Integrated Solutions for Infrastructure Development
Edited by Lau, H. H., Tang, F. E., Ng, C. K., and Singh, A.

ISBN: 978-0-9960437-3-1

RACKING PERFORMANCE OF LIGHT WEIGHT CONCRETE FILLED COLD-FORMED STEEL PANELS

SHAHRZAD KASAEIAN[1], HAMID R. RONAGH[2], and REZA NIKOONEJAD[3]

[1]*School of Civil Engineering, University of Queensland*
[2]*Institute for Infrastructure Engineering, University of Western Sidney*
[3]*Civil Engineering, Azad University of Yazd*

The implementation of Cold-Formed Steel (CFS) as a structural element is almost new. As a light weight material, CFS members in the earthquake impose low inertia force to the structure. So greatly increased worldwide demand for such structures. Due to the thin-walled nature of CFS they are susceptible to buckling. Great efforts had been made to promote the lateral load resistance of CFS panels. The common bracing methods are not capable of economically resisting the high demands imposed on the system in highly seismic regions. In some instances, all panels are to be covered with Oriented Strand Board or steel sheeting in order to adequately address the anticipated earthquake load, and this renders the system too expensive. A promising method to combat this deficiency is to fill the cavity in between the panel studs with concrete. Results showed that a panel made in such a manner was able to resist the lateral loads three times more effectively than a similar configuration panel but with strap bracing. The strength, ductility and earthquake response factor of such a system were the major concerns. The experimental tests were performed on a 1.2 m × 2.4 m wall with three different configurations of studs and tracks.

Keywords: Thin-walled, Lateral load, Strength, Ductility, Earthquake response factor.

1 LITERATURE REVIEW

The construction of structures framed with CFS panels are becoming more common throughout the world. These panels are obtained through the assemblage of internal and external profiles (studs) with lipped channel cross-sections. The studs interconnect at each end by members (tracks) with un-lipped channel cross sections, in such a way as to realize a Cold-Formed Steel Frame Panel (CFSP). Panels with one- or two-sided sheathing have been widely used to construct the external and internal walls (CFSW) of buildings. For external walls, the panels support the structural load according to building regulations. For internal walls, some are load bearing, whereas others are only used for partitioning purposes and thus, are only required to offer secondary structural support. The main problem with these walls is their low brace capacity which hinders their use in medium to high seismic regions despite their advantageous light weight. Bracing capacities, provided by normal bracing methods such as steel straps or gypsum board cladding, are in the range of 2-7.4 kN/m (Vieira *et al.* 2012). For example, according to AISI S213-07, for a sheathed CFSP with ½” gypsum board on one side and screws placed 200 mm on the edges and 300 mm within the board, its nominal shear capacity under seismic loads is 230 lb/ft (3.35 kN/m) and once factored with the capacity reduction factor of 0.6, an ultimate capacity of 2 kN/m would be the result. However, while making it double-sided gives about 4 kN/m, it is still small. Although decreasing the screw spacing to 100 mm can create approximately twice this capacity, it is impractical and too labor intensive. With steel straps a similar problem exists; strap capacity is small and cannot be relied upon as it is often limited to the capacities of the end connecting screws. An alternative to these would be using an expensive Oriented Strand Board (OSB) that allows higher capacities (12 kN) to be achieved. However, transferring these forces through the bottom plate of the panel to the foundation is very much questionable and would often hinder the use of the theoretically available capacity (Fülöp and Dubina 2004, Landolfo *et al.* 2004).

Comparing the above lateral strength to the demand, a sharp contrast becomes evident. For a typical 300 m^2 two storey cold-formed house in a seismic region with a base acceleration of around 0.3 g, one would expect base shears of around 80 to 120 kN. Base shear could rise to 160 kN for a three storey. It is then difficult to control these lateral seismic forces through sheathed panels with low lateral capacities of 4 kN/m. One would need 40 m long panels to resist 160 kN. In many instances, this creates an architectural nightmare as openings must be cut short. Therefore, study a new structural wall is needed. As studs to evaluate the lateral load carrying capacity of concrete filled CFSP. As studs naturally form a good support for the formwork and placement of concrete, this research focuses on the study of the behavior of concrete filled CFS wall. The main behavior that was studied is the maximum lateral load bearing capacity of the specimens, their deformation behavior, the earthquake response factor (R factor) and different failure modes of system.

2 MATERIAL AND METHODS

The nine concrete filled cold-formed steel wall (CFCFSW) specimens, which are represented in three different configurations (A, B and C) shown in Table 1, were tested in the structures laboratory at University of Taft, using a testing frame designed specifically for in-plane shear loading. The cold-formed steel properties are provided in Table 2. All walls were 2400 × 1200 mm in size with light weight concrete (shown in Figure 1). The light weight concrete properties and mix design are tabulated in Table 3. All the members were connected using wafer head self-drilling/self-tapping screws. The screws shear and tensile strength are 3.3 kN and 3.8 kN, respectively.

Table 1. CFCFSW configuration.

Specimen type	**Corner studs (single)**	**Middle stud (single)**	**Noggin**	**Number**
A	√	-	-	3
B	√	√	-	3
C	√	√	√	3

Table 2. Cold-formed steel properties.

Grade (MPa)	**Thickness** (mm)	**E Modulus** (GPa)	$\mathbf{F_y}$ (MPa)	$\mathbf{F_u}$ (MPa)	$\boldsymbol{\varepsilon_y}$ (%)	$\boldsymbol{\varepsilon_u}$ (%)	$\mathbf{F_u/F_y}$
550	1	169	592	617	0.45	2.86	1.04

Table 3. Light weight concrete properties.

F'c (MPa)	Specific Weight (kN/m^3)	Aggregate	Cement	Slump (mm)
13	15	LECA	Portland	110

All the specimens were installed next to the side studs in the middle of the tracks on both sides of the wall between the upper (fixed) and lower loading rigid beams of the laboratory frame, using four M16 high strength bolts, which were located in frame before the pouring of the concrete. Measurements consisted of bottom wall displacement, acceleration of the loading beam assembly, the shear load at the top of the wall, as well as the uplift force in the hold down anchor rods. The LVDTs, strain gauges, load cells and accelerometer were connected to a computer and the Lab View Signal Express Software to obtain the load-displacement curve for each specimen.

Figure 1. Light weight (LECA) concrete-filled cold formed steel wall specimen in the test frame.

3 LOADING PROTOCOL

The cyclic loading regime that has been used in this study is based on Method B of ASTM-E2126 Standard (Table 4). The Method B test protocol is intended to produce data that sufficiently describes elastic and inelastic cyclic properties, and the typical failure mode that is expected in earthquake loading.

Table 4. Cyclic loading regime, Method B - ASTM E2126-07.

Pattern	Step	Minimum Number of Cycles	Amplitude, % Δ_m
	1	1	1.25
	2	1	2.5
1	3	1	5
	4	1	7.5
	5	1	10
	6	3	20
	7	3	40
2	8	3	60
	9	3	80
	10	3	100
	11	3	Additional increments of 20 (until wall failure)

*Where Δ_m is the ultimate displacement, monotonic which is the displacement corresponding to the failure limit state in a monotonic test.

4 RESULTS

4.1 Lateral Load Capacity

Light weight concrete is a brittle material, especially in tension, so naturally it cannot sustain large deformation like the CFS does. However, at small drifts a considerable amount of lateral resistance from the use of CFCFSW was found. As shown in Figure 2, the hysteretic response of the CFCFSW demonstrates pinching behavior with stiffness and strength degradation. The main failure mode in these specimens was the crack and rupture of concrete in connection with studs. The failure continued by stud-to-track connecting screw pull-out and rivet tilting and then caused some distortional buckling with wing rupture in the lower track. Thereafter, the lateral load was reduced and the frame lost its load bearing capacity. According to the experimental tests, it is crystal clear that middle stud and noggin not only do not play any important role in the lateral force capacity of CFCFSW but depending on the separation caused in the concrete which dropped the lateral force capacity and ductility of the wall. Overturning decreased the lateral load capacity in specimens Type A and Type B. The limit of the lateral strength associated with overturning depends on the presence of hold down anchor rods. In test specimens the rods are located in the bottom corners and in specimens Type B and C the lateral load transferred in two parts of the concrete separately, the tension in the middle of the track did not transfer properly and subsequently caused track distortion and concrete cracking and rupture.

To make the comparison between the proposed wall and conventional walls, in Table 5 the lateral load capacity, which AISI-S213 provides, is presented. The walls are a limited configuration of gypsum board sheathed and oriented strand board sheathed CFS panels. The mentioned shear walls can have openings to resist wind and seismic loads. The lateral load resistance of this wall can be reduced by multiplying in C_a (presented in Table 6).

Table 5. Nominal shear strength (Rn) for wind and seismic loads for shear walls faced with gypsum board or fiberboard. (kN/m) (United States and Mexico).

Assembly Description	Maximum Aspect ratio (h/w)	Fastener Spacing at panel Edges/Field (inches)						
		7/7	4/4	4/12	8/12	4/6	3/6	2/6
1/2" gypsum board on one side of wall; studs max. 24" o.c.	2:1	4.3	6.3	4.4	3.4	-	-	-
1/2" fiberboard on one side of wall; studs max. 24" o.c.	1:1	-	-	-	-	6.3	9.1	9.9

Table 6. Shear resistance adjustment factor (C_a).

	Maximum Opening Height Ratio				
	1/3	½	2/3	5/6	1
Percent Full-Height Sheathing	Shear Resistance Adjustment Factor				
10%	1	0.69	0.53	0.43	0.36
20%	1	0.71	0.56	0.45	0.38
30%	1	0.74	0.59	0.49	0.42
40%	1	0.77	0.63	0.53	0.45
50%	1	0.8	0.67	0.57	0.5
60%	1	0.83	0.71	0.63	0.56
70%	1	0.87	0.77	0.69	0.63
80%	1	0.91	0.83	0.77	0.71
90%	1	0.95	0.91	0.87	0.83
100%	1	1	1	1	1

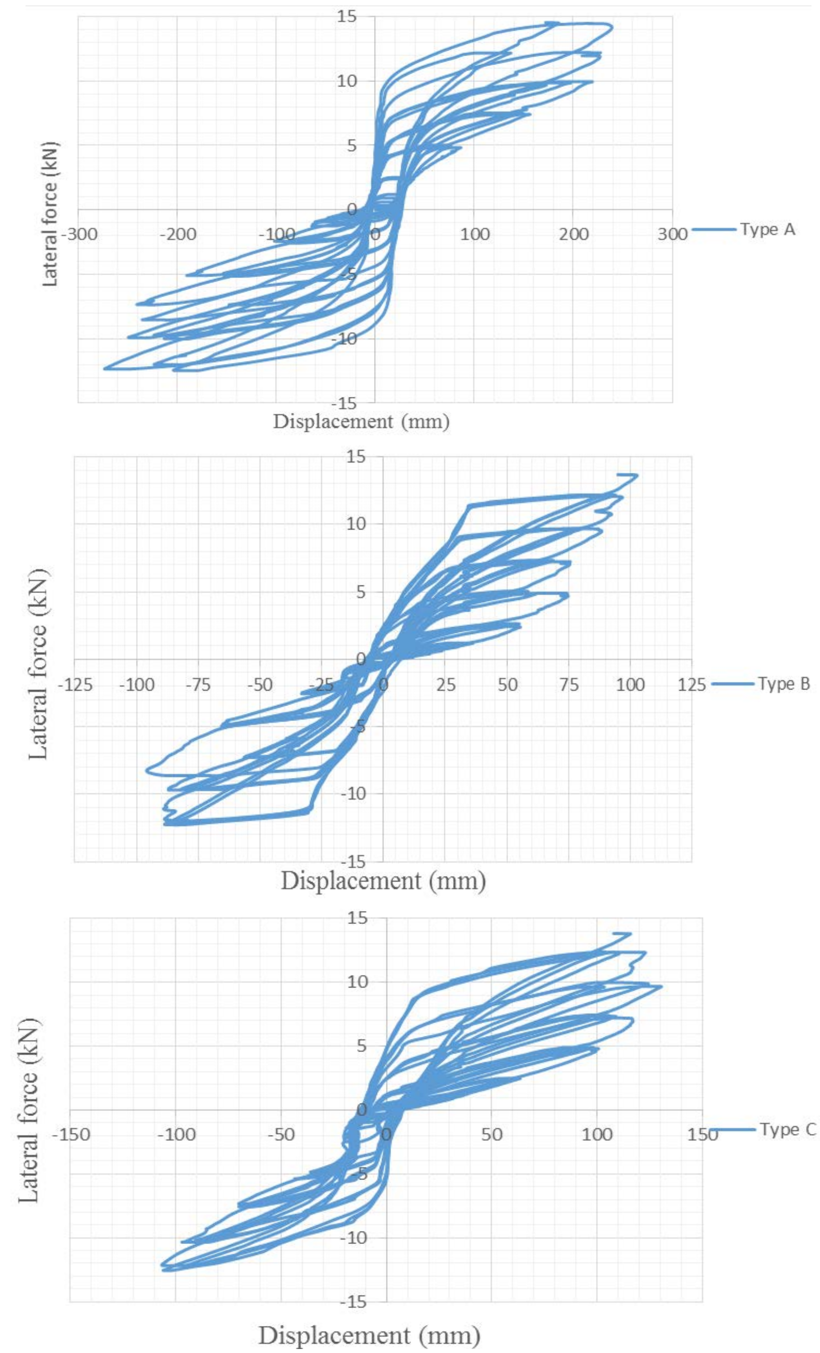

Figure 2. Hysteretic diagram of test specimens (Types A, B and C).

4.2 Earthquake Response Factor (R Factor)

To calculate the response factor, the hysteresis curve was utilized, using the FEMA 356 method. The results are shown in Table 7. Each provided result is the mean value of three specimens with the same configuration.

The achieved R factor illustrates the high ductility of CFCFSW in comparison with conventional CFS walls that are provided in AS/NZS4600 and the National Research Council of Canada.

Table 7. R factor.

Spec.	**R**
A	8.11
B	6.21
C	7.25

5 CONCLUSION

The two most important parameters that can be discovered from Figure 2 and Tables 4 and 5 are the higher lateral force capacity and ductility of CFCFSW in comparison with conventional CFS walls. The only configuration that has almost the same lateral capacity as CFCFSW is OSB sheathed CFSW. However, construction of a CFS wall filled with concrete is much easier than a CFS wall sheathed with OSB that has 2inch fastener spacing in field. On the other hand, some countries that are located in the desert do not have access to wood and the only way to achieve a desired lateral resistance is to apply other materials. Thus, for an effective construction of a light weight building that can resist earthquake and wind load properly, filling of cold formed steel panels with light weight concrete is suggested.

References

American Iron and Steel Institute, AISI-S213, North American Standard for Cold-Formed Steel Framing – Lateral Design 2007.

American Iron and Steel Institute, AISI S213-07, North American Standard for Cold-Formed Steel Framing – Lateral Design, Washington DC, USA, 2007.

AS/NZS4600, Cold-formed steel structures, Australian Building Codes Board, 2005.

ASTM-E2126-07, Standard test methods for cyclic (reversed) load test for shear resistance of walls for buildings, p. 13, USA, 2007.

FEMA-356, Prestandard and commentary for the seismic rehabilitation of buildings NEHRP, USA: Building Seismic Safety Council, 2000.

Fülöp, L. A. and Dubina, D., Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading, *Thin-Walled Structures*, 42, 339-349, 2004.

Landolfo, R., Corte, G. D., and Fiorino, L., Testing of sheathed cold-formed steel stud shear walls for seismic performance evaluation, *13th World Conference on Earthquake Engineering Vancouver*, BC, Canada. 2004.

National Research Council of Canada, National Building Code of Canada, 12th Edition, Ottawa, On, Canada, 2005a.

Vieira Jr., L. C. M. and Schafer, B.W., Lateral stiffness and strength of sheathing braced cold-formed steel stud walls, *Engineering Structures*, 37, 205-213, 2012.