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ROTATIONAL RESTRAINT AND DISTORTIONAL BUCKLING IN COLD-FORMED STEEL FRAMING SYSTEMS

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Abstract

A series of cantilever tests on joist/stud-sheathing assemblies were conducted to determine the rotational restraint that sheathing provides to the flange of a cold-formed steel floor joist or stud. This rotational restraint, characterized by the stiffness, k_{ϕ}, can partially or fully retard the distortional buckling mode. Distortional buckling, common in conventional cold-formed steel members such as the lipped channel, is characterized by significant rotation of the flange at the flange/web juncture. Cantilever tests were conducted for different thicknesses, depths, and flange widths, of the cold-formed steel member, and for two fastener types, and three sheathing types: plywood, oriented strand board (OSB), and gypsum board. The testing demonstrated that the rotational stiffness may be decomposed into two parts: connector, and sheathing. The connector stiffness is due to the rotation of the fastener in the flange of the cold-formed steel member, and is most significantly influenced by the thickness of the cold-formed steel member itself. The sheathing stiffness is due to bending of the sheathing itself, and may be highly variable. The results formed the basis for a new design method adopted in American standards (AISI-S210-10) for incorporating restraint into design strength predictions for the distortional buckling mode.

Key words: Distortional buckling. Cold-formed steel. Rotational restraint. Cantilever test.

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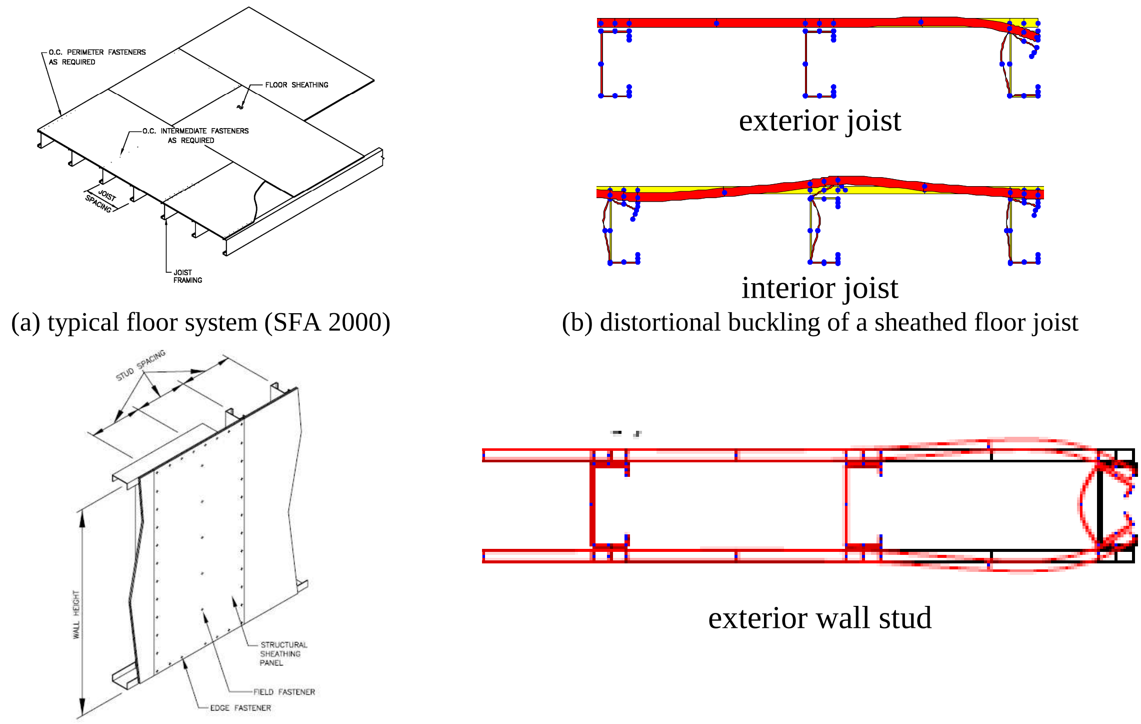
1 INTRODUCTION

Cold-formed steel members typically suffer from three potential modes of instability: local, distortional, and global buckling. The North American Specification for the Design of Cold-Formed Steel Members (AISI-S100 2007) has followed other global standards in recently adopting design methods for distortional buckling limit states. One unique aspect of the AISI-S100 adopted method is the explicit inclusion of a rotational restraint (k_ϕ) that may preclude or retard distortional buckling. Examples of such rotational restraint are common in cold-formed steel framing, for instance the sheathing in a typical floor system (Figure 1a) or wall stud (Figure 1c). It may be shown (Schafer et al. 2007) that most systems have a rotational restraint that will partially retard distortional buckling; but do not provide a high enough k_ϕ to preclude distortional buckling altogether. Therefore, it is necessary to quantify the available rotational restraint against distortional buckling so that it may be used in design. This rotational restraint is investigated here using experimental methods.



(a) typical floor system (SFA 2000)

(b) distortional buckling of a sheathed floor joist

(c) typical wall stud (SFA 2000)

(d) distortional buckling of sheathed wall stud

Figure 1: Distortional buckling of a floor system, sheathing provides rotational restraint

In the early 1980's in the United States the Metal Building Manufacturer's Association (MBMA) examined available rotational restraint in their systems: purlins fastened through insulation to metal deck. MBMA developed the "F" test (MRI 1981, Hausler and Pabers 1973) which later was formalized as a test standard AISI TS-1-02 (AISI 2002). The test uses a small cantilevered segment of panel with a purlin attached, and pulls on the free flange of the purlin such that a moment and rotation is induced at the panel-purlin connection. This test provides an estimate of the panel-purlin rotational restraint (k_ϕ) and was intended to provide reliable estimates of the bracing provided by the panel to restrict lateral-torsional buckling of the purlin, and to restrict rolling of the Z-section as it attempts to respond in its principal plane. LaBoube summarized the available MBMA testing with metal panels and demonstrated the critical role of purlin thickness on the available rotational restraint (LaBoube 1986). The important role of thickness in the conducted tests (as opposed to purlin depth, deck thickness, insulation, etc.) suggests that the panel-purlin connection flexibility, and local flange deformations at the connection, played a dominant role.

The restraint provided by metal deck was specifically examined in the context of distortional buckling (Yu 2005, Yu and Schafer 2007). The existing MBMA tests were found to provide a conservative prediction of developed restraint and in the commentary to the AISI Specification (AISI-S100 2007) suggested for use as the k_ϕ to partially restrict distortional buckling. However, no equivalent data for cold-formed steel framing systems is available. The tests reported herein use the "F" or "cantilever" test to examine cold-formed steel framing systems: cold-formed steel members sheathed with oriented strand board (OSB), plywood, or gypsum board as might exist in cold-formed steel floors, walls, and ceilings. Beyond this paper, detailed test reports are available (Schafer et al 2007, Guan and Schafer 2008).

2 TEST SETUP AND STIFFNESS DETERMINATION

The basic test setup for measuring the sheathing rotational restraint is shown in Figure 2. The setup is similar to that used in AISI TS-1-02, (AISI 2002) but has been modified and expanded to reflect the specific needs of this testing program. Based on the load measured from the load cell at the end of the hydraulic actuator, P, the moment, per unit width, imposed on the connection is calculated as

$$M = (P/w)h_o \tag{1}$$

This definition for M is exact for the undeformed state, but becomes approximate for higher vertical deflections, Δ_v. In the conducted tests the width ‘w’ is 54 in. (1372 mm). The total rotation, θ_2, of the sheathing-connector-joist assembly considers only Δ_v and h_o where:

$$\theta_2 = \tan^{-1}(\Delta_v/h_o) \tag{2}$$

Based on these definitions for M and θ_2 the rotational stiffness is defined as

$$k_{\phi 2} = M/\theta_2 \tag{3}$$

where $k_{\phi 2}$ has units of (force•distance/length)/radian or simply force/radian.

(a) line drawing of test setup (b) photo during test of plywood sheathed specimen

Figure 2: Test setup for rotational restraint measurement

2.1 Component stiffness measurements

Total rotation, θ_2, is the simplest and most direct measurement of the rotation; however it is worth recognizing that this rotation derives from several factors, as illustrated in Figure 3. The rotation of the assembly consists of rotation θ_w of the sheathing (wood), and rotation θ_c at the connector – in addition, since measurement is made at the free flange and not directly at the connection, rotation θ_s due to bending of the steel joist and rotation θ_L due to the loading apparatus (straps, turnbuckle. etc) also occur. Figure 3 depicts these component rotations along with overall definitions of the rotation, θ, θ_1, and θ_2 as shown.

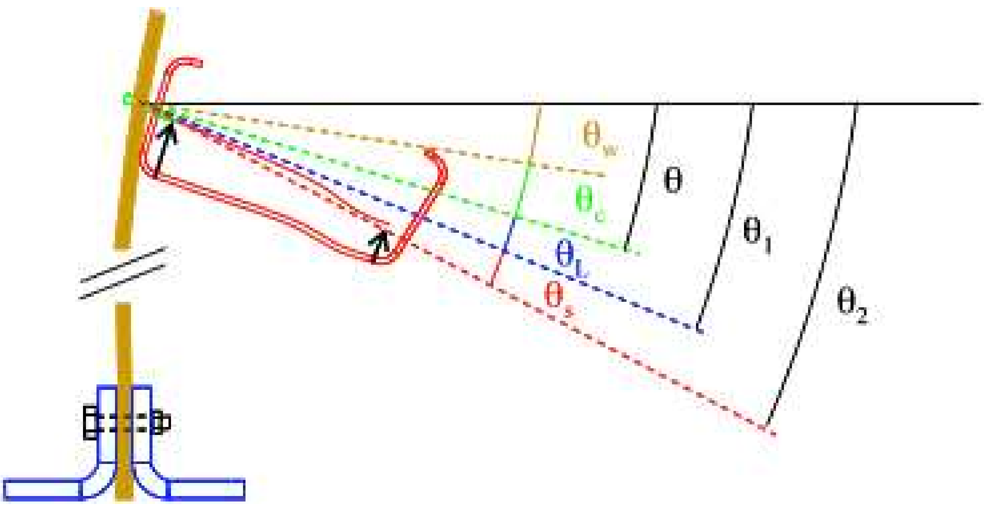


Figure 3: Breakdown of rotation at the joist-sheathing juncture

The sheathing deformation, θ_w, may be removed from the total rotation by assuming a simple beam theory model for the sheathing and measuring the horizontal displacement, Δ_h. The lateral deflection at the point of moment application in the linear elastic range assuming standard beam theory for the sheathing deformation is:

$$\Delta_h = ML^2/(2EI_w) \quad (4)$$

and the rotation at the point of moment application is

$$\theta_w(\text{at } \Delta_h) = ML/(EI_w) \quad (5)$$

Using Eq. 4 and 5 the sheathing rotation is defined in terms of the measured horizontal displacement as

$$\theta_w = 2\Delta_h/L$$

The rotational stiffness of the sheathing (wood) may be determined via:

$$k_{\phi w} = M/\theta_w = M/(2\Delta_h/L) \quad (6)$$

The simplest definition of the connector rotation, θ_{c2}, assumes that only the sheathing rotation is removed from the total rotation, i.e.:

$$\theta_{c2} = \theta_2 - \theta_w \quad (7)$$

which results in a connector stiffness of:

$$k_{\phi c2} = M/\theta_{c2} = M/(\theta_2 - \theta_w) \quad (8)$$

This definition for connector stiffness lumps the testing rig flexibility (θ_L) and joist bending (θ_S) into the connection flexibility. More involved models for the connector stiffness, where θ_L and θ_s are removed, are explored in Schafer et al. 2007 and Guan and Schafer 2008. In particular, Guan and Schafer 2008 demonstrate that θ_S can be significant and worthy of separation in deeper, thinner, sections.

The developed component stiffness model is consistent with a spring in series model (Figure 4) and thus the rotational stiffnesses are related by:

$$k_{\phi 2}=(1/(1/k_{\phi c2} + 1/k_{\phi w})) \qquad (9)$$

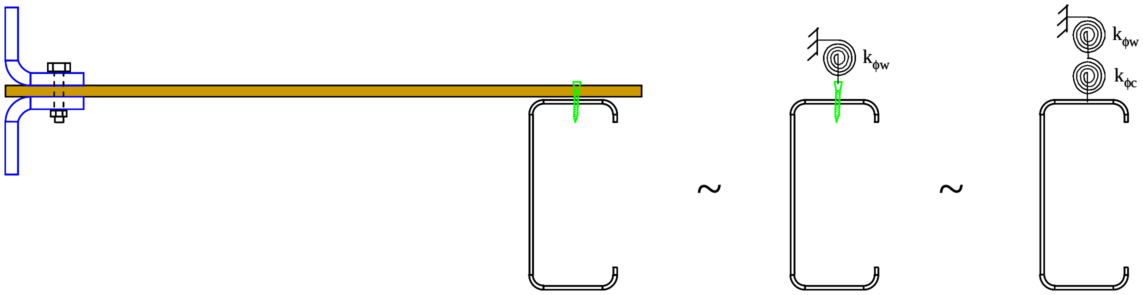


Figure 4: Illustration of rotational springs for sheathing and fastener

2.2 Test parameters

A total of 36 tests were initially conducted to investigate the rotational restraint of sheathed cold-formed steel assemblies – the parameters studied are summarized in Table 1. The majority of the testing was conducted with Plywood sheathing. Specific details of the specimens, fasteners, construction (including flaws) are provided in Schafer et al. (2007).

Table 1: Parameters of conducted rotational restraint tests

conducted tests												
Sheathing -->	Plywood					OSB		Gypsum				
Joist Spacing (L) -->	12"				24"	24"		12"		24"		
Fastener # -->	6		10		6	6	10	6	10	6	10
Fastener Spacing -->	6"	12"	6"	12"	12"	12"	12"	12"	12"	12"	12"
362S162-33		1						1			
362S162-68		1						1			
800S200-54	2	4		1	3	1	1	1	1	1	1
800S250-54		1		1							
800S200-97			1	4			1		2		
1200S200-54		2				1					
1200S200-97				2			1				

(joist designation in SSMA 2001 nomenclature, www.ssma.com: e.g., 362S162-33, web depth=3.62 in. (92 mm), flange width = 1.62 in. (41 = 0.033 in. (0.84 mm), 1" = 25.4mm)

3 EXPERIMENTAL RESULTS

To provide an overview of the conducted experiments, results for tests on an 800S200-54 joist with #6 fasteners spaced 12 in. (25 mm) on-center attached to OSB, plywood, and gypsum sheathing (24 in. (610 mm) long, 54 in. (1372 mm) wide) are provided in Figure 5. The stiffness (k_ϕ) results (slope of the M-θ lines) indicates that OSB provides the most robust response, plywood can undergo significant rotation, but is much more flexible than OSB, and gypsum provides a stiff response, but with very low rotation capacity.

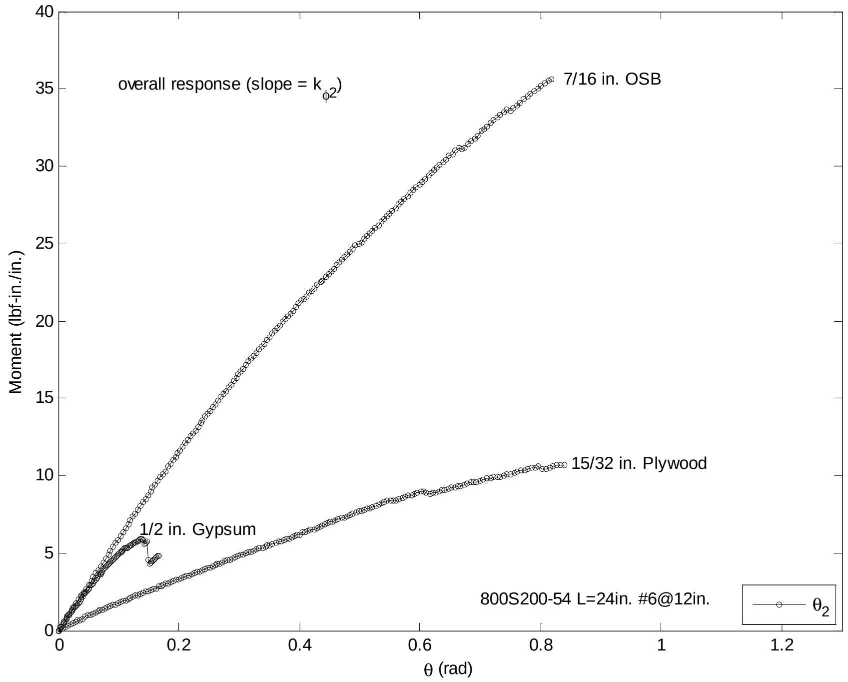


Figure 5: Typical moment-rotation results for overall stiffness (1 lbf = 4.448 N)

The measured rotational restraint from the conducted tests ($k_{\varphi2}$) is summarized in Table 2. While Eq. (3) provides the fundamental relationship for determining $k_{\phi2}$ significant details remain in selection of $k_{\phi2}$ from the recorded data. The raw results (P and Δ_v) are sampled at 10 Hz, resulting in approximately 18,000 points per test. A 100-point moving average is used to down-sample the data. The initial rotational restraint $k_{\varphi2}$ is found by linear regression on the M-θ_2 curve for M<0.4M_{peak}, where M_{peak} is the maximum recorded moment in the test. Note, M_{peak} is the moment at 6 in. of Δ_v displacement or the failure moment, whichever is less. Selection of 0.4M_{peak} was based on trial-and-error with a goal of providing repeatable, reliable stiffness measurements within rotational ranges of practical interest. The 6 in. limit reflects the maximum stroke of the actuator, and also results in a reasonable rotation limit for the tested specimens.

Table 2: Rotational restraint $k_{\phi 2}$ from tests (1lbf = 4.45 N)

$k_{\phi 2}$ (lbf-in./in./rad)											
Sheathing -->	Plywood					OSB		Gypsum			
Joist Spacing (L) -->	12"				24"	24"		12"		24"	
Fastener # -->	6		10		6	6	10	6	10	6	10
Fastener Spacing -->	6"	12"	6"	12"	12"	12"	12"	12"	12"	12"	12"
362S162-33		40						75			
362S162-68		42						94			
800S200-54	41	34		33	18	57	44	76	60	53	58
800S250-54		53		43							
800S200-97			47	44			66		58		
1200S200-54		34				44					
1200S200-97				59			75				

(1) average values reported when multiple tests conducted

The results of Table 2 may be used directly in the distortional buckling design method of AISI-S100-07 (e.g., see k_ϕ in Eq. 3.1.4-10 in the Standard). However, as discussed further below the $k_{\varphi 2}$ results for gypsum board are somewhat misleading as little moment or rotation can be sustained in these connections before failure. Separation of the rotational restraint into the sheathing and connector component stiffness provides significantly more insight.

The sheathing and connector component stiffness was determined for the same three tests of Figure 5 using Eq.'s (6) and (8). The results are provided in Figure 6. Figure 6 shows that the difference between the plywood sheathed specimens and the OSB and gypsum sheathed specimens is due to the plywood itself, not the connection stiffness. In fact, the connection stiffness for all three specimens ($k_{\phi c2}$), which have nominally the same joist dimension, joist thickness, fastener size, and fastener spacing are quite similar (same slope) despite having different attached sheathing types (OSB, plywood, and gypsum board).

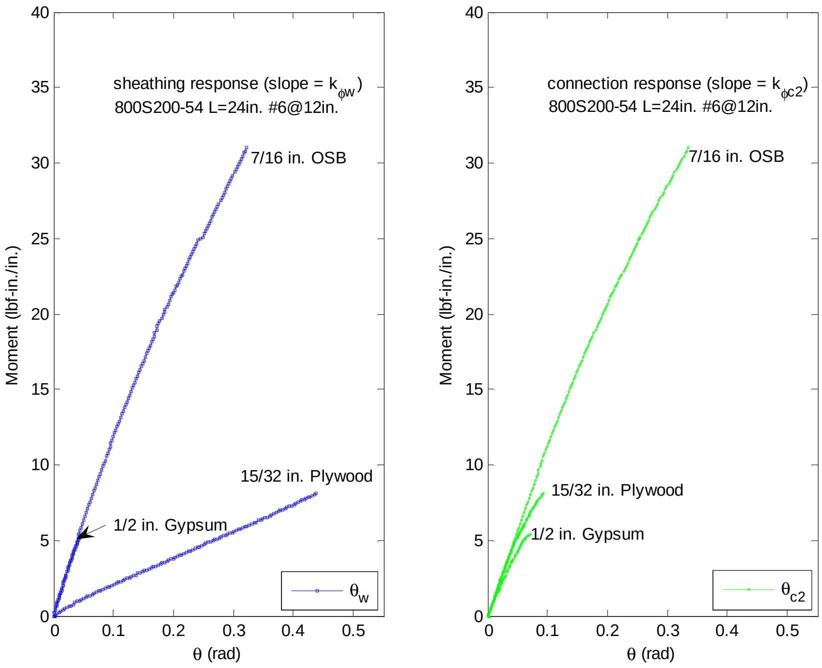


Figure 6: Typical moment-rotation results for sheathing & connection stiffness (1lbf = 4.45N)

Results for the component stiffness in all of the conducted tests are provided in Table 3(a) for the sheathing and (b) for the connector. Full experimental results: all construction details and full P-Δ and M-θ response for every test are provided in Schafer et al. (2007).

Table 3: Average measured component rotational stiffness (1lbf = 4.448 N, 1 in. = 25.4 mm)

(a) sheathing average stiffness results

$k_{\phi w}$ (lbf-in./in./rad)											
Sheathing -->	Plywood					OSB		Gypsum			
Cantilever (L) -->	12"				24"	24"		12"		24"	
Fastener # -->	6		10		6	6	10	6	10	6	10
Fastener Spacing -->	6"	12"	6"	12"	12"	12"	12"	12"	12"	12"	12"
362S162-33		78						295			
362S162-68		72						300			
800S200-54	63	56		51	21	117	101	295	285	128	138
800S250-54		98		66							
800S200-97			58	59			112		378		
1200S200-54		60				89					
1200S200-97				82			122				

(b) connection average stiffness results

$k_{\phi c2}$ (lbf-in./in./rad)											
Sheathing -->	Plywood					OSB		Gypsum			
Cantilever (L) -->	12"				24"	24"		12"		24"	
Fastener # -->	6		10		6	6	10	6	10	6	10
Fastener Spacing -->	6"	12"	6"	12"	12"	12"	12"	12"	12"	12"	12"
362S162-33		81						100			
362S162-68		102						137			
800S200-54	116	109		97	137	113	77	103	77	91	99
800S250-54		116		124							
800S200-97			269	167			159		144		
1200S200-54		78				85					
1200S200-97				215			195				

The highly flexible response for plywood sheathed assemblies, as provided in Figure 5, is somewhat misleading, as in actual practice significant variability was observed in the response. For example, at the same cantilever length (L) the sheathing stiffness varies considerably for the plywood sheathed specimens, as given in Table 3(a). As will be shown later, the variability in the connection stiffness is definable, and largely a function of joist thickness. Table 3(b) also provides an examination of fastener type (#6 vs. #10) and fastener spacing (6 in. vs. 12 in.) in plywood sheathed specimens.

The OSB sheathed results of Table 3 show the enhanced sheathing stiffness typified in Figure 5, and that the connector stiffness values are largely similar to the plywood sheathed specimens; particular for a given joist thickness. In one of the OSB sheathed specimens a pull-through failure was observed, thus indicating the possibility of this failure mode in OSB. However, the observed pull-through failure did not occur until approximately 0.5 rad (29 deg.), which is well beyond the anticipated rotational demands in distortional buckling up to and including collapse. Note, none of the plywood specimens failed in this manner up through Δ_v=6in.

The response of the joists sheathed with gypsum was significantly different than the OSB or plywood sheathed specimens: at low rotations the fasteners pulled-through the gypsum board and failed the specimens (Figure 7). Pull-through occurred at a mean rotation of 0.1 rad (6 deg.) but in one nominally similar specimen (across only 8 specimens) occurred at 0.01 rad (0.6 deg.), and order of magnitude lower. The observed behavior suggests that while gypsum board may be able to resist distortional buckling of walls and ceilings at service loads, as it has significant initial stiffness, it is likely unreliable at ultimate strength levels as it has inadequate rotation capacity.

(a) large separation between joist and gypsum board (b) pull-through failure and fracture of gypsum board

Figure 7: Response of 800S200-54 joist sheathed to gypsum board with #10s @ 12 in. (305 mm)

4 ANALYSIS OF STIFFNESS AND PROPOSED DESIGN MODEL

The $k_{\phi 2}$ results of Table 2 may be used directly in the distortional buckling design method of AISI-S100-07; however, only a limited number of cases would be covered with such a direct experimental approach. In this section a proposed design method is presented for extending the application of the experimental results. The proposed method relies on the separation of $k_{\phi 2}$ into the sheathing rotational stiffness, $k_{\phi w}$, and the connection rotational stiffness, $k_{\phi c2}$. The sheathing rotational stiffness relies on a simple mechanical model of the bending engaged during distortional buckling of a framing system and industry standard values are recommended for the material properties. The connector rotational stiffness relies on the empirical observation that thickness of the cold-formed steel member is the dominant parameter in determining the connection stiffness. The design method explained herein was adopted in American standards (AISI-S210 2010) and design examples demonstrating its practical use are also available (Schafer 2008).

4.1 Sheathing stiffness compared with industry tables values

With Eq. 4 the displacement, Δ_h, and the load, P, may be used to back-calculate the experimentally observed sheathing bending rigidity EI_w. The observed EI_w are compared to industry provided values in Table 2. The results indicate that the measured values are generally consistent with industry provided values, but industry provided values are typically more conservative than the average measured response.

The relationship between the bending rigidity (EI_w) and the sheathing rotational stiffness ($k_{\phi w}$) is depicted in Figure 8 where it is shown to be a function of joist spacing and location. The expressions for interior and exterior joists given in Figure 8 are recommended for design.

Table 4: Sheathing bending rigidity

(a) sheathing stiffness determined from testing

	EI_w (lbf-in.^{2}/ft of panel width)				
	mean	C.O.V.	n	min	max
Plywood*	9000	0.3	27	4000	14000
OSB*	31000	0.1	5	26000	35000
Gypsum	41000	0.1	7	37000	43000

*stress perpindicular to strength axis

(b) sheathing stiffness available from standards

	EI_w (lbf-in.^{2}/ft of panel width)	
	mean	source
32/16 Plywood*	8100	APA, Panel Design Spec. (2004)
24/16 OSB*	16000	APA, Panel Design Spec. (2004)
32/16 OSB*	25000	APA, Panel Design Spec. (2004)
Gypsum (min)	18000	Gypsum Assoc, GA-235-01, (2001)
Gypsum (max)	48000	Gypsum Assoc, GA-235-01, (2001)

*stress perpindicular to strength axis

(1 lbf-in.^{2}/ft = 9.476 kN-mm^2/m)

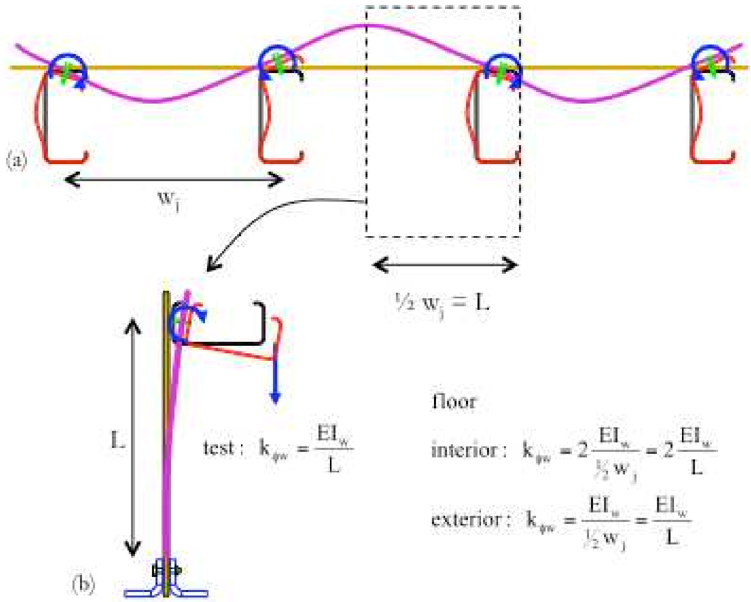


Figure 8: Sheathing stiffness for interior and exterior joists and comparison to conducted tests

4.2 Connection stiffness and design simplification

The average connection stiffness, measured in the testing reported here, is provided in Table 3. The two parameters found to have the most influence on the connection rotational stiffness are joist thickness and fastener spacing (see Schafer et al. 2007 for additional analysis and discussion on this point). From a practical standpoint industry has shown a reluctance to move towards fastener spacing less than 12 in. (305 mm) on center, so the focus of the results are on the 12 in. (305 mm) on-center tests. For those tests, joist thickness is varied from 0.033 in. (0.84 mm) to 0.097 in. (2.46 mm) and the resulting measured connection rotational stiffness is reported in Figure 11.

Figure 9 shows that an empirical relationship exists between the joist thickness and the connection rotational stiffness, largely independent of sheathing type (sheathing influence is captured through $k_{\phi w}$). The empirical relationship in Imperial units is:

$$k_{\phi c2} = 0.00035Et^2 + 75 \tag{10}$$

where: $k_{\phi w}$ = sheathing rotational stiffness in units of lbf-in./in. width / radian, E = 29,500,000 psi, and t = nominal joist thickness in inches. Eq. 10 has no mechanical basis, and is merely a mathematical convenience. To date, the simple dimensionally consistent mechanical models that have been investigated (see both Schafer et al. 2007 and Guan and Schafer 2008) lead to poor correlations with the data; thus the above has been developed.

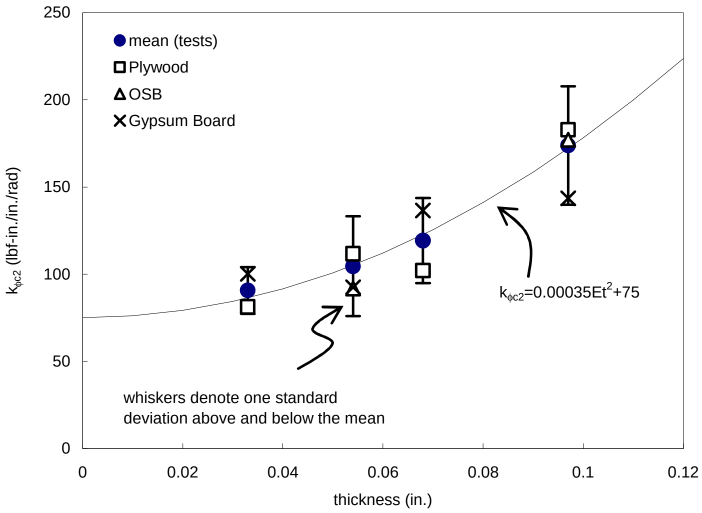


Figure 9: Connection rotational stiffness as a function of joist thickness

Comparison of the design method with the measured total rotational stiffness is provided in Table 5. Use of average tested values for the sheathing material leads to relatively high standard deviations for the plywood, but given the variability of plywood this seems acceptable. Simplification of the connection stiffness to values based on the thickness of the joist increases the variability of the predictive method for OSB and gypsum, but leaves the average test-to-predicted values within acceptable ranges. Use of Eq. 10 for $k_{\phi c2}$ is statistically equivalent to using the average tabled values for connection stiffness. Use of design values for the sheathing bending rigidity (i.e., based on APA or GA tables) introduces conservatism and increases variability of the predictive method, but is nonetheless recommended for design practice at this time.

Table 5: Test-to-predicted ratio for total rotational stiffness $k_{\phi 2}$

		plywood		OSB		gypsum board	
$k_{\phi w}$	**$k_{\phi c2}$**	**ave.**	**st. dev.**	**ave.**	**st. dev.**	**ave.**	**st. dev.**
Table 4a	tested values	0.97	0.21	1.00	0.06	1.00	0.02
Table 4a	thickness only*	0.98	0.22	0.97	0.14	0.92	0.16
Table 4a	Eq. 10	0.98	0.22	0.97	0.14	0.92	0.16
Table 4b, min values	Eq. 10	1.03	0.23	1.47	0.26	1.30	0.21

* connection rotational stiffness is determined from the average tested values for a given joist thickness

5 COMPLEMENTARY TESTING AND ANALYSIS

In addition to the main body of experimental work described herein and used to support the newly adopted rotational restraint provisions in American Standards (AISI-S210 2010), additional complementary testing and analysis has recently been completed. This complementary works covers: additional testing to support an ongoing project related to sheathing braced design of wall studs, additional testing to further explore the role of fastener spacing on the results, and additional analysis to investigate second-order effects present in the testing and its subsequent analysis.

5.1 Supplementary testing related to sheathing braced design of walls

The first three authors of this paper are currently involved in a multi-year project to examine the design of wall studs braced by sheathing (see, e.g Vieira and Schafer 2009). A component of the sheathing resistance derives from the rotational restraint, as discussed in this paper. Thus, a series of tests focused on the specific details used in that testing have recently been completed. The studs are 362S162-68's (SSMA 2001 nomenclature) throughout. Two types of sheathing are employed: OSB (7/16 in., rated 24/16, exposure 1) and gypsum (½ in. Sheetrock). Number 6 screws (Simpson #6 x 1 5/8'') were used to connect to the gypsum boards and number 8 screws (Simpson #8 x 1 15/16'') to connect to the OSB boards. The boards were kept in an environmental chamber for seven days at a temperature of 20 C and 65% humidity. The studs were connected to the boards every 12 in. o.c.. The results are provided in Table 6.

Table 6: Complementary stiffness tests on 362S162-68 studs (Stiffness reported in units of lbf-in./in./rad.)

Test	$k_{\phi 2}$	$k_{\phi w}$	$k_{\phi c2}$	$k_{\phi\text{-}10\%Mmax}$
BBB-GYP-12-6-6-01	68	283	90	77
BBB-GYP-12-6-6-03	78	-	-	67
BBB-GYP-12-6-6-04	79	255	115	79
BBB-GYP-12-6-6-05	58	193	82	52
average	70.8	243.7	95.7	68.9
COV	0.14	0.19	0.18	0.18

Test	$k_{\phi 2}$	$k_{\phi w}$	$k_{\phi c2}$	$k_{\phi\text{-}10\%Mmax}$
BBB-OSB-12-8-6-02	81	288	113	103
BBB-OSB-12-8-6-06	64	201	95	85
BBB-OSB-12-8-6-07	67	212	98	86
BBB-OSB-12-8-6-08	69	243	97	91
average	70.3	236.0	100.8	91.4
COV	0.11	0.17	0.08	0.09

For the connection stiffness, Eq. 10 predicts a $k_{\phi c2}$ of 123 lbf-in./in./rad, which may be compared with the mean measured values of 96 lbf-in./in./rad in the gypsum and 101 lbf-in./in./rad in the OSB, as reported in Table 6. Noting that the standard deviation on the original data was 24 lbf-in./in./rad the measured connection stiffness in these tests is approximately 1 standard deviation below the average values in the tests of Table 3b.

For the sheathing stiffness $k_{\phi w}$ is determined following Figure 8 and the appropriate industry standard EI_w values, i.e., Table 4b. For gypsum, $k_{\phi w}$ is expected to be between 125 lbf-in./in./rad and 333 lbf-in./in./rad (from min and max values reported by GA 2001) which may be compared with an average measured $k_{\phi w}$ of 243 lbf-in./in./rad. The limited rotational capacity of gypsum sheathed specimens was again noted. For OSB, $k_{\phi w}$ is expected to be 111 lbf-in./in./rad for stress perpendicular to strength axis (as-tested here) and 541 lbf-in./in./rad for stress parallel to the strength axis, which may be compared with an average measured $k_{\phi w}$ of 236 lbf-in./in./rad. The APA (2004) values are again shown to provide a conservative estimate.

A surprising result from Table 6 is that the mean sheathing stiffness was slightly higher for gypsum than OSB. However, this conclusion is based in part on how the stiffness is determined; in particular, the criteria that the moment must be less than 40%M_{peak} leads to a much smaller θ range for determining $k_{\phi w}$ in the gypsum specimens, than in the OSB specimens. As Table 6 notes, if one instead uses 10%M_{peak}, the OSB values are higher than the gypsum board. Thus, one must take some care in interpreting the data and it is noted that the nonlinearity of the actual M-θ response can indeed influence the results.

5.2 Fastener Spacing

The results of Table 3b indicate that tighter fastener spacing will increase the connection stiffness, but to what extent is somewhat inconclusive. The restraint increased from 167 to 269 lbf-in./in./rad (61%) when the spacing was halved from 12 in., down to 6 in. in a 0.097 in. thick joist with plywood sheathing, but only from 109 to 116 lbf-in./in./rad (6%) in a similarly configured 0.054 in. joist. Although there is reluctance to move away from 12 in. spacing in floors, in walls tighter spacing is common. Therefore, a means to handle tighter fastener spacing, and the increased rotational restraint developed, is desired.

A small set of additional tests were conducted to further examine fastener spacing (Guan and Schafer 2008). The same joist dimension that lead to the small 6% increase in fastener spacing (800S200-54) was re-examined, this time with OSB sheathing (variability in the plywood sheathing stiffness can make it difficult to isolate the exact influence of the connection stiffness across specimens) and at fastener spacing of 12, 6 and 3 in.. The resulting $k_{\phi c2}$ stiffness is 102, 198 and 246 lbf-in./in./rad respectively. This is remarkably well approximated by the simple expression:

$$\frac{k_{\phi c}^{*}}{k_{\phi c}} = -2\left(\frac{s^{*}}{s}\right) + 3 \qquad (11)$$

where $k_{\phi c}^{*}$ is the connection rotational restraint at any spacing s^{*} which is less than 12 in. (305 mm), and $k_{\phi c}$ is the rotational restraint at 12 in. spacing (i.e., Eq. 10). Though Eq. 11 is linear, it recognizes that the increase for tighter fastener spacing does not explicitly follow a “tributary width” rule and in fact there is diminishing increase in the strength for tighter spacing (in Eq. 11 the maximum increase above the 12 in. value is 3 times).

5.3 Second-order effects

As illustrated in Figure 10 a number of potential nonlinear (second-order) effects exist in the cantilever tests conducted herein, including P-Δ moments and the loading direction. Analysis was conducted in Guan and Schafer (2008) to determine the extent to which these effects influence the determination of the rotational restraint.

Decomposition of the sheathing stiffness from the connection stiffness relies on the small angle approximation that rotation of the wood θ_w equals $2\Delta_h/L$. In Guan and Schafer (2008) this approximation is shown to be accurate in the necessary rotation ranges when compared with large deflection analyses performed on the cross-sections tested.

In the cantilever tests, the moment arm which drives the rotational demand (Figure 10d) is assumed to always be $P \cdot h_o$. This ignores the small rotation of the section (as the load remains vertical). It is shown in Guan and Schafer (2008), for the studied sections, that this assumption results in an average stiffness error of 2.5%. The simplification is warranted.

As reported herein the second-order $P\text{-}\Delta_h$ moment (Figure 10c) delivered to the sheathing is ignored in determining the sheathing rotation. It is shown in Guan and Schafer (2008), that this assumption results in an average stiffness error of 7.5%, though not insignificant, it is still believed that ignoring the $P\text{-}\Delta_h$ demand is a worthy simplification.

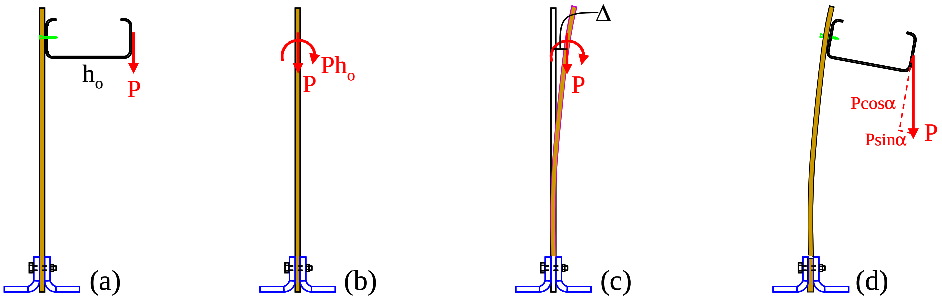


Figure 10: Large deformation effects in cantilever test (a) test (b) sheathing with forces (c) P-Δ moment (d) load direction

6 FUTURE WORK

A number of additional tests and basic research could be performed to improve upon the design method and findings presented herein. *Additional testing* at 0.033 in. (0.84 mm) thickness joists or thinner and 0.097 in. (2.46 mm) or thicker are needed. A series of tests focused on fastener location are needed to account for the practical situation of joists which have two pieces of sheathing attached to the same flange in the location of a sheathing joint. Durability over time is also an unknown. *Additional modeling and/or analytical studies* are needed, for example, further study of the impact of the distortional buckling half-wavelength, and associated sine wave deformations vs. the conservative uniform deformations employed in the testing is needed. *Additional studies to improve design method*: a design methodology that incorporates strength, likely through (a) determining a rotation demand then (b) determining the forces developed in such a demand and finally (c) checking those forces against pull-through failure is needed. Initial work in this direction is provided in Guan and Schafer (2008). Analytical work is needed to determine the rotation demand in distortional buckling. Further consideration of reliability (beyond Guan and Schafer 2008) is also needed. While the preceding list represents a significant amount of additional work, the findings presented herein provide support for a workable design method that immediately allows floors and other framing systems to benefit from the rotational restraint provided by sheathing.

7 CONCLUSIONS

Distortional buckling of cold-formed steel members in bending can be significantly retarded, or even altogether precluded, depending on the rotational restraint provided by sheathing or other attachments to the compression flange. A series of cantilever tests on sheathed joists was conducted to assess the rotational stiffness provided by plywood, OSB, and gypsum board sheathing to typical cold-formed steel joists in use in North America. The tests indicate that plywood and OSB can provide beneficial restraint, but gypsum has inadequate rotational capacity due to a pull-through failure which occurs at low strength and rotation. The traditional cantilever testing protocol (AISI TS-1-02) was successfully extended to include additional displacement measurements which were then used to separate the rotational stiffness into a sheathing component and a connection component. Evaluation of the connection stiffness indicated that joist thickness and fastener spacing are the most influential variables for predicting the available stiffness. A simple design method for predicting the component stiffness values was developed and shown to provide reasonable and conservative agreement with the conducted tests. This design method is recommended for use in the design of cold-formed steel framing systems where sheathing partially restraints distortional buckling and has been recently adopted in American standards.

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