A Simplified Method of Evaluating Lateral Strengths of Shear Wall Panels with Cold Formed Steel Framing

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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. Martinez and Xu (2006) presented an analytical method to determine the lateral strength of the shear wall panel based on the analogy of designing eccentrically-loaded steel-bolted moment connections. The method takes into account the factors that affect the behaviour and the strength of the shear wall panel, such as material properties, geometrical dimensions and construction details. However, since an iterative process was adopted to determine the instantaneous center of panel rotation, the associated computational effort may hinder the efficiency of the method. A simplified method is proposed herein with no need of the iterative process. Lateral strengths of different sheathing wall panels obtained from the proposed method were compared with recent experimental investigations. The comparisons demonstrate that the results obtained by the proposed simplified method are in good agreement with those of the tests; therefore, the method is recommended for engineering practice.

Introduction

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Shear wall panels, as the one of the primary structural components in building lateral load resisting systems, have been extensively used in cold formed steel framing of low- and mid-rise residential construction, particularly in seismic applications. In practice, cold formed steel studs are generally designed to support vertical loads, while the sheathing is considered to resist lateral loads. However, the lateral strength of the shear wall panel cannot be determined alone by the strength of the sheathing because of the complexity of the interaction among the sheathing, the studs, and the fasteners. Thus, predicting the lateral strength of shear wall panels presents a great challenge for structural engineers.

Martinez and Xu (2006) presented an analytical method to determine the ultimate lateral strength of the shear wall panel based on the analogy of designing eccentrically-loaded steel-bolted moment connections. The method takes into account the factors that affect the behaviour and strength of shear wall panels, such as material properties, thickness and geometry of sheathing and studs, and construction details such as size and spacing of the sheathing-to-stud fasteners. However, since an iterative process was adopted to determine the so-called instantaneous centre of panel rotation, the associated computational effort with iterative process may hinder the efficiency of the method. It is proposed herein to eliminate the iterative process from the foregoing method so the lateral strengths of CFS shear wall panels can be determined in a simpler fashion. In addition, revisions have made to the evaluation process to improve the efficiency of the simplified method. Consequently, the computational effort is greatly reduced without much affecting the accuracy of the results. The effectiveness of the proposed simplification is validated by the comparison of test results of different shear wall panels from different investigators (AISI 2004, Rogers *et al* 2004, and Serrette *el at* 2002) to the lateral strengths obtained from the proposed method.

In this study, only in-plane lateral loading and the behaviour of SWP are considered. The failure of a shear wall panel subjected to in-plane lateral loading at the ultimate strength state occurs when the panel has no further strength to resist lateral loads. According to the tests that have been carried out, the predominant failure mode of the shear wall panel is the failure of the sheathing (Rogers *et al* 2004, Serrette *et al* 2002, Fulop and Dubina 2004). It is observed that the failure is often initiated at sheathing-to-framing connections for the most common sheathing materials such as plywood, oriented strand board, and gypsum wall board. The failure of the sheathing is evident due to rupture of the sheathing-to-framing connections, and in some cases the sheathing could be separated completely from the frame, as observed in the tests. However, in the case that the thickness of steel studs are relatively thin (e.g., thickness ≤ 33 mils [0.84mm]), the failure of a shear wall panel may be initiated by the buckling of the studs even though the studs are braced by the sheathing. The failure of steel studs can also occur when sheathing is applied on the both sides of the frame. Applying the sheathing on both sides of the frame or doubling the sheathing thickness enhances the panel lateral strength which amplifies the compressive force in the studs and may result in stud failure in compression prior to the sheathing failure.

The lateral strength of shear wall panels associated with sheathing failure

The lateral strength of shear wall panels which is contributed by the assembly of sheathing and steel framing studs can be expressed as

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

where P_S is the lateral strength associated with sheathing. In the case that sheathing is provided on both sides of a shear wall panel, its lateral strength is given by

$$P_S = \sum_{i=1}^{2} P_{S,i} \qquad \text{Eq. (2)}$$

where $P_{S,i}$ (i = 1, 2) are the lateral strengths of the sheathing presented on side 1 and 2 of the panel, respectively. The lateral strength contribution associated with steel framing studs, P_F, can be determined as

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

where K_F is the lateral stiffness associated with the framing studs, and Δ is the lateral deflection of the sheathing impending the failure at the ultimate lateral load level. Compared to the sheathing, the framing studs contribute little to the ultimate lateral strength of SWP, because 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, and 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

$$\Delta = \frac{P_S}{K_S} \qquad \text{Eq. (5)}$$

Substituting Eq. (5) into Eq. (3) yields

$$P_F = \frac{K_F}{K_S} P_S \qquad \text{Eq. (6)}$$

and substituting Eq. (6) into Eq.(1), the lateral strength of the shear wall panel is given by

$$P_R = \left(1 + \frac{K_F}{K_S}\right) P_S \qquad \text{Eq. (7)}$$

where K_S is the effective sheathing stiffness that can be calculated as

$$K_S = \frac{G_S A_S}{1.2h} \alpha_V + \frac{3 E_S I_S}{h^3} \alpha_B \qquad \text{Eq. (8)}$$

where E_S and G_S are the Young's and shear modulus of the sheathing, respectively; h is the height of the shear wall panel; α_V and α_B are stiffness reduction coefficients for shear and bending deformation, respectively; A_S and I_S are the cross sectional area and moment of inertia of the sheathing, defined as

$$A_S = t_S l \,, \qquad I_S = \frac{t_S l^3}{12} \qquad \text{Eq. (9a, b)}$$

in which t_S is the sheathing thickness, and l is the width of SWP.

As the lateral strength of the shear wall panel is computed at its imminent state of failure, at this point the lateral stiffness of the shear wall panel is substantially less than its initial elastic stiffness. In addition to the inelastic behaviour, the degradation of the lateral stiffness primarily contributed to the failure of the sheathing-to-framing connections as evidenced by the experimental tests (Rogers *et al* 2004, Serrette *et al* 2002). The sheathing stiffness reduction coefficients, α_V and α_B, introduced herein to account for effects the connection failure as functions of number of effective screws and screw spacing, are calibrated based on the test results (Rogers *et al* 2004) as follows:

$$\alpha_V = \left(\frac{C_u}{3.3 \cdot n_C}\right)^{1.8} \cdot \left(\frac{6in}{s_C}\right), \qquad \alpha_B = \left(\frac{6}{C_u}\right)^2 \cdot \left(\frac{6in}{s_C}\right)^{\frac{1.3 \cdot n_C}{C_u}} \qquad \text{Eq. (10a,b)}$$

where s_C is the edge screw spacing in inches; n_C is the total number of screws used to fasten sheathing to steel framing; C_u is the ultimate strength coefficient representing the number of effective screws of the shear wall panel at imminent state of failure and to be determined in the next section.

Lateral strength of sheathing

Considering the analogy between the shear wall panel and the eccentrically loaded bolted steel connection, in both cases the loads are applied eccentrically, and the strength reduction is primarily result of the failures of the connections or fasteners initiated at locations which are far from the centre of rotation. In this study, 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 centre 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_{Si} (i=1, 2) is evaluated as

$$P_{S,i} = C_u V_r \, \eta \;; \qquad (i = 1, 2) \qquad \text{Eq. (11)}$$

where V_r is the strength of a single sheathing-to-framing connection that 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. C_u is the ultimate strength reduction coefficient associated with the eccentrically applied load. η is the strength modification factor accounting for the variation of the height-to-width ratio of the shear wall panel,

$$\eta = \sqrt{8.0 - \frac{h}{l}} - 1.45 \geq 0 \qquad \text{Eq. (12)}$$

The evaluation of C_u involves the determination of the so-called instantaneous centre of rotation of the fastener group as shown in Figure 1. It is understood that the iterative process is introduced to achieve the moment equilibrium with respect to the instantaneous center of rotation between the moments associated with the applied force and the resistant forces of the fasteners (Brandt 1982, Martinez and Xu 2006). For moment associated with the fastener forces, it is evaluated based on individual fastener force and its distance to the instantaneous centre. The fastener force is associated with the deformation of the connection which is linearly proportional to its distance to the instantaneous centre of rotation as proposed by Brandt (1982) based on test results. Instead of evaluating the force for each fastener which involves the iterative process of updating the location of the instantaneous centre of rotation, the simplified method proposed herein adopted a constant force for all fasteners. Thus, the moment associated with the fastener forces can be evaluated without iterations.

The value of the constant force, 0.93 percent of the sheathing-to-stud connection strength is obtained from the calibration of the results with the iterative process.

Simplified procedure to calculate the ultimate strength reduction coefficient

For the eccentrically loaded fastener group shown in Figure 1, the components of the distance from fastener *i* to the elastic centre of the fastener group are

$$d_{xi} = x_{Ci}\,; \qquad d_{y_i}^{(0)} = y_{Ci} \qquad \text{Eq. (13 a, b)}$$

where x_{Ci}, y_{Ci} are the coordinates of the fasteners with respect to the elastic centre. The simplified method of evaluating the strength reduction coefficient C_u is described as follows:

Step 1. Compute the polar moment of inertia of the fastener group with respect to the elastic centre of rotation and the moment associated with the applied unitary force P_x,

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

$$M_0 = P_x e_y^{(0)} \qquad \text{Eq. (15)}$$

Step 2. Calculate the distance between the instantaneous centre of rotation and the elastic centre of rotation, and evaluate the eccentricity of the applied unitary force with respect to the instantaneous centre of rotation,

$$\delta_y = \left(P_x / n_C\right)\left(J / M_o\right) \qquad \text{Eq. (16)}$$

$$e_y = e_y^{(0)} + \delta_y \qquad \text{Eq. (17)}$$

Evaluate the moment the associated with the applied unitary force,

$$M_p = P_x e_y \qquad \text{Eq. (18)}$$

Step 3. Compute the distance between each fastener and the instantaneous centre of rotation,

$$d_{yi} = y_{Ci} + \delta_y \qquad \text{Eq. (19)}$$

$$d_i = \sqrt{\left(x_{Ci}\right)^2 + \left(d_{y_i}\right)^2} \qquad \text{Eq. (20)}$$

Calculate the moment the associated with the fasteners,

$$M = 0.93\sum_{i=1}^{n_C} d_i \qquad \text{Eq. (21)}$$

Step 4. Compute the strength reduction coefficient,

$$C_u = \left| \frac{M}{M_p} \right| \qquad \text{Eq. (22)}$$

The results obtained from the foregoing simplified procedure are compared with that of using the iterative procedure (Martinez and Xu 2006) for 34 shear wall panels listed in Table 1 to 4. It is found that the maximum difference between the two procedures is less than 2.5%.

The lateral strength of shear wall panels associated with frame failure

In resisting the applied lateral load, end framing studs of a shear wall panel experience either tension or compression against the overturning of the panel as shown in Figure 2, while the studs between the end ones carry much less load. Thus, the failure of steel framing studs of shear wall panels is primarily associated with the failure of the end stud in compression. The lateral strength of shear wall panels associated with frame failure can be obtained as

$$P_{fc} = \frac{l}{h} P_n \qquad \text{Eq. (23)}$$

where *l* and *h* are the length and height of the shear wall panel, respectively. P_n is the nominal compressive strength of the end stud evaluated in accordance with Chapter D of the North American Specification for the Design of Cold-formed Steel Structural Members (S136-01, 2001). Recommended by Telue and Mahendran (2001) through their experimental investigation, the effective length factors associated with the end stud can be K_x=0.75, K_y=K_t=0.10 and K_x=0.75, K_y=0.10, and K_t=0.20 for end studs with sheathing presented on both and one sides of SWP, respectively. In the case that the lateral strength of a shear wall panel governed by failure of the end stud, P_{fc} is less than the value of P_R computed by Eq. (1), then the lateral strength of the shear wall panel is

$$P_R = P_{fc} \qquad \text{Eq. (24)}$$

Results comparison between analytical and experimental investigations.

Experimental results (Rogers *et al* 2004, and Serrette *el at* 2002) and published values of the shear wall panels (AISI 2004) are used to validate the accuracy of the proposed simplified method of evaluating the ultimate lateral strength of shear wall panels. As not all properties are reported in the foregoing literature, the material properties adopted in the evaluation may not be identical as those the tested materials. In this study, the geometric gross properties of the steel studs were computed based on the cross-section dimensions reported in each literature. For the material properties of steel being used in the calculations, unless it is specified in the individual case, the yield strength and Young's modulus are 33 ksi (230 MPa) and 29500 ksi (203000 MPa), respectively. For sheathing material the following material properties are used in the evaluation, shear modulus of elasticity for Oriented Strand Board (OSB), Douglas Fir Plywood (DFP) and the Canadian Softwood Plywood (CSP) are 134 ksi (925 MPa), 120 ksi (825 MPa), and 72 ksi (497 MPa), respectively (Okasha, 2004), while the modulus of elasticity associated with OSB (OSB, 1995), DFP and CSP (CANPLY, 2003) are 1438 ksi (9917 MPa), 1515 ksi (10445 MPa) and 1070 ksi (7376 MPa), respectively.

Shown in Table 1 and 2 are the comparisons of the lateral strengths of shear wall panels predicted based on the simplified method and test results reported by Rogers et al (2004). The three different sheathing materials investigated are OSB, DFP, and CSP with thicknesses of 7/16 in. (11 mm), 1/2 in. (12.5 mm) and 1/2 in. (12.5 mm) respectively. The C-shape cold formed steel studs were 362S162-44mils (92S41-1.12mm), spaced 24 in. (610 mm) in the centre, and double C-shape back-to-back studs were placed at the ends of the panel. The sheathing was attached on one side of the panel using No. 8 screws at every 12 in. (305 mm) in the field. Three edge screw spacing, 3 in. (76 mm), 4 in. (102 mm) and 6 in. (152 mm), were investigated for shear wall panels with height of 8 ft. (2438 mm) and length of 4 ft. (1219 mm) as shown in Table 1 and for the SWP with length of 8 ft. (2438 mm) in Table 2. Two edge screw spacing, 4 in. (102 mm) and 6 in. (152 mm), were tested for shear wall panels with height of 8 ft. (2438 mm) but length of 2 ft. (609 mm) as shown in Table 2. The maximum difference between predicted and tested results shown in Table 1 and 2 is 10% and 15%, respectively.

Table 3 shows the comparison between the results of the predicted and those tested by Serrette *et al* (2002). The shear wall panel dimensions were 4 ft. (1219 mm) by 8 ft. (2438 mm). OSB sheathing was fastened on one side or both sides of the panel using No. 8 or No. 10 screws. The screw spacing was 2 in. (51 mm) on the edge and 12 in. (305 mm) in the field of the sheathing. The framing steel studs investigated in the two tests were 350S162 (89S41 mm) with thicknesses of 54 mils (1.37mm) and 68 mils (1.73 mm), and yielding strength of 59 ksi (407 MPa) and 56 ksi (386 MP*a*), respectively. The studs were spaced at 24 in. (610 mm) on center, and double studs were placed at the ends of the shear wall panels. The ultimate lateral strengths of the tests shown in Table 3 are the average values obtained from two specimens, tested under reversed cyclic loading protocol. As shown in Table 3, the predicted results are in excellent agreement with the test results.

Table 4 presents the comparison on the predicted lateral strengths of shear wall panels with OSB sheathing to that are published in the *Standard for Cold-Formed Steel Framing-Lateral Design* (AISI 2004). The length and height of the panels are 4 ft. (1219 mm) and 8 ft. (2438 mm), respectively. The C-shape steel stud designation is 350S162, and the four different steel thicknesses that are listed in the standard are 33 mils (0.838 mm), 43 mils (1.092 mm), 54 mils (1.372 mm), and 68 mils (1.727 mm). Double studs are used for the end studs. OSB sheathing was attached on one side of the panel using No. 8 or No. 10 screws at every 12 in (305 mm) in the field. Four edge screw spacing are 2 in. (51 mm), 3 in. (76 mm), 4 in. (102 mm) and 6 in. (152 mm). Table 4 shows a good correlation between predicted and test results. However, compared to the results presented in Tables 1 and 2, a larger value of standard deviation is observed which may result from the difference of OSB material properties between the tested and that used for the calculation.

Conclusions

In current practice, the lateral strengths of shear wall panels with cold formed steel framing stipulated in AISI standard (AISI 2004) are determined primarily from experimental tests. As only a limited number of configurations of the shear walls panels have been tested, practitioners are restricted in their design to those that are available in the design standard. Certainly, an analytical method of evaluating the lateral strength of shear wall panels is in urgent need for practitioners. The simplified method presented in this paper is practical and comprehensive and can be used to evaluate the lateral strength of shear wall panels with different sheathing and framing materials, panel dimensions, and construction details such as fastener spacing. The comparisons made on the results obtained from the proposed method and the experimental tests carried out by different investigators have shown good agreement between the evaluated and tested results. In addition, the simplified method has significantly less computational effort than the iterative one (Martinez and Xu 2006). For the 34 experimental tests listed in Tables 1 to 4, it is found that the maximum difference of the predicted lateral strength of SWP between using the iterative and simplified procedure is less than 2.5%. Therefore, the proposed simplified method is recommended for engineering practice.

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Appendix – References

AISI. Standard for Cold-Formed Steel Framing-Lateral Design. American Iron and Steel Institute, Washington, DC; 2004.

AISI. Shear Wall Design Guide. American Iron and Steel Institute, Technical Data RG-9804, Washington, DC; 1998.

Brandt, G. D. Rapid Determination of Ultimate Strength of Eccentrically Loaded Bolt Groups. Engineering Journal, American Institute of Steel Construction; Second quarter 1982.

CANPLY. Plywood Design Fundamentals. Canadian Plywood Association, Vancouver, BC; 2003.

Fulop, L., Dubina, D. Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading Part I: Experimental research. Thin-Walled Structures, 2004; 42:321-338.

Okasha, A. F. Performance of steel frame / wood sheathing screw connections subjected to monotonic and cyclic loading. University of McGill, MSc Thesis; 2004.

Martinez J., Xu L. Determination of the Lateral Strength of Shear Wall Panels with Cold Formed Steel Frames. Proceeding of the International Conference on Advances in Engineering Structures, Mechanics & Construction, Waterloo, ON, Canada; May 2006.

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

Rogers, C. A., Branston, A. E., Boudreault, F. A., Chen, C. Y. Steel Frame / Wood Panel Shear Walls: Preliminary Design Information for Use with the 2005 NBCC. In: Proceedings of the 13th Word Conference on Earthquake Engineering, Vancouver, BC; 2004.

S136-01. North American Specification for the Design of Cold-formed Steel Structural Members. Canadian Standards Association, S136-01; September 2002.

Serrette, R. L., Morgan, K. A., Sorhouet, M. A. Performance of Cold-Formed Steel-Framed Shear Walls: Alternative Configurations. Santa Clara University, Final Report LGSRG-06-02; 2002.

Telue, Y.K., Mahendran, M. Behaviour of cold-formed steel wall frames lined with plasterboard. Journal of Constructional Steel Research, 2001; 57:435-452.

Appendix – Notation

Δ	Shear wall panel lateral deformation
α_V, α_B	Sheathing stiffness reduction coefficients for shear and bending
δ_y	Distance between the elastic centre and the instantaneous centre of rotation
η	Shear wall panel strength modification factor
A_S	Sheathing cross sectional area
C_u	Ultimate strength coefficient
d_i	Distance from the screw *i* to the elastic centre of rotation
d_{xi}, d_{yi}	x and y components of the distance between screw *i* and the elastic centre of rotation
E_F	Steel framing studs Young's modulus
E_S	Sheathing Young's modulus
e_y	y component of the distance from the load to the elastic centre of rotation
G_S	Sheathing shear modulus of elasticity
h	Shear wall panel height
i	Number of the screw in consideration
I_F	Moment of inertia of steel stud
I_S	Moment of inertia of sheathing
J	Polar moment of inertia of the fastener group
K_F	Lateral stiffness associated with steel framing studs
K_S	Lateral stiffness associated with sheathing
K_t	Effective length factor of steel stud for torsion
$K_{x,y}$	Effective length factor of steel stud for x and y axes, respectively.
l	Shear wall panel length
M	Moment strength of the fasteners group
M_o	Moment associated with the unitary force about the elastic centre of rotation
M_p	Moment produced by the unitary force about the instantaneous centre of rotation
n_C	Total number of screws on the panel, used to attach the sheathing

P_F	Steel framing studs lateral strength
P_{fc}	Shear wall panel lateral strength due to failure of the end stud
P_n	Nominal compressive strength of the end stud
P_R	Shear wall panel lateral strength
P_S	Sheathing lateral strength
P_x	Lateral unitary force applied in the location of the actual force
s_C	Screw spacing on the edge of the panel
t_S	Sheathing thickness
V_r	Strength of a single sheathing-to-framing connection
x_{Ci}, y_{Ci}	Coordinates of the *i* screw with respect the elastic centre of rotation

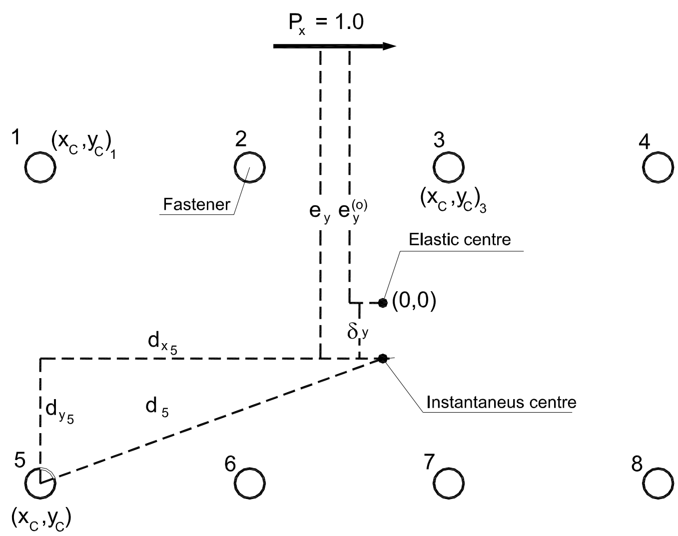

Figure 1 Fastener arrangement notation

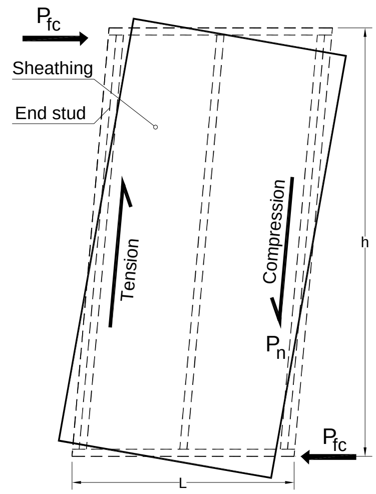

Figure 2 Panel rotation and force distribution

Table 1. Comparison between predicted and tested results (Rogers, 2004)

Assembly description	Sheathing material and thickness in	Edge screw spacing In	Lateral strength, plf: Test Rogers (2004)	Lateral strength, plf: Predicted Sheath-ing	Lateral strength, plf: Predicted Stud	Ratio Pred[1]/Test
Stud: 362S162-44	OSB 7/16" one side	6	904	839		0.93
Screw size: No. 8		4	1322	1234		0.93
Field screw spacing: 12"		3	1610	1628		1.01
Wall $l \times h$: 2'×8'	DFP 1/2" one side	6	1096	1057		0.96
Sheathing $l \times h$: 4'×8'		4	1631	1557	2000	0.95
		3	2035	2056		1.01
	CSP 1/2" One side	6	870	854		0.98
		4	1137	1252		1.10
		3	1720	1648		0.96
					Average	0.98
					Standard deviation	0.05

[1] "Pred" is the smaller predicted strength based on sheathing and stud failures.

Table 2. Comparison between predicted and tested results (Rogers, 2004)

Assembly description	Sheathing material and thickness in	Edge screw spacing in	Lateral strength, plf			Ratio $\frac{\text{Pred}}{\text{Test}}$
Stud: 362S162-44 Screw size: No. 8 Field screw spacing: 12”			Test Rogers (2004)	Predicted		
				Sheath-ing	Stud	
Wall *l* × *h*: 2’×8’ Sheathing *l* × *h*: 2’×8’ one side	OSB 7/6	6	857	760		0.89
		4	1261	1074		0.85
	CSP 1/2	3	836	788		0.94
		3	1233	1103	2000	0.89
Wall *l* × *h*: 8’×8’ Sheathing *l* × *h*: (2) 4’×8’ one side	CSP 1/2	6	932	906		0.97
		4	1405	1282		0.91
		3	1802	1658		0.92
					Average	0.91
					Standard deviation	0.04

Table 3. Comparison between predicted and tested results (Serrette, 2002)

Assembly description Wall *l* × *h*: 8’×8’ Sheathing *l* × *h*: (2) 8’×8’	Screw size	Stud thick. mils (mm)	Lateral strength, plf			Ratio $\frac{\text{Pred}}{\text{Test}}$
			Test Serrette (2002)	Predicted		
				Sheath-ing	Stud	
OSB 7/16” sheathing **one side** Stud: 362S162 Screw spacing (in) Edge: 2; Field: 12	No. 8	54 (1.37)	2356	2397	3963	1.02
	No.10	68 (1.73)	3081	2851	5085	0.93
OSB 7/16” sheathing **two sides** Stud: 362S162 Screw spacing (in) Edge: 2; Field: 12	8	54 (1.37)	4177	4763	4145	0.99
	10	68 (1.73)	5244	5659	5303	1.01
					Average	0.99
					Standard deviation	0.05

Table 4. Comparison between analytical and tested results (AISI, 2004)

Assembly description	Screw size	Stud thickness mils	Edge screw spacing in	Lateral strength, plf			Ratio Pred / AISI (2004)
				AISI (2004)	Predicted		
					Sheath-ing	Stud	
Sheathing: OSB 7/16” one side Stud: 350S162 Wall $l \times h$: 4’×8’ Sheathing $l \times h$: 4’×8’	No. 8	33	6	700	830	1451	1.19
			4	915	1225		1.34
		43	6	825	839	2047	1.02
			4	1235	1235		1.00
			3	1545	1629		1.05
			2	2060	2416		1.17
		54	6	940	848	2713	0.90
			4	1410	1245		0.88
			3	1760	1640		0.93
			2	2350	2428		1.03
	No.1	68	6	1232	1020	3584	0.83
			4	1848	1491		0.81
			3	2310	1960		0.85
			2	3080	2896		0.94
Average							1.00
Standard deviation							0.15