Evaluating the LRFD Factor for Cold-formed Steel Compression Members

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Abstract

This paper summarizes recent work to determine if the LRFD resistance factor for cold-formed steel compression members can be increased above its current value of ϕ_c=0.85. An experimental database of 675 concentrically loaded columns with plain and lipped C-sections, plain and lipped Z-sections, hat sections and angle sections, including members with holes was compiled. The predicted strength of each specimen was calculated with the AISI-S100-07 Main Specification and Direct Strength Method (DSM). Test-to-predicted strength statistics were employed with the first order second moment reliability approach in AISI-S100-07 Chapter F to calculate the resistance factors. The observed trends demonstrate that DSM is a more accurate strength predictor than the current Main Specification, especially for columns with partially effective cross sections. Serious consideration should be given to replacing the Main Specification with DSM, which would provide improved prediction accuracy and a viable rationale for increasing the resistance factor. The test-to-predicted strength ratios for columns with plain and lipped angle cross-sections exhibit a high coefficient of variation and become increasingly conservative with increasing global slenderness. Fundamental research on the mechanics of angle compression members is needed to improve existing design methods.

Introduction

The American Iron and Steel Institute (AISI) implemented the load and resistance factor (LRFD) design approach for cold-formed steel members in 1991 (AISI 1991), with the strength limit state for columns defined as:

$$P_u \leq \phi_c P_n \,. \tag{1}$$

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The required column strength (factored demand) is P_u, and the available strength (resistance) is the nominal capacity, P_n, multiplied by a resistance factor, ϕ_c. The resistance factor reduces nominal capacity based on the likelihood of deleterious variations in column geometry and material properties during fabrication (Nowak 2000). The resistance factor also compensates for bias and variability in a strength prediction approach, and when derived with formal structural reliability theory as in the case of the LRFD approach, can be tuned to produce designs with a uniform probability of failure (Hsiao et al. 1990).

The current AISI LRFD resistance factor of ϕ_c=0.85 was established in 1991, more than 20 years ago, based on a cold-formed steel column database of 264 specimens. Since then, major changes to the AISI Specification have been implemented, including modifications to the column curve predicting global capacity (AISI 1996), the incorporation of a distortional buckling limit state (AISI 2007), and the addition of the Direct Strength Method (AISI 2004; Schafer 2002), which considers cross-section connectivity in the capacity calculation. These changes have improved strength prediction accuracy, however corresponding gains in design efficiency could still be achieved by reevaluating the LRFD resistance factor.

This research takes a fresh look at the AISI LRFD resistance factor, exploring the viability of raising ϕ_c above 0.85, and presents resistance factors by cross-sectional slenderness (i.e., partially or fully effective), by ultimate limit state, and by cross-section type. The research is conducted with an expanded column experiment database containing 675 tests, including C-sections, Z- sections, hat sections and angle sections, as well as columns with and without holes. Resistance factors are calculated for both the AISI Main Specification and Direct Strength Method strength prediction approaches, and code revisions are recommended that have the potential to improve design efficiency and cost competitiveness of cold-formed steel columns.

LRFD resistance factor calculation

Resistance factors in this study are calculated with the first order second moment structural reliability approach described in AISI-S100-07 Chapter F (AISI 2007; Galambos 1998; Hsiao et al. 1990):

$$\phi = C_\phi \left(M_m F_m P_m\right) e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P {V_P}^2 + V_Q^2}} \quad . \qquad (2)$$

A detailed derivation of Eq. (2) is provided in Ganesan (2010). The reliability index, β_o, has been established as 2.5 for LRFD cold-formed steel member design in the United States and Mexico, which corresponds to a probability of failure of approximately 6 in 1000 columns. The coefficient of variation (COV) of the applied loading is assumed as V_Q=0.21 for a dead load to live load ratio of 5 to 1 and the LRFD calibration coefficient C_ϕ=1.52, see Ganesan (2010) for details.

Bias and variability in the predicted column capacity are accounted for with a material factor *M* (related to steel yield stress), a fabrication factor *F* (related to column dimensions), and a professional factor *P* (quantifies the accuracy of capacity predictions relative to tests). For cold-formed steel columns, the steel yield stress is typically higher than the minimum specified, and therefore the mean of the material factor is M_m=1.10 with a COV of V_M=0.10 (Hsiao et al. 1990). The column nominal dimensions are assumed to be unbiased, and therefore F_m=1.00 with a COV of V_F=0.05. The statistics for the professional factor, P_m and V_P, will be calculated with the column test database and prediction methods introduced in the following sections.

Column test database

A cold-formed steel column test database was assembled to facilitate the calculation of the professional factor statistics P_m and V_m in Eq. (2). The database, summarized in Table 1, contains the original 264 columns considered in the 1991, excluding eccentrically loaded columns tests (Loh and Peköz 1985), and including concentrically loaded column data from several experimental programs conducted over the past 30 years, for a total of 675 column tests. Plain and lipped C-sections (with and without holes), Z-sections, plain and lipped angle sections, and hat sections are represented in the database. (Dimension notation for each cross-section type is provided in Figure 1.) Built-up I-sections (DeWolf et al. 1974; Weng and Pekoz 1990) and box sections (DeWolf et al. 1974) have not been considered. Tested boundary condition details and column dimensions for all experimental programs considered is provided in Ganesan (2010).

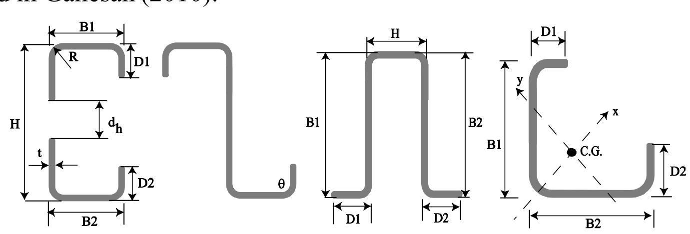

Figure 1 Cross-section dimension notation

Table 1 Cold-formed steel column test database

Reference	Section type	Holes	n	B/t		H/t		D/t		D/B		d_h/H		λ_c	
				min	max	min	max	min	max	min	max	min	max	min	max
Thomasson 1978	Lipped C		13	69	159	207	472	14.0	32.4	0.2	0.2	---	---	0.9	1.2
Loughlan 1979	Lipped C		33	30	80	91	226	10.9	32.8	0.4	0.4	---	---	0.6	1.1
Dat 1980	Lipped C		43	19	23	33	41	8.3	10.1	0.4	0.4	---	---	0.4	1.9
Desmond et al. 1981	Lipped C		7	26	30	37	39	2.2	8.9	0.1	0.3	---	---	0.1	0.2
Desmond et al. 1981	Hat		11	51	51	42	42	7.5	29.9	0.2	0.6	---	---	0.2	0.4
Ortiz-Colberg 1981	Lipped C	✓	32	21	33	46	72	6.7	10.4	0.3	0.3	0.1	0.5	0.2	1.4
Ortiz-Colberg 1981	Lipped C		11	21	33	46	72	6.6	10.4	0.3	0.3	---	---	0.2	1.4
Mulligan 1983	Lipped C		37	33	100	64	355	7.4	21.3	0.2	0.2	---	---	0.1	1.1
Wilhoite et al. 1984	Plain Angles		7	23	23	---	---	---	---	---	---	---	---	1.9	2.0
Sivakumaran 1987	Lipped C	✓	42	26	32	58	118	7.9	9.8	0.3	0.3	0.2	0.6	0.2	0.2
Sivakumaran 1987	Lipped C		6	26	32	58	118	7.9	9.8	0.3	0.3	---	---	0.2	0.2
Polyzois et al. 1993	Plain Z		13	30	51	77	137	---	---	---	---	---	---	0.2	0.5
Polyzois et al. 1993	Lipped Z		72	35	56	76	137	2.4	36.2	0.1	0.7	---	---	0.1	0.4
Miller and Peköz 1994	Lipped C		43	17	40	43	175	5.2	9.0	0.2	0.3	---	---	0.2	2.8
Miller and Peköz 1994	Lipped C	✓	37	19	40	47	173	5.7	9.5	0.2	0.3	0.4	0.8	0.1	3.0
Moldovan 1994	Plain C		35	20	35	20	53	---	---	---	---	---	---	0.1	1.2
Moldovan 1994	Lipped C		29	19	46	32	65	6.3	13.7	0.2	0.4	---	---	0.1	1.0
Abdel-Rahman and Sivakumaran 1998	Lipped C	✓	8	22	33	80	108	6.9	10.3	0.3	0.3	0.3	0.4	0.1	0.2
Young and Rasmussen 1998a	Lipped C		12	25	34	66	66	8.0	8.6	0.2	0.3	---	---	0.2	1.7
Young and Rasmussen 1998b	Plain C		14	25	34	64	67	---	---	---	---	---	---	0.2	2.0
Popovic et al. 1999	Plain Angles		12	11	22	---	---	---	---	---	---	---	---	0.9	1.8
Pu et al. 1999	Lipped C	✓	30	43	65	82	122	13.3	20.0	0.3	0.3	0.2	0.4	0.1	0.1
Pu et al. 1999	Lipped C		6	43	65	82	122	13.3	20.0	0.3	0.3	---	---	0.1	0.1
Shanmugam and Dhanalakshmi 2001	Plain Angles		3	20	63	---	---	---	---	---	---	---	---	1.6	4.9
Young and Hancock 2003	Lipped C		42	21	68	41	68	4.7	7.4	0.1	0.2	---	---	0.7	0.9
Young 2004	Plain Angles		24	38	62	---	---	---	---	---	---	---	---	3.0	5.1
Chodraui et al. 2006	Plain Angles		4	25	25	---	---	---	---	---	---	---	---	1.7	2.0
Young and Chen 2008	Lipped Angles		25	44	84	---	---	9.1	17.4	0.2	0.2	---	---	0.4	4.2
Moen and Schafer 2008	Lipped C		12	36	43	92	139	7.8	11.1	0.2	0.3	---	---	0.3	0.7
Moen and Schafer 2008	Lipped C	✓	12	37	42	91	146	8.3	12.2	0.2	0.3	0.2	0.4	0.3	0.8

Strength prediction methods

AISI Main Specification

Column capacity is predicted in the AISI Main Specification (AISI 2007) as the minimum capacity corresponding to three ultimate limit states - global buckling, local-global buckling interaction, and distortional buckling:

$$P_n = min(A_g F_n , A_e F_n , P_{nd}) , \tag{3}$$

where A_g is the column gross cross-sectional area, A_e is the column effective cross-sectional area including the local buckling influence, element by element, with the effective width method (Peköz 1987; Von Karman et al. 1932), F_n is the global buckling column strength (stress), and P_{nd} is the distortional buckling column capacity. The original 1991 LRFD development did not consider distortional buckling, however ϕ_c=0.85 was demonstrated to be viable and conservative resistance factor for this limit state (Schafer 2000).

The design expressions for global column strength, F_n, in 1991 were consistent with the American Institute of Steel Construction (AISC) column curve at that time (AISC 1986):

$$F_e > F_y/2\,,\ \ F_n = \left(1 - F_y/4F_e\right)$$
$$F_e \le F_y/2\,,\ \ F_n = F_e\,. \tag{4}$$

The critical elastic global buckling stress, F_e, is the minimum of the critical elastic flexural, torsional, or flexural-torsional column buckling stress and F_y is the steel yield stress. The LRFD resistance factor of ϕ_c=0.85 was established based on Eq. (3).

The global buckling column curve was modified in the 1996 AISI Specification (AISI 1996) to its present form based on research by Peköz (1992) which coincided with updates to the AISC LRFD Specification (AISC 1993):

$$\lambda_c \le 1.5,\ \ F_n = \left(0.658^{\lambda_c^{\,2}}\right)F_y\,,$$
$$\lambda_c > 1.5,\ \ F_n = \left(0.877F_y\right)/\lambda_c^{\,2}\,, \tag{5}$$

where $\lambda_c=(F_y/F_e)^{0.5}$ is the column global slenderness. A comparison of Eq. (4) and Eq. (5) in Figure 1 demonstrates that for the same λ_c, the 1996 AISI column curve (also the current AISI-S100-07 column curve) predicts a lower capacity than the 1991 AISI column curve, with a maximum difference of 10%.

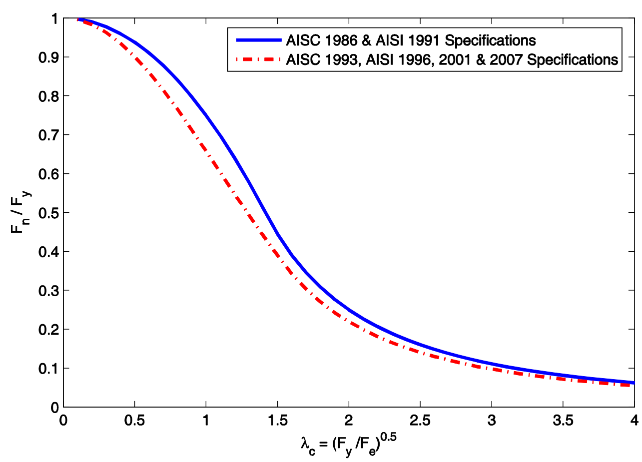

Figure 2 History of AISI and AISC global buckling column curves

AISI Direct Strength Method

Introduced in 2004, the AISI Direct Strength Method (DSM) represents an important advance in cold-formed steel design as it employs elastic buckling behavior of a general cross-section including cross-section connectivity to predict column strength. The column capacity is calculated in a similar fashion to the AISI Main Specification, considering the minimum of three ultimate limit states – global buckling, local-global buckling interaction, and distortional buckling:

$$P_n = min\left(P_{ne}, P_{n\ell}, P_{nd}\right), \tag{6}$$

where P_{ne}, $P_{n\ell}$, P_{nd} are the nominal capacities for global, local and distortional buckling failures respectively. The global buckling capacity (P_{ne}) and distortional buckling capacity (P_{nd}) are equivalent for the Main Specification and DSM, however the local buckling capacity, $P_{n\ell}$, is calculated including the effects of cross-section connectivity instead of element by element with the effective width method.

AISI capacity prediction for angle columns

AISI-S100-07 states that the axial capacity of partially effective concentrically loaded angle columns should be calculated including a demand moment of $\phi_c P_n L/1000$ from an initial column out-of-straightness imperfection (Peköz 1987; Popovic et al. 1999) applied about the *y*-axis shown in Figure 1. The angle column capacity, P_n, is calculated with an interaction equation (AISI 2007):

$$\frac{\phi_c P_n}{\phi_c P_{no}} + \frac{\phi_c P_n L/1000}{\phi_b M_{ny}} \leq 1.0, \tag{7}$$

where P_{no} is the nominal axial capacity and M_{ny} is the flexural strength of the gross cross-section about the *y*-axis (see Figure 1).

Resistance factor results

The tested capacities, P_{test}, from the experimental programs in Table 1 (675 columns in total) and the predicted capacities, P_n, from the AISI Main Specification and Direct Strength Method approaches presented in the previous section, define the professional factor, $P=P_{test}/P_n$. The predicted column specimen strengths (P_n) for the 675 column specimens were computed using custom MATLAB code (Mathworks 2009) validated with the AISI Design Manual (AISI 2008). The mean and COV of the professional factor, P_m and V_P, are input into Eq. (2) to calculate the LRFD resistance factor, ϕ_c. The resistance factor is presented in the following sections considering different groupings of test data: all data, columns with and without holes, columns by cross-section type, columns with partially or fully effective cross-sections, and by predicted ultimate limit state. Columns inside and outside of the code dimensional limits described in Table 2 for the Main Specification and in Table 3 for DSM are evaluated. The different groupings facilitate comparisons between the Main Specification and DSM and are used to identify trends in prediction accuracy that can be leveraged with future code revisions to improve design efficiency.

Table 2 Main Specification dimensional limits

Cross-section element	Column dimension	Limiting range
Stiffened compression element with longitudinal edge connected to web/flange	Flat-width-to-thickness (w/t)	$w/t \le 60$
Stiffened compression element with both longitudinal edges connected stiffened elements	Flat-width-to-thickness (w/t)	$w/t \le 500$
Unstiffened compression element	Flat-width-to-thickness (w/t)	$w/t \le 60$
Uniformly compressed stiffened element with circular holes	Depth of hole-to-flat width (d_h/w)	$0.5 \ge d_h/w \ge 0$
↓	Flat-width-to-thickness (w/t)	$w/t \le 60$
Uniformly compressed stiffened element with non circular holes	Center-to-center hole spacing (s)	$s \ge 24$ in.
	Clear distance from hole at ends (s_{end})	$s_{end} \ge 10$ in.
	Depth of the hole (d_h)	$d_h \le 2.5$ in.
	Length of the hole (L_h)	$L_h \le 4.5$ in.
↓	Depth of hole-to-out-to-out-width(d_h/w_o)	$d_h/w_o \le 0.5$
Uniformly compressed stiffened element with simple lip edge stiffener	Lip Angle (θ)	$140^o \ge \theta \ge 40^o$

Table 3 DSM prequalified dimensional limits

Cross-section dimension	Lipped C-section	Lipped Z-section	Hat section
Web height-to-thickness (H/t)	$H/t < 472$	$H/t < 137$	$H/t < 50$
Flange width-to-thickness (B/t)	$B/t < 159$	$B/t < 56$	$B/t < 20$
Lip width-to-thickness (D/t)	$4 < D/t < 33$	$0 < D/t < 36$	$4 < D/t < 6$
Web height-to-flange width (H/B)	$0.7 < H/B < 5.0$	$1.5 < H/B < 2.7$	$1.0 < H/B < 1.2$
Lip width-to-flange width (D/B)	$0.05 < D/B< 0.41$	$0.00 < D/B< 0.73$	$D/B = 0.13$
Lip Angle (θ)	$\theta = 90^o$	$\theta = 50^o$	---

Resistance factors considering all columns in database

The test-to-predicted statistics and resistance factors considering all column data, except angle columns, are summarized in Table 4. (Note that the resistance factor for angle columns is presented in a later section.) The resistance factor calculated considering all data is ϕ_c=0.85 for the Main Specification and ϕ_c=0.87 for DSM, confirming the viability of ϕ_c=0.85 currently established in AISI-S100-07. (Note that the DSM resistance factor does not include specimens with holes as there are currently no DSM provisions to predict the capacity of columns with holes). Both Main Specification and DSM resistance factors are unaffected by column specimens outside their respective dimensional limits (see Table 2 and Table 3). This insensitivity is at least partially attributed to the small number of column specimens exceeding the dimensional limits in this study. An examination of broader applicability for the prediction methods is warranted based on the results though, especially DSM, which currently specifies ϕ_c=0.80 for columns outside prequalified limits.

Table 4 Resistance factors for all columns in the database (except angles)

Prediction method	Classification	Test-to-predicted statistics Mean (P_m)	SD	COV (V_p)	ϕ_c	n
Main Spec	All columns †	1.06	0.18	0.17	0.85	448
	All columns	1.06	0.18	0.17	0.85	600
DSM	All columns*	1.05	0.15	0.14	0.87	390
	All columns	1.04	0.15	0.15	0.87	439

† Within Main Spec dimensional limits, refer toTable 2

* Within DSM prequalified limits, refer to Table 3

Resistance factors for columns with and without holes

Resistance factors for columns with and without holes are provided in Table 5 for the Main Specification. Strength predictions for columns with holes are conservative considering specimens inside and outside AISI dimensional limits (P_m=1.17 and P_m=1.16 respectively, see Table 5). The conservative predictions result in resistance factors that are near or above unity. The Main Specification resistance factor for columns without holes provided in Table 5 facilitates a meaningful comparison to the DSM resistance factor (also for columns without holes) in Table 3. For the Main Specification, ϕ_c=0.83, and for DSM, ϕ_c=0.87. The higher DSM resistance factor results from a lower test-to-predicted COV, demonstrating the improved prediction accuracy of DSM achieved in the local buckling analysis including interaction between connected cross-section elements. (Remember, distortional buckling and global buckling prediction equations are the same for both approaches).

Table 5 Main Specification resistance factors, columns without and with holes

Classification	Test-to-predicted statistics Mean (P_m)	SD	COV (V_p)	ϕ_c	n
Columns without holes†	1.04	0.18	0.17	0.83	397
Columns without holes	1.03	0.18	0.18	0.82	439
Columns with holes †	1.17	0.14	0.12	1.01	51
Columns with holes	1.16	0.15	0.13	0.98	161

† Within Main Spec dimensional limits, refer to Table 2

Resistance factors by cross-section type

Resistance factors calculated per cross-section type for both the Main Specification and DSM are provided in Table 6. The COV increases with the number of specimens for each cross-section, resulting from the statistical variability created by considering multiple experimental programs within each cross-section group. This sensitivity to the number of experimental programs considered makes it difficult to draw definitive conclusions from the cross-section data representation. For example, the hat section column tests were all performed by one researcher (Desmond et al. 1981) resulting in a conservative test-to-predicted mean (P_m=1.34 for the Main Specification), while the lipped Z-section tests, also performed by one researcher (Polyzois et al. 1993), results in an unconservative test-to-predicted mean (P_m=0.88 for the Main Specification). For both cross-sections groups, the COV is low because there is no statistical influence across experimental programs. For cross-sections with a larger number of tests, for example the lipped C-section, P_m is near unity because experimental bias is averaged across multiple test programs. The results in Table 6 demonstrate that without large quantities of data, it is difficult to specify meaningful resistance factors per cross-section type. It can be concluded that DSM is a more accurate strength predictor of lipped C-section columns than the Main Specification (compare ϕ_c=0.90 versus ϕ_c=0.83 in Table 6). The improved DSM prediction accuracy can be observed in the tighter band of test-to-predicted data around P_{test}/P_n=1 plotted for each cross-section (compare Figure 3a to Figure 3b).

Table 6 Resistance factors by cross-section type

Prediction method	Classification	Test-to-predicted statistics Mean (P_m)	SD	COV (V_p)	ϕ_c	n
Main Spec	Plain C	1.10	0.13	0.12	0.95	49
	Lipped C†	1.06	0.18	0.17	0.85	252
	Lipped C	1.04	0.18	0.17	0.83	294
	Plain Z	1.12	0.07	0.06	1.02	13
	Lipped Z	0.88	0.10	0.11	0.76	72
	Hat Sections	1.34	0.08	0.06	1.21	11
DSM	Plain C	1.03	0.13	0.13	0.88	49
	Lipped C*	1.07	0.15	0.14	0.90	245
	Lipped C	1.06	0.16	0.15	0.88	294
	Plain Z	1.12	0.07	0.06	1.02	13
	Lipped Z	0.94	0.11	0.11	0.81	72
	Hat Sections	1.24	0.05	0.04	1.13	11

† Within Main Spec dimensional limits, refer to Table 2

* Within DSM prequalified limits, refer to Table 3

Resistance factors considering partially and fully effective sections

Resistance factors for columns with partially effective cross-sections (A_e<A_g or $P_{n\ell}$<P_{ne}) and fully effective cross sections (A_e=A_g or $P_{n\ell}$=P_{ne}), excluding columns with holes, are presented in Table 7. The DSM resistance factor is 10% higher than the Main Specification for partially effective cross-sections (compare

ϕ_c=0.89 to ϕ_c=0.81 in Table 7), emphasizing that DSM provides improved strength prediction accuracy over a wide range of cold-formed steel columns sensitive to local buckling. The DSM and Main Specification resistance factors for fully effective sections are consistent (compare ϕ_c=0.83 to ϕ_c=0.81 in Table 7) because the same prediction equations are used in both approaches for global and distortional buckling.

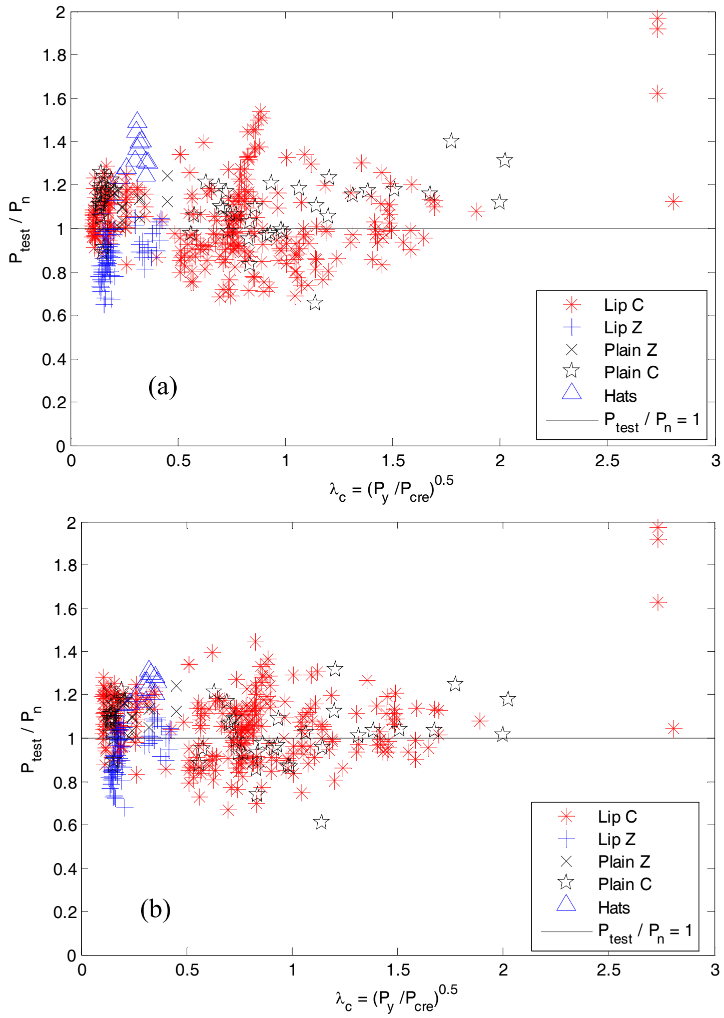

Figure 3 Test-to-predicted ratio as a function of global slenderness for (a) the AISI Main Specification and (b) the AISI Direct Strength Method. Columns with holes are not shown.

Table 7 Resistance factors for partially and fully effective columns

Prediction method	Classification	Test-to-predicted statistics Mean (P_m)	SD	COV (V_p)	ϕ_c	n
Main Spec	Fully effective †	1.04	0.20	0.19	0.81	104
	Fully effective	1.04	0.20	0.19	0.81	104
	Partially effective†	1.00	0.17	0.17	0.81	293
	Partially effective	1.02	0.18	0.17	0.82	335
DSM	Fully effective *	1.07	0.21	0.19	0.83	65
	Fully effective	1.04	0.20	0.19	0.81	109
	Partially effective*	1.04	0.14	0.13	0.89	325
	Partially effective	1.05	0.14	0.13	0.89	330

† Within Main Spec dimensional limits, refer to Table 2

* Within DSM prequalified limits, refer to Table 3

Fully effective: $A_e=A_g$ or $P_{n\ell}=P_{ne}$ Partially effective: $A_e<A_g$ or $P_{n\ell}<P_{ne}$

Resistance factors by limit state

Both the AISI Main Specification and DSM relate column capacity to three limit states: global buckling or yielding of the cross-section, local-global buckling interaction, and distortional buckling as described in Eq. (3) and Eq. (6). Grouping the column data by these limit states, and excluding columns with holes to provide a fair comparison between the Main Specification and DSM, results in the resistance factors provided in Table 8.

Table 8 Resistance factors by ultimate limit state

Prediction method	Limit state	Test-to-predicted statistics Mean (P_m)	SD	COV (V_p)	ϕ_c	n
Main Spec	Global buckling or yielding †	1.04	0.21	0.20	0.81	92
	Global buckling or yielding	1.04	0.21	0.20	0.81	92
	Local-global buckling interaction †	0.98	0.17	0.17	0.78	235
	Local-global buckling interaction	1.00	0.19	0.19	0.79	265
	Distortional buckling †	1.09	0.11	0.10	0.96	70
	Distortional buckling	1.09	0.10	0.10	0.96	82
DSM	Global buckling or yielding*	1.06	0.22	0.20	0.81	59
	Global buckling or yielding	1.03	0.20	0.19	0.80	103
	Local-global buckling interaction *	1.03	0.15	0.14	0.87	235
	Local-global buckling interaction	1.03	0.15	0.14	0.87	236
	Distortional buckling*	1.07	0.10	0.09	0.94	96
	Distortional buckling	1.08	0.10	0.09	0.95	100

† Within Main Spec dimensional limits, refer to Table 2

* Within DSM prequalified limits, refer to Table 3

The most accurate strength predictor is distortional buckling, with ϕ_c=0.96 for the Main Specification and ϕ_c=0.94 for DSM. Local-global buckling interaction is predicted much more accurately by DSM (compare ϕ_c=0.87 versus ϕ_c=0.78 in Table 8) which is consistent with the results in Table 7 for columns with partially effective cross-sections. The global buckling resistance factor is the same for DSM and the Main Specification (ϕ_c=0.81). An increase in the resistance factor for the distortional buckling limit state to ϕ_c=0.95 is a valid consideration for a future code revision, as is the replacement of the current Main Specification approach with DSM, which could lead to better prediction accuracy and a higher resistance factor.

Resistance factors for angle columns

Angle columns have been treated separately from the other columns in this study because of their highly variable test-to-predicted statistics summarized in Table 9. Figure 4 demonstrates that the Main Specification and DSM strength predictions become overly conservative as global slenderness, λ_c, increases for both plain angles and lipped angles. The additional PL/1000 moment required by AISI-S100-07 for partially effective angle cross-sections (Popovic et al. 1999) causes the prediction to be even more conservative.

Table 9 LRFD resistance factors for angle columns

		Test-to-predicted statistics				
Prediction method	Classification	Mean (P_m)	SD	COV (V_p)	ϕ_c	n
Main Spec	Plain Angles	3.13	2.42	0.77	0.69	50
	Lipped Angles	2.00	0.91	0.46	0.93	25
	All Angles	2.76	2.11	0.76	0.62	75
	Angles with $\lambda c \leq 2$	1.35	0.36	0.26	0.93	38
DSM	Plain Angles	3.02	2.09	0.69	0.81	50
	Lipped Angles	1.97	0.97	0.49	0.84	25
	All Angles	2.67	1.86	0.69	0.71	75

The low resistance factors indicate that fundamental research on angle columns is needed, with a concerted effort to identify the source of post-buckling capacity currently neglected by the global buckling column curve for slender angle columns. Prediction accuracy improves when $\lambda_c \leq 2$ (ϕ_c=0.93, see Table 9), potentially supporting a higher resistance factor for stockier angle columns, however even for this case, the high test-to-predicted variability (V_p=0.26) is shrouded by conservative predictions (P_m=1.35).

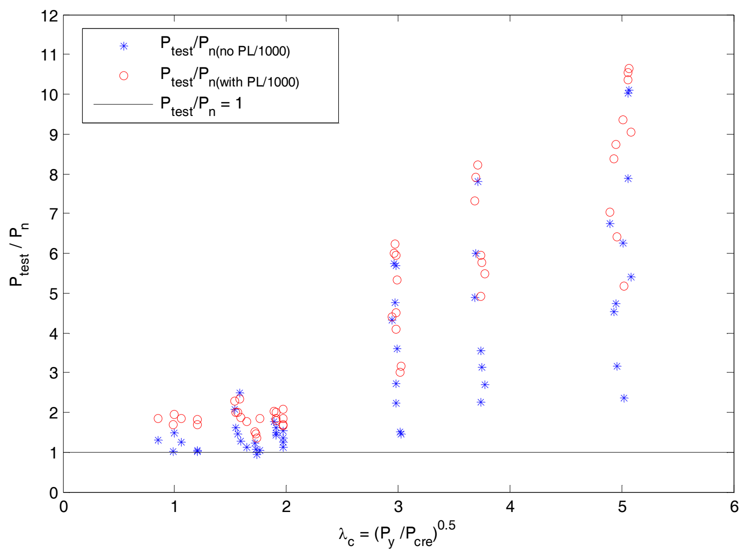

Figure 4 Comparison of angle column predicted strengths with and without PL/1000

Conclusions

LRFD resistance factors for cold-formed steel columns were calculated with a first order second moment reliability approach to identify potential AISI code modifications that could improve design efficiency and cost competitiveness. A test database of 675 cold-formed steel columns was assembled, and test-to-predicted statistics were obtained for the AISI Main Specification and Direct Strength Method. Different groups of test-to-predicted data were considered to evaluate trends in the resistance factors based on prediction method, columns with and without holes, columns with partially or fully effective cross-sections, and columns failing in global buckling, local-global buckling interaction, or distortional buckling.

The AISI Direct Strength Method is a more accurate predictor of local buckling capacity, and a replacement of the current Main Specification with DSM should be seriously considered, as well as an expansion of the DSM prequalified dimensional limits. Distortional buckling is the most accurately predicted strength limit state, and consideration of an increased resistance factor to ϕ_c=0.95 is warranted. Main Specification and DSM capacity predictions for angles columns were found to be overly conservative and highly variable relative to tested values. Research is needed to identify the source of angle column post-buckling strength.

References

Abdel-Rahman, N., and Sivakumaran, K. S. (1998). "Effective design width for perforated cold formed steel compression members." *Canadian Journal of Civil Engineering*, 25, 315-330.

AISC. (1986). *Load and resistance factor design specification for structural steel buildings.*, Chicago, IL.

AISC. (1993). "Load and Resistance Factor Design Specification for Structural Steel Buildings." American Institute of Steel Construction, Chicago, IL.

AISI. (1986). "Cold-Formed Steel Design Manual." American Iron and Steel Institute, Washington, DC.

AISI. (1991). "LRFD Cold-Formed Steel Design Manual." American Iron and Steel Institute, Washington, DC.

AISI. (1996). "Cold-Formed Steel Design Manual." American Iron and Steel Institute, Washington, DC.

AISI. (2001). *North American Specification for the Design of Cold-Formed Steel Structural Members and Commentary*, American Iron and Steel Institute, Washington, D.C.

AISI. (2004). *Supplement to the North American Specification for the Design of Cold-Formed Steel Structural Members, Appendix 1*, American Iron and Steel Institute, Washington, D.C.

AISI. (2007). *AISI-S100-07, North American Specification for the Design of Cold-Formed Steel Structural Members*, American Iron and Steel Institute, Washington, D.C.

AISI. (2008). "Cold-Formed Steel Design Manual." American Iron and Steel Institute, Wash., DC.

Chodraui, G. M. B., Shifferaw, Y., Malite, M., and Schafer, B. W. "Cold-formed steel angles under axial compression." *Eighteenth International Specialty Conference on Cold-Formed Steel Structures: Recent Research and Developments in Cold-Formed Steel Design and Construction,* Orlando, FL, United States, 285-300.Dat, D. T. (1980). "The Strength of Cold-Formed Steel Columns." Cornell University Department of Structural Engineering Report No. 80-4, Ithaca, NY.

Desmond, T. P., Pekoz, T., and Winter, G. (1981). "EDGE STIFFENERS FOR THIN-WALLED MEMBERS." *ASCE Journal of Structural Division*, 107(2), 329-353.

DeWolf, J. T., Peköz, T., and Winter, T. (1974). "LOCAL AND OVERALL BUCKLING OF COLD-FORMED MEMBERS." *ASCE Journal of Structural Division* , 100(10), 2017-2036.

Galambos, T. (1998). *Guide to Stability Design Criteria for Metal Structures, 5th Edition*, John Wiley & Sons, New York, NY.

Ganesan, K. (2010). "Resistance Factor for Cold-Formed Steel Compression Members," M.S. Thesis, Virginia Tech, Blacksburg.

Hsiao, L.-E., Yu, W.-W., and Galambos, T. V. (1990). "AISI LRFD method for cold-formed steel structural members." *ASCE Journal of Structural Engineering New York, N.Y.*, 116(2), 500-517.

Loh, T. S., and Peköz, T. (1985). "Combined Axial Load and Bending in Cold-Formed Steel Members." Cornell University Department of Structural Engineering Report, Ithaca, NY.

Loughlan, J. (1979). "Mode Interaction in Lipped Channel Columns under Concentric or Eccentric Loading," Ph.D. Thesis, University of Strathclyde, Glasgow.

Mathworks. (2009). "Matlab 7.8.0 (R2009a)." Mathworks, Inc., www.mathworks.com.

Miller, T. H., and Peköz, T. (1994). "Unstiffened strip approach for perforated wall studs." *ASCE Journal of Structural Engineering*, 120(2), 410-421.

Moen, C. D., and Schafer, B. W. (2008). "Experiments on cold-formed steel columns with holes." *Thin-Walled Structures*, 46, 1164-1182.

Moldovan, A. (1994). "Compression tests on cold-formed steel columns with monosymmetrical section." *Thin-Walled Structures*, 20(1-4 pt 2), 241-252.

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, NY.

Nowak, A. S., Collins, Kevin R. (2000). *Reliability of Structures*, McGraw Hill, NY.

Ortiz-Colberg, R. A. (1981). "The Load Carrying Capacity of Perforated Cold-Formed Steel Columns," M.S. Thesis, Cornell University, Ithaca, NY.

Peköz, T. B. (1987). "Development of a Unified Approach to the Design of Cold-Formed Steel Members." American Iron and Steel Institute, Washington, D.C.

Peköz, T., and Sümer Ö. (1992). "Design provisions for cold-formed steel columns and beam columns." American Iron and Steel Institute, Washington, DC.

Polyzois, D., and Charnvarnichborikarn, P. (1993). "Web-flange interaction in cold-formed steel z-section columns." *Journal of Structural Engineering*, 119(9), 2607-2628.

Popovic, D., Hancock, G. J., and Rasmussen, K. J. R. (1999). "Axial compression tests on cold-formed angles." *ASCE Journal of Structural Engineering*, 125(5), 515-523.

Pu, Y., Godley, M. H. R., Beale, R. G., and Lau, H. H. (1999). "Prediction of ultimate capacity of perforated lipped channels." *ASCE Journal of Structural Engineering*, 125(5), 510-514.

Schafer, B. W. (2000). "Distortional buckling of cold-formed steel columns." American Iron and Steel Institute, Washington, D.C.

Schafer, B. W. (2002). "Local, distortional, and Euler buckling of thin-walled columns." *ASCE Journal of Structural Engineering*, 128(3), 289-299.

Schafer, B. W., and Ádàny, S. (2006). "Buckling analysis of cold-formed steel members using CUFSM: conventional and constrained finite strip methods." *Eighteenth International Specialty Conference on Cold-Formed Steel Structures*, Orlando, FL.

Shanmugam, N. E., and Dhanalakshmi, M. (2001). "Design for openings in cold-formed steel channel stub columns." *Thin-Walled Structures*, 39, 961-981.

Sivakumaran, K. S. (1987). "Load capacity of uniformly compressed cold-formed steel section with punched web." *Canadian Journal of Civil Engineering*, 14, 550-558.

Thomasson, P. O. (1978). "Thin-Walled C-Shaped Panels in Axial Compression." Swedish Council for Building Research, Report: ISBN-91-540-2820-5 Sweden.

Weng, C. C., and Pekoz, T. (1990). "Compression tests of cold-formed steel columns." *ASCE Journal of Structural Engineering*, 116(5), 1230-1246.

Young, B. (2004). "Tests and Design of Fixed-Ended Cold-Formed Steel Plain Angle Columns." *ASCE Journal of Structural Engineering*, 130(12), 1931-1940.

Young, B., and Chen, J. (2008). "Column tests of cold-formed steel non-symmetric lipped angle sections." *Journal of Constructional Steel Research*, 64(7-8), 808-815.

Young, B., and Hancock, G. J. (2003). "Compression tests of channels with inclined simple edge stiffeners." *Journal of Structural Engineering*, 129(10), 1403-1411.

Young, B., and Rasmussen, K. J. R. (1998a). "Design of lipped channel columns." *Journal of structural engineering New York, N.Y.*, 124(2), 140-148.

Young, B., and Rasmussen, K. J. R. (1998b). "Tests of fixed-ended plain channel columns." *ASCE Journal of Structural Engineering*, 124(2), 131-139.

Von Karman, T., Sechler, E. F., and Donnell, L. H. (1932). "Strength of thin plates in compression." *American Society of Mechanical Engineers -- Transactions -- Applied Mechanics*, 54(2), 53-56.