PROPOSAL COVER SHEET

TO:	Jay Larson Director, Construction Standards Development American Iron and Steel Institute 3810 Sydna Street Bethlehem, PA 18017-1048
FROM:	Marge Dolly Research Projects Officer The Johns Hopkins University W-400 Wyman Park Building 3400 N. Charles Street Baltimore, MD 21218-2686
PRINCIPAL INVESTIGATOR:	Benjamin Schafer Assistant Professor Department of Civil Engineering The Johns Hopkins University Latrobe 203 3400 N. Charles Street Baltimore, MD 21218
PROJECT TITLE:	Sheathing Braced Design of Wall Studs
TYPE OF APPLICATION:	New Grant
AMOUNT REQUESTED FROM SPONSOR:	$188,727
PROPOSED STARTING DATE:	September 1, 2006

PRINCIPAL INVESTIGATOR

Benjamin Schafer
Assistant Professor
E-Mail: schafer@jhu.edu
Phone: (410) 516-7801
Fax: (410) 516-7473

Signature of Principal Investigator

JHU INSTITUTIONAL REPRESENTATIVE

Marge Dolly
Research Projects Officer
E-Mail: mdolly@jhu.edu
Phone: (410) 516-4093
Fax: (410) 516- 7775

Signature of Institutional Representative

SHEATHING BRACED DESIGN OF WALL STUDS

Abstract

The objective of this research is to develop a reliable design methodology for light-frame wall systems that properly accounts for the behavior of wall studs sheathed by similar, dissimilar, and one-sided sheathing. The developed design methodology will be straight forward in its application, and prescriptive if necessary, such that the method may be readily adopted by the American Iron and Steel Institute – Committee on Framing Standards as a key component to a new edition of the Wall Stud Standard. A combined plan of full-scale experimentation and computational modeling is proposed to develop the new methodology. The full-scale experiments will take advantage of a new testing rig that is being developed in the Johns Hopkins University Structural Testing Laboratory through National Science Foundation funding. This rig will allow for cold-formed steel walls (and members) to be loaded in compression, as well as in-plane bending and/or shear, and out-of-plane bending. The focus of the testing proposed here will be compression and a limited examination of out-of-plane bending (as would develop due to lateral load on the wall). The computational modeling will augment the testing such that (a) further wall system parameters may be studied and (b) the basic behavior of sheathing wall systems may be better understood. Two basic mechanical models have been proposed and used by the AISI standards at different times: the first assumes the sheathing provides a shear diaphragm, the second includes only the local "spring" restraint provided at fastener locations. Both testing and computational modeling are required to further develop and validate the most appropriate mechanical model for predicting the behavior of wall studs with similar, dissimilar, and one-sided sheathing. A three-year project is proposed for addressing these issues.

1. Background

The use of a variety of different sheathing types in light-frame wall systems is a common and beneficial practice. An ideal sheathing serves as bracing for C-section cold-formed steel wall studs in both compression and bending. Such bracing restricts global movements associated with torsion and translation of the cross-section, thus greatly boosting the global compressive strength through restriction of flexural and torsional-flexural buckling, and global bending strength through restriction of lateral-torsional buckling. Sheathing may also serves as a restraint on the cross-section movements associated with distortional buckling.

It is common in light-frame wall systems to have dissimilar sheathing on the two sides of the studs; e.g., plywood on the external face of a wall and gypsum on the internal face. This condition was not directly envisioned in the original research on sheathed wall systems that was conducted at Cornell in the 1970's, which instead focused on corrugated metal as the sheathing. Further, the mechanical models generally employed in the design methods used through the years relied on the sheathing being identical on the two sides. As a result, in current design, when considering dissimilar sheathing the weaker of the two must be assumed to exist on both faces. Due to the fact that the stiffness provided by the different sheathing types can be quite different, forcing the engineer to design assuming the weakest sheathing exists on both sides ignores a great deal of potentially beneficial stiffness; and may lead to assumed behavior which is wholly inconsistent with actual performance.

Two basic mechanical models have been put forth for the behavior of sheathed wall systems: the shear diaphragm model and the local spring model. The shear diaphragm model assumes the sheathing acts as a diaphragm when the studs attempt to flexurally buckle, as a result the shear stiffness of the sheathing is the most important parameter. In its typical implementation the shear diaphragm model is cumbersome, to say the least, but was used by AISI for a number of years. The local spring model assumes that most of the stud-sheathing deformation (and thus flexibility) is confined to the fastener location and thus the fastener details: diameter, pitch, etc., and stud and sheathing thickness drive the stiffness - which largely must be considered experimentally. Neither model can fully explain all the behavior observed in existing testing or computational models. In earlier work (Schafer and Hiriyur 2002, included as an appendix to this proposal) we summarized the basic state of knowledge with regard to the behavior of sheathed walls.

- 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) provide a simple way to assess elastic buckling capacity of unperforated studs with one sided and/or dissimilar sheathing.

(Schafer and Hiriyur 2002)

2. Statement of Work

The objective of this research is to develop a reliable design methodology for light-frame wall systems that properly accounts for the behavior of wall studs sheathed by similar, dissimilar, and one-sided sheathing. As detailed in the RFP the sheathing types to be considered include: plywood, OSB, gypsum board, and an unsheathed strap & block detail. The goal of the research is to provide a design method that is simple, and even completely prescriptive if necessary, for conventional details, but based on mechanically sound principles such that extrapolation to other details and configurations is reliable and possible. To achieve this goal a combined research effort of experimental and computational work is proposed. It is proposed to divide the work into 4 areas as detailed in the work plan and schedule of Table 1. The areas include: literature summary, Phase 1 testing, Phase 2 testing, and computational modeling and design methods.

Table 1 Work plan and schedule

	Year 1			Year 2			Year 3		
	Fa	Sp	Su	Fa	Sp	Su	Fa	Sp	Su
Literature summary									
summarize existing design methods									
summarize existing test data									
perform preliminary computational modeling									
finalize "control configuration" for testing									
provide predictions for "control configuration"									
Fabrication of testing rig*									
Phase 1 Testing									
Control configuration w/ varied sheathing									
specimen fabrication									
testing									
post-processing and data reduction									
comparison with design methods									
recommendations and summary									
Phase 2 Testing									
Fastener spacing, stud thickness, stud spacing									
specimen fabrication									
testing									
post-processing and data reduction									
comparison with design methods									
recommendations and summary									
Computational modeling and design methods									
modifications/development of design model									
development of computational FEA model									
validation studies									
parametric studies and prediction									
comparison of design methods									
ballots for AISI-COFS									
design examples									

* the testing rig employed in this proposal is being developed primarily through already awarded NSF funding to the PI

The literature summary includes bringing together the literature on all existing design methods as well as existing test methods. As detailed in Schafer and Hiriyur (2002) the primary test database consists of testing conducted at Cornell, first my Simaan and Peköz (1976) and later by Miller and Peköz (1993, 1994); subsequent to this testing Miller has conducted additional work, as has Mahendran in Australia, and researchers in Eastern Europe and China. This data will be drawn together to provide an initial summary of the available results and to compare the existing data to proposed and existing design methods. Further, the available literature will be used to begin examination of light-frame walls by computational methods such as finite strip and nonlinear finite element analysis. This literature summary combined with consultation with a COFS advisory committee will be used to determine an appropriate "control configuration" for testing of the wall systems with different sheathing configurations.

Due to the inherently complex nature of the stud-sheathing interaction, as well as the uniqueness of stud-track boundary conditions, it is anticipated that experimental testing will be a primary tool for exploration of the behavior of light-frame wall systems. It is proposed that the testing be broken into two phases: Phase 1 focuses on a control configuration with varied sheathing details, Phase 2 examines the impact of other potentially influential variables such as fastener spacing, and stud spacing, stud depth, stud thickness and stud material grade.

A preliminary proposal for the test matrix of Phase 1 testing is provided in Table 2. The control configuration is initially proposed to be an 8' x 8' panel with 362S162-68 (50ksi) studs spaced 12 in. on center, and sheathing as detailed in Table 2 fastened 12 in. on center. Due to a rather high expectation of variability in the performance it is anticipated that 3 tests with each sheathing configuration should be conducted which results in a minimum of 27 planned tests to explore the control configuration in similar, dissimilar, and one-sided sheathing configurations.

Table 2 Preliminary proposal for Phase 1 testing: varying sheathing types on a control wall configuration

		SIDE B				
	sheathing	1/2" plywood w/ #8 screws	7/16" OSB w/ #8 screws	1/2" gypsum w/ #6 screws	5/8" gypsum w/ #6 screws	strap & block
SIDE A	1/2" plywood w/ #8 screws					
	7/16" OSB w/ #8 screws		*no tests planned*		*no tests planned*	
	1/2" gypsum w/ #6 screws				*no tests planned*	
	5/8" gypsum w/ #6 screws				*no tests planned*	*no tests planned*
	strap and block					*no tests planned*

control configuration - for every sheathing highlighted above the following will be tested
8' x 8' panel with 362S162-68 (50ksi) with 12 in. stud spacing and 12 in. fastener spacing

Development of the full Phase 2 testing matrix will be completed subsequent to the literature summary, Phase 1 testing, and approval of the COFS advisory committee. However, an initial proposal for the test matrix is provided in Table 3. For the first study it is proposed to hold the stud spacing at 12 in., but vary the sheathing fastener spacing between 6 in. and 12 in. for plywood in a one-sided sheathing configuration and plywood with gypsum sheathing on the far side. Additionally, the stud section and grade would be varied from 362S162-33 (33 ksi) up to 800S200-97 (50ksi) as detailed in the left portion of Table 3. For the second study in Phase 2 a limited number of sections: 362S162-33 (33 ksi), 362S162-68 (50 ksi) and 800S200-97 (50 ksi), are proposed to be examined under the same sheathing configurations (see right portion of Table 3). The resulting tests provide the experimental data necessary to validate or reject design methods as they relate to section geometry and grade, fastener details, stud spacing, and sheathing type and geometry.

Table 3 Preliminary proposal for Phase 2 testing: varying fastener spacing, stud depth, stud thickness, stud grade and stud spacing around the control configuration

Study	Fastener Spacing and Stud "t"
Sheathing	(a) plywood - (b) strap & block
	(a) plywood - (b) 1/2" gypsum
Loading	axial and axial + bending

fastener spacing-->		6"		12"	
stud spacing-->		12"	24"	12"	24"
stud	**yield**				
362S162-33	33				
362S162-54	33				
362S162-68	33				
362S162-54	50				
362S162-68	50			C*	
800S200-54	50				
800S200-97	50				
C* control config., tested for all sheathing types					

Study	Stud Spacing
Sheathing	(a) plywood - (b) strap & block
	(a) plywood - (b) 1/2" gypsum
Loading	axial and axial + bending

fastener spacing-->		6"		12"	
stud spacing-->		12"	24"	12"	24"
stud	**yield**				
362S162-33	33				
362S162-54	33				
362S162-68	33				
362S162-54	50				
362S162-68	50			C*	
800S200-54	50				
800S200-97	50				
C* control config., tested for all sheathing types					

The rig that will be employed for the testing is being developed as part of the PI's National Science Foundation (NSF) CAREER grant. The rig is designed to apply axial load, in-plane shear, and major and minor axis bending independently to both ends of a cold-formed steel specimen. This is completed with 6 actuators attached to load beams above and below a specimen (4 vertical actuators at the corners of the load beam and two angled actuators at one end to apply the in-plane shear). This allows for a full investigation of cold-formed steel beam-columns (the original intent), but the rig had been envisioned from its inception to also allow for testing light-frame walls as well, as discussed in this proposal. A schematic for a simplified version of the testing rig (angled actuators are not shown since in-plane shear is not the focus here) for use in this grant is shown in Figure 1. The schematic only shows the actuators that will be connected to the reaction floor, an additional set of actuators attached to a large reaction frame enveloping the specimen are used to allow for independent control at the base of the specimen as well as the top.

Money has been secured for the envisioned testing rig, the hydraulic actuators have been donated to the University, and a post-doctoral student is beginning in September 2006 to complete the design and bring the testing rig online. Nonetheless, the rig is a relatively time consuming endeavor, what with 12 actuators to control. Given the work left to complete in design, fabrication, and control software testing cannot commence until Spring 2007, as indicated on the schedule given in Table 1.

Some limitations of the testing rig that are anticipated at this time: wall heights must be under 10 ft, wall length must be less than 10 ft, specimen depth must not be greater than 12 in.. Advantages of this testing rig include the ability to test the walls in compression and out-of-plane bending. This allows for a conservative simulation of lateral loads (i.e. as a constant bending moment). Also, the testing rig has the ability to perform follow-on work with the addition of in-plane shear to understand the inter-relationship between shear-axial-bending actions in light-frame shear walls. Such an extension is readily possible with this testing rig, but is not proposed at this time.

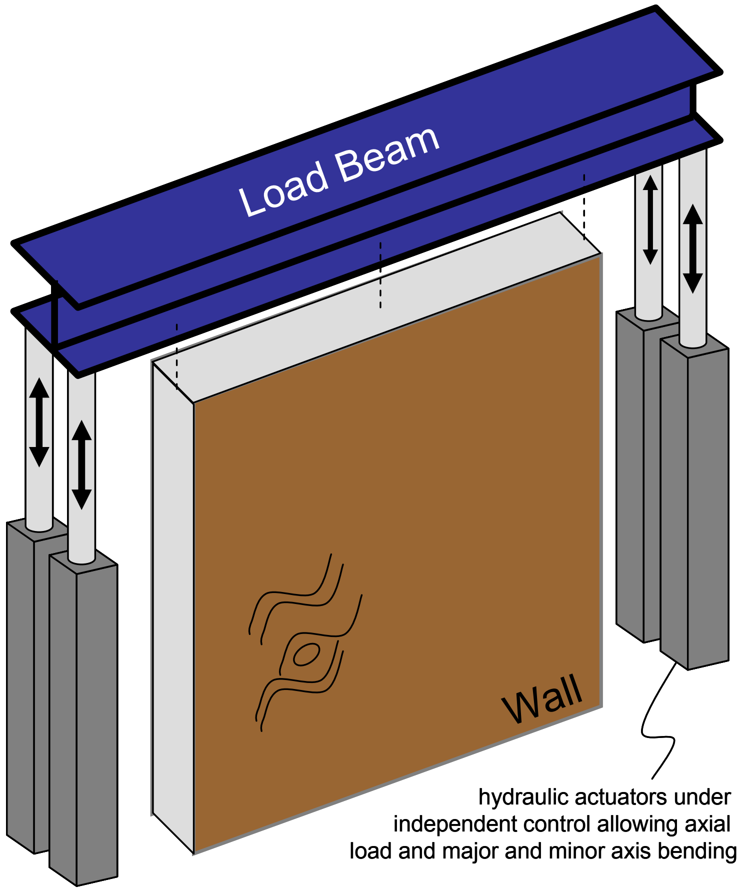

Figure 1 Schematic of loading rig for testing

Computational modeling and development of design methods is the final component of the proposed work, as envisioned in Table 1. The attached paper of Schafer and Hiriyur (2002) demonstrates how the developed design methods can be explored with even relatively simple computational models. Such studies need to be extended so that the shear diaphragm model, local stiffness model, or an alternative approach can be properly developed. The literature summary detailing all existing testing will be used as the starting place for complete development of a nonlinear finite element model of a light-frame wall systems. The developed model can be used to explore the wall systems as testing is being conducted, and thus serve to inform the instrumentation plan, testing matrix, etc. Further, once validated the computational model can be further employed for parametric studies beyond the tested configurations. The PI has had good success with a balance of experimental and computational efforts for better understanding the complicated behavior of cold-formed steel systems and then using that understanding to develop simplified design methods for engineers.

The final component of this phase of the project will be the comparison of the test data (and nonlinear FEA results as appropriate) to the available and developed design methods. In addition, formal Specification language will also be developed for use in the new edition of the AISI-COFS Wall Stud Standard. It is envisioned by the PI that the mechanics required to properly predict the strength of walls with dissimilar or one-sided sheathing will be non-trivial; therefore it is expected at this time that two design methods will be developed, (a) a simplified and prescriptive method which may be used within relatively strict bounds, and (b) a more general purpose method (or approach) that can be used far outside the bounds of the prescriptive treatment. It is expected that both methods will be of use to the engineers and manufacturers that the AISI-COFS standards help to support.

3. Work Product

Interim progress reports will be provided to the AISI-COFS at their biannual meetings, and a Final Report upon completion of the project. A sample ballot including proposed specification provisions, commentary, and a design example will be included in the final report. All test data and reports will be made available in electronic format to AISI and other researchers.

4. Budget

The budget for the project is provided in Table 4. It is important to state that if only partial funding is available this budget and the statement of work would allow us to determine the most productive level of effort and then revise this proposal to match. For instance, if only the testing was pursued, and the project initiated in the Spring of 2007, it is likely that only the first two years of funding would be required. No formal co-funding has been secured by the PI; although the costs for development of the testing rig are almost entirely offset by an existing National Science Foundation grant.

Table 4 Budget

			1-Sep-06 31-Aug-07		1-Sep-07 31-Aug-08		1-Sep-08 31-Aug-09	**Total**
Personnel								
Ben Schafer	Salary[1]	(1/2 month)	4583		4813		5053	14449
	Fringe	33%	1513	33%	1588	33%	1668	4768
Grad Student	Stipend[1]	(Full Year)	21000		22050		23153	66203
	Tuition[2]	20%	6324	20%	6577	20%	6840	19741
	Health Ins.[3]		1425		1482		1541	4448
Equipment and Supplies								
	Equipment[4]		6000		0		0	6000
	Supplies[5]		4050		5700		1500	11250
Total Direct Costs			44895		42210		39754	126859
Base for F&A Indirect Costs			31146		34151		31373	96670
Indirect Costs		64%	19933	64%	21856	64%	20079	61869
Total			**64828**		**64066**		**59833**	**$ 188,727**

(1) assumed 5% salary increase per annum
(2) PhD graduate students pay only 20% of tuition and no F&A charges
(3) No F&A is charged on health insurance for graduate students
(4) Equipment costing greater than $5K has no F&A charges
(5) Assumes donated studs, estimated number of tests, per test disposable cost of $150

Please note that the budget request includes $6000 of equipment in year 1. This cost will be used to provide the necessary modifications to the beam-column testing rig for performing the tests proposed here and to offset the large cost of development for this testing rig.

5. Personnel

Ben Schafer, Associate Professor, Department of Civil Engineering. Johns Hopkins University is the Principal Investigator for the project. His research is specialized primarily in the behavior and design of cold-formed steel structures. He serves on the AISI Committee on Specifications and the AISI Committee on Framing Standards where he has performed several research projects for AISI and authored new Specification provisions for the Standards. He also serves as President of the Light Gauge Steel Engineering Association and Chairmen of the Structural Stability Research Council Task Group on Thin-walled Construction. He has authored over 15 journal publications and numerous conference papers related to cold-formed steel behavior and design.

6. References

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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.

Schafer†, B.W., Hiriyur, B. (2002). "Analysis of Sheathed Cold-Formed Steel Wall Studs." Proceedings of the Sixteenth International Specialty Conference on Cold-Formed Steel Structures, Orlando, FL. 501-513.

Sixteenth International Specialty Conference on Cold-Formed Steel Structures
Orlando, Florida U.S.A., October 17-18, 2002

Analysis of Sheathed Cold-Formed Steel Wall Studs

Benjamin W. Schafer[1], Badri Hiriyur[2]

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 (AISI) 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 'I'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. 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 that 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

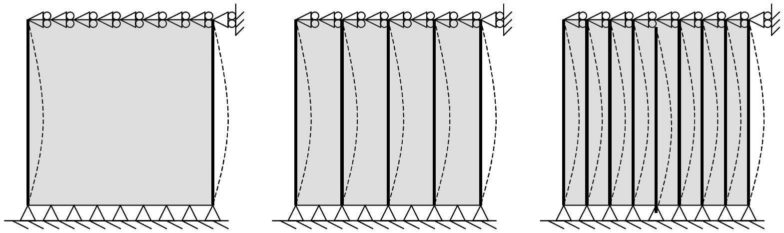
1. Assistant Professor, Johns Hopkins University, 203 Latrobe Hall, Baltimore, MD 21218, USA (schafer@jhu.edu)
2. Graduate Research Assistant, Johns Hopkins University, 304 Latrobe Hall, Baltimore, MD 21218, USA hiriyur@jhu.edu)

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.



(a) "studs" on ends only (b) "studs" 24 in. o.c. (c) "studs" 12 in. o.c.

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 ½ 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 that 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.

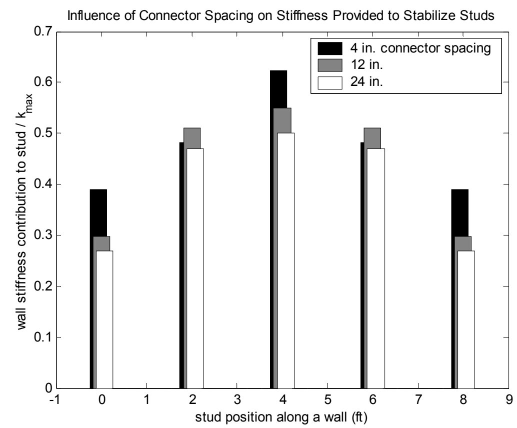


Figure 2 Influence of connector spacing on stiffness provided to stabilize studs

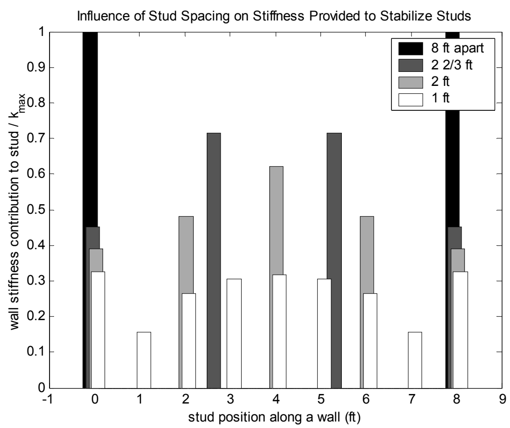


Figure 3 Influence of stud spacing on stiffness provided to stabilize studs for an 8' x 8' panel

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.

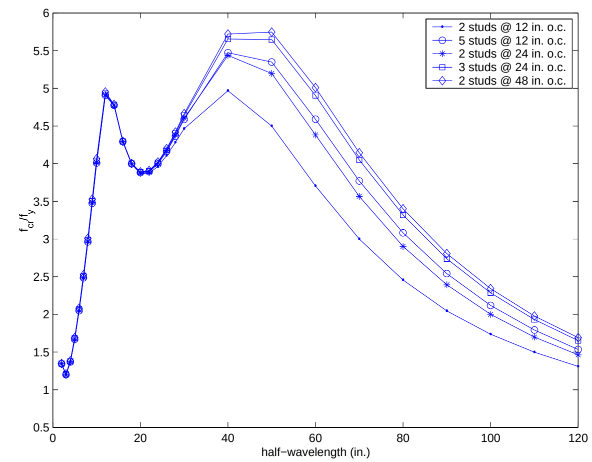


Figure 4 Influence of stud spacing on FTB for 362S162-68 studs with steel t=0.018 sheathing on both sides

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 ½ 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 in 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.

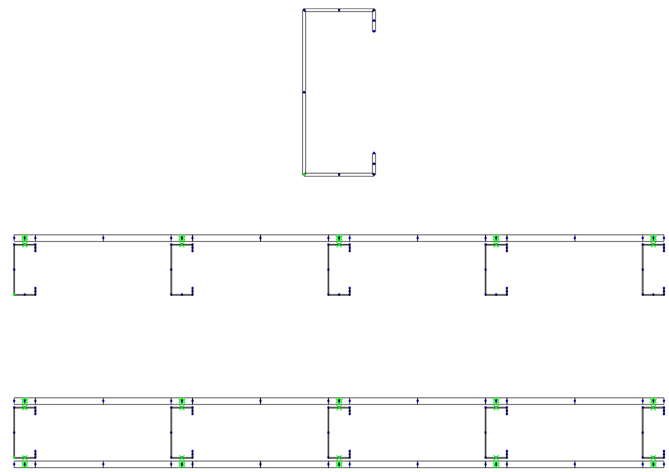


Figure 5 Finite strip models of unsheathed, sheathed on one side, and sheathed both sides

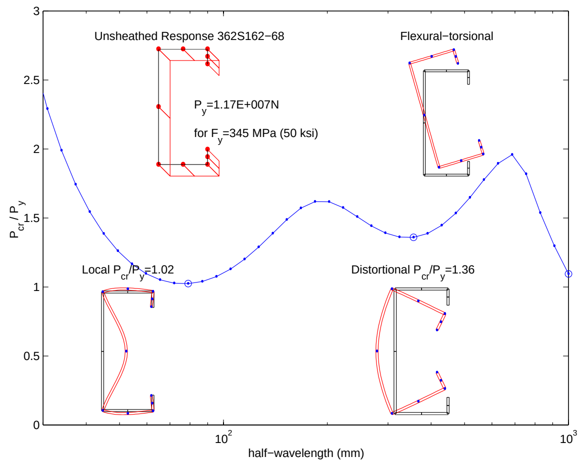


Figure 6 Unsheathed elastic buckling response of an SSMA 362S168-68

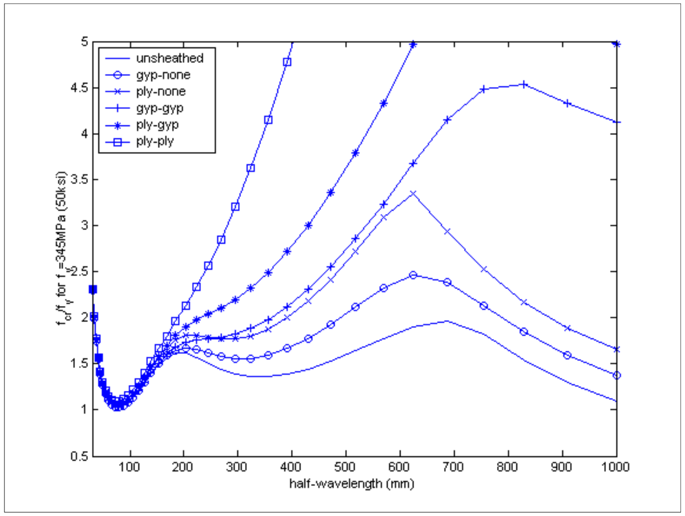


Figure 7 Comparison of elastic buckling response for different sheathing attachments and types

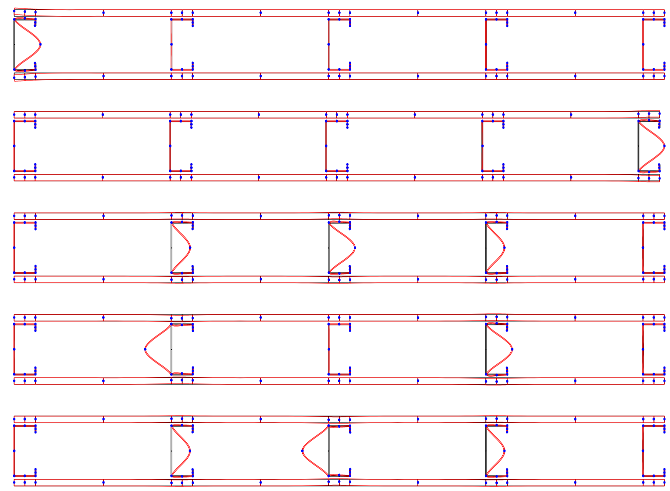


Figure 8 First 5 local modes for the ply-ply model (note f_{cr}/f_y varies from 1.09 to 1.12)

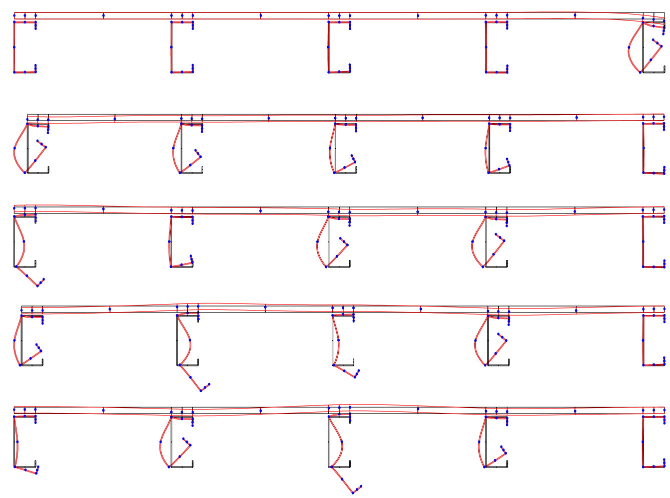


Figure 9 First 5 distortional modes for the ply-none model (note f_{cr}/f_y varies from 1.77 to 1.84)

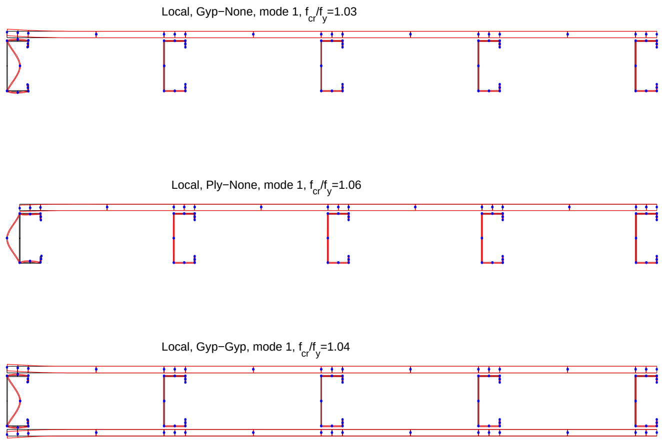


Figure 10 Role of sheathing in local buckling mode

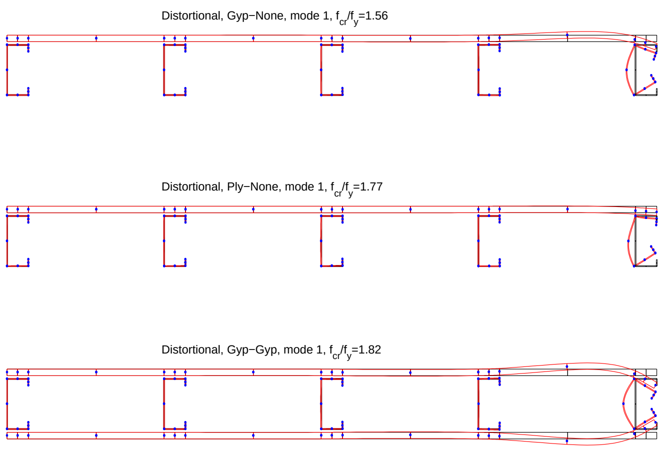


Figure 11 Role of sheathing in distortional buckling mode

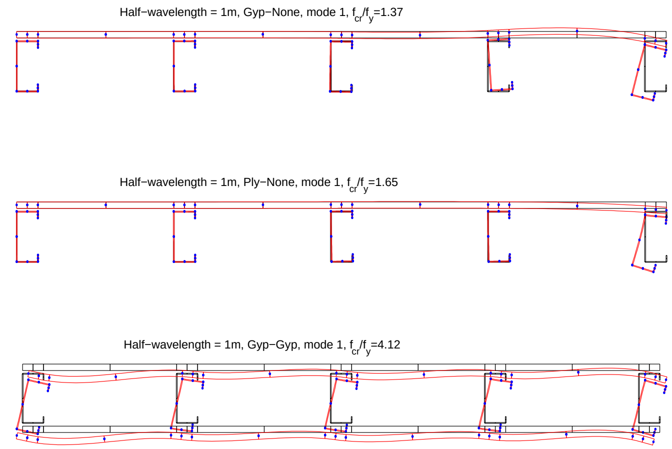


Figure 12 Role of sheathing in long wavelength buckling modes

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 <u>yet</u> 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.

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