12th Canadian Conference on Earthquake Engineering / 12ème Conférence Canadienne du Génie Parasismique
June 17-20, 2019 / 17-20 juin 2019
Château Frontenac, Québec, QC.
CCEE 2019

Cold-Formed Steel Framed Shear Wall Resistance Factors

Zhidong Zhang[1], C.A. Rogers[2], B.W. Schafer[3]

[1] Graduate Research Assistant, Dept. of Civil Engineering, Johns Hopkins University, Baltimore MD, USA.
[2] Associate Professor, Dept. of Civil Engineering, McGill University, Montreal, QC, Canada.
[3] Professor, Dept. of Civil Engineering, Johns Hopkins University, Baltimore MD, USA.

ABSTRACT

The objective of this paper is to re-assess the resistance factor for cold-formed steel-framed shear walls for both Canadian Limit States Design and American Load and Resistance Factor Design in light of a recently compiled database of cold-formed steel shear wall tests and consistent with current seismic practice established in the North American Standard for Seismic Design of Cold-Formed Steel Structural Systems (AISI S400-15). Overall, a target reliability consistent with LSD and LRFD must be selected. Then, on the capacity side establishing the resistance factor requires determining the statistics of the nominal strength prediction compared with the predicted strength (the professional factor), as well as the bias and variance with respect to material and fabrication factors. The recently compiled database of 700 monotonic and cyclic tests, spanning shear walls with wood structural panel sheathing, steel sheet sheathing, and strap bracing, form the basis for the re-evaluation of the professional factor. Nominal strength predictions differ in the U.S. and Canada even for the same test data, and not all tests include specimens that are valid per current design, so care must be taken in the comparisons. On the demand side establishing the reliability requires determining the bias and variance in the selected load case. Seismic demands have seen significant change in the U.S. ASCE 7 and in Canada's NBCC and traditional reliability calculations for resistance factors as codified, e.g., in the North American Specification for Cold-Formed Steel Structural Members (AISI S100-16) have limitations. Complications with establishing resistance factors for seismic load cases are noted, nonetheless a series of resistance factors are provided and compared with current provisions and recommendations are made for improvement and future work.

Keywords: cold-formed steel framed shear wall; test database; reliability analysis; expected strength; resistance factor.

INTRODUCTION

Buildings framed from cold-formed steel (CFS) typically rely on shear walls to provide lateral resistance against seismic demands. In both the U.S. and Canada the nominal strength of CFS-framed shear walls is provided in the North American Standard for Seismic Design of Cold-Formed Steel Structural Systems (AISI S400-15) [1] For CFS walls sheathed with wood structural panels (WSP) or steel sheet (SS) sheathing the nominal strength in AISI S400 is provided in tabular form – i.e., without an analytical strength approximation. Engineers must still ensure that the studs are adequate for all applicable load cases and size the studs, hold-downs, etc. using the country specific principles of capacity-based design as prescribed in AISI S400. For the seismic load case sizing of the shear wall uses either the Canadian Limit States Design (LSD) or American Load and Resistance Factor Design (LRFD) – either of which require that the available strength (ϕV_n) be greater than the load effect from the factored seismic demand in an appropriate load case (i.e. from ASCE 7 [2] or NBCC [3]). where ϕ is the resistance factor and V_n is the nominal shear wall strength.

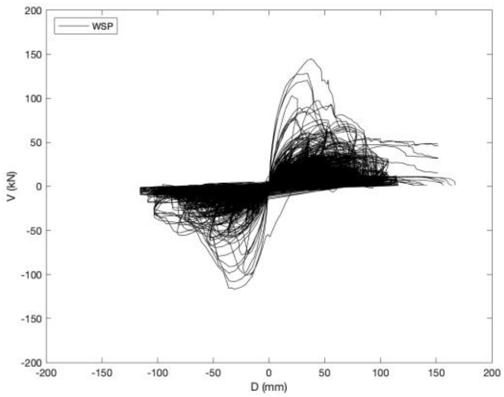
As described in the commentary to [1] the resistance factor, ϕ, employed in AISI S400 was selected for use in the U.S. when only a small number of WSP shear walls had been tested. At that time it was decided to use a factor consistent with CFS diaphragms – i.e. $\phi = 0.6$. This resistance factor has remained in place since the late 1990's despite 100's of additional shear wall tests being conducted. In the U.S. the nominal strength, V_n, is the mean of the peak tested strength from shear walls.

In Canada, the resistance factor was selected more recently, but embedded in the decision is a key difference between the nominal strength tabulated for the U.S. and Canada. The Canadian provisions employ an equivalent energy elastic plastic (EEEP) model for the shear walls. The EEEP model provides a consistent means to define the initial elastic stiffness and the nominal strength for the design model against the nonlinear tested response. However, the nominal strength in the EEEP model is less (potentially considerably less) than the peak tested strength. Using a target reliability, β, of 2.5 a $\phi = 0.7$ was derived for WSP and SS shear walls in Canada under wind load and extended to seismic design by engineering judgment [1].

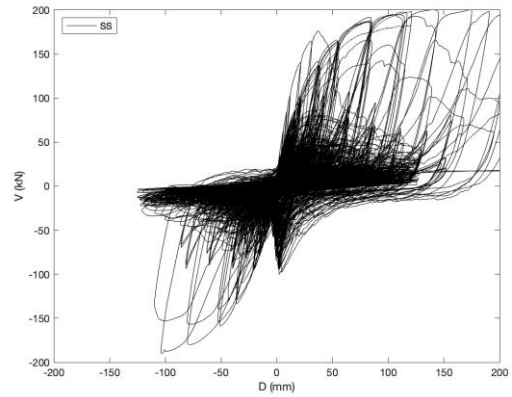
The objective of this work is to examine what current data and understanding suggests about the appropriate resistance factor for use in the U.S. and Canada in the seismic design of CFS shear walls. To achieve this goal a recently assembled database of CFS shear wall tests is examined to determine statistics on the test-to-predicted performance of the AISI S400 nominal strength tables. This is followed by examination of the resistance factor calculation for seismic design and calculation of a range of potentially appropriate resistance factors.

SHEAR WALL DATABASE

Recently, a database of CFS shear wall tests has been assembled. The database is introduced in [4] and has been expanded for this work to currently include 700 CFS shear wall tests assembled from 29 primary references [5]-[33]. The database includes all major features of the tested specimens and the complete load-displacement response of the walls. For example, the available load-displacement data for WSP and SS shear walls is provided in Figure 1.



(a)



(b)

Figure 1. Load-displacement response available in database for (a) WSP and (b) SS CFS shear walls

STATISTICS OF CURRENT SHEAR WALL PREDICTIONS

Using the shear wall database we determined the test-to-predicted (peak test strength / nominal strength in AISI S400) ratio for each entry matching the strength tables of AISI S400 for WSP and SS shear walls. For example, Table 1a, provides the tabulated nominal strength for WSP shear walls in the U.S. A single entry in the table implies a great deal of restrictions on a given shear wall, only those tested walls matching all criteria are included. For example in Table 1a,b,c, first row, first column, only 3 tests match all of the criteria – they have a mean test-to-predicted ratio of 1.44 and a coefficient of variation (CoV) of 0.08. See [1] for additional discussion on why the test-to-predicted ratio is not always 1.0 even though the basic philosophy of the U.S. nominal strength is to use the tested strength.

Wood Structural Panel (WSP) Sheathed Shear Walls

The test-to-predicted ratio statistics for WSP shear walls are provided in Table 1 for the U.S. and Table 2 for Canada. The mean test-to-predicted ratio for the U.S. is 1.14 with a CoV of 15%. There is systematic differences in the U.S. statistics between the accuracy for plywood and OSB sheathed shear walls that could be considered. The mean test-to-predicted ratio for Canada is 1.29 with a CoV of 14%. There is no systematic differences in the Canadian statistics between the accuracy for plywood (CSP or DFP) and OSB sheathed shear walls.

Steel Sheet (SS) Sheathed Shear Walls

The test-to-predicted ratio statistics for SS shear walls are provided in Table 3 for the U.S. and Table 4 for Canada. The mean test-to-predicted ratio for the U.S. is 1.16 with a CoV of 20%. The mean test-to-predicted ratio for Canada is 1.63 with a CoV of 19%. The relatively high Canadian test-to-predicted ratio is attributed to the use of EEEP for the nominal strength, and also to the specific manner in which the ratios are calculated herein. AISI S400 generally places a minimum on the stud and track thickness – in the tabulated comparisons data that meet or exceeded that limit are included. Thus, while the original tabulated strength number may have only considered 33 mil studs, the test-to-predicted ratio includes 33-mil or greater studs. In some cases tests have been performed with thicker studs subsequent to tabulation of the provided strength values.

Table 1. Test-to-predicted statistics for wood structural panel (WSP) shear walls in the U.S.

(a) nominal shear strength, lbf/ft, for wood structural panel shear walls (AISI S400-15)

Assembly	Max Aspect Ratio	Perim. screw spacing (in.)				Stud & Track (mils)	Screw
		6	4	3	2		
15/32 in. Structural 1 (4-ply)	2:1	780	990	-	-	33 or 43	8
	2:1	890	1330	1775	2190	43 or 54	8
						68	10
7/16 in. OSB	2:1	700	915	-	-	33	8
	2:1	825	1235	1545	2060	43 or 54	8
	2:1	940	1410	1760	2350	54	8
	2:1	1230	1850	2310	3080	68	10

(b) mean bias against peak strength: (peak test strength)/(nominal strength)

Assembly	Max Aspect Ratio	Perim. screw spacing (in.)				Stud & Track (mils)	Screw
		6	4	3	2		
15/32 in. Structural 1 (4-ply)	2:1	1.44	1.70	-	-	33 or 43	8
	2:1	1.23	1.26	1.30	1.22	43 or 54	8
						68	10
7/16 in. OSB	2:1	1.16	1.42	-	-	33	8
	2:1	1.06	0.96	1.06	1.22	43 or 54	8
	2:1	1.23	-	0.91	1.10	54	8
	2:1	-	-	-	1.06	68	10

(c) supplemental statistics (coeff. of variation of test/nominal, count)

Assembly	Max Aspect Ratio	Perim. screw spacing (in.)				Stud & Track (mils)	Screw
		6	4	3	2		
15/32 in. Structural 1 (4-ply)	2:1	(0.08,3)	(0.01,3)	-	-	33 or 43	8
	2:1	(0.08,3)	(0.01,3)	(0.03,9)	(0.04,2)	43 or 54	8
						68	10
7/16 in. OSB	2:1	(0.16,5)	(0.05,2)	-	-	33	8
	2:1	(0.21,8)	(0.01,3)	(0.06,8)	(0.06,4)	43 or 54	8
	2:1	(0.12,2)	-	(0.08,3)	(0.01,2)	54	8
	2:1	-	-	-	(0.05,2)	68	10

Ensemble statistics for WSP: mean test/nominal=1.14, COV=15%, n=62

Table 2. Test-to-predicted statistics for wood structural panel (WSP) shear walls in Canada

(a) nominal strength, kN/m, for wood structural panel shear walls (AISI S400-15)

Assembly	Max Aspect Ratio	Perim. screw spacing (mm)			Stud & Track (mils)	Screw
		150	100	75		
9.5 mm CSP	2:1	8.5	11.8	14.2	43 min	8
12.5 mm CSP	2:1	9.5	13	19.4	43 min	8
12.5mm DFP	2:1	11	17.2	22.1	43 min	8
9mm OSB	2:1	9.6	14.3	18.2	43 min	8
11mm OSB	2:1	9.9	14.6	18.5	43 min	8

(b) mean bias against peak strength: (peak test strength)/(nominal strength)

Assembly	Max Aspect Ratio	Perim. screw spacing (mm)			Stud & Track (mils)	Screw
		150	100	75		
9.5 mm CSP	2:1	1.24	1.31	1.35	43 min	8
12.5 mm CSP	2:1	1.33	1.29	1.40	43 min	8
12.5mm DFP	2:1	1.27	1.32	1.34	43 min	8
9mm OSB	2:1	1.21	1.21	1.18	43 min	8
11mm OSB	2:1	1.18	1.19	1.29	43 min	8

(c) supplemental statistics (coeff. of variation of test/nominal, count)

Assembly	Max Aspect Ratio	Perim. screw spacing (mm)			Stud & Track (mils)	Screw
		150	100	75		
9.5 mm CSP	2:1	(0.06,3)	(0.03,3)	(0.16,4)	43 min	8
12.5 mm CSP	2:1	(0.05,9)	(0.19,10)	(0.06,10)	43 min	8
12.5mm DFP	2:1	(0.08,3)	(0.01,3)	(0.03,7)	43 min	8
9mm OSB	2:1	(0.04,3)	(0.06,3)	(0.05,3)	43 min	8
11mm OSB	2:1	(0.34,9)	(0.01,3)	(0.06,8)	43 min	8

Ensemble statistics for WSP: mean test/nominal=1.29, COV=14%, n=87

Table 3. Test-to-predicted statistics for steel sheet (SS) sheathed shear walls in the U.S.

(a) nominal shear strength, lbf/ft, for steel sheet shear walls (AISI S400-15)

Steel Sheet	Max Aspect Ratio	Perim. screw spacing (in.)				Stud Blocking	Stud & Track (mils)	Screw
		6	4	3	2			
0.018 in.	2:1	390	-	-	-	No	33 (min)	8
0.027 in.	2:1	647	710	778	845	No	33 (min)	8
	2:1	-	1000	1085	1170	No	43 (min)	8
0.030 in.	2:1	910	1015	1040	1070	No	43 (min)	8
	2:1	-	-	-	1355	Yes	43 (min)	10
0.033 in.	2:1	1055	1170	1235	1305	No	43 (min)	8
	2:1	-	-	-	1505	Yes	43 (min)	10
	2:1	-	-	-	1870	No	54 (min)	8
	2:1	-	-	-	2085	Yes	54 (min)	10

(b) mean bias against peak strength: (peak test strength)/(nominal strength)

Steel Sheet	Max Aspect Ratio	Perim. screw spacing (in.)				Stud Blocking	Stud & Track (mils)	Screw
		6	4	3	2			
0.018 in.	2:1	1.25	-	-	-	No	33 (min)	8
0.027 in.	2:1	1.05	1.31	-	1.17	No	33 (min)	8
	2:1	-	1.06	-	1.28	No	43 (min)	8
0.030 in.	2:1	1.05	-	-	-	No	43 (min)	8
	2:1	-	-	-	1.03	Yes	43 (min)	10
0.033 in.	2:1	1.08	1.06	-	1.28	No	43 (min)	8
	2:1	-	-	-	1.13	Yes	43 (min)	10
	2:1	-	-	-	1.06	No	54 (min)	8
	2:1	-	-	-	1.01	Yes	54 (min)	10

(c) supplemental statistics (coeff. of variation of test/nominal, count)

Steel Sheet	Max Aspect Ratio	Perim. screw spacing (in.)				Stud Blocking	Stud & Track (mils)	Screw
		6	4	3	2			
0.018 in.	2:1	(0.17,7)	-	-	-	No	33 (min)	8
0.027 in.	2:1	(0.01,2)	(0.21,5)	-	(0.29,5)	No	33 (min)	8
	2:1	-	-	-	(N/A,1)	No	43 (min)	8
0.030 in.	2:1	(0.13,5)	(N/A,4)	-	-	No	43 (min)	8
	2:1	-	-	-	(0.01,2)	Yes	43 (min)	10
0.033 in.	2:1	(0.01,2)	(0.01,2)	-	(0.21,4)	No	43 (min)	8
	2:1	-	-	-	(0.15,8)	Yes	43 (min)	10
	2:1	-	-	-	(0.01,2)	No	54 (min)	8
	2:1	-	-	-	(0.07,2)	Yes	54 (min)	10

Ensemble statistics for SS: mean test/nominal=1.16, COV=20%, n=60

Table 4. Test-to-predicted statistics for steel sheet (SS) sheathed shear walls in Canada

(a) nominal shear strength, lbf/ft, for steel sheet shear walls (AISI S400-15)

Steel Sheet	Max Aspect Ratio	Perim. screw spacing (mm)				Stud Blocking	Stud & Track (mils)	Screw
		150	100	75	50			
0.46 mm	2:1	4.1	-	-	-	No	33 (min)	8
0.46 mm	2:1	4.5	6.0	6.8	7.5	No	43 (min)	8
0.68 mm	2:1	6.5	7.2	7.9	8.7	No	33 (min)	8
0.76 mm	4:1	8.9	10.6	11.6	12.5	No	43 (min)	8
0.84 mm	4:1	10.7	12	13	14	No	43 (min)	8
0.46 mm	2:1	7.4	9.7	11.6	13.5	Yes	43 (min)	8
0.76 mm	2:1	11.7	14.3	-	-	Yes	43 (min)	8
0.76 mm	2:1	-	-	19.9	23.3	Yes	54 (min)	8

(b) mean bias against peak strength: (peak test strength)/(nominal strength)

Steel Sheet	Max Aspect Ratio	Perim. screw spacing (mm)				Stud Blocking	Stud & Track (mils)	Screw
		150	100	75	50			
0.46 mm	2:1	1.74	-	-	-	No	33 (min)	8
0.46 mm	2:1	1.67	2.11	-	1.73	No	43 (min)	8
0.68 mm	2:1	1.53	1.86	-	1.80	No	33 (min)	8
0.76 mm	4:1	1.56	-	-	-	No	43 (min)	8
0.84 mm	4:1	1.58	1.55	-	1.73	No	43 (min)	8
0.46 mm	2:1	1.30	1.31	-	1.26	Yes	43 (min)	8
0.76 mm	2:1	1.45	1.51	-	-	Yes	43 (min)	8
0.76 mm	2:1	-	-	-	1.38	Yes	54 (min)	8

(c) supplemental statistics (coeff. of variation of test/nominal, count)

Steel Sheet	Max Aspect Ratio	Perim. screw spacing (mm)				Stud Blocking	Stud & Track (mils)	Screw
		150	100	75	50			
0.46 mm	2:1	(0.17,7)	(-,-)	(-,-)	(-,-)	No	33 (min)	8
0.46 mm	2:1	(0.24,3)	(-,1)	(-,-)	(0.27,3)	No	43 (min)	8
0.68 mm	2:1	(0.01,2)	(0.24,4)	(-,-)	(0.34,3)	No	33 (min)	8
0.76 mm	4:1	(0.11,7)	(-,6)	(-,-)	(-,9)	No	43 (min)	8
0.84 mm	4:1	(0.02,4)	(0.05,4)	(-,-)	(0.17,14)	No	43 (min)	8
0.46 mm	2:1	(-,1)	(-,1)	(-,-)	(-,1)	Yes	43 (min)	8
0.76 mm	2:1	(-,1)	(-,1)	(-,-)	(-,-)	Yes	43 (min)	8
0.76 mm	2:1	(-,-)	(-,-)	(-,-)	(-,1)	Yes	54 (min)	8

Ensemble statistics for SS: mean test/nominal=1.63, COV=19%, n=73

RESISTANCE FACTOR CALCULATION

For both LSD and LRFD design the capacity must be greater than the demand:

$$\phi R_n \geq c \sum \gamma_i Q_i \tag{1}$$

where ϕ is the resistance factor, R_n is the nominal strength, Q_i is load i, γ_i is load combination factor for load i, c converts the applied load to a load effect, and i is over all considered loads (dead, live, earthquake, etc.). The first-order reliability method implemented in [1] can be solved for the resistance factor via:

$$\phi = C_\phi M_m F_m P_m e^{-\beta\sqrt{V_M^2+V_F^2+V_P^2+V_Q^2}} \tag{2}$$

where C_ϕ is the load combination dependent effect, M_m and V_m are the mean and CoV of the material factor, F_m and V_F are the mean and COV of the fabrication factor, P_m and V_P are the mean and CoV of the professional factor – i.e. the test-to-predicted ratios, V_Q is the CoV of the load effect, and β is the target reliability. In Chapter K of [1] a method is provided for test-based determination of ϕ. In the Chapter K approach C_ϕ =1.52 (U.S.) and 1.42 (Canada) and V_Q =0.21. As detailed in [34] this implies one specific combination of dead and live load, and that gravity load is the controlling load combination. For other load combinations the load effect changes, although this is typically ignored [34]provides additional combinations of C_ϕ and V_Q that can be employed – for example in Western North America in a high seismic zone C_ϕ =1.33 and V_Q =0.38 while in lower seismic zones consistent with Eastern central North America C_ϕ =1.72 and V_Q =0.17. It is worth mentioning that the reliability of seismic load combinations is complicated by a number of factors – most notably the fact that the design considers complete system performance including nonlinearity and damage – see [35] for a thorough discussion of the challenges. Note, the U.S. recommended method for determining seismic reliability: FEMA P695 states that "resistance factors calibrated for use with common gravity load combinations are recommended for use". It is unclear if this is a rational conclusion for a component specifically designed to resist lateral load. To calculate ϕ here we assume M_m=1.0 and V_M=0 due to the fact that material variability is embedded directly in the test results (i.e. embedded in P_m and V_P) and F_m=1.0 and V_F=0.05 consistent with [1].

Under these assumptions and assuming an appropriate range of target reliabilities (LRFD member β target = 2.5 and connections target = 3.5, LSD targets are 3 and 4 respectively) the developed ϕ factors are provided in Table 5. The cases where the current resistance factor would not meet the target reliability are shaded in Table 5 – essentially only the uncertainty associated with high seismic demands is potentially problematic. If the recommendations of FEMA P695 and AISI S100 Chapter K are followed then the selected ϕ factors may be overly conservative.

Table 5. Calculated resistance factors as a function of target reliability for CFS sheathed shear walls.

Sheathing	**Cϕ, V_Q method**	**β=2.5**	**β=3.0**	**β=3.5**	**β=4.0**
WSP	AISI S100/CSA S136 Chapter K	0.90	0.79	0.69	0.61
U.S.	Ref. [34] Low Seismic	1.10	0.98	0.87	0.77
ϕ=0.6	Ref. [34] High Seismic	0.54	0.44	0.36	0.29
WSP	AISI S100/CSA S136 Chapter K	0.96	0.85	0.74	0.65
Canada	Ref. [34] Low Seismic	1.26	1.13	1.01	0.90
ϕ=0.7	Ref. [34] High Seismic	0.62	0.50	0.41	0.34
Steel Sheet	AISI S100/CSA S136 Chapter K	0.84	0.73	0.63	0.54
U.S.	Ref. [34] Low Seismic	1.02	0.90	0.78	0.69
ϕ=0.6	Ref. [34] High Seismic	0.52	0.42	0.34	0.27
Steel Sheet	AISI S100/CSA S136 Chapter K	1.13	0.98	0.85	0.73
Canada	Ref. [34] Low Seismic	1.46	1.29	1.13	0.99
ϕ=0.7	Ref. [34] High Seismic	0.74	0.60	0.49	0.39

shading indicates current phi factor would not meet this target reliability

CONCLUSIONS

The seismic design of cold-formed steel wood structural panel and steel sheet sheathed shear walls employ tabulated values for nominal strength based on tested shear walls. Though provided in the same standard the tabulated strengths for the U.S. and Canada employ different philosophies for determination of the nominal strength. A database of tested shear walls provides a means to assess the tested capacity vs. the predicted (tabulated) capacity and examine the mean bias and variation in the tabulated strengths. Structural reliability is generally assured through the use of appropriate resistance factors. The existing resistance factor for CFS shear walls in the U.S. and Canada is assessed against different target reliabilities and assumptions regarding the load effects. The analysis shows that existing resistance factors are adequate or conservative under traditional assumptions, but high seismic load cases may deserve further study.

ACKNOWLEDGMENTS

This material is partially based upon work supported by the U.S. National Science Foundation under Grant No. 1663348. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the author(s) and do not necessarily reflect the views of the U.S. National Science Foundation.

REFERENCES

[1] AISI S400 (2015) North American Std. for Seismic Design of Cold-Formed Steel Structural Systems. American Iron & Steel Institute.
[2] ASCE 7-16 (2016). *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. American Society of Civil Engineers. ASCE/SEI 7-16, ISBN (print): 9780784414248ISBN (PDF): 9780784479964
[3] NRCC (2015). National Building Code of Canada (NBCC). National Research Council Canada
[4] Ayhan, D., Baer, S., Zhang, Z., Rogers, C.A., Schafer, B.W. (2018). "Cold-Formed Steel Framed Shear Wall Database." Proc. of the Int'l Spec. Conf. on Cold-Formed Steel Structures. St. Louis, MO. 15pp.
[5] Boudreault, FA. (2005) "Seismic analysis of steel frame / wood panel shear walls." Master's Thesis, McGill Univ., June.
[6] Branston, A.E. (2004) "Development of a design methodology for steel frame / wood panel shear walls", Master's, McGill Univ., June.
[7] Chen, C.Y. (2004) "Testing and performance of steel frame / wood panel shear walls", Master's Thesis, McGill Univ., August.
[8] Rokas, D. (2006) "Testing and evaluation of light gauge steel frame / 9.5 mm CSP wood panel shear walls", Master's, McGill Univ.
[9] Blais, C. (2006) "Testing and analysis of light gauge steel frame / 9 mm OSB wood panel shear walls", Master's Thesis, McGill Univ.
[10] Hikita, K. (2006) "Combined gravity and lateral loading of light gauge steel frame / wood panel shear walls", Master's, McGill Univ.
[11] Al-Kharat, M., Rogers, C.A. (2005) "Testing of light gauge steel strap braced walls", Project Report No. 1, McGill University.
[12] Al-Kharat, M., Rogers, C.A. (2006) "Inelastic performance of screw connected light gauge steel strap braced walls", Project Report No. 2, McGill University, Montreal, Canada, December.
[13] Comeau, G. (2008) "Inelastic performance of welded cold-formed steel strap braced walls", Master's Thesis, McGill Univ., June.
[14] Velchev, K. (2008) "Inelastic performance of screw connected cold-formed steel strap braced walls", Master's Thesis, McGill Univ.
[15] Morello, D. (2009) "Seismic performance of multi-storey structures with cold-formed steel wood sheathed shear walls", Master's Thesis, McGill Univ., May.
[16] Balh, N., Rogers, C.A. (2010) "Development of seismic design provisions for steel sheathed shear walls", Project Report, McGill University, Montreal, Canada, January.
[17] Ong-Tone, C. (2009) "Tests and evaluation of cold-formed steel frame / steel sheathed shear walls", Master's Thesis, McGill Univ.
[18] DaBreo, J., (2012) "Impact of gravity loads on the lateral performance of cold-formed steel frame / steel sheathed shear walls", Master's Thesis, McGill Univ., September.
[19] Lu, S. (2015) "Influence of gypsum panels on the response of cold-formed steel framed shear walls", Master's Thesis, McGill Univ.
[20] Rizk, R. , Rogers, C.A. (2018) "HIGHER STRENGTH COLD-FORMED STEEL FRAMED / STEEL SHEAR WALLS FOR MID-RISE CONSTRUCTION" Project Report, McGill University, Montreal, Canada
[21] SANTOS, V. , Rogers, C.A. (2018) "HIGHER CAPACITY COLD-FORMED STEEL SHEATHED AND FRAMED SHEAR WALLS FOR MID-RISE BUILDINGS: PART 1" Project Report, McGill University, Montreal, Canada
[22] BRIERE, V. , Rogers, C.A. (2018) "HIGHER CAPACITY COLD-FORMED STEEL SHEATHED AND FRAMED SHEAR WALLS FOR MID-RISE BUILDINGS: PART 2" Project Report, McGill University, Montreal, Canada
[23] Serrette, R., Enchalada, J., Hall, G., Matchen, B., Nyugen, H., Williams, A. (1997) "Additional shear wall values for light weight steel framing", Report No. LGSRG-I-97, Department of Civil Engineering, Santa Clara University, Santa Clara, California, USA, March.
[24] Yu, C., Vora, H., Dainard, T., Tucker, J., Veetvkuri, P. (2007) "Steel sheeted options for cold-formed steel framed shear walls assemblies providing shear resistance", Report No. UNT-G76234, Dept. of Eng. Tech., Univ. of North Texas, Denton, Texas, USA.
[25] Yu, C., Chen, Y. (2009) "Steel sheeted options for cold-formed steel framed shear walls assemblies providing shear resistance - Phase 2", Report No. UNT-G70752, Department of Engineering Technology, University of North Texas, Denton, Texas, USA, October.
[26] Elhaji, N. (2005) "Cold-formed steel walls with fiberboard sheathing-shear wall testing", Summary Test Report, NAHB Research Center, Inc., Upper Malboro, Maryland, USA, September.
[27] Kochkin, V., Hill, R. (2006) "Cyclic testing of fiberboard shear walls with varying aspect ratios", Report, NAHB Research Center, Inc., Upper Malboro, Maryland, USA, March.
[28] Liu, P., Peterman, K.D., Schafer, B.W. (2012) "Test report on cold-formed steel shear walls", Research Report, CFS-NEES project: NSF-CMMI-1041578: NEESR-CR: Enabling Performance-Based Seismic Design of Multi-Story Cold-Formed Steel Structures, USA.
[29] Kelly A. Morgan and Mark A. Sorhouet, Reynaud L. Serrette (2002) "Performance of CFS Framed Shear Walls - Alternative Configurations", Research Report, Report No. LGSRG-06-02, Dept. of Civil Eng., Santa Clara University, Santa Clara, CA, USA.
[30] Hoang Nguyen, Georgi Hall and Reynaud Serrette(1996) "Shear Wall Values for Light Weight Steel Framing", Research Report, Report No. LGSRG-3-96, Department of Civil Engineering, Santa Clara University, Santa Clara, California, USA, March.
[31] CoLA-UCI, 2001. "Report of a Testing Program of Light-Framed Walls with Wood-Sheathed Shear Panels", Final Report to the City of Los Angeles Department of Building and Safety, Dept. of Civil and Env. Eng., University of California, Irvine, CA.
[32] Yu, C., Vora, H., Dainard, T., Tucker, J., and Veetvkuri, P., (2007). "Steel Sheet Sheathing Options for Cold-Formed Steel Framed Shear Wall Assemblies Providing Shear Resistance", Research Report RP07-3, American Iron and Steel Institute, Washington, D.C.
[33] Salenikovich and Dolan, 1999, Revised 2007. Salenikovich, A.J. and Dolan, J.D., "Monotonic and Cyclic Tests of Long Steel-Frame Shear Walls with Openings, Research Report RP99-2, American Iron and Steel Institute, Washington, D.C.
[34] Zeinoddini, V.M., Schafer, B.W. (2014). "Impact of Load Combinations on Structural Reliability Determined from Testing Cold-Formed Steel Components." *Structural Safety*, 48 25-32 (DOI: 10.1016/j.strusafe.2013.10.006)
[35] Ellingwood, B. R. 2001. "Earthquake Risk Assessment of Building Structures." Reliability Engineering and System Safety 74 (3): 251-262. doi:10.1016/S0951-8320(01)00105-3.