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Tenth International Specialty Conference on Cold-formed Steel Structures
St. Louis, Missouri, U.S.A., October 23-24, 1990

STUDIES ON THE BEHAVIOR OF COLD-FORMED STEEL WALL STUD ASSEMBLIES

Thomas H. Miller[1] and Teoman Pekoz[2]

ABSTRACT

The overall behavior under axial loading of cold-formed steel wall stud assemblies is investigated. Wall stud assemblies consist of several wall studs acting as beam-columns, top and bottom tracks to restrain the ends of the studs and may include both diaphragm and discrete bracing.

Particular emphasis is placed on the study of the following: 1) the strengthening effects of wallboard sheathing, channel bridging and strap bracing, 2) effective lengths for buckling of braced and unbraced studs, 3) loading eccentricity effects on axial strength, and 4) the influence of widely-spaced rectangular perforations on strength and failure modes.

Experimental efforts include: 1) stub column tests to examine local buckling behavior, 2) individual long column testing to study the interaction of local and overall buckling, 3) cantilever connection tests to determine the stiffnesses and failure modes of typical stud-to-joist connections, 4) wallboard fastener connection tests to observe the behavior of connections between wallboard and studs using self-drilling screws, 5) flat-ended column testing to estimate loading eccentricities for wall studs and tracks bearing directly on concrete floors, and 6) wall stud assembly tests to observe the behavior of the overall system including the effects of bracing elements.

The test results are compared to predictions based on the 1986 American Iron and Steel Institute specification for the design of cold-formed steel wall studs.

BACKGROUND

Cold-formed steel wall studs are used in both curtain wall construction and for load-bearing walls in residential, commercial and industrial facilities. As the primary vertical structural elements, they have been used in buildings as high as eight stories, but more typically in structures of less than five stories.

Generally cold-rolled as lipped-channel sections, load-bearing wall studs, the subject of this study, are available in depths ranging from 2-1/2" (64 mm) to 6" (152 mm), and are formed from sheets 14 to 20 gage (normally (0.0747")(1.90 mm) to (0.0359")(0.91 mm), respectively) in thickness. Typical floor to ceiling spans range from 8' (2.4 m) to 16' (4.9 m), and studs are usually spaced 12" (305 mm), 16" (406 mm) or 24" (610 mm) on center and sheathed to form the interior and exterior walls of the building. Axial capacities of individual wall studs are normally in the 1 kip (4.5 kN) to 20 kip (89 kN) range.

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First conceived as an alternative to timber construction, light-gage steel wall studs have high strength-to-weight ratios and achieve this efficiency through their shapes as assemblages of thin, usually stiffened, elements. Steel wall studs are easily and quickly installed either on the construction site with self-drilling screws or as shop-fabricated, welded wall panels. Pre-cut perforations through the web simplify the placement of bracing and the subsequent passage of utilities (water, telephone, electrical conduits) within the wall as the construction proceeds.

INTRODUCTION

Primary emphasis in the project is to investigate the current American Iron and Steel Institute [1986] specification methods as applied to the design of wall studs through comparisons with observed behavior. Overall failure modes examined in the AISI specification are weak-axis flexural buckling, torsional-flexural buckling, and failure under combined axial load and bending. Local buckling in combination with each of these overall failures is modeled in the specification using an effective width approach. Finally, wall studs with wallboard bracing are also checked for column buckling between wallboard fasteners, overall buckling (flexural and torsional-flexural) with shear diaphragm bracing, and for shear failure of the sheathing.

The experimental investigation addresses the behavior of both individual components and the overall wall stud system as outlined below:

1) 45 stub column tests to study local buckling,
2) 48 individual long column tests to investigate the interaction of local buckling with overall buckling modes,
3) 7 stud-joist connection tests to determine the stiffnesses and failure modes of typical connection details,
4) 11 wallboard fastener connection tests to study the local behavior at the screw connections between the stud and the wallboard material,
5) 7 flat-ended column tests to estimate the loading eccentricity at the ends of studs with tracks bearing flat on a level surface, and
6) 24 wall assembly tests to investigate the overall behavior of the system with studs, tracks and bracing combined as in actual construction.

The following are limitations to the scope of the experimental effort:

1) Axial loadings, including eccentric, are studied.
2) Two lipped-channel stud types are tested :
   a) 3-5/8" (92 mm) deep, 14 gage (1.90 mm) thickness stud, and
   b) 6" (152 mm) deep, 20 gage (0.91 mm) thickness stud.
3) Maximum stud length tested is 8' (2.4 m).
4) Connections between studs and track members restraining the stud ends in the test specimens are made using self-drilling screws.
5) Stud bracing techniques which are examined include:
   a) Steel straps attached with screws to each flange at mid-height,
   b) A channel passing through the web at mid-height, and
   c) Gypsum wallboard on each side of the wall stud assembly.

The two types of studs span the range of dimensions commonly available. The 6", 20 gage stud is very susceptible to local buckling, due to the large width-to-thickness ratio of the web. The more "stocky" 3-5/8", 14 gage section, on the other hand, is much less subject to local buckling. Nominal dimensions for both sections are shown in Figure 1.

Descriptions, procedures, results and detailed comparisons between the experimental and predicted behavior for each of the test series are included in Miller [1990]. Stub columns are tested as specified in the AISI [1986] recommended Test Procedures. Long column tests are accomplished using special end fixtures, described in Mulligan [1983], and providing free rotation at each end of the stud about one axis and fixity about the other. The wall assembly tests form the focus of this paper and are described briefly here to provide additional background for the comparisons and conclusions to follow.

WALL STUD ASSEMBLY TESTS

The structural behavior of the overall wall stud system is examined in the wall assembly test series. Assemblies are tested without bracing, with mid-height channel bridging, with mid-height strap bracing or with gypsum wallboard attached to both sides of the wall. A typical setup for a five-stud wall assembly test is shown in Figure 2 (both two-stud and five-stud assemblies are tested). The wall stud assembly tests are conducted as follows:

1) To provide more realistic loading conditions, alignment of the load is accomplished geometrically rather than with a strain balancing method.

2) The loading is applied incrementally and equally to each stud in the assembly. Valves allow the loading to be discontinued for studs already failed, while continuing to load the remaining wall studs.

3) Overall racking (shear) motions and motions of the top of the assembly out of the plane of the wall are both restrained by the test fixture.

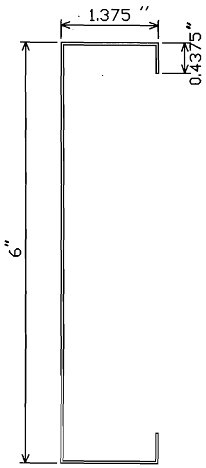
4) Metal shims are placed between the ends of the studs and the top and bottom tracks to provide more uniform bearing conditions.

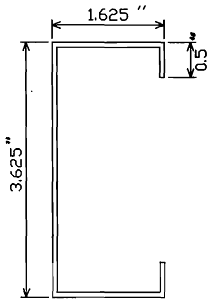
5) Horizontal strap bracing consists of 2" (51 mm) wide, 18 gage (0.0478") (1.21 mm) steel straps connected with a single #8 self-drilling screw to the center of each flange of the stud at mid-height. The two straps are attached at their ends to rigid test bay columns. The straps are installed as in typical construction practice with some slack present.

6) Mid-height channel bridging is connected to the wall studs as shown in Figure 3. The bridging is anchored at the ends to test bay columns.

7) Gypsum wallboard sheathing is attached to the flanges of the studs on both sides of the wall with #8 self-drilling, bugle head screws at 12" (305 mm) spacing on center.

8) Tracks to match the stud gage and depth are attached at each end of the stud with two #8 self-drilling screws (one centered on each flange).





3-5/8", 14 gage Wall Stud

6", 20 gage Wall Stud

Figure 1 Nominal Dimensions of Stud Sections

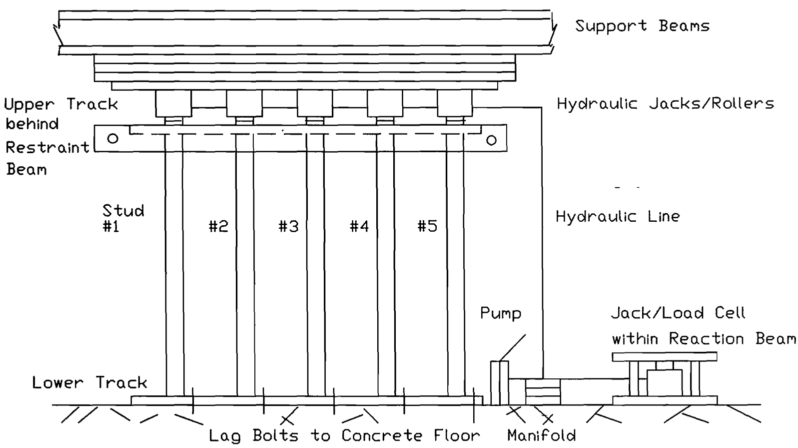


Figure 2 Typical Wall Assembly Test Setup

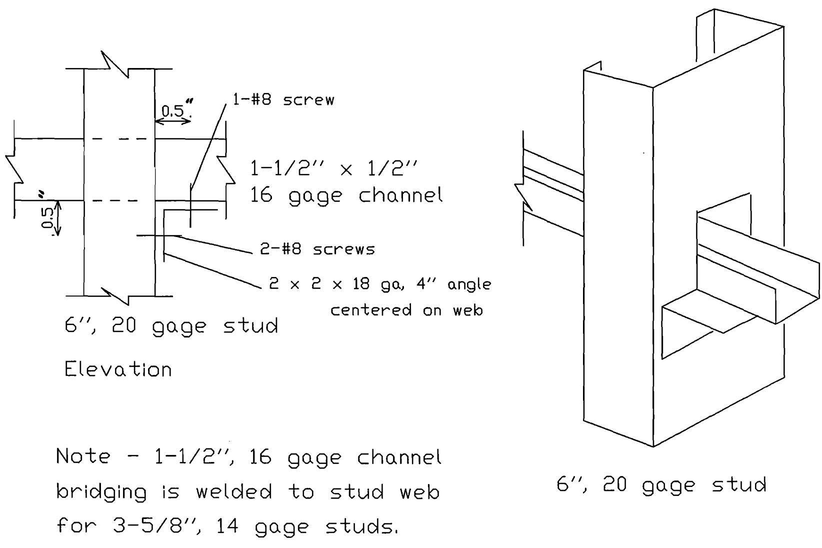


Figure 3 Mid-height Channel Bridging Details

EFFECTS OF BRACING ELEMENTS

The wall stud assembly tests are conducted to study the behavior of the overall structural system used to construct load-bearing walls from cold-formed steel wall studs. Predictions of the behavior of assemblies with several different types of bracing elements are made using the AISI [1986] specification. The performances of mid-height strap bracing, channel bridging and gypsum wallboard sheathing are evaluated. Moreover, attention is given to the AISI treatment of sheathed wall studs and the strengths of these wall assemblies with studs at different spacings.

HORIZONTAL STRAP BRACING AND CHANNEL BRIDGING

In Table 1, the experimental strengths of unbraced, unperforated wall stud assemblies are compared to those of unperforated assemblies with strap bracing and those with channel bridging at mid-height. The following conclusions can be drawn from this table:

1) The experimental failure loads for similar unperforated studs with mid-height strap bracing and with mid-height channel bridging are nearly equal (within 5 percent). Thus, in terms of axial load capacity for the 6" and 3-5/8" studs in the wall assembly configuration, the two types of bracing are of roughly equivalent effectiveness.

2) Strap bracing and channel bridging are both effective in raising the failure loads for 6" and for 3-5/8" studs above those for the unbraced cases. Increases of at least 25 percent for the 6" studs and at least 60 percent for the 3-5/8" studs are observed experimentally.

Table 1 also includes predicted strengths based on the AISI [1986] specification. Comparisons of these predictions with experimental results for the unsheathed wall assembly tests are outlined below:

1) Accurate or slightly conservative predictions for unperforated studs are made using the assumed effective lengths factors (indicated in Table 1) if one accounts for a small eccentricity (such as that due to loading at the gross section centroid versus the centroid of the effective section at failure).

2) The AISI [1986] specification accurately predicts the torsional-flexural failure mode of the wall assemblies without sheathing.

3) Strength predictions based on a concentric loading (through the centroid of the effective section at failure) for unperforated 3-5/8" wall stud assemblies actually loaded at the gross centroid are quite accurate. However, they are 25% unconservative for 6" stud walls.

Additional experimental observations from the tests of assemblies without wallboard include the following:

1) Twist is very effectively restrained at mid-height by the strap bracing. The channel bridging passing through the web also restrains the mid-height twist, but may not be quite as effective, particularly for the 6" studs with flanges further from the channel restraint.

Table 1 Bracing Effectiveness Comparisons for Unperforated Two-Stud Wall Assembly Tests

Test#	Bracing Type		Exp. Ult. Load (kips)	Pred. Ult. Load (kips) (a)	Pred. Ult. Load (kips) (b)	Exp/ Pred (a)	Exp/ Pred (b)
6", 20 gage Studs:							
WT13	None	(1)	3.00	3.66	2.80	0.82	1.07
		(2)	2.88	3.71	2.82	0.78	1.02
WT14	Mid-height Straps	(1)	3.78	5.49	3.87	0.69	0.98
		(2)	4.18	5.43	3.76	0.77	1.11
WT24	Mid-height Channel	(1)	3.84	5.32	3.84	0.72	1.00
		(2)	3.81	5.24	3.80	0.73	1.00
3-5/8", 14 gage Studs:							
WT15	None	(1)	9.54	9.98	9.98	0.96	0.96
		(2)	10.40	9.83	9.83	1.06	1.06
WT16	Mid-height Straps	(1)	17.38	17.13	16.11	1.01	1.08
		(2)	17.38	17.19	16.18	1.01	1.07
WT17	Mid-height Channel	(1)	16.71	17.22	16.18	0.97	1.03
		(2)	16.71	17.19	16.18	0.97	1.03

Notes:
(1) and (2) indicate stud numbers (left to right) in an assembly. Experimental and predicted ultimate loads are given for each of the two studs in a wall assembly.
(3) All studs are 8' (2.4 m) high, with top end pinned about the strong axis and loaded through a roller welded to a plate resting on the upper track. The bottom track is lag bolted to the concrete floor of the test bay.
(4) Predicted failure loads are calculated using the 1986 edition of the AISI Specification with the following assumptions for effective length factors for overall L = 8' (2.4 m) :

K_x=0.7 (for all tests)
K_y=0.5 (when unbraced) K_y=0.35 (when braced)
K_t=0.7 (when unbraced) K_t=0.44 (when braced)

(5) Prediction (a) assumes a concentric load through the centroid of the effective section at the AISI predicted failure load. (b) assumes an eccentric load through the gross centroid. In all of the tests, the loading plate at the top was geometrically centered on the gross centroid.
(6) All failures and failure predictions are torsional-flexural.

2) Deflections in the plane of the wall at mid-height are effectively restrained at mid-height by either the straps or the channel bridging.

3) Straps do not provide much restraint to deflection out of the plane of the wall. The flexurally stiffer channel provides more resistance.

4) Local buckling of the web under the load (and occasionally at the bottom track) has been observed for many of the 6" stud wall assemblies. Although this local buckling does not lead to immediate failure of the stud, it may create loading eccentricities (by shifting the effective centroid away from the web) which do lower the eventual failure load.

5) Torsional-flexural failures are observed in these assemblies due to the pinned condition about the strong axis of the section at the top of the wall.

DIAPHRAGM BRACING EFFECTS

Briefly, the evaluation of the testing of wall stud assemblies with diaphragm bracing (gypsum wallboard sheathing) is summarized:

In the current AISI [1986] specification, a shear diaphragm model based on the work of Simaan [1973] is used to predict the strength of sheathed wall studs. Using this method, increased stud spacing increases the overall shear rigidity of the wall by forcing more sheathing material to be deformed per stud. Greater stud spacing results in increased strength predictions for both the overall diaphragm-braced buckling modes and for the mode involving a shear failure of the sheathing itself. Only the failure mode consisting of buckling between fasteners is independent of stud spacing.

In this project, a limited number of tests on gypsum wallboard-braced studs are performed at different stud spacings. The strength of wallboard-braced studs is not observed to be very sensitive to stud spacing. In addition, contrary to the shear diaphragm model, the deformations of gypsum wallboard panels (in tension) are seen to be very localized at the fastener locations. Moreover, the failure mode is not accurately predicted by the AISI [1986] specification. The shear failure of the sheathing predicted would occur at the ends of the studs where shear deformations are greatest. However, the observed failures occur closer to mid-height, and usually initiate at a perforation location. They involve a torsional-flexural failure (with local buckling concentrated at the perforation) accompanied by an unzipping of the wallboard fasteners and cracking of the wallboard due to the overall deformations. Screw shear failures, screws pulled through the wallboard and wallboard tearing failures (at edges) are observed.

Although the limited test data are insufficient to develop an improved behavioral model for sheathed wall studs, the observed behavior from the wall assembly tests does support a review of the shear diaphragm model as applied to gypsum wallboard sheathing. Other materials may be adequately modeled by the shear diaphragm approach, where the deformations are not highly localized at the fasteners. Moreover, the current approach does provide conservative predictions for the 12" (305 mm) and 24" (610 mm) stud spacings tested in this project if one assumes a concentric loading (through the centroid of the effective section at failure) and accounts for the presence of perforations.

EFFECT OF LOADING ECCENTRICITY ON AXIAL STRENGTH

In the series of tests on individual long columns with well-defined end conditions, a strong influence of loading eccentricity on wall stud strength is observed.

First, a parameter study is conducted examining the sensitivity of predicted strength based on the AISI [1986] specification to loading eccentricity for each of the long column tests. Several examples from this study are shown in Figures 4 and 5.

In Figure 4, the effect of eccentricity about the strong axis on the AISI-predicted failure load is presented for a typical long column test of the 3-5/8", 14 gage stud section. The positions of the gross section centroid and effective section centroid at failure for a concentric axial load are shown, and the two centroids coincide due to symmetry about the strong axis. The plot is drawn over the full range of eccentricities within the outside dimensions of the cross-section. Note the symmetry of the plot reflecting the section symmetry about the strong axis, and the rather gradual decay in strength with eccentricity.

Figure 5 shows the much stronger sensitivity of the AISI-predicted failure load to eccentricities about the weak axis. The failure load is plotted against the initial eccentricity of the load about the gross section centroid. Note that the positions of the gross section centroid and effective section centroid at failure for a concentric axial load do not coincide along this axis.

Conclusions based on the parameter study follow:

1) Predicted strengths based on the AISI [1986] specification for unperforated long columns with loads applied at the gross centroid (treated as eccentric loadings) can be as much as 30 percent below concentrically-loaded strengths (assumed loaded at the effective section centroid at failure).

2) Moreover, eccentricities about the weak axis (through the gross centroid) as small as 0.1" (2.5 mm) can reduce the AISI-predicted failure load by as much as 40 percent below the concentric (effective centroid) failure load. Eccentricities of this magnitude are certainly possible in both testing with geometric alignment and in field construction.

Next, the predicted strengths are compared to the experimental results from the long column tests.

Figure 6 shows graphically the unconservative nature of concentric strength predictions (with loading assumed at the effective section centroid at failure) and the improved predictions resulting from the incorporation of a small loading eccentricity. In this plot, the unperforated long column test results for both stud types are compared to predicted strengths over a range of dimensionless slenderness ratios. The slenderness ratio, $(F_y/F_e)^{1/2}$, is determined at the elastic flexural buckling stress, F_e calculated using the mean yield stress, F_y, and section properties for the long column tests. Unperforated stub column test results are also plotted, and fall quite close to the predicted column curves as expected.

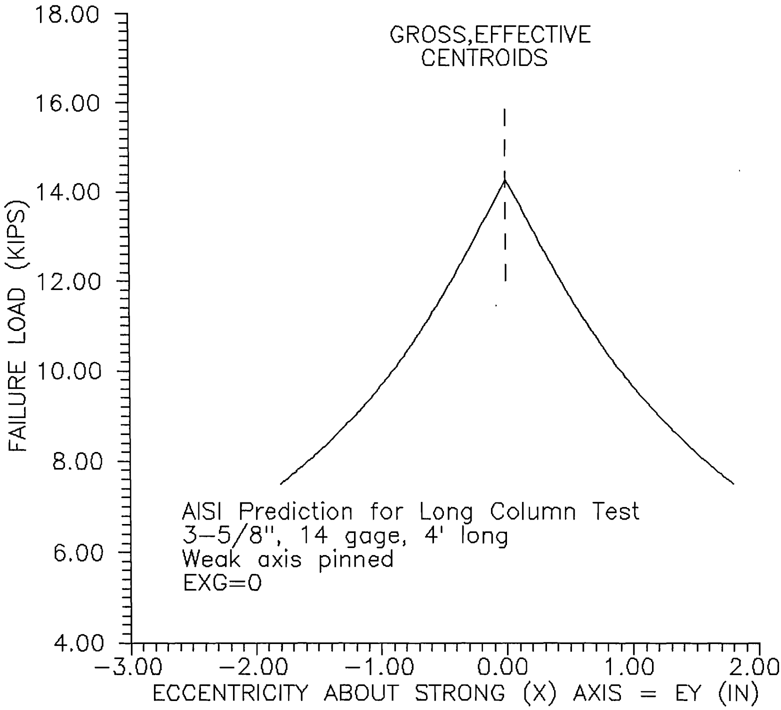


Figure 4 Effect of Eccentricity about the Strong Axis on AISI-Predicted Strength for 3-5/8", 14 gage Long Column Test

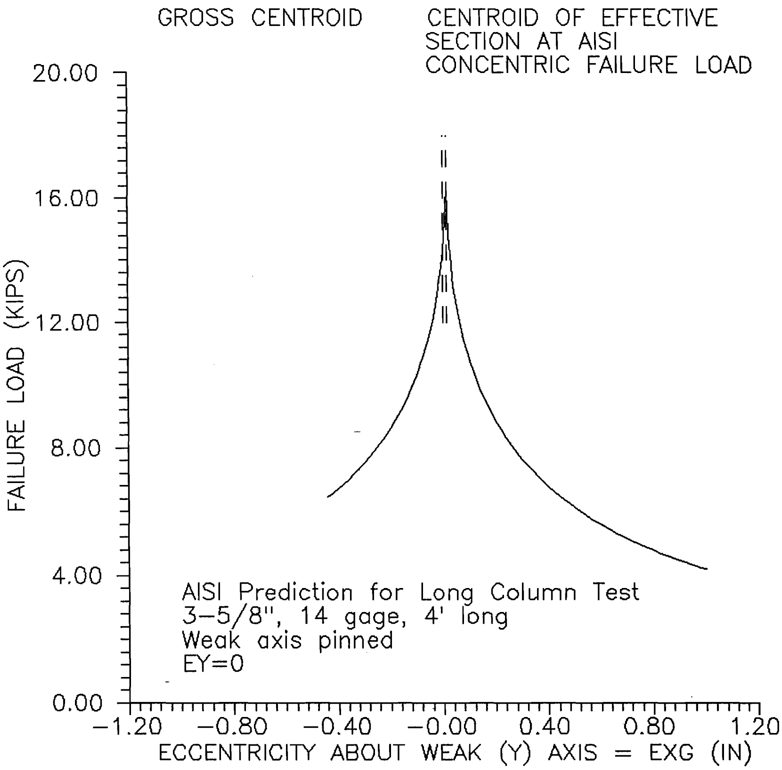


Figure 5 Effect of Eccentricity about the Weak Axis on AISI-Predicted Strength for 3-5/8", 14 gage Long Column Test

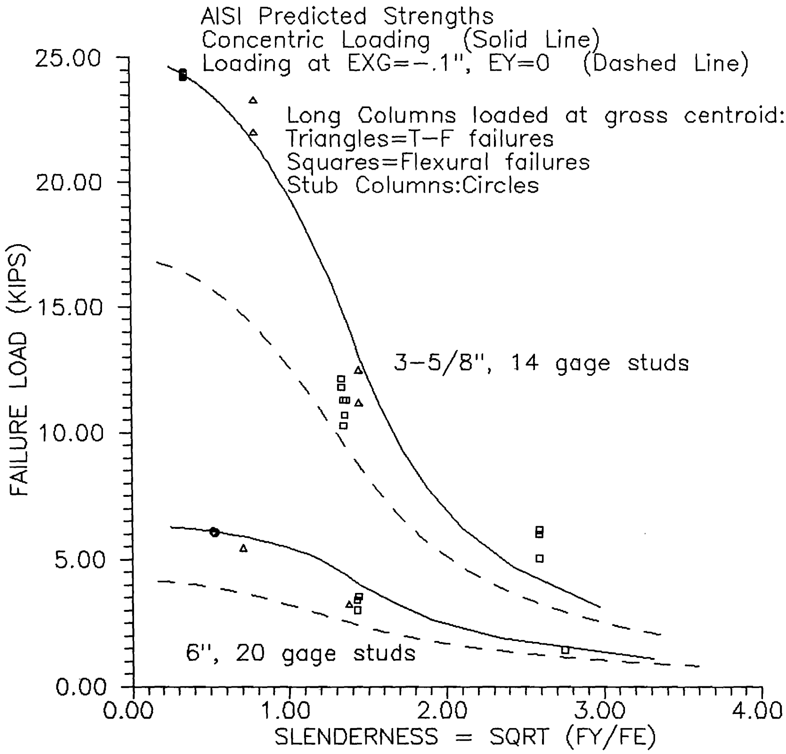


Figure 6 **Comparison of Experimental and Predicted Strengths for Unperforated Long Columns**

Comparisons between the predictions and observed behavior from the long column tests yield the following conclusions/observations:

1) Long column test failure modes are accurately predicted by the AISI [1986] specification.

2) Accounting for the shift in effective centroid due to local buckling using the AISI interaction equation provides improved, but sometimes still unconservative, predictions for unperforated long columns loaded through the centroid of the gross section. Moreover, these unconservative predictions are due in large part to unintentional small loading eccentricities.

3) Loading eccentricities appear to be more significant than and may mask perforation effects on the strength of long columns.

Finally, the cantilever tests of stud-joist connections further emphasize the importance of considering loading eccentricities, and show that very large eccentricities (up to 4-6" (102-152 mm)) can result from joist moments transferred into the studs. This places the effective axial loading well outside the stud section dimensions and greatly reduces the strength.

FLAT-ENDED STUD ECCENTRICITIES

A series of flat-ended column tests is conducted to simulate field conditions where top and bottom tracks bear directly on concrete floors. The primary objective is to obtain an estimate of the loading eccentricity for this case.

The flat-ended columns are tested in the same hydraulic testing machine as the individual long columns. However, the special fixture to allow free end rotation about one axis is not used in these tests. Instead, the columns are fitted with short (12" (305 mm) long) sections of track (matching the stud depth and gage) at both ends and simply placed flat-ended between the fixed heads of the machine. Positioning consists of rough geometric centering in the test machine and leveling of the column to insure verticality.

Flat-ended column test predictions based on the AISI [1986] specification are evaluated using several different assumptions for end fixity. Effective lengths for buckling based on complete end fixity, pinned conditions at the ends, as well as the AISC [1980] recommended effective lengths for design are all studied.

The conclusions based on the limited data of this pilot test series follow:

1) Concentric predictions (assuming a loading at the centroid of the effective section at failure) are unconservative for flat-ended columns (with tracks bearing directly on a flat surface) if complete end fixity is assumed. Conservative predictions are made if an eccentric loading at the gross section centroid is assumed.

2) Effective lengths adopted from the AISC [1980] specification provide a reasonably good prediction for flat-ended studs assuming a concentric loading and ignoring the perforations. Some additional conservatism can be provided by including the effect of perforations or some small loading eccentricity.

3) The most conservative assumptions for effective lengths (assuming pinned ends) result in quite conservative flat-ended column predictions even if a concentric loading is assumed.

EFFECT OF PERFORATIONS

The analytical treatment of rectangular perforations is not specifically provided in the AISI [1986] specification. Only circular perforations, with narrowly defined geometric parameters, are covered. The Test Procedures section of the AISI [1986] design manual does present a methodology for using stub column test results for both perforated and unperforated sections in determining the effective section properties at various stress levels up to ultimate. Similarly, the RMI [1985] specification also details a stub column approach for including local buckling effects and determining effective cross-sectional areas.

When stub column results are not available for a section with rectangular web perforations, an alternative, analytical method is proposed. An extension of the work of Davis and Yu [1972], the method uses a simple modification of the unified effective width approach already applied to cold-formed steel sections and models the web of the wall stud as two unstiffened elements, one on either side of the perforation. Essentially, this assumes that for local buckling calculations, the perforation extends over the length of the stud.

In the effective section calculations, the effective widths of the unstiffened elements are determined at the stress level of interest. For concentric loadings, (and bending about the weak-axis, which also causes uniform web compression) the web strips are treated using the AISI [1986] specification as uniformly-compressed, unstiffened elements. For bending about the strong-axis of the section, the web strips are treated as unstiffened elements subjected to a stress gradient.

The detailed treatment of web perforations in the wall stud strength predictions is outlined below:

1) The "unstiffened strip method" is used to determine the effective area of the web for concentric axial load strength determinations.

2) Combined bending and axial load is modeled using the AISI [1986] interaction equations. The moment capacities are determined using two unstiffened strips to model the web.

3) Weak-axis flexural buckling stresses and torsional-flexural buckling stresses are based on gross section properties.

Eccentrically-loaded wall studs, as defined by the AISI [1986] specification, are loaded at a location other than the effective section centroid at failure. Using the "unstiffened strip approach", the location of the effective section centroid is readily determined since the effective section is well defined. However, if stub column test results are used, the locations of the effective section centroids in both the stub column test and in the wall stud strength prediction are unknown.

A simple and conservative approximation of the location of the effective section centroid at any stress level can be made by assuming that only the web is ineffective:

$$e = x_g * ((A_g/A_e)-1)),$$

where, e = distance between gross and effective section centroids,

x_g = distance between gross section centroid and web centerline,

A_g = gross cross-sectional area,

A_e = area of effective section at stress level of interest.

This approximation is exact for the majority of wall stud sections available from various manufacturers, as most studs are proportioned so that the flanges and lips are fully effective according to the AISI specification. For those sections which have partially effective flanges or lip stiffeners, the method is conservative in overestimating the eccentricity.

Observations of the effects of widely-spaced, rectangular perforations on the strength of the wall stud sections tested in the project follow:

1) The effective width equations of the AISI specification generally provide good predictions of the failure loads for unperforated stub columns of the two section types studied. They are slightly conservative (4%) for the 3-5/8", 14 gage section and about 8% unconservative for the 6", 20 gage stud.

2) A rectangular perforation (1-1/2" x 2-1/2") (38 mm x 64 mm) decreases the experimental strength 10 percent for 3-5/8", 14 gage stub columns, but has almost no effect on the strength of 6", 20 gage stub columns.

3) Predictions for perforated stub columns assuming unstiffened strips on either side of the perforation are generally conservative (10%) for both stud types.

4) For the wall assemblies with strap bracing, a reduction in the experimental failure load (torsional-flexural) appears to be produced by the perforations. Failures occur at the perforations, and decreases in strength of 16 percent from the unperforated stud experimental strengths are seen for both the 3-5/8" 14 gage studs and 6", 20 gage studs. Predicted concentric strength reductions due to the perforations of 23% and 15% for the 3-5/8" and 6" studs, respectively, result from the application of the "unstiffened strip method".

CONCLUSIONS

Based on the present study, the following recommendations for the design of cold-formed steel wall studs are made:

1) Gypsum Wallboard-Braced Studs - The current shear diaphragm-based approach should be limited to applications where stud spacings are no greater than 24" (610 mm) . Moreover, the use of steel channel bridging and strap bracing should be encouraged wherever possible.

2) Effective Lengths for Buckling of Flat-Ended Wall Studs - The following effective lengths are recommended for flat-ended wall studs.

Unbraced: $K_x=K_y=K_t= 0.65$
Braced at Mid-Height: $K_x= 0.65$ $K_y=K_t= 0.4$

Note : All of the above effective length factors are applied to the overall height of the wall stud.

3) Eccentrically Loaded Studs - The strength of lipped-channel, pin-ended wall studs is extremely sensitive to loading eccentricities about the weak-axis as small as 0.1" (2.5 mm). Sources of potential loading eccentricity include: the shift in effective centroid due to local buckling, non-uniform bearing conditions at the ends of the studs, misalignment during construction, initial imperfections in the studs and moments introduced through connections to joist framing. In actuality, all wall studs are eccentrically loaded, and some eccentricity should be used in all design calculations because of its significant effect. The total eccentricity assumed in a given design should be based on construction details (tracks, shims, joists), construction practice (alignment specification), and the extent of local buckling expected in the studs.

4) Wall Studs with Widely-Spaced Rectangular Perforations - Perforation effects should be included in the design, especially if a perforation is located at the center of an unbraced length. Perforations can be treated using either the AISI [1986] or RMI [1985] stub column approaches if stub column tests have been performed. The approach assuming unstiffened strips on either side of the perforation appears to be a conservative method if stub column data are not available.

APPENDIX--REFERENCES

AISC, American Institute of Steel Construction [1980], Manual of Steel Construction, Eighth Edition, Chicago.

AISI, American Iron and Steel Institute [1986], Cold-Formed Steel Design Manual, Washington, D.C.

Davis, C.S. and Yu, W. [1972], "The Structural Behavior of Cold-Formed Steel Members with Perforated Elements," Department of Civil Engineering, University of Missouri-Rolla, Rolla, Missouri, May.

Miller, T. H. [1990], "Behavior of Cold-Formed Steel Wall Stud Assemblies Subject to Eccentric Axial Loads," Ph.D. thesis, Cornell University, Ithaca, N.Y., January.

Mulligan, G.P. [1983], "The Influence of Local Buckling on the Structural Behavior of Singly-Symmetric Cold-Formed Steel Columns," Ph.D. thesis, Cornell University, Ithaca, N.Y., May.

RMI, Rack Manufacturers Institute [1985], "Specification for the Design, Testing and Utilization of Industrial Steel Storage Racks," Pittsburgh, PA.

Simaan, A. [1973], "Buckling of Diaphragm-Braced Columns of Unsymmetrical Sections and Application to Wall Stud Design," Ph.D. thesis, Cornell University, Ithaca, N.Y., May.

APPENDIX--NOTATION

The following symbols are used in this paper:

A_e = area of effective section at stress level of interest,
A_g = area of gross cross-section,
EXG = eccentricity of loading in the x-direction about the weak (y) axis through the centroid of the gross section,
EY = eccentricity of loading in the y-direction about the strong (x) axis,
e = distance between the centroids of the gross and effective sections,
F_e = elastic flexural buckling stress,
F_y = yield stress of the steel,
K_t = effective length factor for torsion,
K_x = effective length factor for bending about the x-axis,
K_y = effective length factor for bending about the y-axis,
L = overall length of the wall stud,
t = section thickness,
x_g = distance between gross section centroid and web centerline.