{i+1}) L h_i \sin 2\alpha \quad \textbf{(9)}$$

where all terms are as previously defined.

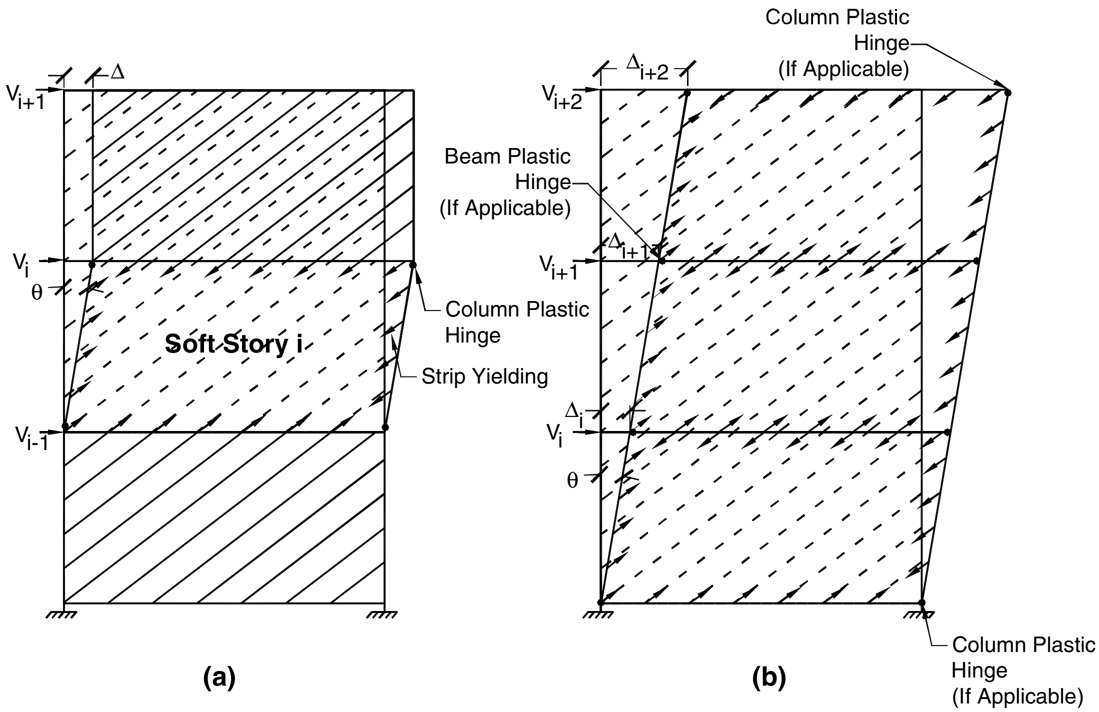

Figure 3 (a) Soft-Story Collapse Mechanism (b) Uniform Yielding Mechanism

The reader is referred to Berman [8] for equations for multi-story SPSW with rigid beam-to-column connections. After examining the results of several different pushover analyses for multistory SPSW with simple or rigid beam-to-column connections (using a single 3-story frame geometry, with arbitrarily selected beams, columns, and plate thicknesses), it has been observed that the actual failure mechanism is typically somewhere between a soft-story mechanism and uniform yielding of the plates on all stories. Finding the actual failure mechanism is difficult by hand, therefore, a computerized pushover analysis should be used. However, the mechanisms described above will provide a rough estimate of the ultimate capacity. They will also provide some insight as to whether a soft story is likely to develop (by comparing the ultimate capacity found from the soft story mechanism with that of the uniform yielding mechanism).

COMPARISON WITH EXPERIMENTAL RESULTS

To validate ultimate strengths predicted by Eqs. (6) for the plastic analysis of single story frames with simple or rigid beam-to-column connections, a comparison is made with results obtained experimentally by others (Table 1). The experimental results given for multistory specimen are either those for the first story shear (in the case of Driver [7]) or they are the total base shear in cases where loading was applied to the top story only (Elgaaly [9], and Caccese [10]). Furthermore, no results are given tests on SPSWs that had openings. As shown in Table 1, on average Eq. (6) predicts an ultimate load capacity for steel plate shear walls with true pin or semi-rigid beam-to-column connections that is 5.9% below the experimentally obtained values. Note that Cases 7 and 11, included in Table 1 for completeness, were not included in the average because their ultimate failure was due to column instability or problems with the test setup. The final three cases in Table 1 show that Eq. (6) is conservative for use in calculating ultimate strengths of SPSW with rigid beam-to-column connections. Generally, the equations derived from plastic analysis of the strip model are conservative for calculating the expected ultimate strength of steel plate shear walls regardless of beam-to-column connection type. Although, for capacity design, care must be taken to account for the extra strength provided by the boundary frame.

Table 1 Comparison of Experimental Results with Plastic Analysis

Case	Study	Specimen ID	No. Stories	h (mm)	L (mm)	t (mm)	F_y (Mpa)	α (deg.)	V_{uexp} (kN)	V_{uexp} (kN) Eq. (6)	% Error for Eq. (6)
(i) Simple (Physical Pin) Beam-to-Column Connections											
1	Timler [3]	-[3]	1	2500	3750	5	270.8	42.7	2698	2531	-6.2
2	Roberts [11]	SW2	1	370	370	0.83	219	45.0	35.1	33.6	-4.2
3		SW3	1	370	370	1.23	152	45.0	38.2	34.6	-9.5
4		SW14	1	370	450	0.83	219	45.0	44.5	40.9	-8.1
5		SW15	1	370	450	0.83	219	45.0	45.3	42.1	-7.1
(ii) Semi-Rigid Beam-to-Column Connections (Web-Angle or Other)											
6	Berman [12]	F2	1	1829	3658	0.91	221	44.8	364	367.8	1.1
7	Elgaaly [9]	SWT11[2]	2	1118	1380	2.28	239	41.5	370	373.1	0.9
8		SWT15	2	1118	1380	2.28	239	41.3	426	372.9	-11.3
9	Caccese [10]	S22	3	838	1244	0.76	256	42.2	142	120.4	-15.2
10		S14	3	838	1244	1.9	332	40.2	356	386.8	8.7
(iii) Rigid Beam-to-Column Connections											
11	Lubell [13]	SPSW1[1]	1	900	900	1.5	320	36.9	210	207.3	-[3]
12		SPSW2	1	900	900	1.5	320	36.9	260	207.3	-[3]
13	Driver [7]	-[3]	4	1927	3050	4.8	355.4	41.1	3080	2578	-[3]

[1] Testing stopped due to failure of lateral bracing
[2] Testing stopped due to column buckling
[3] Not applicable

IMPACT OF DESIGN PROCEDURE ON ULTIMATE STRENGTH OF SPSW

CAN/CSA S16-01 Approach

The procedure given for preliminary sizing of plates in CAN/CSA-S16-01 is simple but results in designs that may not be consistent with the demands implicit in the seismic force modification factor *R*. The transition from the equivalent story brace model (used for preliminary proportioning and to select the amount of steel in the infill plates) to the multi-strip model (used for final analysis) may change the ultimate capacity and shape of the pushover curve for the structure being designed, while the *R*-factor is not revised.

In the equivalent story brace model, the ultimate capacity of the wall is only a function of the brace area, yield stress, and the bay geometry (aspect ratio). The story shear can be used to size the equivalent brace for each story by using simple statics (recall Fig. 1a). Then Eq. (1) can be employed to relate the brace area to the plate thickness which, along with the strip spacing, yields the strip area for the detailed strip model. However, these two models will not produce the same ultimate capacity unless the aspect ratio of the bay is 1:1. To demonstrate this consider a single story SPSW (with simple beam-to-column connections) as shown in Fig. 2a. Let the aspect ratio of the bay be equal to the bay width over the story height. Using the same design base shear, beam and column sizes (selected to remain elastic for all cases), the area of the equivalent story braces were found for several aspect ratios. From these, the plate thicknesses were found as described above and the detailed strip models were developed using Eq. (2) to find the angle of inclination for the strips.

Pushover analyses of all resulting SPSW were then conducted and the resulting ultimate strengths of the various walls, designed to resist the same applied lateral loads, were compared. Fig. 4a shows a plot of the base shear (normalized by dividing out the design base shear used to find the area of the equivalent story brace) versus percent story drift for several SPSW of different aspect ratios, obtained from pushover analyses of the strip models and equivalent story brace models. The resulting ultimate capacity of the strip model is below the capacity of the equivalent story brace model for all aspect ratios, except 1:1 for which it is the same. The difference between the capacity of the strip model and equivalent story brace model increases as the aspect ratio further deviates from 1.0. Fig. 4b shows how the difference between the strip model capacity and the equivalent story brace model capacity changes with the aspect ratio of the bay. At an aspect ratio of 2:1 (or 1:2 since the results are symmetric in that sense) the strip model is only able to carry 80% of the base shear for which it should have been designed.

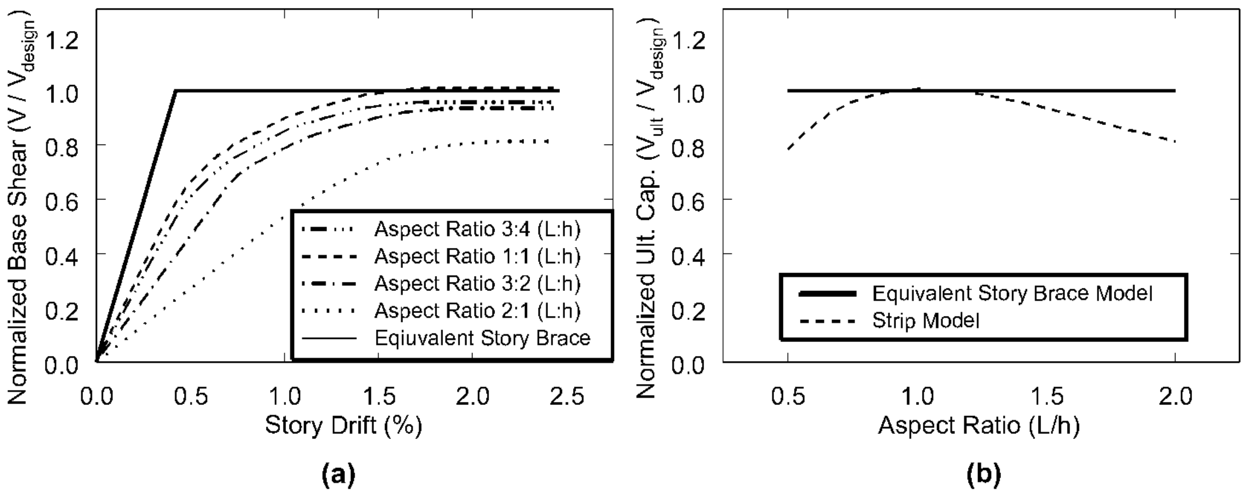

Figure 4 (a) Normalized Pushover Curves Similar SPSW of Different Aspect Ratios (b) Comparison of Strip and Equivalent Story Brace Models Ultimate Capacities

Plastic Analysis

Using the results of the plastic analyses described previously, the infill plates of steel plate shear walls can be sized to consistently achieve the desired ultimate strength. The procedure is simple, even for a multistory SPSW, and neglecting the contribution of plastic hinges in beams and columns will always give a conservative design in the case of rigid beam-to-column connections. The proposed procedure requires the designer to:

(A) Calculate the design base shear, and distribute it along the height of the building as described by the applicable building code;

(B) Use the following equation to calculate the minimum plate thicknesses required for each story:

$$t = \frac{2 V_s \Omega_s}{F_y L \sin 2\alpha} \tag{10}$$

where, Ω_s is the system overstrength described below and V_s is the design story shear found using the equivalent lateral force method;

(C) Develop the strip model for computer (elastic) analysis using Eq. (2) to calculate the angle of inclination of the strips;

(D) Design beams and columns according to capacity design principles (to insure the utmost ductility) or other rational methods using plate thicknesses specified (in case those exceed the minimum required for practical reasons);

(E) Check story drifts against allowable values from the applicable building code;

Note that Eq. (10) is identical to Eq. (6) but modified to account for the proper relationship between the equivalent lateral force procedure and *R*, the seismic force modification factor. Recommended system overstrength values are between 1.1 and 1.5 based on the results from various pushover analyses [8].

CYCLIC TESTING OF LIGHT-GAUGE STEEL PLATE SHEAR WALLS

Prototype and Specimen Design

Using the MCEER Demonstration Hospital from Yang [14], prototype steel plate shear walls were designed as seismic retrofits. This hospital is a four story, steel framed, structure in a region of high seismicity. The original lateral load resisting system is moment frames on the end walls, which are shown to be insufficient by Yang. Light-gauge steel plate shear walls were selected as the retrofit option for investigation because they would minimize the demand on the surrounding framing, avoiding further column strengthening, and they would develop smaller forces in the connection of the infill to the surrounding frame, proving from more connections alternatives.

The equivalent lateral force procedure was employed to calculate seismic loads and infill thicknesses were selected on the basis that one bay of every frame orientated in the north-south direction would receive infills. This allowed the plates to be thin and insured that the existing framing would be sufficient to resist the moments induced by the tension field action of the infills. Two different prototypes were designed, one using a 20 Gauge (1.0 mm) flat infill and the other using a 22 Gauge (0.75 mm) corrugated infill with a corrugation profile equal to that of type B steel deck (these are first story thicknesses only). The corrugations were orientated at 45° to match the angle of inclination of the tension field for the flat infills.

Using the first story infill thicknesses found in the prototype designs, three single story light gauge steel plate shear walls were designed. Specimens F1 and F2 utilized the flat infills of 20 Gauge material. Specimen C1 used 22 Gauge type B metal deck as the infill. Coupon tests showed the yield stress (F_y) of the infill materials to be 150 MPa, 225 MPa and 325 MPa for specimens F1, F2, and C1 respectively. Infill connections to the boundary frame were accomplished using intermediate angles or WT shapes and industrial strength epoxy (specimens F1 and C1) or welding (specimen F2) and are discussed in further detail in Berman and Bruneau [12]. The boundary frames for the specimens were designed to remain elastic during testing and utilized web-angle beam-to-column connections similar to those in the MCEER Demonstration Hospital. Columns and beams were selected to be W310x143 and W460x125 respectively. To allow for actuator capacity, the height (1830 mm) and width (3660 mm) of the specimens were set at ½ the height and width of the prototypes and the aspect ratio of 0.5 (h/L) was maintained. Figures 5a and 5b show specimens C1 and F2 prior to testing.

Test Setup

Each specimen was mounted on a stiff beam which was pretensioned to the strong floor of the Structural Engineering and Earthquake Simulation Laboratory (SEESL) at the University at Buffalo. A 1100 kN static actuator was mounted between the specimen and stiff reaction frame available in the SEESL as shown in Figure 6.

Instrumentation consisted of strain gauges mounted on each face of the infill of each specimen. A minimum of 4 gauges were placed at the center of the infills (one at each of 0°, 45°, 90° and 135° from horizontal) so that the direction of the principal stresses could be obtained. Four additional clusters of two strain gauges (orientated at 45° and 135° from horizontal) were placed at mid-height and even intervals across the length of the infills to measure the variation in strain over the infills. Strain gauges were also placed at the center of the beams and columns (two gauges on each flange, one on each side of the web) so that the axial forces and moments could be obtained. Strain gauge results are discussed in detail in Berman [12]. Magnetic Strictive Transducers (Temposonics) were placed at the quarter points and top and bottom of the north

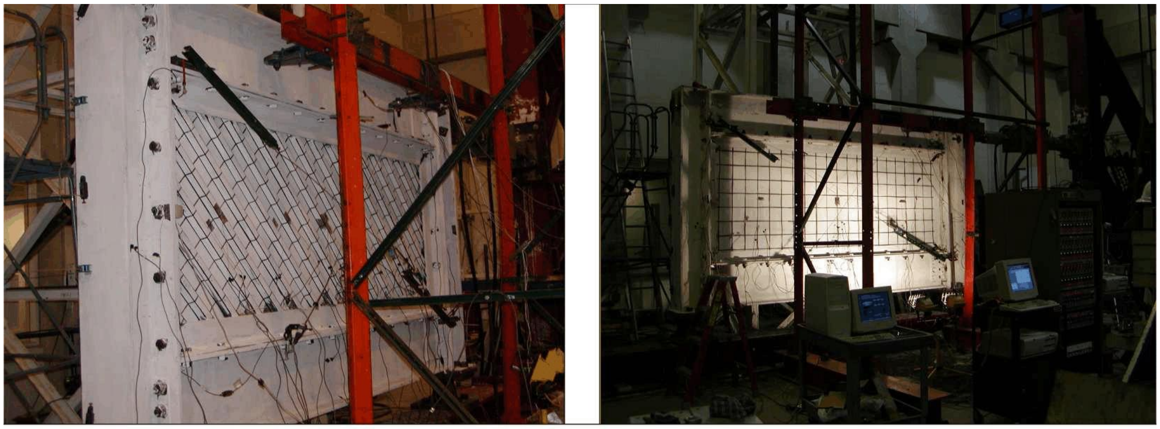

Figure 5 Specimens Prior to Testing; (a) C1 (b) F2

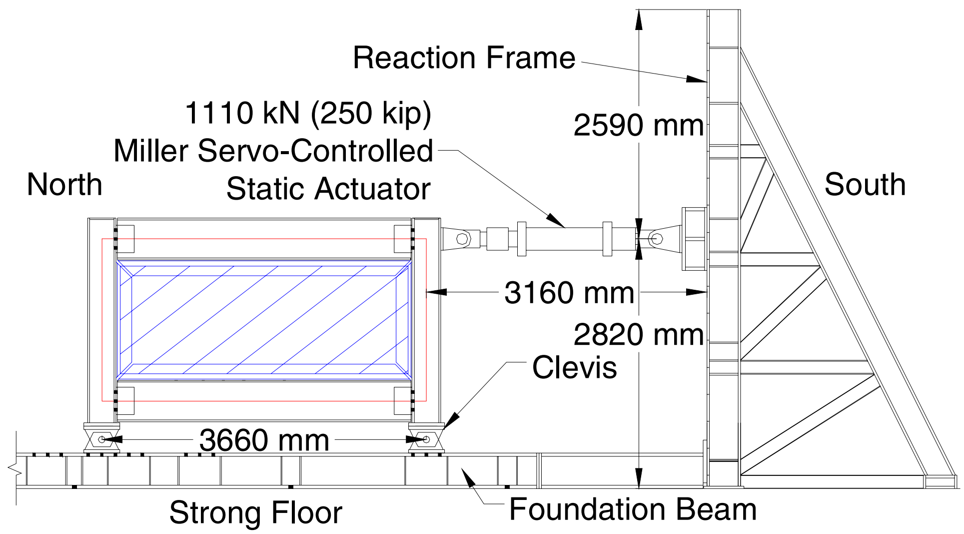

Figure 6 Test Setup

column of the specimens to detect any pull-in of the column resulting from the formation of the diagonal tension field in the infills. Loading was carried out in accordance with ATC 24 [15].

Experimental Observations and Results

Specimen F1 suffered a premature failure of the epoxy connecting the infill to the top beam prior to the infill material reaching yield. This was likely the result of insufficient epoxy coverage and the hysteresis for specimen F1 showed that there was no significant energy dissipation.

Specimen C1 was tested successfully to 33.5 mm of displacement ($4\delta_y$ and 1.83% drift). The yield displacement was found to be 8.1 mm (0.44% drift) and the corresponding yield base shear was 505 kN. The ultimate failure mode was fracture of the infill at locations of repeated local buckling shown as lines 1, 2 and 3 in Figure 7. This local buckling was the result of the compressive forces in corrugations under negative loading. Fig. 8a shows the local and global buckling of the infill at $-3\delta_y$ and Fig. 8b shows the fractures along line 2 near the end of testing.

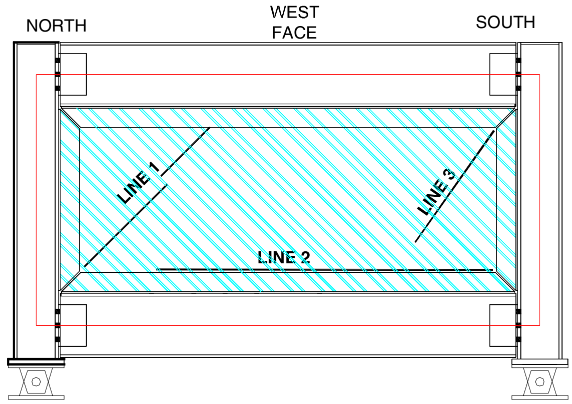

Figure 7 Lines of Local Buckling
Specimen C1

Figure 8 Specimen C1; (a) Buckling of Infill (b) Fracture Along Line 2

The experimentally obtained hysteresis curves for specimen C1 are shown in Figure 9a. The strength degradation of the specimen following the cycles at $3\delta_y$ (1.35% drift) is a direct result of the fractures in the infill.

From the strain gauge data it was apparent that the epoxy formed an effective bond between the infill and boundary frame, since the strain was uniform across the infill. Additionally, it was found that the moments in the beams and columns did not exceed 16% of the yield moment, taken as S_xF_y (956 kN m for the beams and 753 kN m for the columns), where S_x is the elastic section modulus. Therefore, it seemed that this infill was effective in keeping the demands on the framing members low.

The most desirable behavior was obtained from specimen F2. Pinched but stable behavior was obtained to a displacement of 67.0 mm ($12\delta_y$ and 3.65% drift) which demonstrates the ductile behavior that can be achieved with steel plate shear walls. The yield displacement and base shear were found to be 5.1 mm (0.29% drift) and 350 kN, and ultimate failure resulted from fractures in the corners of the infill. These fractures appeared early in the testing ($2\delta_y$) but did not have a significant impact on the capacity of the specimen until $12\delta_y$, when they had to grown to the size shown in Figure 10. Similar fractures were observed in all four corners of the infill, propagating from the ends of fillet welds that connected the infill to the WTs. The experimental hysteresis curves for specimen F2 are shown in Figure 9b.

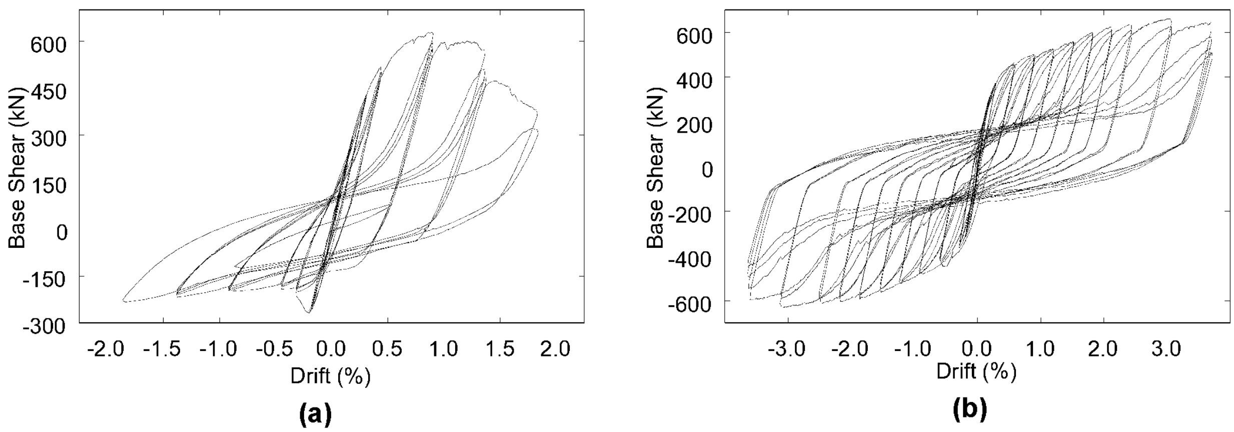

Figure 9 Specimen Hysterses; (a) Specimen C1 (b) Specimen F2

From strain gauge data it was observed that there was very little variation in the strain across the infill, indicating that the entire infill participated in dissipating energy. Maximum moments in the beams and columns did not exceed 7% of the yield moments.

Figure 10 Fracture of Infill - Specimen F2

CONCLUSIONS

The CAN/CSA S16-01 recommended procedure for the analysis and design of steel plate shear walls has been reviewed and instances where this procedure can lead to unconservative designs with lower than expected ultimate capacity have been identified. Plastic collapse mechanisms for single and multistory SPSW with simple and rigid beam-to-column connections have been investigated and simple equations that capture the ultimate strength of SPSW have been developed and compared with experimental results reported by others with reasonable agreement. Using the results of these plastic analyses a new procedure for the sizing of the infill plates has been proposed. The proposed procedure allows the engineer to control the ultimate failure mechanism of the SPSW, and directly accounts for structural overstrength.

Light-gauge steel plate shear walls have been shown to be a viable seismic retrofit option for buildings. Substantial ductility and stiffness can be achieved with these types of infills. From the experimental results for three specimens, there is no substantial advantage to using infills with corrugated profiles in spite of their enhanced buckling strength. The failure mode for the specimen utilizing a corrugated infill was determined to be fracture of the infill at locations of repeated local buckling, while for the specimen with a flat infill and welded connection to the boundary frame, fracture of the infill occurred at a drift of $12\delta_y$. Moreover, until that drift level was reached there was no strength degradation which shows these systems can be stable up to large drifts. The moments in the beams and columns were shown to be small for all specimen and the variation of the strain across the infills was insignificant.

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