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DIRECT STRENGTH PREDICTION OF COLD-FORMED STEEL MEMBERS USING NUMERICAL ELASTIC BUCKLING SOLUTIONS

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ABSTRACT

Current design of cold-formed steel members is unduly complicated. Part of this complication arises from the need to perform elastic buckling calculations by hand. Also, complications occur in determining the effective width and resulting effective properties of members. Further, as cross-sections become more optimized (e.g., through the introduction of longitudinal stiffeners) both the elastic buckling and effective width calculations become markedly more complex. In order to investigate alternatives to current design a large amount of experimental data on flexural members of varying geometry is collected. The use of numerical elastic buckling solutions for the entire member, is investigated as an alternative to current practice. Employing strength curves on the entire member, similar to the effective width strength curves for an element, it is found that a “direct strength” approach is a reliable alternative to current design. Such an approach leads to complete flexibility in cross-section geometry, thus greatly increasing the ability to optimize cold-formed steel members. Conservative limitations of the direct strength approach are also addressed.

INTRODUCTION

The thin-walled nature of typical cold-formed steel members makes stability a primary concern. Most modern design specifications (e.g., AISI (1996)) account for stability issues by using an effective width approach. The expression for empirical determination of the effective width includes the elastic buckling stress of an element and the applied compressive stress on the element. For simple cross-sections (e.g., a hat with no intermediate stiffeners) this approach is relatively straightforward. However, for more modern, optimized, cross-sections the calculation of the elastic buckling stress and the calculation of the effective properties can become complicated. A simple design method that avoids the complexity of current methods, yet is general enough to be applicable to modern optimized cross-sections is needed.

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ELASTIC BUCKLING SOLUTIONS

A necessary first step in cold-formed steel member design is calculation of the elastic buckling solution. Design specifications typically use idealized plate buckling solutions. For example, the k = 4 solution for a simply supported plate in pure compression, is employed for local buckling of the compression flange. These simple solutions ignore interaction amongst elements, also unusual member geometry, such as the addition of longitudinal stiffeners, is difficult to incorporate. Further, new buckling modes that arise with more complex member geometry (e.g., distortional buckling) are difficult to predict using these simple, classical methods.

The result of these complications is that existing design specifications actively hamper the development of more optimized member geometry. Reliance on hand methods insures that in the few cases in which design rules are developed for more optimized cross-sections, the resulting guidelines are even more complicated. The hand calculation of elastic buckling requires a great deal of effort and complication, even though reliable computational methods now exist. Modern computational methods: finite element (example programs: ABAQUS, ANSYS, STAGS), finite strip (example programs THIN-WALL, CUFSM) may all be used to determine the elastic buckling solution. In this work, the “semi-analytical” finite strip method, as implemented in CUFSM, is employed.

COLLECTED EXPERIMENTAL DATA

Experimental data is collected from 17 researchers for a total of 574 flexural members. The member types investigated include laterally braced channels and zees, as well as hats and trapezoidal decks. The members include different types, locations, and numbers of longitudinal stiffeners. The geometry of the collected members is summarized in Table 1. Finite strip analysis is completed on every member to determine the elastic buckling solution. The slenderness of each member is determined from the minimum critical buckling moment (M_{cr}) and the yield moment (M_y). In Figure 1, the slenderness is plotted versus the experimental bending strength (M_{test}) normalized by M_y. Despite the vastly differing member geometry clear trends exist.

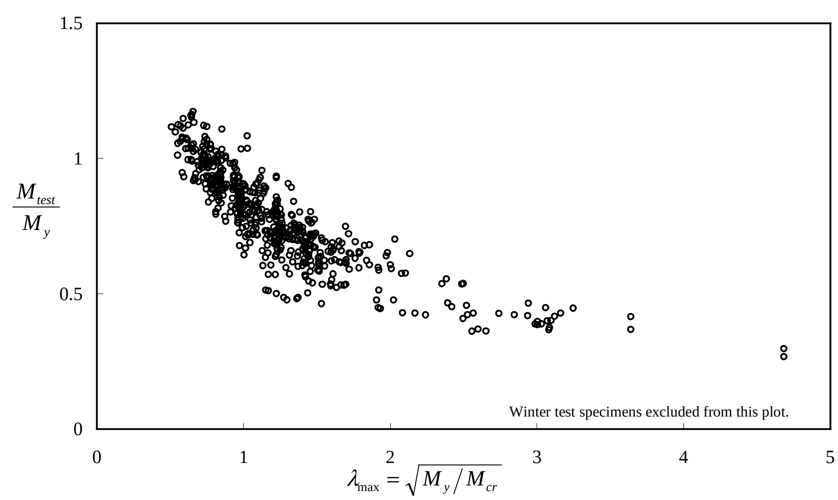

Figure 1: Slenderness vs. strength for the experimental data

TABLE 1

DESCRIPTION OF THE EXPERIMENTAL DATABASE FOR FLEXURAL MEMBERS

Researcher	Lipped C	Unlipped C	Sloping Lip C	Lipped C + Web Stiffener	Hat	Hat + 1 Interm. Stiffener	Hat + 2 Interm. Stiffener	Hat + 3 Interm. Stiffener	Hat + 4 Interm. Stiffener	Trapezoidal Deck	Trap. + 1 Int. Stiffener	Trap. + 1 Int. St. + 1 Web St.	Trap. + 1 Int. St. + 2 Web St.	Trap. + 2 Int. St. + 2 Web St.	Z	Z with unsloped lips	Grand Total
Acharya (1997)							30	32	32								94
Bernard (1993)										6	12						18
Cohen (1987)	14		22														36
Desmond (1977)						22											22
Ellifritt et. al. (1997)	5														5		10
Hoglund (1980)										6	53	16	4	19			98
Konig (1978)					4	15	13										32
LaBoube and Yu (1978)	32																32
Moreyra (1993)	9																9
Papazian (1994)						2	6	6	6								20
Phung and Yu (1978)	6			42													48
Rogers (1995)	50	9															59
Schardt and Schrade (1982)															8	29	37
Schuster (1992)	5																5
Shan (1994)	29																29
Willis and Wallace (1990)	4														6		10
Winter (1946)					15												15
Grand Total	154	9	22	42	19	39	49	38	38	12	65	16	4	19	19	29	574

PREDICTION METHODS

Based on the trend shown in Figure 1, three new prediction methods are introduced. The methods are also compared to the current AISI Specification (1996) approach. The AISI Specification for a cold-formed steel flexural member is based on the unified effective width approach of Peköz (1987). For members that do not undergo lateral-torsional buckling the nominal bending strength (M_n) is calculated as:

$$\left(M_n\right)_{AISI} = S_{eff}\, f_y \tag{1}$$

The effective section modulus (S_{eff}) is determined by finding the effective moment of inertia and effective neutral axis location. These are determined once the effective width (b_e) of the component elements is calculated, via:

$$b_e = \rho b\,,\ \rho = \left(1 - 0.22/\lambda\right)/\lambda\,,\ \lambda = \sqrt{f_y / f_{cr}}\ \text{ and } \lambda > 0.673 \text{ or } \rho = 1 \tag{2}$$

Three "direct strength" methods are investigated as alternatives to the "effective width" approach. The first alternative method employs the same reduction factor, ρ, as in Eqn. 2, but applies ρ to the entire section, such that:

$$\left(M_n\right)_1 = \rho M_y = \left(\rho S_g\right) f_y = S_g\left(\rho f_y\right) \tag{3}$$

Though ρ in Eqn. 3 is applied directly to M_y, in some cases it may be simpler to think of ρ as providing a reduced section modulus, or a reduced stress. To determine ρ in Eqn. 3, λ, (see Eqn. 2) applies to the entire cross-section. If λ < 0.673 then $M_n = M_y$. For cases in which λ > 0.673, Eqn. 3 may be rewritten to give the direct strength prediction:

$$\left(M_n\right)_1 = \sqrt{\frac{M_{cr}}{M_y}}\left(1-0.22\sqrt{\frac{M_{cr}}{M_y}}\right)M_y \tag{4}$$

The second prediction method provides an additional reduction for failures in the distortional mode. Hancock (1995) and Schafer (1997) observed decreased post-buckling capacity in distortional failures. Further, the distortional mode may control failure even when elastic buckling predicts otherwise. Therefore, a correction can be made in determining M_{cr}. The selected approach based on Schafer (1997) is:

$$\left(M_n\right)_2 = \sqrt{\frac{M_{cr}^*}{M_y}}\left(1-0.22\sqrt{\frac{M_{cr}^*}{M_y}}\right)M_y \tag{5}$$

$$M_{cr}^* = \min\left(\left(M_{cr}\right)_{local}, R_d\left(M_{cr}\right)_{dist}\right) \tag{6}$$

$$R_d = \frac{1.17}{\lambda_d+1}+0.3 \text{ for } \lambda_d > 0.673, \text{ where } \lambda_d = \sqrt{M_y/\left(M_{cr}\right)_{dist}} \tag{7}$$

The third, and final, direct strength prediction method is based on the form of Eqn. 4, modified to better agree with the available experimental data. If λ < 0.776 then $M_n = M_y$. For cases in which λ > 0.776 then:

$$\left(M_n\right)_3 = \left(\frac{M_{cr}}{M_y}\right)^{0.4}\left(1-0.15\left(\frac{M_{cr}}{M_y}\right)^{0.4}\right)M_y \tag{8}$$

PERFORMANCE OF PREDICTION METHODS

The strength curves for the direct strength method: M_1 (Eqn. 4), M_2 (Eqn. 5), and M_3 (Eqn. 8) are shown in Figure 2. M_2 only differs from M_1 in cases where distortional buckling controls. For these cases the reduced strength curve for M_2 is plotted in Figure 2. Direct comparison of the adequacy of the proposed approaches as well as the existing AISI Specification is provided in Figure 3 and Table 2.

On average the AISI Specification performance is reasonable. However, the variation is high (i.e., wide scatter in the test to predicted ratio plot of Figure 3). The results by researcher indicate consistently unconservative strength predictions for certain classes of channels and zees. Also, the prediction of members with multiple intermediate stiffeners in the compression flange is quite poor. The AISI Specification has no design rules for members with web stiffeners. Therefore, no AISI data is reported for the work of Hoglund, or Phung and Yu.

The first direct strength approach, M_1, appears relatively conservative for all the members tested. The variation in the predictions is lower than the AISI Specification. Figure 3 shows that the prediction becomes progressively more conservative as slenderness increases. Use of the standard reduction factor (ρ) on the whole member works reasonably well on a wide class of members. The M_2 approach is poorer than the M_1 approach. The method is based on the observation that distortional buckling has a lower post-buckling capacity than local buckling. This conclusion is not borne out by this experimental data. The M_3 approach performs well, excepting the conservative predictions for the Winter (1946) data. The method is markedly more accurate and reliable than the other direct strength approaches and even better than the AISI Specification. A simple direct strength approach is viable for this class of cold-formed steel flexural members.

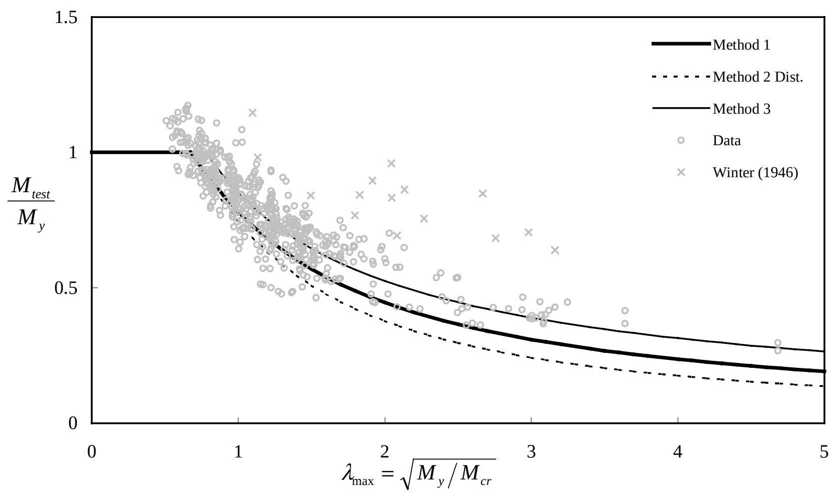

Figure 2: Strength curves and experimental data

TABLE 2

TEST TO PREDICTED RATIOS OF DIFFERENT PREDICTION METHODS

	average of test to predicted ratio				standard dev. of test to predicted ratio				
	$\frac{M_{test}}{M_{AISI}}$	$\frac{M_{test}}{M_1}$	$\frac{M_{test}}{M_2}$	$\frac{M_{test}}{M_3}$	$\frac{M_{test}}{M_{AISI}}$	$\frac{M_{test}}{M_1}$	$\frac{M_{test}}{M_2}$	$\frac{M_{test}}{M_3}$	count
Acharya (1997)	1.03	1.15	1.24	1.04	0.32	0.16	0.22	0.13	94
Bernard (1993)	1.04	1.22	1.33	1.01	0.12	0.19	0.26	0.11	18
Cohen (1987)	1.14	1.12	1.15	1.02	0.09	0.12	0.13	0.10	36
Desmond (1977)	1.07	1.06	1.11	0.94	0.07	0.07	0.08	0.07	22
Ellifritt et. al. (1997)	0.78	0.89	0.97	0.81	0.10	0.11	0.13	0.10	10
Hoglund (1980)		1.06	1.06	0.99		0.13	0.12	0.09	98
Konig (1978)	1.01	1.18	1.26	0.96	0.23	0.33	0.35	0.18	32
LaBoube and Yu (1978)	1.02	1.14	1.14	1.04	0.08	0.12	0.13	0.10	32
Moreyra (1993)	0.82	0.96	0.96	0.88	0.09	0.10	0.10	0.09	9
Papazian (1994)	0.90	1.11	1.15	1.03	0.17	0.12	0.13	0.11	20
Phung and Yu (1978)		1.15	1.21	1.03		0.10	0.12	0.07	48
Rogers (1995)	1.01	1.15	1.22	1.05	0.12	0.11	0.17	0.08	59
Schardt and Schrade (1982)	1.05	1.15	1.17	1.03	0.10	0.07	0.09	0.07	37
Schuster (1992)	0.82	1.03	1.03	0.94	0.04	0.04	0.04	0.04	5
Shan (1994)	0.97	1.08	1.12	0.99	0.13	0.12	0.16	0.10	29
Willis and Wallace (1990)	1.00	1.07	1.08	0.98	0.07	0.07	0.07	0.07	10
Winter (1946)	1.10	1.84	1.84	1.54	0.08	0.32	0.32	0.21	15
Grand Total	1.02	1.14	1.18	1.02	0.16	0.14	0.16	0.10	574

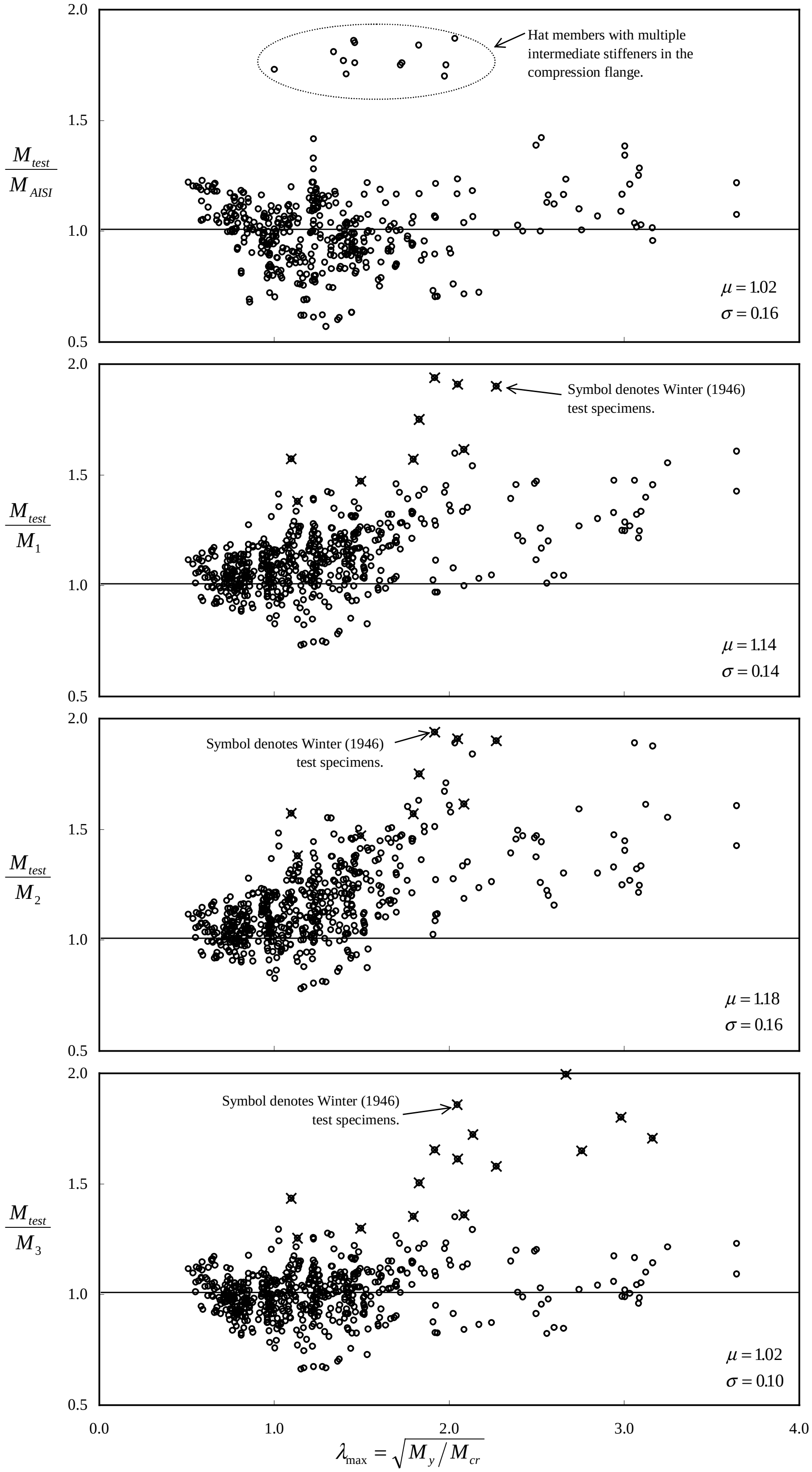

Figure 3: Test to predicted ratios for AISI and direct strength methods

LIMITATIONS OF DIRECT STRENGTH METHODS

Multiple Minima in the Elastic Buckling Solution

When determining the elastic buckling solution numerically many different modes will exist. One philosophy is to simply take the minimum buckling moment (stress) and use that to determine the slenderness of the member. This seemingly rational decision has some rather serious ramifications. First, it assumes that the strength and failure mode are governed by the mode of behavior consistent with the minimum elastic buckling moment (stress). Experimental data shows that when two buckling modes compete the final failure mode may not be consistent with the elastic minimum. Second, this encourages an optimum design in which several different modes occur at the same elastic buckling moment (stress). Experimental evidence shows this is a poor optimum, due to strength considerations and that it invites problems with coupled instabilities. The M_2 method is an attempt to alleviate these concerns. However, the approximations made for this approach appear to be excessively conservative.

Effective Width vs. Direct Strength

One obvious limitation to the direct strength method is the conservative predictions for the Winter test specimens. The basic geometry of these specimens is shown in Figure 4. These hat sections illustrate an important conceptual difference between the direct strength approach and the effective width approach. Consider that as the flange becomes more slender the results for the two methods approach Figures 4b and 4c. For the effective width approach the elimination of the compression flange as ρ goes to zero causes a large neutral axis shift and the resulting section is left to carry the load. For the direct strength appraoch as ρ goes to zero so goes the entire strength. A parametric study of hats similar to those of Figure 4 shows that the difference between these two methods is particularly pronounced in specimens similar to those Winter tested. Specimens with low height to flange width (h/b) ratios, and low tension flange width to compression flange width (c/b) ratios, and low reduction factors (ρ) are predicted quite conservatively using a direct strength approach. However, as h/b, or c/b or ρ increase the two prediction methods become similar again. For more typical sections the adequacy of the direct strength approach is evidenced by the fine correlation with existing data.

CONCLUSIONS

Current design methods for cold-formed steel members are unduly complicated and as a result hamper innovation in cross-section design. A significant amount of this complication is due to the need to calculate elastic buckling solutions by hand. Today, these solutions may be completed numerically with great reliability and ease. Further, using numerical methods for the elastic buckling solution the whole member may be treated consistently, rather than just the component elements. A secondary complication of current design methods is the determination of the effective section. Comparison to a large experimental database of flexural members of varying geometry shows that accurate knowledge of the member buckling solution (member slenderness) provides a direct way to get at the member capacity. The current AISI Specification and three alternative direct strength approaches are compared with the large experimental database. It is shown that a direct strength method can provide the same overall average predictive capabilities and lower variation than the existing AISI Specification. The direct strength approach is not however without its limitations, for some specific cross-sections (low h/b, and c/b and ρ) it may become too conservative.

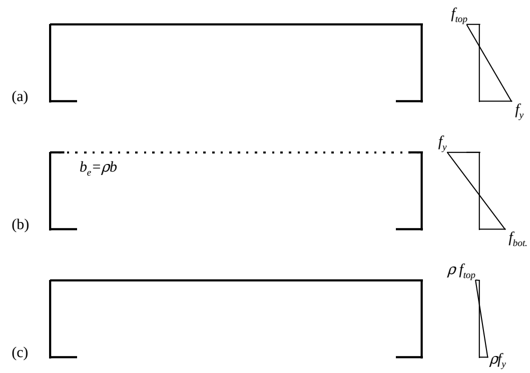

Figure 4: Difference in effective width vs. direct strength methods (a) gross section (b) effective section as ρ approaches 0 (c) direct strength approach as ρ approaches 0.

REFERENCES

American Iron and Steel Institute (1996). *Cold-Formed Steel Design Manual*. American Iron and Steel Institute.

Bernard, E.S. (1993). Flexural Behavior of Cold-Formed Profiled Steel Decking. Ph.D. Thesis, U. of Sydney, Australia.

Desmond, T.P. (1977). The Behavior and Design of Thin-Walled Compression Elements with Longitudinal Stiffeners. Ph.D. Thesis, Cornell University, Ithaca, New York.

Ellifritt, D., Glover, B., Hren, J. (1997). Distortional Buckling of Channels and Zees Not Attached to Sheathing. *Report for the American Iron and Steel Institute*.

Haaijer, G. (1957). Plate Buckling in the Strain-Hardening Range. *Transactions*, *ASCE*. **124**.

Hancock, G.J., Kwon, Y.B., Bernard, E.S. (1994) Strength Design Curves for Thin-Walled Sections Undergoing Distortional Buckling. *J. of Constructional Steel Research* **31:2-3**, 169-186.

Höglund, T. (1980). Design of Trapezoidal Sheeting Provided with Stiffeners in the Flanges and Webs. *Swedish Council for Building Research* **D28:1980**.

König, J. (1978). Transversally Loaded Thin-Walled C-Shaped Panels With Intermediate Stiffeners. *Swedish Council for Building Research* **D7:1978**.

LaBoube, R.A., Yu, W. (1978). Structural Behavior of Beam Webs Subjected to Bending Stress. *Civil Engineering Study Structural Series*, 78-1, Department of Civil Engineering, University of Missouri-Rolla, Rolla, Missouri.

Moreyra, M.E. (1993). The Behavior of Cold-Formed Lipped Channels under Bending. M.S. Thesis, Cornell University.

Papazian, R.P., Schuster, R.M., and Sommerstein, M. (1994). Multiple Stiffened Deck Profiles. *Twelfth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla.

Peköz, T. (1987). *Development of a Unified Approach to the Design of Cold-Formed Steel Members*. American Iron and Steel Institute Research Report CF 87-1.

Phung, N., Yu, W.W. (1978). Structural Behavior of Longitudinally Reinforced Beam Webs. *Civil Engineering Study Structural Series*, **78:6**, Department of Civil Engineering, University of Missouri-Rolla, Rolla, Missouri.

Rogers, C.A., Schuster, R.M. (1995) Interaction Buckling of Flange, Edge Stiffener and Web of C-Sections in Bending. *Research Into Cold Formed Steel, Final Report of CSSBI/IRAP Project*, Department of Civil Engineering, University of Waterloo, Waterloo, Ontario.

Schafer, B.W. (19957). Cold-Formed Steel Behavior and Design: Analytical and Numerical Modeling of Elements and Members with Longitudinal Stiffeners. Ph.D. Thesis, Cornell University, Ithaca, New York.

Schardt, R. Schrade, W. (1982). Kaltprofil-Pfetten. Institut Für Statik, Technische Hochschule Darmstadt, Bericht Nr. 1, Darmstadt.

Schuster, R.M. (1992). Testing of Perforated C-Stud Sections in Bending. Report for the Canadian Sheet Steel Building Institute, University of Waterloo, Waterloo Ontario.

Shan, M., LaBoube, R.A., Yu, W. (1994). Behavior of Web Elements with Openings Subjected to Bending, Shear and the Combination of Bending and Shear. *Civil Engineering Study Structural Series*, **94:2**, Department of Civil Engineering, University of Missouri-Rolla, Rolla, Missouri.

Willis, C.T., Wallace, B. (1990). Behavior of Cold-Formed Steel Purlins under Gravity Loading. *J. of Structural Engineering*, ASCE. **116:8**.

Winter, G. (1946). Tests on Light Beams of Cold-Formed Steel. Forty-fifth Progress Report, Cornell University. (Unpublished)