DETERMINATION OF THE LATERAL STRENGTH OF SHEAR WALL PANELS WITH COLD-FORMED STEEL FRAMES

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Abstract
In current construction practice, lateral strengths of shear wall panels with cold formed steel framing are primarily determined by tests owing to the lack of analytical methods. Meanwhile, the use of numerical methods such as the finite element method has been limited to researchers investigating the behaviour of SWP. Moreover, the finite element method has rarely been employed in design practice to determine the lateral strength of shear wall panels because the modelling is cumbersome. Presented in this paper is an analytical method to determine the ultimate lateral strength of shear wall panels. The method accounts for the factors that affect the behaviour and the strength of shear wall panels, such as material properties and thickness of sheathing, sizes of the C-shape steel studs, spacing of fasteners, and so on. Lateral strengths obtained from the proposed method for sheathing wall panels were compared with those of recent experimental investigations. The results of the comparison demonstrate that the predicted lateral strengths are in good agreement with those of the tests. Therefore, the proposed method is recommended for engineering practice.

Keywords: lateral strength, shear wall panels, cold formed steel framing.

1. Introduction

In the search for new constructive methods and materials for low- and mid-rise residential buildings, where the quality of living, ease of construction, and cost-efficiency could be improved, cold formed steel (CFS) has been an attractive alternative to traditional materials such as timber. In cold formed steel framing construction, the shear wall panel (SWP) is one of the primary lateral load resisting systems, which has been extensively used in seismic applications in North America.

Typically, SWP in cold formed steel wall framing are constructed with vertically spaced and aligned C-shape cold formed steel studs. The ends of the studs are connected to bottom and top tracks of the wall. Sheathing may be present on either one or both sides of the wall with screw fasteners. In practice, studs are generally designed to support vertical loads, while sheathing is considered to resist lateral loads. However, the lateral strength of SWP cannot be determined alone by the strength of the sheathing, as the interaction among the sheathing, the studs, and the fasteners affect both the behaviour and lateral strength of SWP considerably. Generally, a SWP may experience both in-plane gravity and lateral loads as well as out-of-plane wind loads in the case of exterior walls. This study is focused on the evaluation of the lateral strength of SWP, which is the primary function of SWP. The strengths of SWP to resist in-plane gravity and out-of-plane loading are not considered.

To determine the behaviour and lateral strength of SWP, Serrette et al (1997, 2002) and Rogers et al (2004a) have carried out extensive experimental investigations. Gad et al (1999) conducted the

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finite element analysis on SWP, in which the studs and tracks were modelled by beam elements while sheathing was modelled with shell elements and sheathing-to-framing connections were modelled by nonlinear springs. Fulop and Dubina (2004b) conducted a series of tests and proposed a simplified model for determining the lateral strength of SWP based on replacing the sheathing with a pair of equivalent cross-bracing. A tri-linear force displacement relationship calibrated with test results was proposed for using the finite element method of analyzing SWP.

In practice, the lateral strengths of different SWP are available in the AISI design guideline (Brockenbrough, 1998) and standard (AISI, 2004). The values of the nominal lateral strengths of SWP presented in a tabulated form in the Lateral Design Standard of AISI (2004) are convenient to use and primarily determined on the basis of experimental tests, which provide an acceptable degree of confidence to the practitioners. However, as limited by the number of the tests being carried out, the freedom of selecting different sheathing materials, stud sizes and configurations of SWP is restricted as the tabulated values may not be applied or extended to SWP with different materials, configurations and construction details. Therefore, a reliable analytical method for determining the lateral strength of SWP is of importance to promoting cold formed steel framing technology.

Presented in this paper is an analytical method to determine the ultimate lateral strength of shear wall panels. This method accounts for the aspects that affect the behaviour and strengths of SWP associated with dimensions of the panel, material and cross-section properties of both sheathing and steel studs, and as well as the construction details such as the spacing of screw fasteners. The ultimate lateral strengths of different SWP are evaluated with the proposed method and the comparisons are made between the analytical results and the results obtained from recent experimental investigations (Serrette et al, 2002; Rogers et al 2004a; Fulop and Dubina 2004a).

2. Evaluation of the Ultimate Lateral Strength of SWP

2.1. Lateral Strength of SWP

The lateral strength of SWP is primarily contributed by the sheathing and framing studs and can expressed as,

$$P_R = P_{Sf} + P_F \qquad \text{Eq. (1)}$$

where P_{Sf} and P_F are lateral strengths associated with the sheathing and framing studs, respectively. In the case that the sheathing are provided in both sides of SWP, the lateral strength of the sheathing is given by

$$P_{Sf} = P_{Sf,1} + P_{Sf,2} \qquad \text{Eq. (2)}$$

where $P_{Sf,1}$ and $P_{Sf,2}$ are the lateral strength of sheathing presented on side 1 and 2 of the panel, respectively. In addition to the material and cross-section properties of sheathing, the ultimate lateral strength of sheathing are also highly affected by the characteristics and arrangement of sheathing-to-framing connections, which will be discussed in Section 2.2. The lateral strength of framing studs, P_F, can be determined as

$$P_F = K_F \Delta \qquad \text{Eq. (3)}$$

where K_F is the lateral stiffness of the framing studs and Δ is the lateral deflection the SWP impending the failure. Compared to that of the sheathing, the framing studs contribute little to the ultimate lateral strength of SWP, as the lateral stiffness of the studs is insignificant. Therefore, for the reason of simplicity, the elastic lateral stiffness of the framing studs is adopted as

$$K_F = \sum_{studs} \frac{3E_F I_F}{h^3} \qquad \text{Eq. (4)}$$

where E_F and I_F are the Young's modulus and the moment of inertia of the framing studs, respectively. h is the height of the panel.

Considering the compatibility of lateral deformation between sheathing and framing studs prior to the failure of the panel, the relationship between the sheathing strength and the lateral deformation of the panel is,

$$\Delta = \frac{P_{Sf}}{K_{Sf}} \qquad \text{Eq. (5)}$$

where K_{Sf} is the lateral stiffness of sheathing. Substituting Eq. (5) into Eq. (3) yields,

$$P_F = \frac{K_F}{K_{Sf}} P_{Sf} \qquad \text{Eq. (6)}$$

substituting Eq. (6) into Eq.(1), the lateral strength of SWP is

$$P_R = \left(1 + \frac{K_F}{K_{Sf}}\right) P_{Sf} \qquad \text{Eq. (7)}$$

2.2. *Lateral Stiffness and Strength of Sheathing*

To obtain the ultimate lateral strength of SWP as shown in Eq. (7), it remains to evaluate the lateral stiffness, K_{sf} and strength of sheathing, P_{sf}. Experimental investigations have shown that the predominant mode of failure in SWP is initiated at the sheathing-to-framing connections for the most common sheathing materials such as plywood, OSB, and gypsum board, etc. That is, the degradation of both lateral stiffness and strength of such panels are primarily due to the failure of the connections (Serrette et al, 2002; Rogers et al 2004b). Therefore, the failure of the connections has to be accounted for while evaluating both the lateral stiffness and strength of sheathing. To that end, the degraded sheathing stiffness may be calculated as

$$K_{Sf} = \frac{G_S A_C}{1.2h} + \frac{3E_S I_S}{h^3} \qquad \text{Eq. (8)}$$

where E_S and G_S are the Young's and shear modulus of sheathing, respectively; h is the height of the panel; and A_C is the reduced cross sectional area of the sheathing, defined as

$$A_C = t_S d_C n_C \qquad \text{Eq. (9)}$$

in which t_s is the thickness of the sheathing; d_C is the diameter of the screws, n_C is the number of screws along the cross section of the sheathing that is connected to the top collector member, and I_S is the moment of inertia of the reduced cross-section and is given by

$$I_S = n_C \frac{t_S d_C^3}{12} + 2t_S d_C \sum_{i=1}^{n_C/2} (i \cdot s_C)^2 \qquad \text{Eq. (10)}$$

where s_C is the screw spacing at the edge of the panel.

Considering the analogy between SWP and the eccentrically loaded bolted connection, in both cases the loads are applied eccentrically and the strength reductions are as a result of the failures of the connections or fasteners initiated at locations which are far from the centre of rotation. In this research, the inelastic method of evaluating strength of the eccentrically loaded bolted connection proposed by Brandt (1982) is employed and extended to evaluate the ultimate lateral strength of sheathing. Brandt's method involved an iterative process of locating the inelastic instantaneous center of rotation of the bolt group as shown in Figure 1; the ultimate strength of the connection is found when all of the forces (both internal and external) on the connection are in equilibrium. Extended from Brandt's method, the ultimate lateral strength of sheathing, P_{Sfi} (i=1, 2) can be evaluated as

$$P_{Sf,k} = C_u^{(n_x)} V_r \qquad (k = 1, 2) \qquad \text{Eq. (11)}$$

where n_x is the index of the last iteration, and V_r is the strength of a single sheathing-to-framing connection which is determined by the minimum value of the bearing resistance of the sheathing material, the shear resistance of the fastener, and the bearing resistance of the steel stud. The parameter $C_u^{(n_x)}$ is the ultimate strength reduction coefficient for the group of sheathing-to-framing connections and can be evaluated through the following procedure.

The coordinates of the inelastic instantaneous center shown in Figure 1 are given by

$$x_c = x_o + a_x \qquad y_c = y_o + a_y \qquad \text{Eq. (12)}$$

Where x_o and y_o are the coordinates of the elastic center, and a_x and a_y are the x and y components associated with the distance between the inelastic instantaneous center and elastic center of the bolt group and are given by

$$a_x = \frac{P_y J}{n_f M_o} \qquad a_y = -\frac{P_x J}{n_f M_o} \qquad \text{Eq. (13)}$$

where

$$J = \sum_{i=1}^{n_f} \left(x_{s\,i}^2 + y_{s\,i}^2 \right) \qquad \text{Eq. (14)}$$

$$M_o = -P_x e_{yo} + P_y e_{xo} \qquad \text{Eq. (15)}$$

in which: P_x and P_y are components in the x and y directions of the normalized external force, unitary force, P as shown in Figure 1; J is the polar moment of inertia of all the fasteners with respect to the elastic center (x_o, y_o); x_s and y_s are the coordinates of fastener i; n_f is the total number of fasteners within the panel; M_o is the moment produced by the components of the unitary force; and e_{xo} and e_{yo} are the load eccentricities with respect to the elastic center. It is assumed that the farthest fastener from the inelastic instantaneous center has the largest deformation (Brandt, 1982), for wood sheathing, which is taken as 10mm (0.39in) according to tests carried out by Okasha (2004). The deformation of fastener i (Δ_i) is linearly proportional to the distance (d_i) between the fastener and the inelastic instantaneous center as shown in Eq.(20). Furthermore, the normalized force of the fastener is a nonlinear function of the deformation of fasteners (Eq. 21).

Initially, the components of the distance between the inelastic instantaneous center and fastener i for the first iteration (denoted by the superscript in parentheses) can be obtained as

$$d_{xi}^{(1)} = x_{s\,i} - x_c \qquad d_{y_i}^{(1)} = y_{s\,i} - y_c \qquad \text{Eq. (16)}$$

by setting the coordinates of the elastic center to be zero (x_c=0, y_c=0) and substituting Eq. (12) into Eq. (16), we have

$$d_{xi}^{(1)} = x_{s\,i} - a_x \qquad d_{y_i}^{(1)} = y_{s\,i} - a_y \qquad \text{Eq. (17)}$$

Thus, the distance between the instantaneous center and fastener i is

$$d_i^{(1)} = \sqrt{d_{x\,i}^{2\,(1)} + d_{y\,i}^{2\,(1)}} \qquad \text{Eq. (18)}$$

The initial load eccentricities with respect to the inelastic instantaneous center are

$$e_{x_i}^{(1)} = e_{xo\,i} - a_x \qquad e_{y_i}^{(1)} = e_{yo\,i} - a_y \qquad \text{Eq. (19)}$$

The iterative process for determining the inelastic instantaneous center and ultimate shear strength reduction coefficient C_u is described in the following. For the j-th iteration,

the normalized deformation of fastener i, $$\Delta_i^{(j)} = 0.39 d_i^{(j)} / d_{\max}^{(j)} \qquad \text{Eq. (20)}$$

where d_{max} is the distance between the inelastic instantaneous center to the farthest fastener.

The normalized force of fastener i, $$\left(R_i^{(j)} / R_u^{(j)}\right) = \left(1 - e^{-10\Delta_i^{(j)}}\right)^{0.55} \qquad \text{Eq. (21)}$$

Moment of the fasteners normalized forces, $$M^{(j)} = \sum_{i=1}^{n_f} \left(R_i^{(j)} / R_u^{(j)}\right) d_i^{(j)} \qquad \text{Eq. (22)}$$

Moment of the eccentric unitary force, $$M_p^{(j)} = -P_x e_y^{(j)} + P_y e_x^{(j)} \qquad \text{Eq. (23)}$$

The normalized ultimate fastener force, $$R_u^{(j)} = \frac{-M_p^{(j)}}{M^{(j)}} \qquad \text{Eq. (24)}$$

The normalized force components of fastener i,

$$R_{xi}^{(j)} = \left(-d_{y_i}^{(j)} / d_i^{(j)}\right)\left(R_i^{(j)} / R_u^{(j)}\right) R_u^{(j)} \qquad \text{Eq. (25)}$$

$$R_{y_i}^{(j)} = \left(d_{xi}^{(j)} / d_i^{(j)}\right)\left(R_i^{(j)} / R_u^{(j)}\right) R_u^{(j)} \qquad \text{Eq. (26)}$$

The normalized force of fastener i, $$R_i^{(j)} = \sqrt{\left(R_{xi}^{(j)}\right)^2 + \left(R_{y_i}^{(j)}\right)^2} \qquad \text{Eq. (27)}$$

Components of the unbalanced forces, $$F_x^{(j)} = P_x + \sum_{i=1}^{n_f} R_{xi}^{(j)} \qquad \text{Eq. (28)}$$

$$F_y^{(j)} = P_y + \sum_{i=1}^{n_f} R_{y_i}^{(j)} \qquad \text{Eq. (29)}$$

Displacement of the inelastic instantaneous center to its new position,

$$\delta_{ax}^{(j)} = \left(-F_y^{(j)} / n_f\right)\left(J / M_o\right) \qquad \text{Eq. (30)}$$

$$\delta_{ay}^{(j)} = \left(F_x^{(j)} / n_f\right)\left(J / M_o\right) \qquad \text{Eq. (31)}$$

Updated normalized load eccentricities for the next iteration,

$$e_{xi}^{(j+1)} = e_{xi}^{(j)} - \delta_{ax}^{(j)} \qquad \text{Eq. (32)}$$

$$e_{y_i}^{(j+1)} = e_{y_i}^{(j)} - \delta_{ay}^{(j)} \qquad \text{Eq. (33)}$$

Updated distance between the inelastic instantaneous center and fastener i,

$$d_{xi}^{(j+1)} = d_{xi}^{(j)} - \delta_{ax}^{(j)} \qquad \text{Eq. (34)}$$

$$d_{y_i}^{(j+1)} = d_{y_i}^{(j)} - \delta_{ay}^{(j)} \qquad \text{Eq. (35)}$$

$$d_i^{(j+1)} = \sqrt{\left(d_{xi}^{(j+1)}\right)^2 + \left(d_{y_i}^{(j+1)}\right)^2} \qquad \text{Eq. (36)}$$

The ultimate strength reduction coefficient, $$C_u^{(j)} = \left|\frac{M^{(j)}}{M_p^{(j)}}\right| \qquad \text{Eq. (37)}$$

Repeat above procedure until, $$\frac{C_u^{(j)} - C_u^{(j-1)}}{C_u^{(j)}} < \varepsilon \qquad \text{Eq. (38)}$$

where ε is a pre-assigned tolerance for convergence. The iterative process is terminated when the coefficient C_u will be invariant in further iterations, which indicates the equilibrium conditions are satisfied with respect to the updated location of inelastic instantaneous centre. As stated by Brandt (1982), and also found by this study, only a few iterations are required to obtain the ultimate strength reduction coefficient.

Having computed the ultimate strength reduction coefficient, the strength of the sheathing can be calculated based on Eq. (11) and the ultimate lateral strength of SWP can then be determined in accordance with Eq. (7).

3. Results comparison between analytical and experimental investigations

Experimental results (Serrette et al, 2002; Rogers et al 2004a; Fulop and Dubina 2004a) are used to validate the accuracy of the proposed analytical method of evaluating the ultimate lateral strength of SWP. The accuracy of the evaluated strengths are generally correlated with the material properties and the geometric dimensions of the components. As not all properties are reported in the foregoing literature, the material properties adopted in the evaluation may not be matched with those of the tested materials. In this study, both the material and geometric properties of steel studs were based on the values published by the Steel Stud Manufacturers Association (SSMA, 2001), and the material properties of sheathing being used in the calculation will be discussed in each individual case.

Rogers et al (2004) conducted a series of experimental investigations on SWP with three different sheathing materials, Oriented Strand Board (OSB), Douglas Fir Plywood (DFP), and Canadian Softwood Plywood (CSP). The cold formed steel studs were 92S41-1.12mm (358S158-44mils), spaced 610mm (24in) at the center, and double studs were placed at the chords. The sheathing was fastened with No. 8 screws (diameter = 4.06mm) on one-side of the panel. Screw spacing was 305mm (12in) in the field, and the edge spacing varied from 152mm (6in) to 76mm (3in). The length and height of the SWP were 1219mm (4ft) and 2438mm (8ft), respectively. The ultimate lateral strengths shown in Table 1 are the average values obtained from three specimens.

The following material properties are used to evaluate the ultimate lateral strengths of the foregoing SWP. The shear modulus of elasticity for OSB, DFP and CSP are 925MPa, 825MPa, and 497MPa, respectively (Okasha, 2004), while the modulus of elasticity associated with OSB (OSB, 1995), DFP and CSP (CANPLY, 2003) are 9917MPa, 10445MPa, and 7376MPa, respectively. The comparison between the analytical and test results is presented in Table 1.

Table 2 shows the comparison between the results of the proposed method and those of the experimental investigation conducted by Serrette et al (2002). The framing steel studs used in the two tests were 89S41 (350S158) with thicknesses of 1.37mm (54 mils) and 1.73mm (68 mils). The studs were spaced 610mm (24in) at the center, and double studs were placed at the chords. The sheathing material was OSB, and sheathing was presented on one side of the panel. The screw spacings on the edge and in the field of the panel were 51mm (2in) and 305mm (12in), respectively. The SWP dimensions were 1219mm (4ft) by 2438mm (8ft). The ultimate lateral strengths shown in Table 2 are the average values obtained from two specimens. The foregoing material properties associated with OSB sheathing are used to evaluate the strength of the SWP.

Presented in Table 3 is the comparison between the result of the proposed method and that of the experimental investigation conducted by Fulop and Dubina (2004a). The framing steel studs were 152S44-1.57mm (600S175-62mils) with 610 mm (24in) spacing. OSB sheathing was presented

Table 1. Comparison between analytical and tested results (Rogers et al, 2004a)

Shear Wall Panel description	Edge screw spacing (mm)	Lateral Strength (N/m)		Test / Predicted
		Test	Predicted	
OSB sheathing: 11 mm Steel stud 92S41-1.12mm Screw size: No. 8 Field screw spacing: 305mm	152	13257	12423	1.07
	102	19293	18181	1.06
	76	23550	23911	0.98
DFP sheathing: 12.5mm Steel stud 92S41-1.12mm Screw size: No. 8 Field screw spacing: 305mm	152	16010	15636	1.02
	101	23792	22920	1.04
	76	29721	30171	0.99
CSP sheathing: 12.5mm Stud 92S41-1.12mm Screw size: No. 8 Field screw spacing: 305mm	152	12752	12736	1.00
	102	16596	18521	0.90
	76	24880	24266	1.03

Table 2. Comparison between analytical and tested results (Serrette et al, 2002)

Shear Wall Panel description	Screw size (mm)	Stud Thickness (mm)	Lateral Strength (N/m)		Test / Predicted
			Test	Predicted	
OSB sheathing: 11mm Steel stud: 89S41 Screw spacing (mm) Edge: 51; Field: 305	No.8	1.37	34383	35373	0.97
	No.10	1.73	44964	42225	1.07

Table 3. Comparison between analytical and tested results (Fulop and Dubina, 2004a)

Shear Wall Panel description	Screw Size	Lateral Strength (N/m)		Test / Predicted
		Test	Predicted	
OSB sheathing: 10mm Steel stud: 152S44-1.57mm Screw spacing (mm) Edge: 102; Field: 305	d=4.8mm	21882	20120	1.09

on one side of the panel. The screw diameter was 4.6mm and the screw spacings were 102mm (4in) on the edge and 254mm (10in) in the field of the panel. Different from the foregoing two experimental investigations, the dimensions of the panel were 3600mm (≈12ft) by 2440mm (8ft). The ultimate lateral strengths shown in Table 3 are obtained from one specimen. As the material properties of sheathing were not available from the literature, the foregoing properties of OSB are employed in the analytical evaluation.

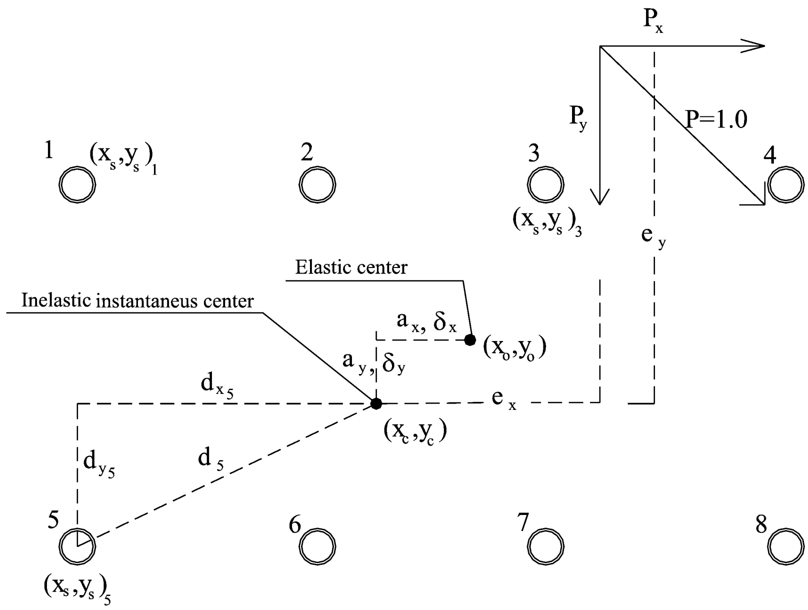

Figure 1. Fastener arrangement notation

4. Conclusions

The utilization of shear wall panels constructed with cold-formed steel and wood sheathing is becoming common practice for low- and mid-rise residential construction. However, analytical methods of evaluating the ultimate lateral strengths of the panels are needed to be developed in order to make cold formed steel systems more attractive to design practitioners. The method presented in this paper is comprehensive and can be used to evaluate the ultimate lateral strengths of SWP with different sheathing and framing materials, as well as panel dimensions and construction details such as fastener spacing. The comparisons made on the results obtained from the proposed method and the experimental investigations have shown good agreement between the evaluated and tested results. Therefore, the proposed method is recommended for engineering practice.

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References

AISI (2004). Standard for Cold-Formed Steel Framing-Lateral Design, *American Iron and Steel Institute*.

Brandt, G. D. (1982). Rapid Determination of Ultimate Strength of Eccentrically Loaded Bolt Groups, *Engineering Journal*, American Institute of Steel Construction.

Brockenbrough, R. L. & Associates (1998). Shear Wall Design Guide, *American Iron and Steel Institute,* Technical Data, RG-9804.

CANPLY (2003). Plywood Design Fundamentals, *Canadian Plywood Association.*

Fulop, L., & Dubina, D. (2004a). Performance of wall-stud cold-formed shear panels under monotic and cyclic loading Part I: Experimental research. *Thin-Walled Structures, 42*, 321-338.

Fulop, L., & Dubina, D. (2004b). Performance of wall-stud cold-formed shear panels under monotic and cyclic loading Part II: Numerical modelling and performance analysis. *Thin-Walled Structures, 42,* 339-349.

Gad, E. F., Chandler, A. M., Duffield, C.F., & Stark, G. (1999). Lateral Behaviour of Plasterboard-Clad residential Steel Frames. *Journal of Structural Engineering*, *ASCE*, January, 32-39.

Okasha, A. F. (2004). Performance of steel frame / wood sheathing screw connections subjected to monotonic and cyclic loading, *Department of Civil Engineering and Applied Mechanics*, *University of McGill*, Montreal, Master's Thesis.

OSB (1995). OSB Design Manual, Design Rated Oriented Strand Board, Structural Board Association.

Rogers, C. A., Branston, A. E., Boudreault, F. A., & Chen, C. Y. (2004a). Light gauge steel frame / wood panel shear wall test data: summer 2003, *Department of Civil Engineering and Applied Mechanics*, *University of McGill*, Montreal, Progress Report.

Rogers, C. A., Branston, A. E., Boudreault, F. A., & Chen, C. Y. (2004b). Steel Frame / Wood Panel Shear Walls: Preliminary Design Information for Use with the 2005 NBCC, *13th Word Conference on Earthquake Engineering*.

Serrette, R. L, Escalada, J., & Juadines, M. (1997). Static Racking Behavior of Plywood, OSB, gypsum, and Fiberbond Walls with Metal Framing, *Journal of Structural Engineering*, *ASCE*, August 1997, 1079-1086.

Serrette, R. L., Morgan, K. A., & Sorhouet, M. A. (2002). Performance of Cold-Formed Steel-Framed Shear Walls: Alternative Configurations. *Light Gauge Steel Research Group, Department of Civil Engineering, Santa Clara University*, Final Report LGSRG-06-02.

SSMA (2001). Product technical information. Steel Stud Manufacturers Association. ICBO ER-4943P, Retrieved January, 12, 2004, http://www.ssma.com