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INFLUENCE OF GYPSUM PANELS ON THE RESPONSE OF COLD-FORMED STEEL FRAMED STRAP-BRACED WALLS

Sophie Lu[1], Colin A. Rogers[2]

Abstract

In cold-formed steel construction the steel frame is supplemented with either diagonal strap braces or structural sheathing panels (typically steel or wood) to provide overall stability to the structural system and to directly transfer lateral wind and seismic loads through to the foundation as per the design provisions found in AISI S240 (2015) and AISI S400 (2015). Gypsum panels are often specified to provide a fire-resistance rating for the CFS frame, as well as to ensure that adequate sound-proofing exists between adjacent rooms or building units. The engineer may choose to rely on this gypsum to provide additional lateral resistance, as permitted in the AISI Standards. However, in the majority of cases the gypsum panels are considered to be non-structural elements of the building specified by the architect, and as such, are not taken into account in the design of the lateral load carrying system. Whether considered in the design process or not, these gypsum panels do augment the shear resistance of the lateral load carrying system. This study was carried out to evaluate the performance of combined strap-braced / gypsum-sheathed wall systems, with the intent of defining a corresponding design approach. Described herein are the findings of the laboratory phase of the project, comprising 35 wall specimens.

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Introduction

In cold-formed steel (CFS) construction the steel frame, which is composed of closely spaced gravity carrying stud members, is supplemented with either diagonal strap braces or structural sheathing panels (typically steel or wood) to provide overall stability to the structural system and to directly transfer lateral wind and seismic loads through to the foundation as per the design provisions found in AISI S240 (2015) and AISI S400 (2015). Gypsum panels are often specified to provide a fire-resistance rating for the CFS frame, as well as to ensure that adequate sound-proofing exists between adjacent rooms or building units. As an example, in the case of 1-hour and 2-hour fire resistance rated assemblies, as required by the National Building Code of Canada (NBCC) (NRCC 2015) for certain occupancy types, it is necessary to install one to two layers of 15.9 mm thick Type X gypsum on both sides of the wall. The engineer may choose to rely on this gypsum to provide additional lateral resistance, as permitted in the AISI Standards. However, in the majority of cases the gypsum panels are considered to be non-structural elements of the building specified by the architect, and as such, are not taken into account in the design of the lateral load carrying system.

On one hand, it is understood that there exists a beneficial structural effect of installing gypsum panels in a CFS framed building; that is, additional shear resistance to lateral loading. On the other hand, since the additional resistance of these panels will likely not be taken into account in design there also exists a detrimental effect. Firstly, given the similar response to lateral in-plane loading of CFS framed structural walls and CFS framed gypsum-sheathed walls it is known that the non-structural gypsum panels will increase the stiffness of the building, which may result in greater seismic loads. Secondly, in current North American seismic design, following AISI S400, CFS framed structures must be designed following a capacity-based approach in which the probable resistance of the fuse element in the seismic force resisting system is used, along with all companion gravity loads, to determine the forces applied to the remaining structural members in the lateral load carrying path. The AISI S400 Standard does not explicitly require the inclusion of the non-structural gypsum sheathing in the calculation of capacity forces in a strap-braced CFS framed shear wall. In all likelihood, the unaccounted for gypsum panels will raise the seismic force levels beyond the probable resistance of the brace, in a strap-braced wall, or the sheathing connections, in a shear wall, resulting in capacity forces that are significantly above those used in design. This may result in force demands on the chord studs, tracks, holdowns, foundations, etc., that are higher than anticipated, and ultimately may cause their failure at overall building drift levels that are lower than, and not consistent with, those expected and used in the development of seismic design code provisions.

Various research programs have been carried out to experimentally investigate the performance of strap-braced walls, e.g. Adham et al. (1990), Serrette & Ogunfunmi (1996), Barton (1997), Gad et al. (1999), Tian et al. (2004), Fülöp & Dubina (2004), Kim et al. (2006), Al-Kharat & Rogers (2007, 2008), Moghimi & Ronagh (2009), Velchev et al. (2010), Macillo et al. (2014), and Iuorio et al. (2014), among others. Similarly, gypsum-sheathed bearing and shear walls have been tested by Klippstein & Tarpy (1992), Serrette et al. (1997), Salenikovich et al. (2000), Bersofsky (2004), Landolfo et al. (2006), Lee et al. (2007), Memari et al. (2008), Moghimi & Ronagh (2009), Morello (2009), Peck et al. (2012), Davies et al. (2011) and Liu et al. (2012), among others. However, a variation of strength and stiffness in previous gypsum sheathed wall tests has been observed; hence, it is difficult to extrapolate the results and foresee how thicker framing and gypsum can affect the load sharing. Furthermore, very few tests with 1 or 2 layers of 15.9 mm gypsum panels are available, and little is known about the interaction of the strap-braced and gypsum-sheathed systems in a single wall.

Thus, in this paper, an experimental program is described, which can be used to complement the existing database of strap-braced and gypsum-sheathed walls. A series of 35 tests on strap-braced walls, gypsum sheathed shear walls and gypsum sheathed bearing walls, having 1-hour and 2-hour fire resistance rating, was completed. A short discussion of the influence of gypsum panels on CFS framed strap-braced walls is provided.

Test program

Thirty-five single-storey walls were tested in the Jamieson Structures Laboratory at McGill University with monotonic and CUREE reversed-cyclic (Krawinkler et al. 2000) displacement-based lateral loading protocols to investigate the effect of 1 to 2-hour fire resistance rated gypsum configuration on the shear behaviour. A 1-hour fire resistance rating for a load-bearing steel assembly is achieved by affixing one layer of 15.9 mm (5/8”) Type X fire resistant gypsum on both sides of the steel frame (ULC, 2006). To construct a 2-hour fire resistant assembly, two layers of 15.9 mm (5/8”) Type X fire resistant gypsum can be affixed to both sides of the steel frame (ULC, 2006). Two main categories of walls were tested: shear walls and bearing walls. Shear walls are designed to resist in-plane lateral load, and thus have holdowns to anchor the studs to the ground. Bearing walls carry gravity loads alone, hence are not designed to resist lateral load, and thus do not have holdowns. Figure 1 contains a photograph of the test setup and a representative gypsum-sheathed / strap-braced test wall. A listing of the test specimens is provided in Figure 2. All the walls were 2.44 m high and 1.22 m long (aspect ratio of 2:1) and the studs were spaced at 406 mm. The walls were installed in a test frame specifically designed for in-plane shear loading. The test frame is equipped with a 250kN MTS dynamic loading actuator with a ±125mm stroke. Out-of-plane movements of the walls were

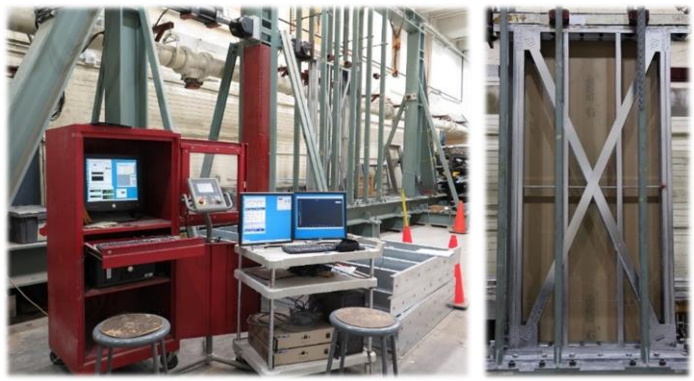


Figure 1 – Shear Wall Test Setup and strap-braced shear wall with gypsum panel

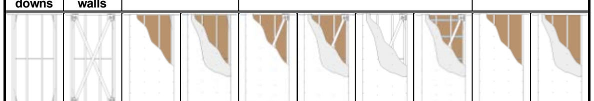
	Test specimens									
	Steel frame with hold-downs	**Strap-braced shear walls**	**Gypsum-sheathed shear walls**		**Gypsum-sheathed strap-braced shear walls**				**Gypsum-sheathed bearing wall**	
										
Name of the specimen	82 A-M	65 A-M 83 A-C	66 A-M 66 B-M 67 A-C 67 B-C	68 A-M 68 B-M 69 A-C 69 B-C	70 A-M 70 B-M 71 A-C 71 B-C	72 A-M 72 B-M 73 A-C 73 B-C	74 A-M 74 B-M 75 A-C 75 B-C	76 A-M 76 B-M 77 A-C 77 B-C	78 B-M 78 C-M 79 A-C 79 B-C	80 A-M 80 B-M 81 A-C 81 B-C
Straps - Thickness: 1.37 mm - Width: 69.9 mm - Grade: 340 MPa	No	Yes	No		Yes				No	
Gusset plates - 177.8 mm x 203.2 mm - Thickness: 1.37 mm - Grade: 340 MPa	No	Yes	No		Yes				No	
Type X Gypsum - 2.44 m x 1.22 m - Thickness: 15.9 mm	NA	NA	1 layer on both sides	2 layers on both sides	1 layer on both sides	2 layers on both sides	2 layers on 1 side	2 layers on 1 side; 2 layers + resilient channel on other side	1 layer on both sides	2 layers on both sides
Chord studs 152 mm x 41 mm x 12.7 mm	Double chord studs put back-to-back - Thickness: 1.37 mm - Grade: 340 MPa								Single chord stud - Thickness: 1.09 mm - Grade: 230 MPa	
Hold-downs Simpson Strong Tie S/HD15S	Yes								No	
Interior studs - 152 mm x 41 mm x 12.7 mm - Thickness: 1.09 mm - Grade: 230 MPa	Spaced at 406 mm o/c									
Tracks - 152 mm x 31.8 mm - Thickness: 1.37 mm - Grade: 340 MPa	Extended tracks (1.52 m long)									

Figure 2 – Listing of CFS framed wall test specimens

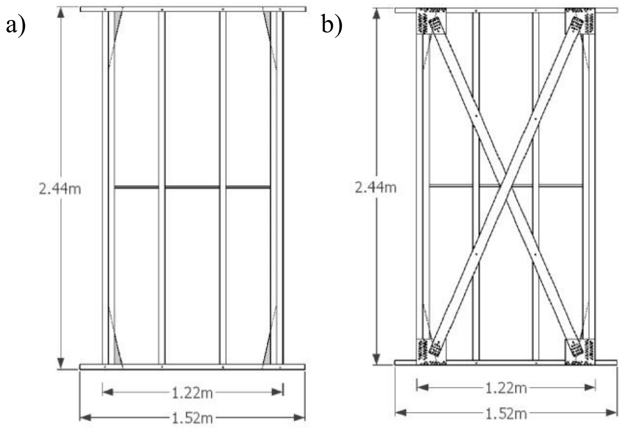


Figure 3 – Shear wall configurations – CFS frame: a) steel frame with holdowns, and b) strap-braced shear wall

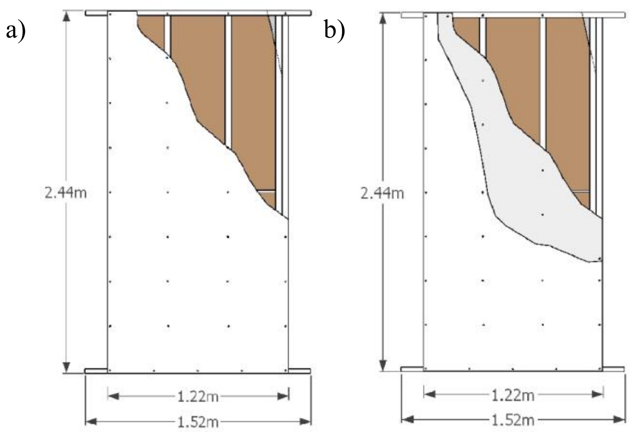


Figure 4 – Shear wall configurations – CFS frame and gypsum panels: a) one layer of gypsum on both sides, and b) two layers of gypsum on both sides

resisted with lateral supports that braced the load beam. One steel frame with hold-downs but no gussets plates or straps (Figure 3) was tested in order to quantify the frame contribution in the lateral resistance of shear walls. Two strap-braced wall with no sheathing (Figure 3) were also tested monotonically and cyclically for comparison purposes. Eight shear walls were sheathed with gypsum only and had no straps or gussets (Figure 4). Sixteen shear walls had straps, gusset plates and gypsum panels (Figure 5). In bearing walls (8 specimens) (Figure 6), no holdowns were used. In all the walls, the screws in each layer of gypsum were spaced at 300 mm o/c. For walls with one layer or for the inner layer of double layer sheathed walls, #6x25 mm (1”) type S drywall screws were used. In the outer layer of double layer sheathed walls, #6x41 mm (1”-5/8) type S drywall screws were used and were staggered with respect to the screws of the inner layer. Since the screws from the outer layer penetrated through the inner layer as well, the inner layer was attached to the frame every 150 mm. Detailed information on the walls’ construction is found in the work of Lu (2015).

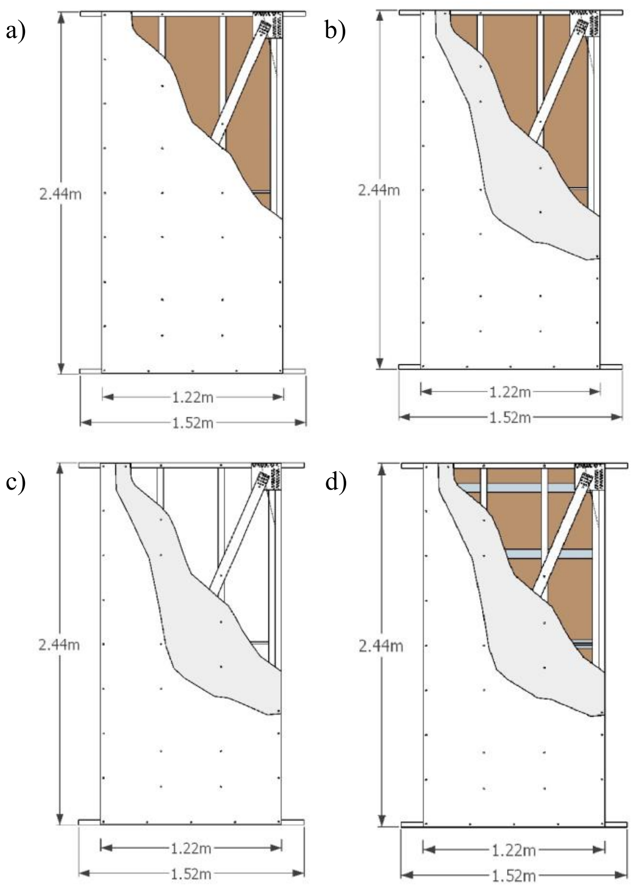


Figure 5 – Shear wall configurations – strap-braced CFS frame and gypsum panels: a) one layer of gypsum on both sides, b) two layers of gypsum on both sides, c) two layers of gypsum on one side, and d) two layers of gypsum on one side and two layers of gypsum and resilient channels on the other side

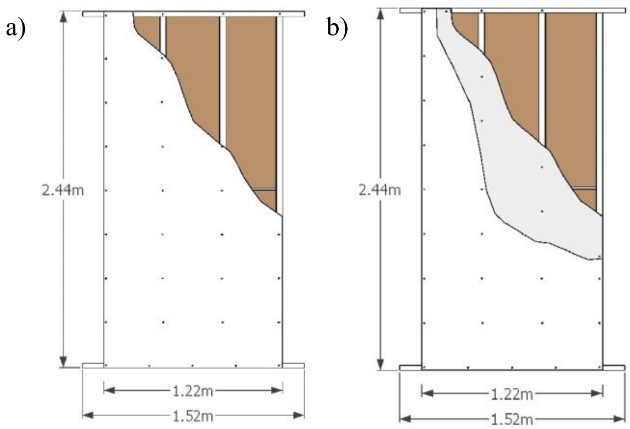


Figure 6 – Gypsum-sheathed bearing walls: a) one layer of gypsum on both sides, and b) two layers of gypsum on both sides

Design of test walls

CFS shear walls need to be designed according to capacity based design principles. To begin with, the fuse element is chosen so that the wall performs in a ductile fashion. The probable resistance is then estimated to design the non-fuse components of the wall. In all test configurations, this force was defined according to the largest fuse configuration resistance, which corresponds to the sum of the probable resistances of the straps and the two layers of gypsum on both sides of the frame. The probable resistance of a gypsum panel was obtained through use of the nominal resistance values in the now retired AISI S213 Standard (2007) (AISI S400 is the current equivalent standard) with a magnification factor of 1.33, and an adjustment for the 300 mm screw spacing that was used (150 mm for inner layer of two layer walls). The probable horizontal resistance of the wall was estimated to be 69 kN (32 kN for straps & 37 kN for gypsum panels). The corresponding vertical force on the chord studs and holdowns was 101 kN. See Lu (2015) for a description of the complete design calculation procedure. In contrast, the components of the bearing wall specimens were not designed with a capacity approach; standard member sizes were used.

Test observations

In all the shear walls (gypsum panels no strap braces), the steel frame was globally undamaged, which is consistent with the design assumption. The CFS components and their fasteners remained elastic except at some localized areas. The lips in the chord studs and interior studs exhibited some minor local distortion. Local web buckling was also observed at the bottom of the interior studs. Some distortional buckling in the chord studs due to bending was also observed. In the walls with strap-braces and gypsum panels, the straps subjected to tension have yielded, the straps subjected to compression have buckled and have provided effectively no resistance, while the steel frame mainly remained elastic. These were the expected member behaviours for the strap-braced walls. In bearing walls, uplift was not restrained by means of holdowns; as such, the tracks and stud-to-track connections were subjected to higher loads than they were in the shear walls. In the bottom corners in tension of some walls this led to the screw bearing failure of the flanges of the tracks or the shear failure of the screw connection between the studs and the track. Localized damage to the tracks and their flanges were also observed.

When lateral in-plane displacement was imposed on the walls, for the most part, the gypsum panels rotated as rigid bodies while the steel frame deformed in shear. The connections between the gypsum panels and the steel frame accommodated this differential displacement by means of bearing / pull through damage in the gypsum and bearing damage in the steel frame, as well as fastener tilting. Due to the differential displacement between the gypsum panels and the steel frame, the holes through which the screws were attached were enlarged. This failure mode is referred to as screw tilting. As the displacements of the wall became larger, the screw head carved into the gypsum, and in some cases pulled entirely through the panel. This failure mode is referred to as pull-through; it was evident at the screw connections along the perimeter of the wall since they were subjected to higher differential displacement. In the specimens tested with a reversed-cyclic protocol, the screw shear failure was not limited to the corners of the walls; rather, several screws failed in shear along the edges of the walls. In the walls with two layers of gypsum on one side and two layers of gypsum and resilient channels on the other side, the side with resilient channels had a different behaviour; failure was concentrated in the resilient channels; the sheathing-to-resilient channel and resilient channel-to-frame connections, as well as the gypsum sheathing remained relatively undamaged. In bearing walls, damage of the sheathing was limited to some screw locations along the perimeter of the panels. In the one-layer gypsum-sheathed bearing walls, screw tilting, screw pull-through, gypsum bearing, gypsum cracking and screw shear were observable. In the two-layer gypsum-sheathed bearing walls, screw pull-through and bearing were visible, along with some screw tilting. See Lu (2015) for photographs and complete descriptions of the walls' failure mechanisms.

Measured test results

The measured properties of each wall were determined for both the monotonic (Figure 7) and reversed-cyclic (Figure 8) tests. When multiple specimens of a wall configuration and loading protocol were tested, the average of the lateral loading response properties were determined. Illustrative shear resistance vs. deformation response graphs of strap-braced / gypsum-sheathed walls are provided in Figure 9.

	Test specimens									
	Steel frame with hold-downs	Strap-braced shear walls	Gypsum-sheathed shear walls		Gypsum-sheathed strap-braced shear walls				Gypsum-sheathed bearing wall	
										
Name of the specimen	82 A-M	65 A-M	66 A-M 66 B-M	68 A-M 68 B-M	70 A-M 70 B-M	72 A-M 72 B-M	74 A-M 74 B-M	76 A-M 76 B-M	78 B-M 78 C-M	80 A-M 80 B-M
F_u (kN)	3.93	31.61	9.60	21.91	37.70	50.04	38.91	40.92	7.64	8.00
$\Delta_{net,u}$ (mm)	125.7	124.5	36.7	64.0	46.6	49.8	53.3	54.0	48.9	38.5
K_e (kN/mm)	0.028	1.48	2.24	2.25	2.27	2.71	2.26	2.13	0.810	0.962
$\Delta_{net,max}$ (mm)	100.0	100.0	61.0	100.0	61.0	61.0	61.0	61.0	53.2	48.7
Normalized energy, Energy / Lateral drift (J/mm)	1.29	26.70	8.35	19.27	30.69	39.66	31.55	32.63	6.02	6.49
F_y (kN)	2.03 (2)	28.58 (1)	8.63 (2)	20.18 (2)	35.17 (2)	46.07 (2)	36.35 (2)	38.33 (2)	6.53 (2)	7.04 (2)
$\Delta_{net,y}$ (mm)	75.52 (4)	31.9 (3)	4.0 (4)	9.1 (4)	15.6 (4)	17.0 (4)	16.1 (4)	18.1 (4)	8.2 (4)	7.4 (4)
Ductility, μ	1.38	3.14	15.88	11.08	3.94	3.59	3.79	3.39	6.50	6.73
R_d	1.33	2.30	5.52	4.59	2.62	2.49	2.56	2.40	3.46	3.51
$\Delta_{y,mod.EEEP}$ (mm)	-	31.24 (5)	13.7	22.9	22.5	25.2	23.0	24.8	22.1	16.4
$K_{e,mod.EEEP}$ (kN/mm)	-	1.01 (5)	0.71	0.96	1.68	1.99	1.69	1.65	0.35	0.49

(1) Yielding force obtained by determining the plateau region
(2) Yielding force obtained with the EEEP method
(3) Yielding displacement corresponding to the point where the plateau region is reached
(4) Yielding displacement defined in the EEEP method
(5) Obtained with the modified EEEP method up to the displacement corresponding to the maximum stroke of the actuator

Figure 7 – Monotonic shear wall test results

Lateral resistance parameters were obtained for each wall specimen when it was possible. The wall resistances (kN) are designated with an identifier beginning with the letter F. In all the specimens, the ultimate resistance was defined as the highest load reached during the test. The corresponding displacement at ultimate resistance is listed as $\Delta_{net,u}$. The in-plane lateral elastic stiffness, K_e, of the wall was calculated as follows:

$$K_e = \frac{F_{0.4u}}{\Delta_{net,0.4u}}$$

where, $F_{0.4u}$ is equal to 40% of the ultimate load F_u, and $\Delta_{net,0.4u}$ is the in-plane lateral displacement of the wall corresponding to $F_{0.4u}$.

	Test specimens									
	Steel frame with hold-downs	Strap-braced shear walls	Gypsum-sheathed shear walls		Gypsum-sheathed strap-braced shear walls				Gypsum-sheathed bearing wall	
										
Name of the specimen	NA	83 A-C	67 A-C 67 B-C	69 A-C 69 B-C	71 A-C 71 B-C	73 A-C 73 B-C	75 A-C 75 B-C	77 A-C 77 B-C	79 A-C 79 B-C	81 A-C 81 B-C
F_u (kN)	-	33.54	9.05	21.07	37.46	49.36	41.04	41.86	7.83	8.73
$\Delta_{net,u}$ (mm)	-	103.3	41.2	58.5	52.7	44.2	48.3	76.8	56.5	30.4
K_e (kN/mm)	-	1.49	3.57	2.25	1.94	2.30	1.96	2.05	0.82	1.05
$\Delta_{net,max}$ (mm)	-	100.0	61.0	61.0	61.0	61.0	61.0	80.5	61.0	53.3
Normalized energy, Energy / Lateral drift (J/mm)	-	28.6	8.1	17.5	29.9	38.7	32.6	34.8	6.4	7.2
F_y (kN) [(1)]	-	32.0	8.3	18.9	35.2	46.5	39.1	39.8	6.8	7.8
$\Delta_{net,y}$ (mm) [(1)]	-	21.56	2.96	8.64	18.21	20.50	20.03	19.63	6.40	7.45
Ductility, μ	-	4.64	27.22	7.34	3.37	3.02	3.07	4.05	10.82	7.26
R_d	-	2.88	7.04	3.68	2.39	2.24	2.26	2.65	4.46	3.65
$\Delta_{y,mod.EEEP}$ (mm)	-	29.7	11.1	20.2	24.4	26.1	24.9	26.1	21.8	14.9
$K_{e,mod.EEEP}$ (kN/mm)	-	1.13	0.86	1.05	1.54	1.90	1.65	1.61	0.36	0.59

[(1)] Obtained with the EEEP method

Figure 8 – Reversed-cyclic shear wall test results

The displacement $\Delta_{net,max}$ was defined depending on the maximum code-based storey drift ratio $\Delta_{max,code}$ = 2.5% drift (61 mm) and the values of the lateral in-plane displacements $\Delta_{net,u}$ and $\Delta_{net,0.8u}$ corresponding respectively to F_u and to $F_{0.8u}$ (post-peak). The resistance of some wall specimens went below $F_{0.8u}$ before reaching $\Delta_{max,code}$. In these cases, the displacement $\Delta_{net,max}$ corresponding to the ultimate failure was taken equal to $\Delta_{net,0.8u}$. Conversely, several walls maintained their resistance beyond the maximum code-based storey drift ratio. Thus, for the walls which reached their maximum capacity F_u at a storey drift greater than 2.5%, showing that they still had a significant lateral resistance at high displacement, a less conservative maximum displacement ($\Delta_{net,max}$ = 100 mm) was chosen. For all the other cases, the displacement $\Delta_{net,max}$ corresponding to the ultimate failure was taken equal to code-based drift limit $\Delta_{max,code}$. Force vs. deformation graphs for each test specimen are available in the work of Lu (2015).

A small plateau region was observed for the monotonic test of the unsheathed strap-braced shear wall specimen (65 A-M). In this specimen, the yielding force, F_y, was taken as the lowest value in the post-yield plateau region. In gypsum-sheathed walls, no yield plateau region was typically observable. An equivalent energy elastic-plastic (EEEP) method was employed to estimate the yield resistance F_y. This equivalent energy approach is based on the assumption that the energy dissipated up to ultimate failure can be represented by a simplified bilinear elastic-plastic curve with the same energy dissipation, which is consistent with data evaluation approach used to obtain Canadian design shear values in AISI S240 and S400. The value of $\Delta_{net,y}$ is the displacement corresponding to the calculated F_y force. The ductility factor μ was determined as the ratio of Δ_{max} / $\Delta_{net,y}$, where Δ_{max} is the displacement corresponding to the failure limit state. The 'test-based' R_d value was determined as follows:

$$R_d = \sqrt{2\mu - 1}$$

The definition of K_e allows for a simple estimate of the elastic stiffness. It is accurate for systems that behave elastically at small displacements and reach their ultimate resistances well within the 2.5% inelastic drift limit. However, when subjected to lateral in-plane loading, gypsum-sheathed walls tend to behave non-linearly at relatively low drifts and the maximum resistance may be reached at high drifts. Thus, an alternate definition for the in-plane lateral elastic stiffness, which takes into account the ductile behaviour of the walls, was considered. This alternate stiffness was based on an EEEP model where the perfectly plastic region is at the level of F_u. Thus, knowing F_u, one could determine $K_{e,mod.EEEP}$ and $\Delta_{y,mod.EEEP}$. See Lu (2015) for example graphs.

Figure 10 shows the additional strength provided by the gypsum panels to a CFS strap-braced wall; the results of the monotonic tests were relied on in this illustrative graph. The test results demonstrated that attaching 15.9 mm-thick Type X gypsum panels to a strap-braced wall could provide 15% (one layer of gypsum on both sides) to 53% (two layers of gypsum on both sides) additional strength. One-layer and two-layer gypsum-sheathed bearing walls exhibited similar ultimate shear resistances because in both cases the steel frame failed at the stud to track connection, while the gypsum and the drywall screws suffered only minor damage.

In design, bearing walls are assumed incapable of efficiently transferring lateral in-plane load (and uplift forces) to the ground since they are constructed without holdowns. Therefore, gypsum-sheathed bearing walls cannot be used as lateral resisting systems. Nevertheless, if the lateral resistance of the bearing walls needs to be considered for the numerical evaluation of the overall dynamic performance

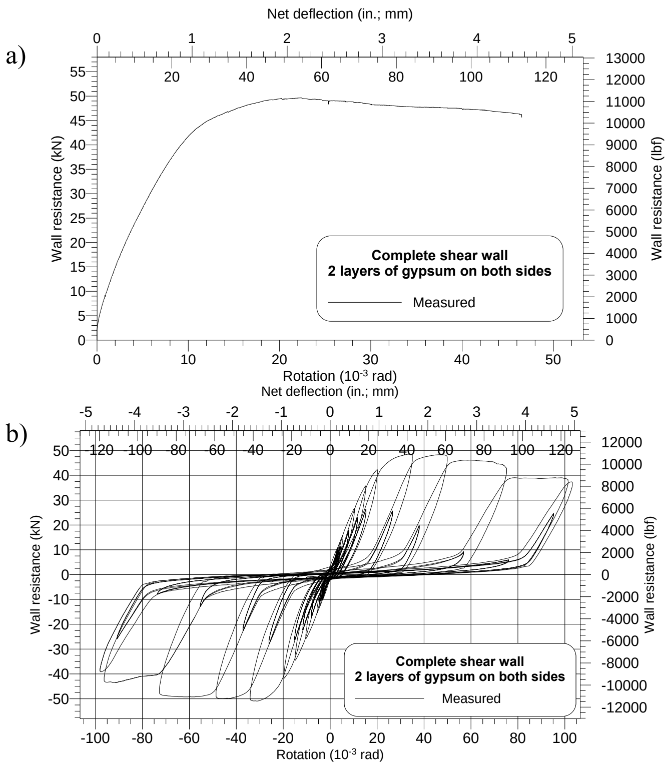


Figure 9 – Typical shear resistance vs. deformation response of strap-braced / gypsum-sheathed wall (Configuration 72 & 73 shown): a) monotonic loading protocol, and b) reversed-cyclic loading protocol

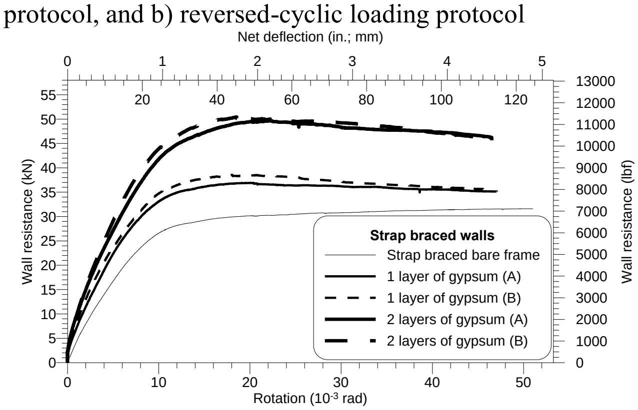


Figure 10 – Shear wall resistance vs. deformation response of monotonic tests showing influence of additional layers of gypsum panels

of building archetypes, then one can use the mean value of the test-based resistances in the determination of representative wall components in the analysis models.

Conclusions

The focus of this paper was to characterize the influence of non-structural gypsum panels, which provide a fire resistance rating from 1 to 2 hr, on the in-plane lateral performance of strap-braced cold-formed steel framed walls. The gypsum provides a substantial increase to the in-plane shear resistance of the walls. The capacity design of the shear wall test specimens (with holdowns) led to the desired behaviour: the fuse elements were able to maintain their strength in the inelastic range while the other structural members in the lateral load carrying path remained mainly elastic. The test results showed that attaching 15.9 mm-thick gypsum panels to a strap-braced wall could provide 15% (one layer of gypsum on both sides) to 53% (two layers of gypsum on both sides) additional strength. In the bearing wall test specimens, for which no capacity design calculations were implemented, the gypsum panels remained mainly undamaged, while the damage was mostly concentrated in the steel frame.

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