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Analysis of Sheathed Cold-Formed Steel Wall Studs

Benjamin W. Schafer¹, Badri Hiriyur²

Abstract

The strength of sheathed wall systems is substantially greater than unsheathed walls with the same studs. Testing on studs sheathed on both sides with dry gypsum board demonstrate strength increases as large as 70% over unsheathed studs (Miller and Peköz 1993, Miller and Peköz 1994). The current AISI Specification for sheathed wall stud systems are complicated, only apply when the sheathing is identical on both sides, have several onerous prescriptive requirements, and the mechanical model employed has been questioned when compared to experimental results. This paper summarizes the deficiencies of, and arguments against, current models and suggests avenues of research for improvements. Analysis discussed herein, directed at answering the many open questions, leads to numerous interesting conclusions about the behavior of sheathed wall systems. Sheathing diaphragm stiffness should not yet be abandoned as the basic mechanical model. Diaphragm stiffness, per stud, is non-uniform and is not solely derived from stud spacing as assumed in AISI (1986) nor is it solely independent of stud spacing as assumed in AISI (1996, 2001). Shear stiffness, either locally of the material, or globally for the diaphragm, is the fundamental manner in which the sheathing resists weak axis buckling of the stud. For sheathing on one-side only, or highly dissimilar sheathing, cross-section distortion plays a role in the behavior and none of the existing (AISII) or proposed (Lee and Miller 2001) models account for this. Finally, freely available numerical tools provide a means to assess elastic buckling capacity of sheathed studs even with one-sided or dissimilar sheathing.

Background

The development of design expressions for sheathed compression members began at Cornell University in the 1940's and was revisited in the 1970's and 1990's. Sheathed compression members have two homes in the AISI Specification:

- AISI C4.4 is for 'C's and 'Z's with one-sided steel sheathing, and uses a prescriptive and empirical treatment based on Simaan and Peköz (1976),
- AISI D4.1 is for 'T's, 'C's and 'Z's with identical two-sided sheathing, and uses a general and theoretical treatment, based on Simaan and Peköz (1976).

No provisions are provided for the most common case in industry: wall studs with dissimilar sheathing (i.e., plywood or OSB on the outside, gypsum on the inside). For one-sided sheathing, provisions are only provided for steel panel sheathing.

The mechanical model of the current AISI (2001) Specification D4 assumes that the sheathing acts as a shear diaphragm in restraining the deformation of the studs. Section D4 is based on Cornell research from the 1960's (Pincus and Fisher 1966, Errera et al. 1967, Apparao et al.

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1969) conducted on columns stabilized by corrugated metal sheathing. This original work was extended to other sheathing types in the 1970's (Simaan et al. 1973, Simaan and Peköz 1976). Recent work has focused on studs with gypsum board sheathing (Miller and Peköz 1993, Miller and Peköz 1994, Lee and Miller 2001a, Lee and Miller 2001b, Telue and Mahendran 2001). Miller's testing lead him to conclude that

“...axial strength [is] independent of stud spacing, reflecting the localized nature of the wallboard deformations rather than the shear diaphragm behavior assumed in the current AISI [1986] specification.” (Lee and Miller 2001a)

Thus, at least for gypsum sheathing, experimental evidence appears to indicate that the Simaan and Peköz (1976) approach may be mechanically inaccurate – Lee and Miller have proposed an alternative, similar in spirit to the 1962 AISI Specification method (AISI 1962). The most recent AISI Specification (1996, 2001) has chosen to continue the use of the shear diaphragm model, but empirically removed the shear stiffness dependence on stud spacing. A stud spacing of 12 in. o.c. is universally assumed, independent of actual stud spacing (see §9, Yu 2001). Based on the limited test data the change is conservative, as the tested data had greater strength at 12 in. o.c. than predicted by AISI (1986).

Shear stiffness and stud spacing

Shear stiffness of the sheathing provides resistance to the in-plane deformation of the stud. Shear stiffness is engaged either locally through deformation of the sheathing material at the screw location, or globally by deforming the entire sheathing as a diaphragm. Without shear stiffness the stud is free to undergo weak-axis flexural buckling. How can these statements be reconciled with the experimental observations that stud strength is largely independent of stud spacing? Preliminary exploratory work with a plane stress finite element model and with a finite strip model of studs and sheathing together help provide some insight on these issues.

Plane stress model

The Simaan and Peköz (1976) model for buckling of sheathed studs determines the resistance of the sheathing by deriving the stiffness of a panel under enforced single half sine waves at the stud locations. The half sine waves are the assumed displacement demand of a stud deforming in weak-axis flexural buckling. Elastic plane stress finite element analysis of an 8' by 8' panel undergoing in-plane enforced displacements in the shape of single half sine waves was investigated in order to better understand the ramifications of this theoretical model.

Brief parametric studies were investigated to examine the role of: stud spacing, connector spacing, and the shear stiffness of the diaphragm. The results of these studies are given in Table 1 and depicted in Figure 1 through Figure 3.

As the number of studs in a given panel increase the total stiffness of the panel to resist deformation increases. For instance, in Figure 1, the total stiffness of (c) is 21% greater than (a). Of course, this increase is offset by the fact that a greater number of studs are being supported, 2 in (a), 9 in (c). This increase in total stiffness available is ignored in the derivations supporting

AISI D4 and helps partially explain why in Miller's testing, 5 studs at 12 in. spacing perform nearly as well as 3 studs at 24 in. spacing.

Table 1 Parametric studies with elastic plane stress panel model

number of "studs"	"stud" spacing	"connector" distance	total panel stiffness (k_{max})*	0.5G stiffness / total panel stiffness
2 studs	8 ft apart	4 in. spacing	2.00	60%
4 studs	2 2/3 ft	4 in.	2.33	51%
5 studs	2 ft	4 in.	2.37	55%
		12 in.	2.16	
		24 in.	1.97	
9 studs	1 ft	4 in.	2.42	54%

* k_{max} is the stiffness available to a single stud in the model with 2 studs 8 ft. apart in an 8' x 8' panel, for the material properties employed in the model this stiffness was 21,850 kip-in., the model is linear elastic and the results are therefore relevant to any elastic material.

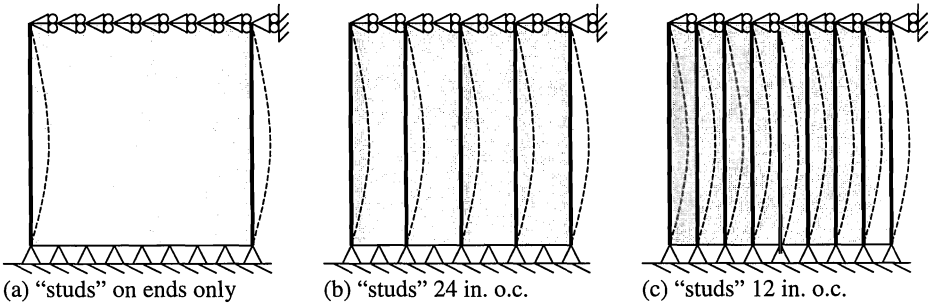


Figure 1 Stiffness analysis of a plate with enforced displacements equal to the buckling mode shape of a stud undergoing weak axis flexural buckling

Figure 3 graphically depicts the stiffness that is available, per stud, as a deformation consistent with weak-axis flexural buckling is enforced. The distribution of stiffness available amongst the studs is not uniform. For example, compare the wall with 2 studs (8 ft apart) to one with 4 studs (2 2/3 ft apart). The stiffness available to the 4 studs is less than for the 2 studs, but not uniformly 1/2 as much. The outside studs see less than 50% of the stiffness available to 2 studs, while the inner studs in the 4 stud wall see nearly 70% of the 2 stud stiffness. Simple laws assuming uniform stiffness distribution miss the real behavior, which is quite a bit more complex, even in a simple model.

The role of connector spacing is influential and can be examined by only enforcing the displacements at assumed connector locations. Figure 2 depicts the, per stud, results for "connector" spacing of 4, 12, and 24 in. o.c. in a model with 5 studs spaced 2 ft apart (Figure 1(b)) in the panel. This simple plane stress model indicates that a 12 in. o.c. connector spacing results in a loss of 8% in the overall stiffness from the ideal case (continuous restraint) and that a 24 in. o.c. connector spacing results in a loss of 16%. However, this loss is not distributed evenly amongst the studs, and for the studs at 2 ft and 6 ft, 12 in. o.c. spacing is better than continuous

restraint! Even this simple elastic plane stress model suggests that the relationship between number of studs in a panel and connector spacing is a relatively complex one. Currently, AISI uses a simple linear correction for connector spacing between 6 and 12 in.

The 8 ft x 8 ft diaphragm was modeled as isotropic and orthotropic. In the orthotropic model G_{xy} can be modeled separately from E_x , E_y , ν_x , and ν_y . For the enforced deformations shown in Figure 1 as $G_{xy} \rightarrow 0$ the stiffness $\rightarrow 0$. Which is really nothing more than saying diaphragm action relies on shear. Shear stiffness is of primary importance in the panels ability to resist weak-axis flexural buckling of the studs.

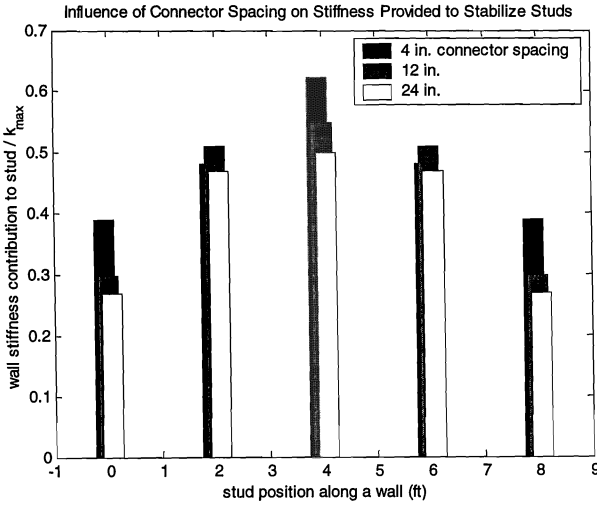


Figure 2 Influence of connector spacing on stiffness provided to stabilize studs

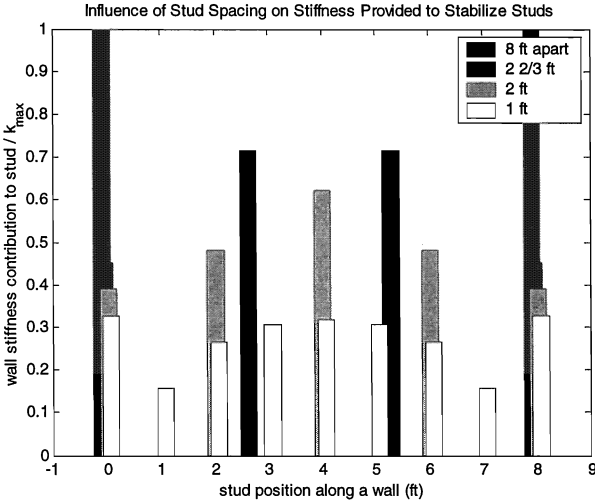


Figure 3 Influence of stud spacing on stiffness provided to stabilize studs for an 8' x 8' panel

Finite Strip Analysis

Finite strip analysis using CUFSM (www.ce.jhu.edu/bschafer/cufsm) of studs rigidly and continuously connected to sheathing was also performed to provide insight into the basic behavior of sheathed wall studs. The geometry of the studs was selected to be SSMA 362S162-68, as tested by Miller and Peköz (1994).

First, let us revisit the issue of the influence of stud spacing on the solution in a simple model. Figure 4 summarizes a series of analysis for sheathed studs, with different numbers of studs and different stud spacing. In this model the sheathing is a rigidly and continuously connected steel sheet of $t=0.018$ in. Local and distortional buckling (the first two minima of Figure 4) are unaffected by the changing stud spacing and numbers; however, the long wavelength flexural – torsional buckling is strongly affected. Consider the response of 2 studs spaced at 12, 24, and 48 in. in Figure 4. For a single panel, the widest stud spacing provides the greatest shear resistance and thus the highest buckling stresses, consistent with the Simann and Peköz model as used in the AISI Specification prior to 1996.

However, 5 studs at 12 in. o.c. performs better than 2 studs at 12 or 24 in. o.c. Further, 2 studs at 24 in. o.c., performs nearly identically to 2 studs at 48 in. o.c. Near independence of the result on stud spacing does not invalidate a shear diaphragm model in favor of a local spring model – as a local model was not used in the finite strip analysis. Rather, it suggests that rules considering only stud spacing (as was done prior to 1996 AISI), or ignoring stud spacing completely (as is currently done in AISI), are oversimplified.

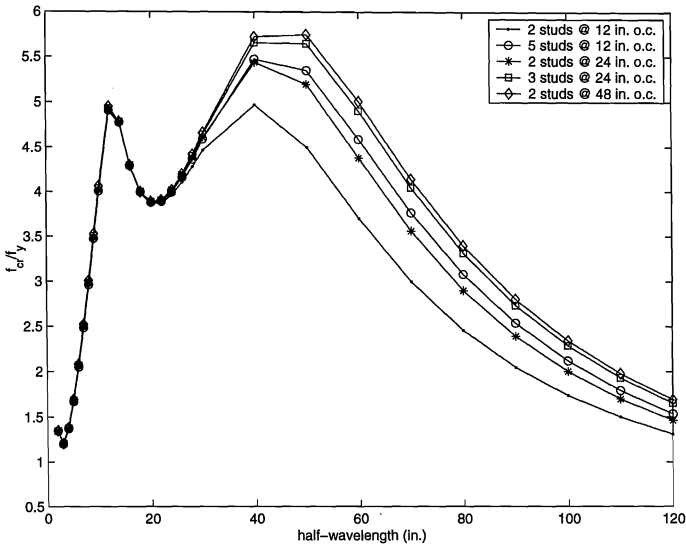


Figure 4 Influence of stud spacing on FTB for 362S162-68 studs with steel $t=0.018$ sheathing on both sides

A more robust elastic finite strip model was generated to investigate the performance of sheathed wall systems with sheathing on one side only or with dissimilar sheathing. The basic models employed are shown in Figure 5. The studs are SSMA 362S162-68 C-sections. As shown in Figure 5, sheathing of plywood, gypsum, or both, on either one side or both sides, connected the five studs. A spacing of 12 in. o.c. was employed for the five studs. Connections between the sheathing and the stud were made at the center of the flanges of the C-sections. At these connections, all the rotational and translational degrees of freedom of the stud were constrained to be compatible with those of the sheathing. Thus, the stud-to-sheathing screws are assumed to be perfect.

Based on general information available in Bodig and Jayne (1982), the material properties for plywood are highly variable with modulus of elasticity, E , ranging from 900 to 1800 ksi and shear modulus, G , ranging from 45 ksi to 90 ksi. Note, the shear modulus is much lower than any isotropic equivalent, $G \neq E/[2(1+\nu)]$. OSB may have slightly higher G values than plywood, but no data was immediately available. For the plywood, a mean value of $E = 1350$ ksi and $G = 67.5$ ksi were used for this study. Little direct material data is available for gypsum board, but based on Sculpt (2002) properties were approximated. Gypsum board was taken to have $E = 100$ ksi and the same E/G ratio as used for the plywood was assumed, therefore $G = 5$ ksi. The thickness of both sheathing materials was assumed to be $\frac{1}{2}$ an inch (12.7 mm) and Poisson's ratio, ν , was assumed to be 0.3 for both sheathing materials.

The finite strip program used for the analysis, CUFSM (2002), allows materials to be modeled as orthotropic, so one unique aspect of this model is that it captures the influence of the low shear stiffness of the sheathing on the solution.

The elastic buckling response of the unsheathed wall stud is given in Figure 6. Elastic critical local buckling is predicted at $1.02P_y$ and elastic critical distortional buckling at $1.36P_y$, where $f_y = 50$ ksi (345 MPa). At longer lengths, for pinned ends, flexural-torsional buckling occurs at a slightly lower buckling stress than weak-axis flexural buckling.

A summary of the analysis results with all the different sheathing conditions investigated is given in Figure 7. The critical loads for local buckling, occurring at half-wavelengths lower than the largest characteristic dimension of the member, are largely independent of restraint conditions. However, distortional buckling and long wavelength buckling (flexural, and flexural-torsional), are highly influenced by the restraint conditions. When the rigidity provided by the sheathing was large, as in the case of ply-ply and ply-gyp, distortional buckling was effectively eliminated.

The finite strip model includes the analysis of the entire wall system: all 5 studs and sheathing. As a result, the conventional modes, for an unsheathed member (Figure 6) now come in groups of 5, as shown in Figure 8 for the model with plywood sheathing on both sides. The first 5 modes for this model are all slightly different local buckling modes, of nearly identical magnitude. In essence, the "local buckling mode" for this wall system is any combination of these first 5 modes, all of which are dominated by local web buckling. Distortional buckling behaves in a similar fashion. For example, the 5 different modes of distortional buckling for plywood on one side and no sheathing on the far side all occur at the same magnitude, as summarized in Figure 9.

One of the 5 local buckling modes is shown in Figure 10 for three different models. The critical loads are essentially the same in the case of local buckling. The unsheathed case has a local buckling stress of $1.02f_y$ while the model with plywood sheathing on both sides had the largest local buckling stress at $1.06f_y$. The analysis supports the experimental observations of others, that local buckling is largely unaffected by screw pattern or sheathing type. This is due primarily to the dominance of web local buckling in the C-Section. Near independence of the local buckling solution on the sheathing is a function of the cross-section selected, if longitudinal stiffeners were formed in the web, the flange restraint would play a greater role, and local buckling would exhibit a greater sensitivity to the sheathing conditions.

One of the 5 distortional buckling modes is shown in Figure 11 for three different models. The distortional modes are of specific importance, because existing and proposed models ignore this mode for sheathed wall systems. The comparison presented in Figure 11 demonstrates key differences between plywood and gypsum sheathing in restricting distortional buckling. The gypsum sheathing provides a small increase in the rotational restraint and therefore boosts the distortional buckling magnitude above the unsheathed case. In the case of distortional buckling with gypsum on both sides, the buckling stress is raised significantly by the presence of the gypsum sheathing; however, the buckling modes show that the sheathing does not stabilize the connected flange in the same manner as the plywood. With plywood sheathing, the connected flange is nearly completely restricted and the distortional mode occurs only in the unrestrained flange. With plywood sheathing on both sides, the distortional mode is effectively removed from

consideration. Thus, as expected, the plywood is far more effective than the gypsum in retarding distortional buckling.

For the long wavelength modes (flexural-torsional buckling and weak-axis flexural buckling), given at 3.3 ft (1m), as shown in Figure 12 and Figure 7, we note the benefits of considering some form of sheathing on both sides of the member even when the sheathing is as weak as gypsum. Design rules that ignore the presence of a weaker sheathing on one side of a stud are discarding a significant amount of strength.

A significantly robust hand (mechanical) model would be required in order to model the large differences between the results given in Figure 7. Therefore, one might anticipate that a “next generation” design specification that desires to incorporate the benefits of dissimilarly sheathed wall systems may use or require numerical analysis in some form. Although the finite strip method ignores some important issues along the length of the studs: perforations in the studs, and finite screw spacing chief among them, it still provides numerous insights on the basic behavior of these systems and may prove an efficient and useful means for improving the design of sheathed wall systems with dissimilar or one-sided sheathing.

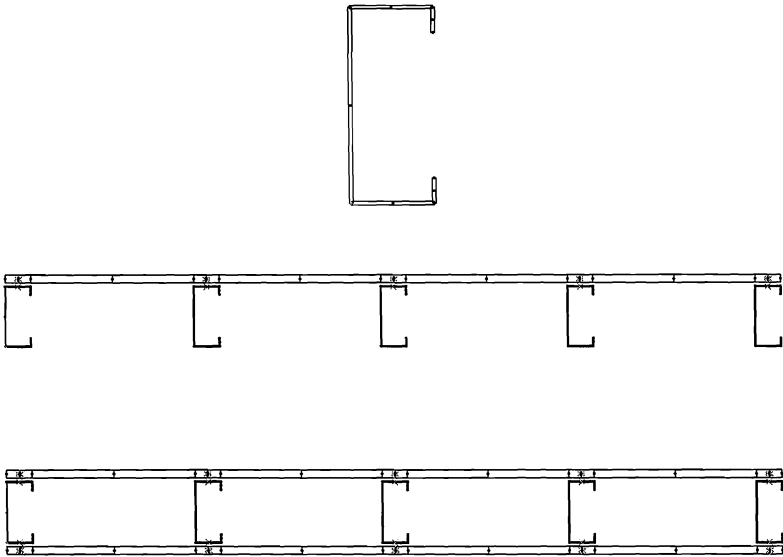


Figure 5 Finite strip models of unsheathed, sheathed on one side, and sheathed both sides

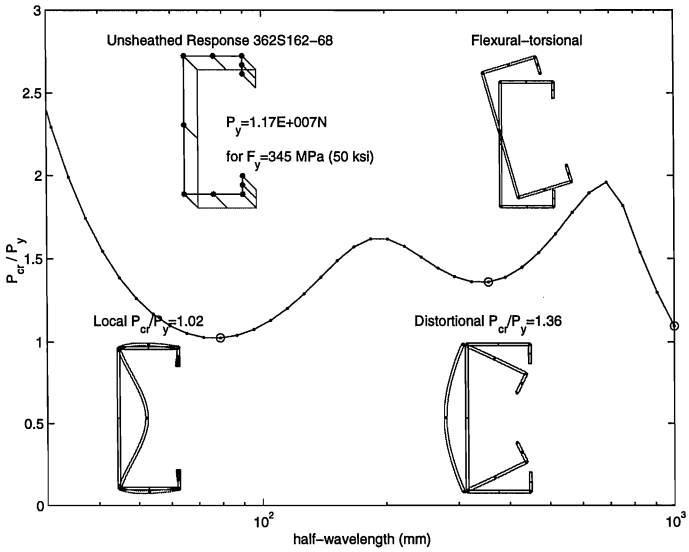


Figure 6 Unsheathed elastic buckling response of an SSMA 362S168-68

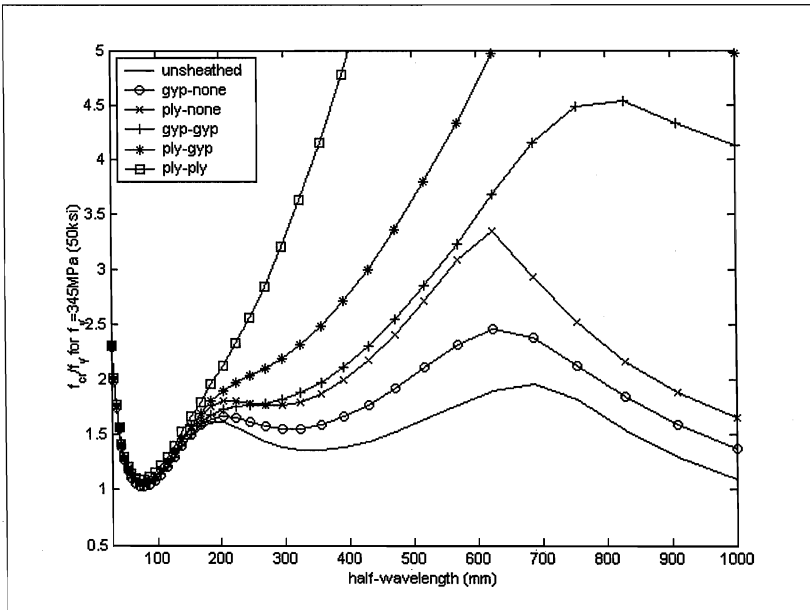


Figure 7 Comparison of elastic buckling response for different sheathing attachments and types

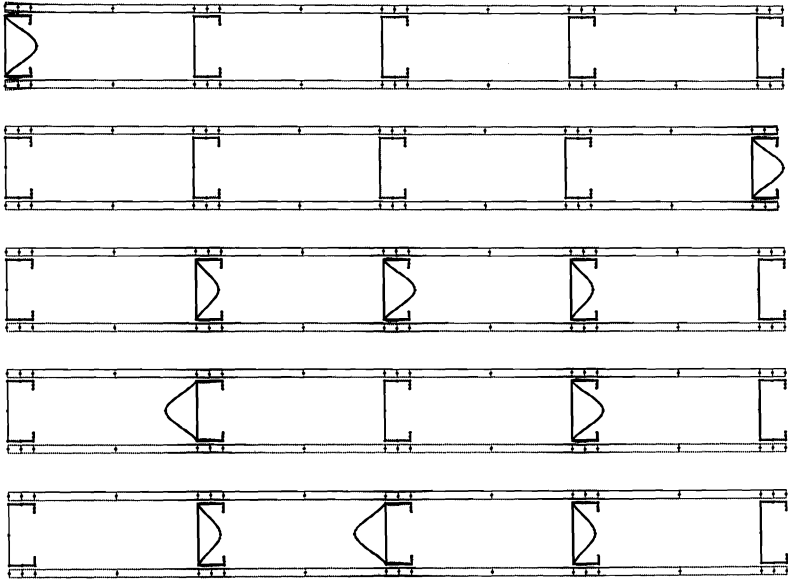


Figure 8 First 5 local modes for the ply-ply model (note f_{cr}/f_y varies from 1.09 to 1.12)

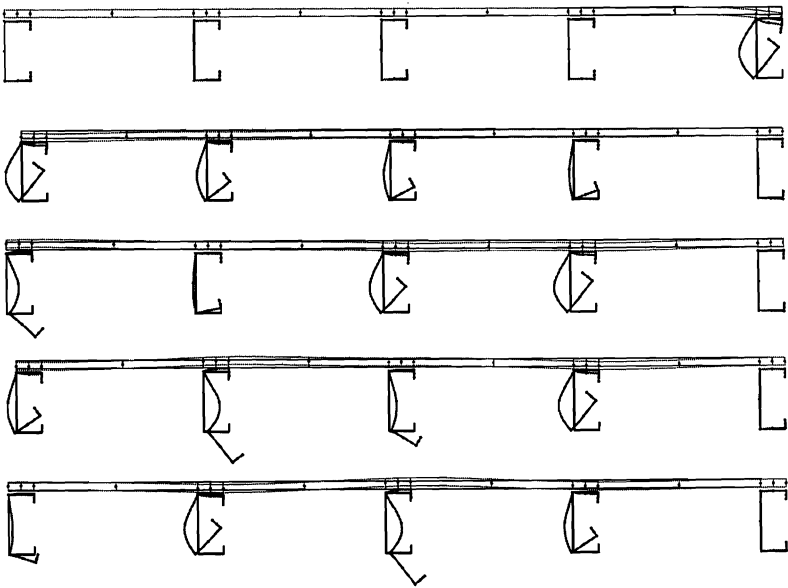


Figure 9 First 5 distortional modes for the ply-none model (note f_{cr}/f_y varies from 1.77 to 1.84)

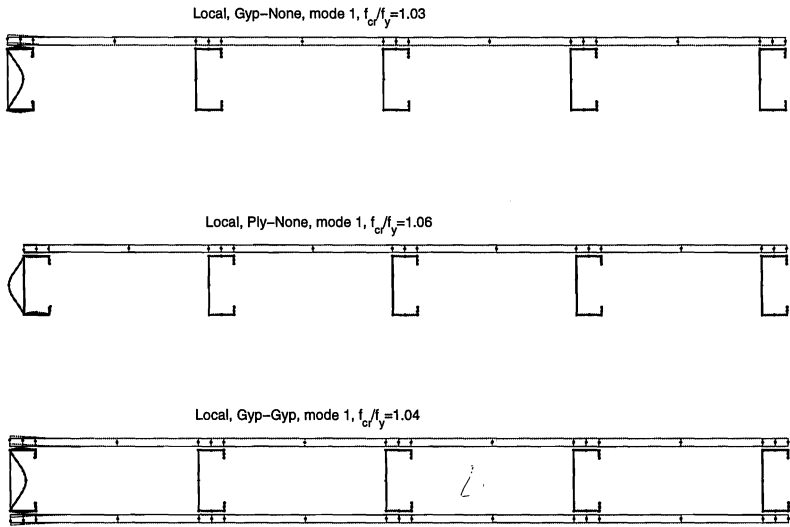


Figure 10 Role of sheathing in local buckling mode

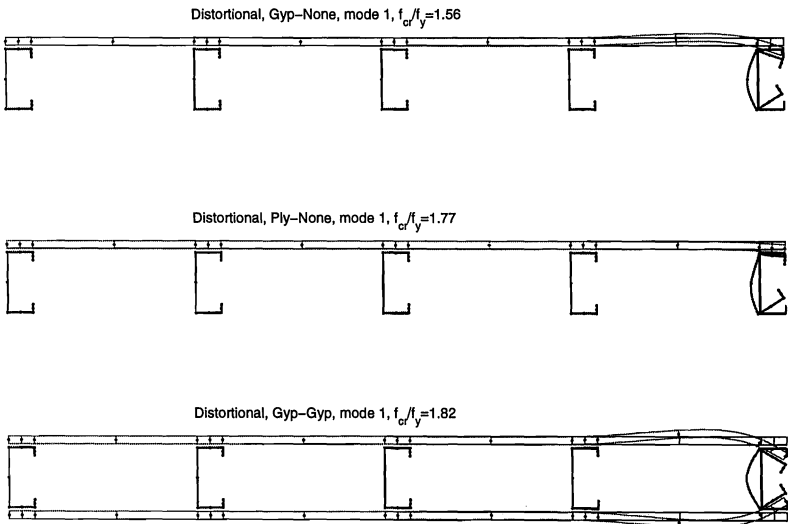


Figure 11 Role of sheathing in distortional buckling mode

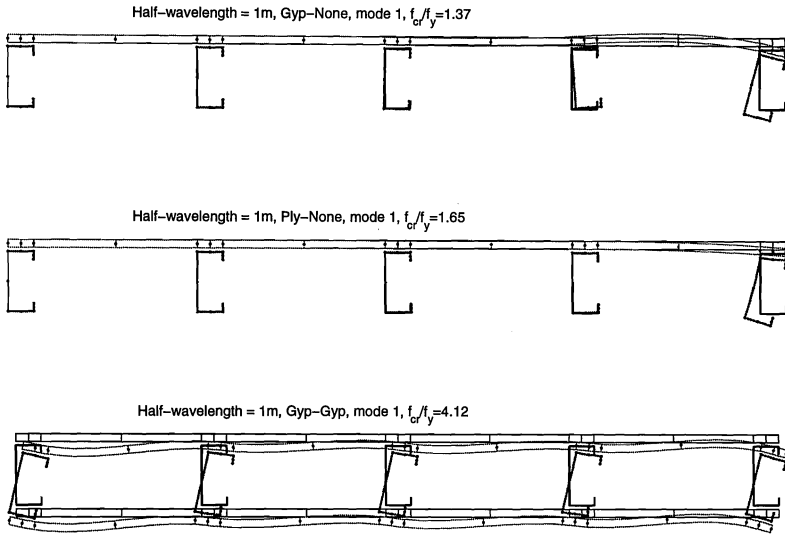


Figure 12 Role of sheathing in long wavelength buckling modes

Conclusions

The behavior and mechanical models related to sheathed wall studs raise a variety of interesting questions, particularly with respect to the role of diaphragm shear stiffness, and stud spacing. Analysis of sheathed wall systems using a plane stress finite element model with imposed displacements, and a finite strip model with the sheathing discretely modeled provides some preliminary answers:

- Diaphragm stiffness should not yet be abandoned as the basic mechanical model for sheathed wall systems in the AISI Specification.
- Diaphragm stiffness, per stud, is non-uniform and is not solely derived from stud spacing (as assumed in AISI 1986) nor is it solely independent of stud spacing (as assumed in AISI 1996,2001).
- Shear stiffness, either locally of the material, or globally for the diaphragm, is the fundamental manner in which the sheathing resists weak axis buckling of the stud.
- Sheathing has little influence on local buckling of wall studs.
- For one-sided sheathing or highly dissimilar sheathing, cross-section distortion (distortional buckling) plays a role in the behavior even at practical lengths and none of the existing (AISI) or proposed (Lee and Miller 2001a) models account for this.

- The presence of sheathing on both sides of a stud, even when the sheathing is dissimilar or relatively weak sheathing (gypsum) still provides significant strength benefits for long-wavelength buckling over considering sheathing on one side only.
- Current numerical tools (e.g. CUFSM 2002) provide a simple way to assess elastic buckling capacity of unperforated studs with one sided and/or dissimilar sheathing.

While much work remains to be done to provide a design method that can easily and fully account for the strength of these systems, currently available numerical tools shed significant light on the problems with current approaches and provide a means to develop new methods that would allow the efficiency of these systems to be realized in practice.

References

- AISI (1962). Specification for the Design of Light Gage Cold-Formed Steel Structural Members. *American Iron and Steel Institute*, Washington, D.C.
- AISI (1986). AISI Specification for the Design of Cold-Formed Steel Structural Members. *American Iron and Steel Institute*. Washington, D.C.
- AISI (1996). AISI Specification for the Design of Cold-Formed Steel Structural Members. *American Iron and Steel Institute*. Washington, D.C.
- AISI (2001). North American Specification for the Design of Cold-Formed Steel Structural Members, June 5, 2001 DRAFT EDITION. *American Iron and Steel Institute*. Washington, D.C.
- Apparao, T.V.S., Errera, S.J., Fisher, G.P. (1969). "Columns Braced by Girts and Diaphragm." *Journal of the Structural Division. ASCE*. 95 (ST5) 965-990.
- Bodig, J., Jayne, B.A. (1982). Mechanics of wood and wood composites, Van Nostrand Reinhold
- CUFSM (2002) www.ce.jhu.edu/bschafer/cufsm
- Errera, S.J., Pincus, G., Fisher, G.P. (1967). "Columns and Beams Braced by Diaphragms." *Journal of the Structural Division. ASCE*. 93 (ST1) 295-318.
- Lee, Y., Miller, T.H. (2001a). "Axial Strength Determination for Gypsum-Sheathed, Cold-Formed Steel Wall Stud Composite Panels." *Journal of Structural Engineering. ASCE*. 127 (6) 608-615.
- Lee, Y., Miller, T.H. (2001b). "Limiting Heights for Gypsum-Sheathed, Cold-Formed Steel Wall Studs." *Practice Periodical on Structural Design and Construction. ASCE*. 6 (2) 83-89.
- Miller, T.H., Peköz, T. (1993). "Behavior of Cold-Formed Steel Wall Stud Assemblies." *Journal of Structural Engineering. ASCE*. 119 (2) 641-651.
- Miller, T.H., Peköz, T. (1994). "Behavior of Gypsum-Sheathed Cold-Formed Steel Wall Studs." *Journal of Structural Engineering. ASCE*. 120 (5) 1644-1650.
- Pincus, G., Fisher, G.P. (1966). "Behavior of Diaphragm-Braced Columns and Beams." *Journal of the Structural Division. ASCE*. 92 (ST2) 323-350
- Sculpt (2002) www.sculpt.com/catalog_98/CastingMaterials/GYPSUMS/gypsum11.htm, visited in January 2002
- Serrette, R.L., Enclada, J., Juadines, M., Nguyen, H. (1997). "Static Racking Behavior of Plywood, OSB, Gypsum, and FiberBond Walls with Metal Framing." *Journal of Structural Engineering. ASCE*. 123 (8) 1079-1086.
- Simaan, A., Peköz, T. (1976). "Diaphragm Braced Members and Design of Wall Studs." *Journal of the Structural Division, ASCE*. 102 (ST1) 77-92.
- Simaan, A., Winter, G., Peköz, T. (1973). Buckling of Diaphragm-Braced Columns of Unsymmetrical Sections and Application to Wall Studs Design. Department of Structural Engineering, School of Civil and Environmental Engineering, *Cornell University*, No. 353.
- Telue, Y., Mahendran, M. (2001). "Behaviour of cold-formed steel wall frames lined with plasterboard." *Journal of Constructional Steel Research. Elsevier*. 57. 435-452.
- Yu, W.W. (2000). *Cold-Formed Steel Design*, 3rd. Ed. , John Wiley & Sons, Inc. 546-566

