12TH CANADIAN CONFERENCE ON EARTHQUAKE ENGINEERING
JUNE 17-20, 2019
CHÂTEAU FRONTENAC
QUÉBEC, QC.

12ÈME CONFÉRENCE CANADIENNE DU GÉNIE PARASISMIQUE
17-20 JUIN 2019

CCEE 2019

Performance of centre-sheathed cold-formed steel framed shear walls: Phase 2

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ABSTRACT

To enter into the construction market for mid-rise buildings, the cold-formed steel (CFS) industry needs a solution to address higher seismic shear forces exceeding 60kN/m. The current AISI S400-15 North American Standard for Seismic Design of Cold-Formed Steel Structural Systems contains only limited design information for steel-sheathed shear walls with low shear resistance values. A new wall configuration in which the sheathing was placed at the mid-line of the framing, denoted the "centre-sheathed" shear wall, was recently developed through a laboratory test program. The centrally confined sheathing resulted in the removal of torsional forces on the frame and a different failure mode that exhibited substantial increase in both shear resistance and ductility in comparison with walls having external sheathing. The initial test walls proved to have shear resistance over four times that currently found in the AISI S400 Standard and could maintain this resistance to drifts exceeding 6%. However, the higher shear forces posed difficulties in designing the framing members and attachments to the foundation given that the structure was composed of cold-formed steel with a maximum thickness of 2.5 mm. Thus, this second laboratory-based study was performed to examine centre-sheathed shear walls with lower shear resistances than those from the previous test program, while maintaining the ability to carry load at high drift levels. In addition, tests were carried out on the bare frame structure to identify the added shear capacity provided by the specially detailed CFS perimeter frame. The present paper contains a summary of the test program, including a report on the measured response of these centre-sheathed shear walls.

Keywords: Cold-formed steel; Steel sheathing; Shear wall; Seismic design; Structural design

INTRODUCTION

The cold-formed steel (CFS) framed steel-sheathed shear wall is a lateral force resisting system used to protect a building from seismic forces. These walls use thinner members and a lighter sheathing than the infill plates used in hot-rolled steel plate shear walls. Both systems rely on a tension-field action from the sheathing, or infill plate, to resist horizontal forces and to dissipate energy; however, CFS shear walls are primarily restricted to low-rise buildings due to their lower capacity. A recent laboratory research program demonstrated the possibility to improve the strength of CFS shear walls to an extent that they could be applied for use in mid-rise buildings, e.g. 5 to 8 storeys, and to compete with hot-rolled steel shear walls.

The AISI S400-15 Standard [1] includes the design provisions for steel-sheathed shear walls in its Section E2. The permitted wall configuration is a steel frame composed of chord members with the sheathing attached, using fasteners such as self-drilling screws, to the outside of the frame. However, the parameters of the wall - such as sheathing thickness, fastener size, and fastener spacing - are limited to values obtained through prior laboratory testing; the design strengths of these preconfigured walls are based on tabulated values. In the USA and Mexico, there is somewhat more freedom to choose the wall's parameters since the shear strength may be calculated based on the Effective Strip Method developed by Yanagi and Yu [2]. This method remains restricted to a range of parameters; it is not permitted in Canada, leaving Canadian designers with only the tabulated values of less than 30kN/m per wall. AISI S400 does permit doubling of the wall strength to 60kN/m if a sheathing panel is attached to both sides of the frame (Figure 1a) but only if the framing members are confirmed to be able to carry the larger corresponding forces.

Attempts have been made to increase the shear resistance of the CFS sheathed shear wall. When sheathing is placed on both sides of the wall, eccentric torsional forces are minimized, resulting in an increase in overall shear resistance. The double-sheathed design was considered by Brière [3] and Santos [4], but tests showed that the overall ductility of this configuration was not improved compared to a one-sided sheathed wall. To further improve the strength and ductility, Brière and Santos tested a new "centre-sheathed" wall configuration (Figure 1b), culminating in a paper published by Brière et al. [5]. In the centre-sheathed wall, the steel sheathing was confined between two sets of chord members. This wall design achieved a much greater level of ductility and a higher energy dissipation given the same total thickness of sheathing as a double-sheathed wall. The strongest specimen by Brière et al. [5] peaked at a strength of 165.7kN/m – more than four times what is available in the AISI S400 Standard. In addition, most walls tested in this configuration were able to reach over 4% lateral drift before strength degradation, whereas the exterior sheathed wall configurations experienced degradation at 1% to 2% drift. While this large jump in strength is promising, it did pose many difficulties in the design of the surrounding frame members: Brière [3] and Santos [4] described the many iterations of reinforcement needed to prevent the 2.5mm thick CFS chord members from buckling prematurely. Furthermore, the centre-sheathed wall is a completely new configuration with no other data to compare.

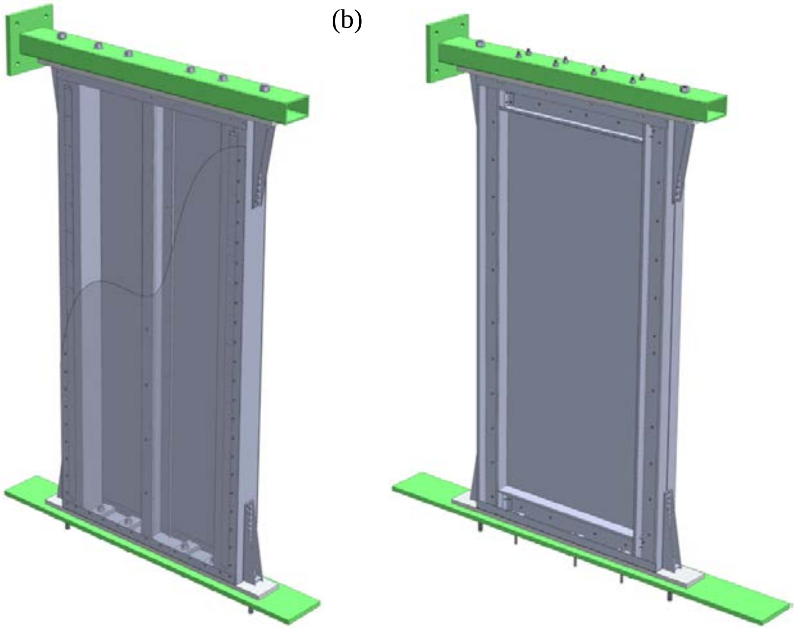

Figure 1. CFS shear walls : (a) double-sheathed configuration with cutaway view, (b) centre-sheathed configuration

To better understand the new wall configuration and to bridge the considerable gap between the existing AISI S400 shear strength values and the maximum values attained by Brière et al. [5], this second laboratory study was completed to examine the various wall parameters with the goal of producing centre-sheathed walls with intermediate strengths and the same excellent ductility. This was also an opportunity to verify if the model of estimating strength used in the previous study remains a good fit for all the tested walls. Furthermore, there was no prior data on the contribution to lateral resistance from the frame itself; in this project, additional frame-only walls were tested to address this shortcoming.

METHOD

Prediction and modelling

The innovative centre-sheathed configuration is not found in any published design standard, thus it required numerical analysis to confirm the adequacy of its member sizes. Each shear wall specimen was designed according to a capacity-based approach in which the ductile fuse was the sheathing-to-fastener connection. The resistance of the wall was entirely determined by the sheathing connections, while the surrounding members in the frame were required to carry the forces associated with sheathing connection failure. The sheathing resisted lateral forces via a tension field action, in which it acted like a thin, wide brace member pulling diagonally on the frame. The vertical members of the frame were subjected to a high axial force and a bending moment when the sheathing develops this tension field. Therefore, the maximum resistance of the sheathing connections must first be estimated for an adequate frame to be chosen. This resistance depended on the number of fasteners participating in the tension field and the bearing resistance of each fastener. Brière [3] and Santos [4] found previously that the tension field in the centre-sheathed configuration did not follow the Effective Strip Model from Yanagi and Yu [2]. Rather, due to the confinement of the sheathing and the high in-plane stiffness of the frame, fasteners across the entire wall participated in the tension field. This was one of the reasons why lateral resistance increased compared to the double-sheathed walls and why additional reinforcement was required for the vertical chord members [5].

Secondly, the resistance of every individual fastener was greater than anticipated. This was explained by the different failure mode that accompanied the new wall configuration. In double-sheathed walls, as well as walls with external sheathing on one side of the frame, the primary failure mode involved the fasteners causing bearing damage to the sheathing until the holes deformed enough that the sheathing, while buckling in shear, pulled up and over the head of the fasteners and became separated from the frame (Figure 2a). This shear failure mode for a 2-ply screw connection followed Eq. 1, found in Section J4.3.1. of the AISI S100 [6] and CSA S136 [7] Standards. In contrast, the confinement of the sheathing in the centre-sheathed wall made it impossible to pull the sheathing over the fasteners. Brière [3] and Santos [4] postulated that the sheathing's failure mode became pure bearing (Figure 2b) and was better represented by Eq. 2, found in Section J3.3.1 of AISI S100 [6] and CSA S136 [7]. This equation was originally meant for connections using bolts rather than screws; the constant C was close to 3.0 and depended on the bolt diameter and member thickness, and the constant m_f was 1.33 for the mid ply of a 3-ply connection. By switching equations, the predicted strength was increased by approximately 48%, which was later verified through testing.

$$P_{nv} = 2.7\ t_1\ d\ F_{u1} \tag{1}$$

$$P_{nb} = C\ m_f\ d\ t\ F_u \tag{2}$$

(a) (b)

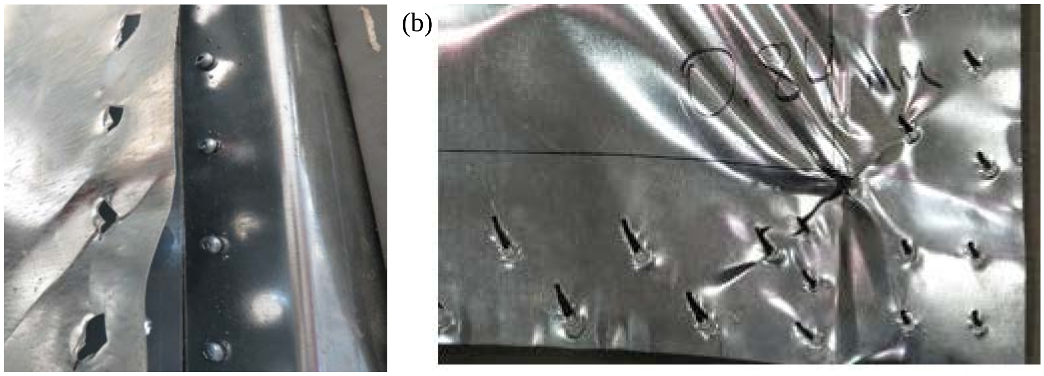

Figure 2. (a) Bearing of exterior sheathing resulting in pullover, (b) bearing of interior confined sheathing (Santos [4])

Following this model, dubbed the Modified Effective Strip Method (MESM), for any given fastener spacing the total number of screws in the wall was known and the bearing resistance of each screw-to-sheathing connection was found via Eq. 2. The tension field acted diagonally from one corner of the wall to the other, resulting in an angle of 63.4º to the horizontal due to the 2:1 aspect ratio of the wall; the horizontal force that the wall can resist was found using this geometry. The forces expected on the frame members were modelled using SAP2000 software; the sheathing was approximated using parallel diagonal strips and a lateral force equal to the predicted resistance of the sheathing connections was applied to the wall to find the corresponding forces on the frame. This was an iterative process because the chord sizes and level of reinforcement must first be assumed. Furthermore, it was unknown how much moment resistance was provided by the frame itself; to be conservative, both a pin-connected and a moment-connected frame were checked in the model. If the frame members could resist the interaction of axial forces, shear, and bending moments according to CSA S136 [7] calculations, then the design for the wall was accepted.

Detailing

As in the previous study, the walls were 2438mm high and 1219mm wide. Depending on strength requirements, reinforcing chords may be added to the vertical members, attached using steel strips and self-drilling screws. While the centre-sheathed tests conducted by Brière [3] and Santos [4] used the final reinforcement configuration denoted R3, some of the lower capacity walls presented herein required only R2 or no reinforcement at all for the chord studs (Figure 3). The prior study also found that a fastener spacing of 150mm for the frame-to-sheathing connections may result in chord stud separation as the sheathing buckles and pushes the frame apart during loading; therefore, in this study all screw spacings were chosen to be less than 150mm. The walls were held to a steel foundation by holdowns and shear anchors; there was a single holdown at each top corner and either one or two holdowns at each bottom corner depending on the anticipated uplift forces. All walls had eight shear anchors at the top track member and six shear anchors at the bottom track member. The corners of the frame were further supported by stiffeners to resist damage at high rotations. Clip angles were used at the corners to help secure the horizontal framing members to the vertical members. The detailing of the wall is illustrated in Figure 4.

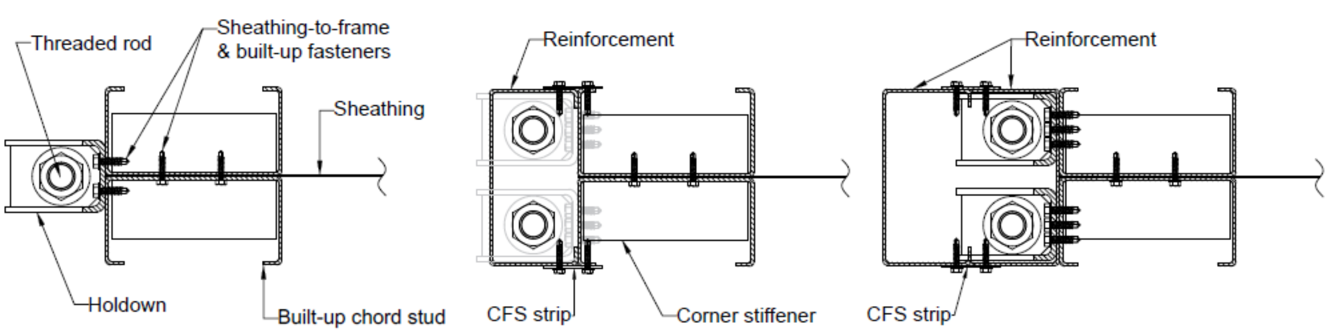

Figure 3. Left to right : no reinforcement, R2, and R3 reinforcement schemes for the chord stud members

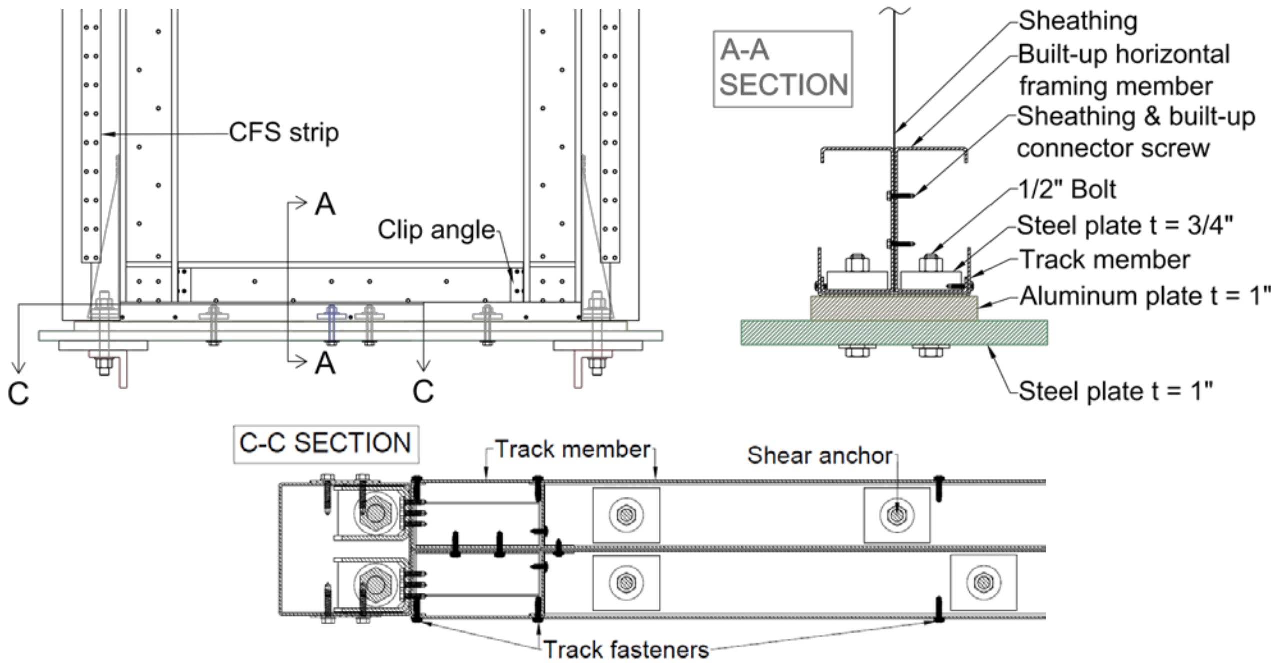

Figure 4. Detailing of centre-sheathed shear wall

New to this present research is the inclusion of three tests involving frame-only walls (designated with "F"). It had been noted in the previous study that the vertical chords buckled locally near the corners at extreme drifts, suggesting that the frame contributed lateral resistance due to moment action, likely aided by the installation of clip angles and stiffeners. This moment resistance added to the total lateral resistance of the wall; the goal of the frame-only tests was to isolate the frame contribution. The frame-only walls followed the same detailing as the full walls but with most of the sheathing removed; only four triangular pieces of sheathing were attached at the corners since it was assumed that the sheathing did contribute to the flexural action of the frame corners (Figure 5). Additional rectangular pieces were distributed along chord members to act as spacers between the frames.

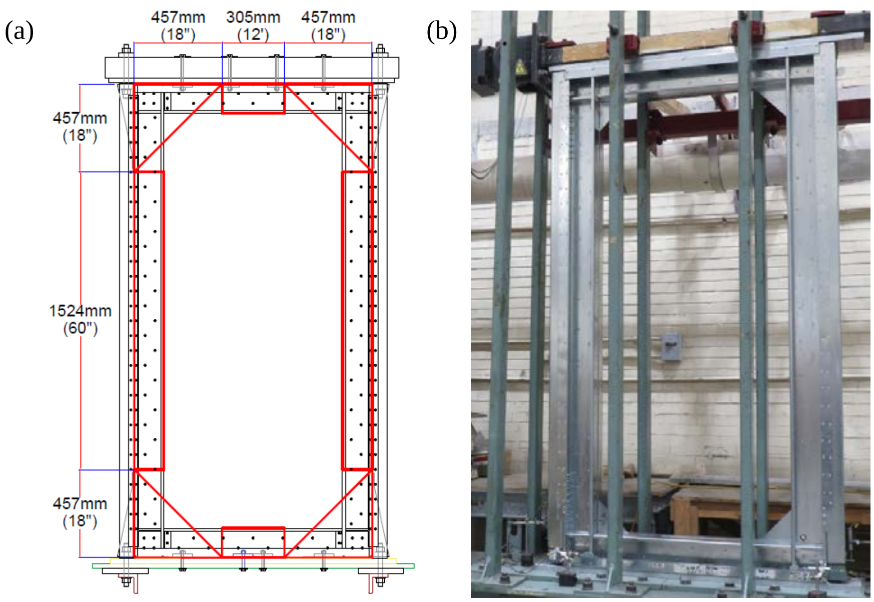

Figure 5. Frame-only shear wall W37F : (a) configuration and dimensions, (b) specimen installed in test frame

Materials

The materials used in this study were the same as those used by Brière et al. [5]. The vertical and horizontal framing members were 2.46mm thick C-shapes 152.4mm × 76.2mm × 15.9 mm. The channel tracks at the top and bottom of the wall were constructed of 156.0mm × 50.8mm C-shapes. The members used to attach the reinforcing chords were 50.8mm wide strips, 1.73mm in thickness. The corner stiffeners were cut from 1.73mm thick C-section members 152.4mm × 50.8mm × 15.9 mm.

All chords, tracks, strips, and stiffeners were made of ASTM A653 Grade 50 (340) steel. On the other hand, the sheathing was obtained from ASTM A653 Grade 33 (230) steel. The holdowns were Simpson Strong-Tie S/HD15S paired with 25.4mm diameter ASTM A193-B7 threaded rods. Shear anchors were 12.7mm ASTM F3125 grade A325 bolts. Hex head self-drilling screws of varying sizes were used as fasteners; sizes #8 and #10 were used on sheathing-to-frame connections; #10 and #12 were used on the frame and reinforcements, and #14 were used on holdowns.

Test protocol

Once mounted on the test frame, the walls were displaced using a 250kN actuator attached at the top of the wall. Shear force was measured using a load cell, while a string potentiometer measured the wall-top displacement, and LVDTs at the bottom corners of the wall measured any slip or uplift. Due to the high ductility of the centre-sheathed wall, a long stroke length was required: all tests in this project followed an asymmetric CUREE cyclic loading protocol with a loading rate of 0.05hz, as per Brière et al. [5]. This asymmetric displacement allowed a displacement of 220mm in the positive direction, which corresponded to approximately 8% drift, and -5mm in the negative direction. Monotonic tests were not performed in this study.

Test schedule

Compared to the walls examined by Brière et al. [5], thinner sheathing and smaller #8 screws were tested to reduce the shear strength. The thickness of the chord members remained as per the previous tests. The type of test “C” designates cyclic loading and R2 & R3 designate the reinforcement of the frame. The shear wall tests performed in this study are listed in Table 1.

Table 1. Shear wall test schedule

Test ID	Sheathing thickness (mm)	Framing thickness (mm)	Screw size (#)	Fastener spacing (mm)	Type of test
W40	0.36	2.46	8	75	C_R2
W32	0.36	2.46	8	100	C
W33	0.36	2.46	8	125	C
W35	0.47	2.46	8	50	C_R3
W36	0.47	2.46	8	75	C_R2
W38	0.47	2.46	8	100	C_R2
W34	0.47	2.46	10	50	C_R3
W37	0.47	2.46	10	75	C_R2
W39	0.47	2.46	10	100	C_R2
W35F	0.47	2.46	8	50	C_R3
W37F	0.47	2.46	10	75	C_R2
W15F	0.84	2.46	10	50	C_R3
W15B*	0.84	2.46	10	50	C_R3
W23B*	1.09	2.46	12	100	C_R3
W25*	0.84	2.46	10	100	C_R3
W26*	1.09	2.46	10	100	C_R3

*Tests performed by Brière et al. [5]. Parameters given here for comparison.

RESULTS

Failure modes

The observations for the centre-sheathed shear wall tests were similar to those reported by Brière [3] and Santos [4]. Under a lateral load, the sheathing developed a tension field along the entire wall (Figure 6a). After each test, the wall was disassembled and bearing damage was observed in the sheathing where the fasteners were connected (Figure 7a). The holes had a diagonal orientation approximately parallel to the direction of the tension field. Some walls exhibited corner tearing at high drift demands, which resulted in a section of the sheathing breaking off entirely (Figure 7b). These levels of bearing deformation were only possible due to the confinement of the sheathing. At extreme levels of drift, only after the peak strength of the wall, the vertical chord members were seen to exhibit local damage near the corners. The frame-only tests showed bearing damage on the triangular corner sheathing pieces similar to the full wall but with no tearing. There was no tension field due to a lack of sheathing (Figure 6b). At large deformations there was also damage to the chord members near the corners.

Strength and ductility

Since all the tests were asymmetric, only the positive displacement range was used for analysis. The energy dissipated by each wall was the total areas under its force vs. deformation hysteresis. A backbone curve was fitted to the data (Figure 8) to aid in the evaluation of shear strength and ductility. The maximum force was considered to be the ultimate state of the wall, with its corresponding deflection used to calculate the drift. The results of all the tests are summarized in Table 2, which includes a comparison to the predicted shear strength values according to the MESM model. The frame-only tests have no prediction with this model because there was no sheathing to develop a tension field.

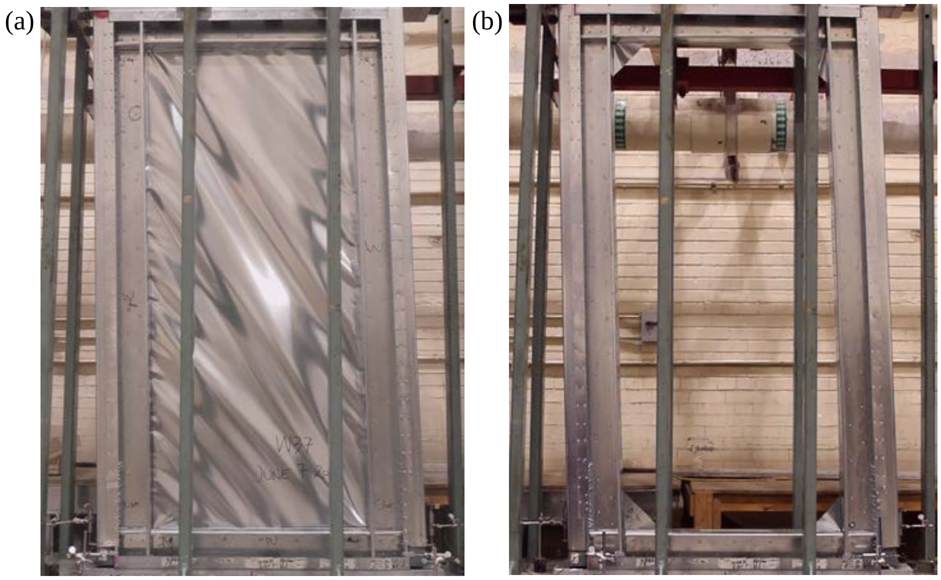

Figure 6. Tension field during testing : (a) Test specimen W35 (b) Test specimen W35F

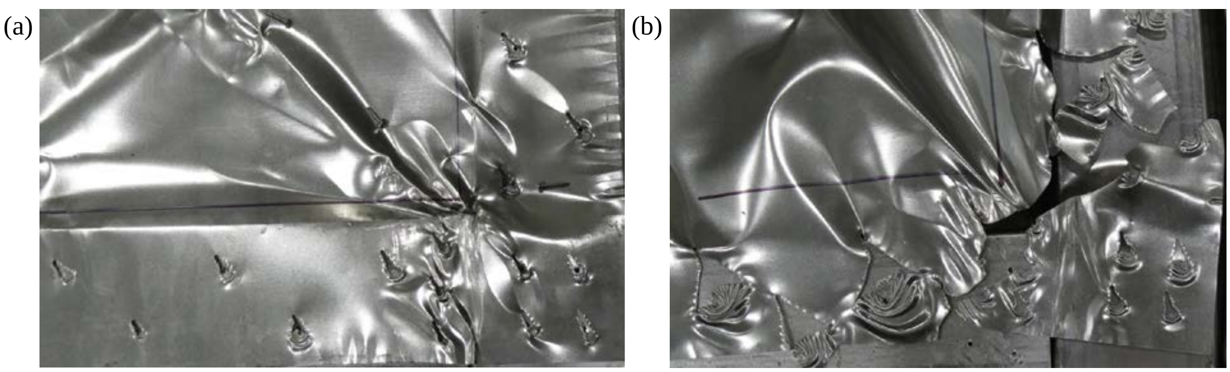

Figure 7. Bearing damage : (a) Specimen W36, (b) Specimen W37 with torn corner due to extensive drift demand

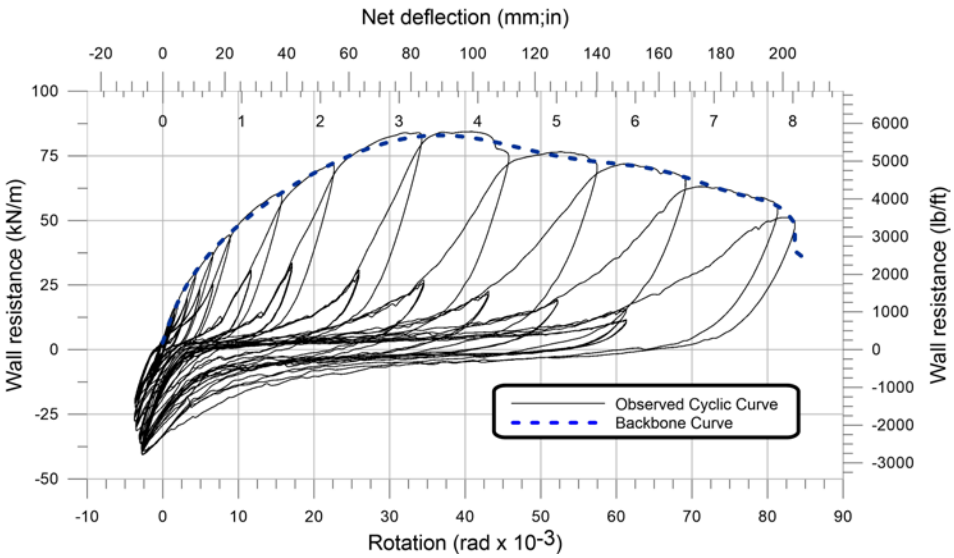

Figure 8. Conversion of W37 force vs. deformation hysteresis to a backbone curve

Table 2. Ultimate strength, deformation, drift, and dissipated energy of the centre-sheathed walls

Test ID	S_u, MESM (kN/m)	S_u, tested (kN/m)	$\Delta_{net, u}$ (mm)	Drift (%)	Energy Total (J)
W40	40.9	68.4	91.5	3.75	33377
W32	33.3	54.8	111.6	4.58	29464
W33	27.3	46.7	91.9	3.77	28753
W35	78.1	91.0	96.4	3.96	50215
W36	57.0	77.1	90.4	3.71	45276
W38	46.5	69.0	94.7	3.88	39429
W34	90.0	90.2	107.6	4.41	51943
W37	65.6	82.9	89.4	3.67	50132
W39	53.5	74.2	95.9	3.93	44567
W35F	-	36.4	109.4	4.49	12277
W37F	-	33.8	119.2	4.89	14903
W15F	-	48.3	93.9	3.85	20562
W15B*	162.2	165.7	160.0	6.56	109013
W23B*	150.3	158.6	121.6	4.99	98377
W25*	96.5	116.7	86.0	3.53	70483
W26*	132.2	145.3	65.7	2.70	61059

*Tests performed by Brière et al. [5]. Values given here for comparison purposes.

The results satisfied the objective of producing walls with intermediate resistances while maintaining high ductility. The thin sheathing and smaller screws reduced the thickness and diameter terms in the bolt bearing equation (Eq. 2), resulting in a lower MESM prediction of shear strength, and the tested strengths were indeed lower than in the previous study. While the various parameters did change the ultimate shear strength of the wall, the general shape of the force vs. deformation hysteresis and its corresponding backbone curve remained comparable across all tests. The new results for the full shear walls were compared to the existing data from Brière et al. [5]. For a fair comparison, only the asymmetrically loaded walls were considered: W15B, W23B, W25, and W26. The tested ultimate shear strengths were graphed alongside the values predicted by the MESM (Figure 9). It was observed that although there was a clear relation, the tested values were systematically higher than expected. The MESM accounted only for the bearing strength of the sheathing and fastener but included nothing about the frame itself. It was suspected that the frame may have contributed the extra resistance that was raising the test values above the prediction.

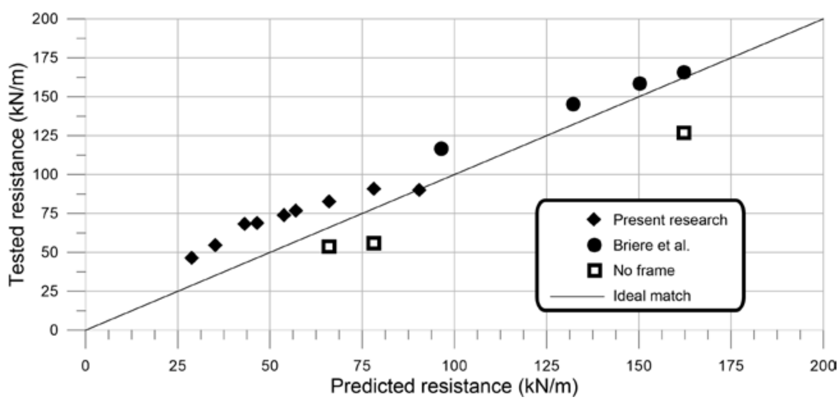

Figure 9. MESM S_u prediction versus tested results

The results of the frame-only tests are summarized in Table 2. These walls had a comparable backbone curve, but were less stiff and typically exhibited a peak shear strength at a larger drift than their corresponding full wall (Figure 10a). To isolate the strength of the sheathing, the results of the frame-only wall were subtracted from those of the full wall. As shown in the backbone curves in Figure 10a, the peak shear strengths did not occur at the same displacement; hence, it may not be entirely accurate to subtract the S_u values. As the CUREE protocol uses displacement as input, it was treated as the independent variable and the force was subtracted along equal displacements on the graph. The resulting curve (Figure 10b) had its own peak value that was taken to be the tested ultimate shear strength of the sheathing alone. These updated "no frame" ultimate resistances are included in Figure 9 and are compared to the original MESM prediction. With the frame removed, the shear strength of the sheathing became lower than predicted. This mismatch may be due to the MESM not taking into account the frame action when the model was initially calibrated, even though the frame action existed for every wall. Furthermore, the sheathing corners used for the frame-only wall tests could have added extra bearing resistance to the wall, resulting in the measured frame-only shear strength appearing higher than if the frame were truly alone; subsequently, the sheath-only strength may have been underestimated. These uncertainties will be addressed in future research; one alternate approach is to examine the shear strength of an individual fastener attached to sheathing arranged in 3-ply and then comparing the result to the MESM prediction for only one fastener.

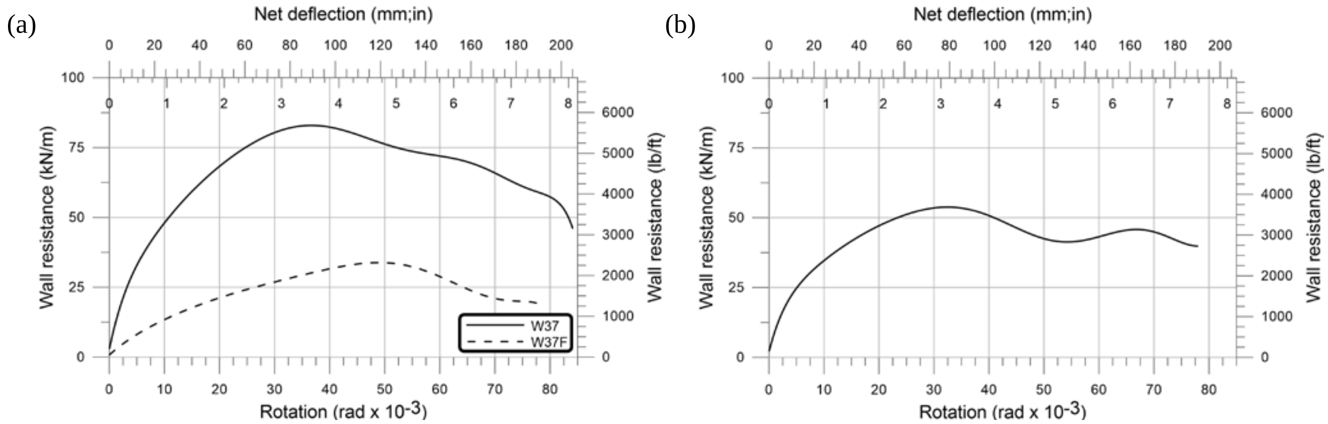

Figure 10. W37 force vs. deformation : (a) full wall and frame-only, (b) Difference between the two backbone curves

CONCLUSIONS

The AISI S400 Standard permits the construction of single and double-sheathed CFS frame shear walls. However, the strength and ductility of the existing system may not be adequate for use in mid-rise buildings; further, design parameters are restricted to previously tested configurations. The centre-sheathed shear wall configuration demonstrates a higher performance, minimizes torsional effects, and exhibits the potential to compete with hot-rolled steel plate shear walls in mid-rise buildings. In this study, shear walls were successfully tested; they showed intermediate strengths while preventing premature failure of the frame in accordance with capacity-based design. More data was collected for the centre-sheathed wall configuration; strengths between 46.7 to 91.0kN/m were possible with ultimate drifts near 4%. The major failure mode for the sheathing was confirmed to be bearing in the mid ply of a 3-ply connection. The Modified Effective Strip Model (MESM) was shown to be a good but imperfect predictor of shear strength for the walls. A systematic error between test values and prediction values was attributed to the frame contribution, which was significant due to the amount of flexural restraint at the corners. To further understand the resistance of the sheathing-to-fastener bearing connection, individual connection testing is recommended. The connection for a self-drilling screw in 3-ply is not addressed in any design standard for cold-formed steel; research is ongoing.

ACKNOWLEDGEMENTS

Financial support for this research was provided by the Canadian Sheet Steel Building Institute (CSSBI) and the Natural Sciences and Engineering Research Council of Canada (NSERC). Gratitude is expressed to the laboratory staff John Bartczak, William Cook, Damon Kiperchuk, and Jorge Sayat, as well as Vincent Brière for explaining the prior research on this topic, and fellow students Albert Charukhchyan, Andrea Iachetta, Keith Lee, and Brice Takou for their support. The materials and tools for the construction of the test specimens were provided by Bailey Metal Products Ltd., Simpson Strong-Tie Company Inc., Ontario Tools & Fasteners Ltd., UCAN, Arcelor Mittal, and DeWALT.

REFERENCES

[1] AISI (2015). “North American Standard for Seismic Design of Cold-formed Steel Structural Systems.” AISI S400-15, American Iron and Steel Institute, Washington, DC, USA.

[2] Yanagi N., and Yu C. (2014). “Effective strip method for the design of cold-formed steel framed shear wall with steel sheet sheathing.” *Journal of Structural Engineering*, 140(4), 1–8.

[3] Brière V. (2017). “Higher capacity cold-formed steel sheathed and framed shear walls for mid-rise buildings: Part 2.” Master’s thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Québec, Canada.

[4] Santos V. (2017). “Higher capacity cold-formed steel sheathed and framed shear walls for mid-rise buildings: Part 1.” Master’s thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, Québec, Canada.

[5] Brière V., Santos V., and Rogers C.A. (2018). “Cold-formed steel centre-sheathed (mid-ply) shear walls.” *Soil Dynamics and Earthquake Engineering*, 114, 253–266.

[6] AISI (2016). "North American Specification for the Design of Cold-formed steel Structural Members.” AISI S100-16, American Iron and Steel Institute, Washington, DC, USA.

[7] CSA (2016). “North American Specification for the Design of Cold-formed Steel Structural Members.” CSA S136-16, Canadian Standards Association, Toronto, Ontario, Canada.