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BRACING STIFFNESS AND STRENGTH IN SHEATHED COLD-FORMED STEEL STUD WALLS

Luiz C. M. Vieira Jr.* and Benjamin W. Schafer*

* Johns Hopkins University
e-mail: luizvieirajr@gmail.com and schafer@jhu.edu

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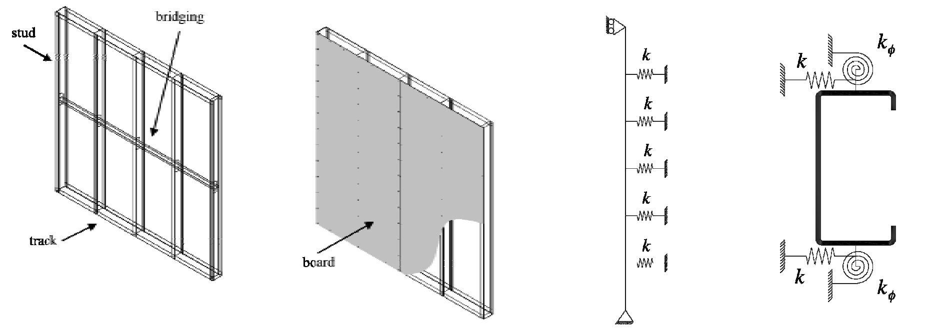
Abstract. *The objective of this paper is to provide the stiffness and strength characteristics for cold-formed steel stud walls stabilized by sheathing. The primary stability resistance is provided by translational (lateral) stiffness supplied at fastener locations along the cold-formed steel studs. This translational stiffness has two parts: local and diaphragm. To analyze the local stiffness, an experimental study consisting of small-scale stud-fastener-sheathing assemblies is conducted. Results are sensitive to the design variables (stud, fastener, sheathing, spacing) as well as the environmental and construction conditions. A simple analytical model for the translational stiffness supplied by the sheathing diaphragm action is proposed and validated. The importance of including both local and diaphragm stiffness is illustrated with a test on a full-scale cold-formed steel stud wall. This paper provides a comprehensive bracing model for sheathing-braced stud walls that includes both local and diaphragm stiffnesses.*

1 INTRODUCTION

This work is part of a larger investigation on walls framed from cold-formed steel studs, Figure 1. A potentially efficient solution to the bracing of the studs in a wall is to utilize the attached sheathing (Figure 1b), as opposed to bridging and blocking (Figure 1a). The sheathing is generally composed of OSB, plywood and/or gypsum board. The connection between the stud and the sheathing is made at regular intervals by fasteners, typically self-drilling screws. The fastener-sheathing system braces the cold-formed steel studs; as such the stiffness and strength of this system is of interest for characterizing its role as a brace.

2 BACKGROUND

Since at least the 1940s, the additional resistance supplied to a cold-formed steel stud due to its connection to sheathing has intrigued researchers and designers. The connection stiffness is typically divided into two parts: rotational and translational (Figure 1d). The rotational stiffness (k_ϕ) is engaged when the stud flange tries to rotate against the sheathing, causing an axial force in the fastener and a compression force at the stud flange edge in contact with the sheathing (creating a moment couple resisting further rotation) [1]. The translational stiffness (k, Figure 1c,d), which is the focus of this paper, is engaged when the stud flange tries to translate (shear) relative to the sheathing resulting primarily in bearing resistance at the fastener location supported by diaphragm resistance of the sheathing as a whole.



a) Bare wall with bridging
b) Sheated wall, no bridging
c) Spring-column model
d) Springs applied to cross-section

Figure 1 – Stud bracing, practice and models

In 1947 George Winter, with his students and colleagues, was the first to formalize the increase of stud capacity due to its connection to sheathing in cold-formed steel studs [2]. In 1962 the AISI Specification [3] incorporated the design method of [2] which Winter expanded in [4]. The approach focused on flexural buckling of the studs, and a companion experimental method for determining the lateral (translational) stiffness (k) of the connector and sheathing. The approach is known as Winter's method, or the local method, and the testing reported herein is motivated strongly by this early work.

In 1976 an alternative to Winter's method was developed by Simaan and Peköz that considered the sheathing as a shear diaphragm both analytically and in terms of the proposed testing [5]. This method was ultimately adopted by the AISI Specification in 1980, and then later abandoned by the AISI Specification in 2004. As reported in [6] comparisons between Winter's local test results and Simaan and Peköz's more involved diaphragm tests are possible, and the differences in stiffness are found to be relatively small. This is explained by the fact that the major contributor to the diaphragm stiffness is local deformations at the fasteners, well captured in Winter's simple shear test.

Recently their has been renewed interest in Winter's method and the related testing; including tests on gypsum board sheathing [7] and cyclic loading with a variety of sheathing conditions [8] and [9]. The bulk of available data has been compiled in [6]. Recent tests, conducted by the authors and provided in a test report [10] are, in part, the subject of this paper.

3 LOCAL MODEL FOR TRANSLATIONAL STIFFNESS AND STRENGTH

3.1 Test setup

Winter's method for determining the translational stiffness of a stud-sheathing assembly employs a simple symmetrical shear test, as illustrated in Figure 2. Two sections of studs are connected by identical sheathing on both sides and then pulled laterally (perpendicular to the long direction of the studs), such that shear must develop in each of the fasteners to resolve the applied tension. In the testing reported here to minimize bending of the studs under the applied force the stud webs are "sandwiched" between 5/16 in. thick steel plate and two angles ¼ in. thick, bolted together at 10 locations along the length of the stud (see Figure 2.c section A-A). The plate is attached to two hot-rolled 1½ by 1½ by ¼ in. angles that are then bolted to a larger structural WT section (WT 11 x 9) which connects to the actuators in the universal test frame. A 100 kip MTS actuator with 6 in. stroke is utilized to apply the loads.

The studs used in the test are 362S162-68's ([11]) throughout. This is the same stud utilized in a larger experimental project on sheathed walls ([12]). The stud spacing, w, fastener spacing, s, and edge distance, e, are varied in the testing. Two types of sheathing are employed: OSB (7/16 in., rated 24/16, exposure 1) and gypsum (½ in. Sheetrock). Number 6 screws (Simpson #6 x 1 5/8'') were used to connect to the gypsum boards and number 8 screws (Simpson #8 x 1 15/16'') to connect to the OSB boards.

Environmental conditions are varied in the conducted testing. Three conditions are considered: humid (saturated), dry, and normal. Humid (saturated) conditions are established by keeping the sheathing inside a tank filled with water for seven days. Dry conditions are established by keeping the sheathing in an oven for seven days at a temperature of 103C. Normal conditions are established by keeping the sheathing in an environmental chamber for seven days at a temperature of 20C and 65% humidity.

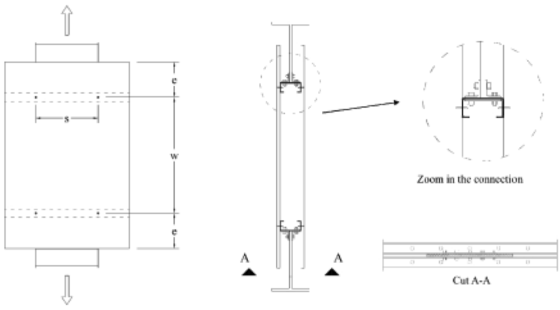


Figure 2 – Test setup design

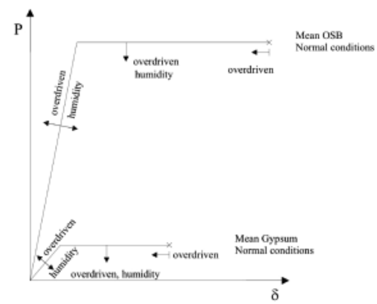


Figure 3 – P-δ of OSB vs. Gypsum

3.2 Results and Discussion

Complete load-displacement response was recorded, but three values are used to focus the discussion: i) maximum load; ii) displacement at maximum load and iii) initial stiffness. The maximum load gives the strength of the connection, the displacement at the maximum load gives a sense of the capacity of the connection to deform (and dissipate energy if needed), while the initial stiffness gives the relationship between load and displacement in the initial response typically appropriate for stability bracing.

A condensed summary of test results is provided in Table 1 see [10] for full results. OSB sheathed test specimens outperform gypsum sheathed specimens by a wide margin. For nominally identical studs, fasteners, and spacing, the lateral stiffness of an OSB sheathed specimen is 3 times greater than gypsum board, the shear capacity in OSB is nearly 7 times greater than gypsum board as the failure mode switches from screw shear to tear out, and the displacement at peak load is 2 times greater in OSB than in gypsum. Stylized load- displacement curves in Figure 3 provide a graphical depiction of the average results and dramatically show the difference between the two sheathing types. As indicated in the figure the impact of humidity and over-driving the fasteners is the same for both sheathing types. Humidity decreases stiffness and strength. Over-driving the fasteners increases stiffness, but decreases strength and deformation capacity. In Table 1 Normal conditions refer to w = 24 in.; s = 4, 12, or 20 in.; e = 6 in.; kept for seven days at a temperature of 20C and 65% humidity. The Overdriven condition has the same w, s, and e but the screw is overdriven by 1/8''. The Humid (saturated) condition has dimensions w = 8 in.; e = 2 in.; and s = 4, 6, 9, 12, and 20 in.; and are kept immersed in water for 7 days.

Table 1 – Condensed summary of test results.

		k initial		Pmax		δ @ Pmax	
		mean (kip/in)	coef. variation	mean (kip)	coef. variation	mean (in)	coef. variation
OSB	Normal Conditions	7.08	0.07	0.58	0.03	0.62	0.02
	Overdriven	9.36	0.10	0.41	0.05	0.34	0.27
	Humid (saturated)	6.32	0.10	0.26	0.04	0.51	0.21
Gypsum	Normal Conditions	2.43	0.02	0.09	0.03	0.34	0.13
	Overdriven	3.49	0.14	0.07	0.02	0.15	0.57
	Humid (saturated)	0.24	-	0.02	-	0.42	-

The lateral support supplied by a connecter-sheathing assembly with dry OSB has greater initial stiffness and ultimate strength than one with humid (saturated) OSB. The cases with humid (saturated) OSB show an average percent reduction of 22% in stiffness and 47% in strength. Under normal conditions the gypsum board has a significant initial stiffness and is able to carry 85.9 lb/fastener. In the humid or dry condition the gypsum board suffers significant stiffness loss and is only able to sustain about ¼ the fastener capacity. Given the low deformation capacity as well, in the dry or humid condition the gypsum board essentially ceases to behave as a structural material. The substantial variation of gypsum board to environmental conditions means its structural properties must be treated with great care in design.

For the studied fastener spacing, neither the initial stiffness nor the strength varied significantly as a function of fastener spacing. This strongly suggests that the developed deformations and failure mechanisms are local to the fastener. During installation it is possible that the fastener will be overdriven, aware of this possibility a series of tests were conducted with fasteners intentionally overdriven by a uniform 1/8 in.. Overdriving the fasteners increases the initial stiffness (32% on average), but at a cost, the strength and deformation capacity decrease (29 and 45% on average, respectively).

From the standpoint of providing bracing for a wall, the increased stiffness is likely beneficial enough (higher stiffness actually lowers the needed bracing forces) to outweigh the decreased capacity, and one is likely to find that over-driving has little impact. However, for a shear wall or other situation where the strength of the fasteners limits the capacity, the increased stiffness only drives more load to the fastener, and the precipitous loss in capacity means it fails earlier. The impact of overdriving the fastener thus depends significantly on the controlling limit state.

The influence of overdriven fasteners was also studied in specimens sheathed with gypsum board. At first glance the conclusions are the same as in the OSB: overdriving increases the stiffness, but decreases the strength and deformation capacity. Indeed this is true, but the strength available in the overdriven case may be as little at 67 lb/fastener or less. It is not clear if this is generally adequate for providing bracing resistance; further study is required to determine if the available strength of overdriven fasteners is sufficient in an actual wall. Similarly, the deformation capacity is reduced down to a range that may make it difficult for the fastener to provide bracing under any meaningful deformations of the stud.

4 DIAPHRAGM MODEL FOR TRANSLATIONAL STIFFNESS

Consider a wall where the studs are buckling in global flexural buckling, as illustrated in Figure 4. If the sheathing deforms in a manner compatible with the flexural buckling it undergoes predominately a shear demand. That is, the sheathing must behave like a shear diaphragm. Simaan and Peköz [5] postulated that this shear deformation of the panel is the key component of bracing resistance that the sheathing supplied to the studs. This observation was used in an energy solution for the stability of the cold-formed steel studs that included the energy from the shear diaphragm (driven by the shear modulus of the sheathing material) and was utilized from 1980-2004 in the AISI Specification.

The Simaan and Peköz analytical model ignored (simplified) the fact that the shear diaphragm must be resolved through the fasteners and only included flexibility from the diaphragm (the sheathing) itself. However, they partially resolved this issue by employing a test method for determining the diaphragm stiffness. The test method, depicted in Figure x, requires forcing a large panel into simple shear and measuring the panel distortion. Since the panel must be connected to a testing rig the local fastener deformations are thus included in determination of the panel shear stiffness. Recent comparisons show that these tests primarily are controlled by local fastener stiffness, as the local stiffness resulting in these tests is similar to that from Winter's tests [6]. The goal here is to isolate the shear diaphragm stiffness from the local stiffness so that both may be considered independently. And determine the stiffness the shear diaphragm (independent of the local fastener stiffness) supplies to the stud at each fastener location.

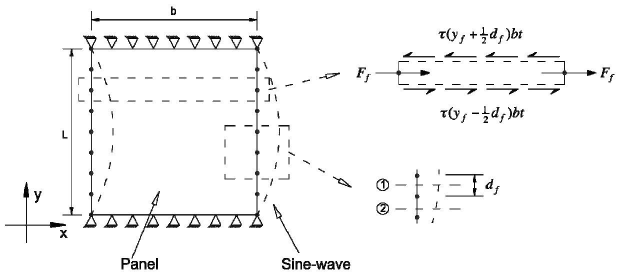


Figure 4 – Plate model

4.1 Analytical model for diaphragm stiffness

Consider again the sheathed wall under flexural buckling of Figure 4. The lateral deformation, u, is:

$$u = \sin(\pi y / L) \tag{1}$$

The stiffness at a fastener location is the force at the fastener, developed from an integration of the shear stress over the tributary area of the fastener, divided by the deformation, u, at the fastener location. Determination of the shear stresses in the sheathing may be understood as if the sheathing has a low shear modulus or the sheathing panel is wide and short, all of which are commonly the case, then the stresses are controlled by shear deflections consistent with diaphragm action

Thus, the shear stress at any point is simply

$$\tau = G\theta \tag{2}$$

From the free body diagram of Figure 4,the force in the fasteners, F_f, at height y_f, with fasteners spaced d_f apart for sheathing of width, b, thickness, t, bracing n studs, is

$$nF_f = \tau(y_f + \tfrac{1}{2}d_f)bt - \tau(y_f - \tfrac{1}{2}d_f)bt \tag{3}$$

where the parentheses indicate the height at which τ is determined. The stiffness at the fastener location is simply

$$k_d = F_f / u_f \tag{4}$$

where u_f is the deformation u at height y_f (the fastener location). Noting $\theta = \mathrm{d}u/\mathrm{d}y$, then Eq. (1) may be differentiated and substituted into Eq. (2), then τ of Eq. (3) may be evaluated at the appropriate locations, and substituted into Eq. (4), leading after simplification to:

$$k_d = \frac{2\pi Gbt}{Ln}\sin\left(\frac{\pi d_f}{2L}\right) \cong \frac{\pi^2 Gbtd_f}{L^2 n} \tag{5}$$

4.2 Finite element comparison of diaphragm stiffness

Physical testing of the diaphragm stiffness does not typically provide a meaningful comparison to k_d, because isolation of k_d from the local fastener stiffness is difficult to impossible in conventionally detailed sheathed walls. To provide examination of the derived expressions a finite element model of a panel undergoing a half sine-wave deformation (i.e., Figure 4) is completed in ABAQUS [13]. The model consists of a plate, 96 in. x 96 in., modeled using linear four-node shell elements (S4R) elements. The boundary conditions are consistent with Figure 4 and include a sine curve applied at every inch in the y direction on both edges (right and left). Assuming 7/16'' OSB, the panel material is modeled as orthotropic with a Young's modulus of 900 ksi [14]. The shear modulus, G, is systematically varied; however, note, the recommended value of G is 45 ksi [14] and for an isotropic homogenous material with ν=0.3, G would be 346 ksi. The developed diaphragm fastener stiffness in the FE model (k_{dFE}) is compared to k_d in Figure 5. At G = 45 ksi, $k_{dFE} = k_d$; thus direct use of k_d is reasonable for practical situations. At G = 346 ksi, $k_{dFE} \sim k_d$; thus k_d is sufficiently accurate up to the isotropic limit.

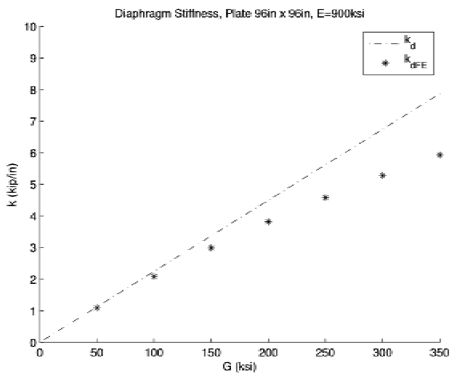


Figure 5 – Diaphragm stiffness compared with FE

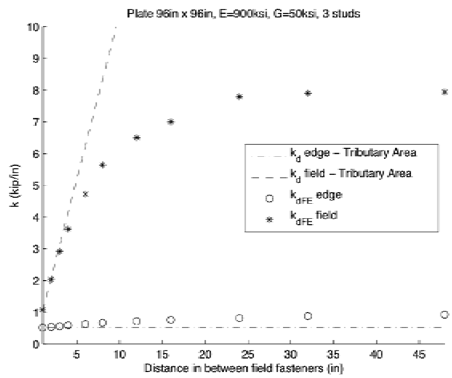


Figure 6 – Limitation of tributary area for k_d

4.3 Multiple studs with fasteners differently spaced

Generalization of Eq. (5) is needed because fastener spacing may not be uniform; in particular, the boundary studs are commonly at a tighter fastener spacing than the studs in the field of the board. It is proposed that the tributary area for each fastener be employed, therefore b/n of Eq. (5) is replaced by w_{tf}, the fastener tributary width, and d_f is the fastener spacing, and may vary:

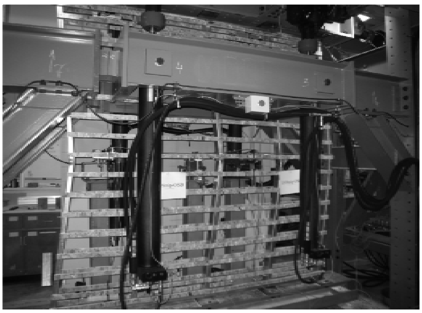
$$k_d = \frac{2\pi G t w_{tf}}{L} \sin\left(\frac{\pi d_f}{2L}\right) \cong \frac{\pi^2 G t d_f w_{tf}}{L^2} \qquad (6)$$

The model of the previous section is exercised to explore the validity of Eq. (6). The edge fastener spacing is 1 in., and the field fastener spacing is varied from 1 in. to 48 in., results are provided in Figure 6. The fastener stiffness is consistent with the tributary area, up to tributary areas in the field that are 6 x greater than the edge tributary area. As the distance between fasteners in the field is increased over this limit the edge fasteners act as if there were no fasteners in the field and the stiffness goes back to the case of only being connected at the edges. The observed limitation is not a practical problem since the relation between tributary areas is typically no greater than 4 x (e.g., 6 in. on the edge, 12 in. in the field).

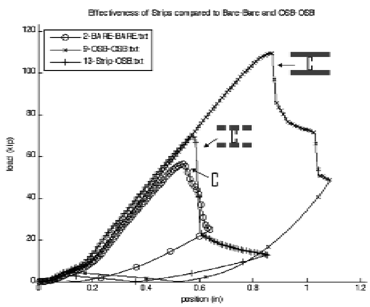
5 FULL SCALE TEST

As reported in [12] the authors tested 8 ft by 8 ft cold-formed steel stud walls comprised of five 362S162-68 (50 ksi) studs spaced 24 in. o.c. and 362T125-68 (50 ksi) track in compression. Without bridging, blocking, or sheathing the walls fail in global flexural buckling at a load of 56.33 kips. When sheathed with OSB on both sides the walls fail in local buckling at a load of 109.55 kips. The enormous difference shows the positive benefit of the sheathing, both in terms of increasing the strength and limiting global buckling failure modes.

Winter's model assumes that the critical bracing stiffness and strength that sheathing supplies to the stud is derived at the fastener location in direct shear. In essence, arguing that only local deformations must be understood to design the brace. Recently, as part of the larger testing program on full-scale sheathing braced walls [12] the authors put this hypothesis to the test. Instead of sheathing with full boards, OSB strips (2 in. wide) were connected to the studs, see Figure 7a. The use of strips negates the shear diaphragm resistance (i.e., k_d). The wall failed in flexural buckling at 69.53 kips, supplying only the local fastener stiffness provided a small increase in axial capacity, but no change in limit state. Thus, providing evidence that sheathing bracing derives from both the local and diaphragm resistance.



a) Full wall test, strips instead of full boards



b) Comparison between loading curves

Figure 7 – Effectiveness of strips compared to Bare-Bare and OSB-OSB

6 COMBINED LOCAL AND DIAPHRAGM MODEL

Sheathing provides bracing to studs. Characterization of this bracing has proven historically difficult, in part because the two competing models for the explanation of the bracing: local and diaphragm, draw such different conclusions on the behavior. For example, diaphragm stiffness is influenced strongly by the stud spacing and the fastener spacing; while the local stiffness is not. Full-scale testing has provided, what to date was considered contradictory evidence, sometimes indicating these variables are important sometimes not. However, if one realizes that the local stiffness is in series with the diaphragm stiffness then the explanation becomes clear. If local stiffness is low enough (and just as importantly diaphragm stiffness high enough) one will only see the local stiffness in the response and stud and fastener spacing will be largely irrelevant. Conversely, if local stiffness is high enough, say for example from a welded specimen with a steel sheet (and diaphragm stiffness low enough) then only the diaphragm stiffness will be important and stud spacing will be enormously important. Mathematically this may be handled by realizing k of Figure 1 may be approximated as

$$k = \left(\frac{1}{k_l} + \frac{1}{k_d} \right)^{-1} \tag{7}$$

where k_l is determined experimentally, and k_d utilizes Eq. 5 or 6. For bracing strength the local model (and its associated testing) includes the most critical strength limiting failure modes: bearing, tilting, edge pull-out, and screw shear. Failure of the sheathing itself, in shear, and not at the connector location is possible (e.g. in a shear wall), but is generally not an expected failure mode for sheathing only acting as bracing. Thus, the testing for k_ℓ also may be used to determine the bracing strength along with the Specification equations [15].

7 FUTURE WORK

The developed bracing model is currently being compared with existing full-scale tests. An extensive finite element study, where the bracing performance can be varied to illustrate the concepts herein is underway. Extension to studs undergoing flexural-torsional buckling has begun. Development of a comprehensive design approach for sheathing braced design is the ultimate objective of this work.

8 CONCLUSIONS

Sheathing may provide adequate bracing for axial load bearing cold-formed steel stud walls. The sheathing may be considered to “brace” the studs at fastener locations. The sheathing bracing derives from both local fastener deformations and global shear diaphragm behavior. Experiments on the local fastener stiffness (and strength) indicate the relative stiffness and strength difference between fasteners attached to studs through OSB and gypsum. The sensitivity, particularly of gypsum, to environmental and installation conditions is illustrated in the local fastener testing. Analytical formulae are provided and verified for the shear diaphragm stiffness. A full-scale wall test using strips of OSB sheathing instead of a full OSB board is completed to demonstrate that the sheathing bracing derives from *both* the local and diaphragm stiffness, not just the local stiffness (or the diaphragm stiffness) as had been classically assumed. A combined bracing model whereby the local and diaphragm bracing are treated as two springs in series is proposed for modeling sheathing-braced studs.

9 ACKNOWLEDGMENTS

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