research report

Overhang Effects on End-One-Flange Web Crippling Capacity of Cold-Formed Steel Members

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Civil Engineering Study 02-1
Cold-Formed Steel Series

Final Report

Overhang Effects on End-One-Flange Web Crippling Capacity of Cold-Formed Steel Members

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Preface

Often a roof purlin rests on a structural frame or endwall with a segment of the purlin cantilevered over the endwall. The *North American Specification for the Design of Cold-Formed Steel Structural Members* (2001) stipulates that if the overhang is less than 1.5 times the flat-width of the web of the cross section, h, the web crippling capacity must be determined assuming an end-one-flange loading condition. However, if the overhang is equal to or greater than 1.5h, the web crippling capacity is defined as an interior-one-flange loading condition. The application of the 1.5h limit creates a discontinuity in loading and imposes a potentially conservative design limit when the overhang is less than 1.5h.

An experimental investigation was initiated at the University of Missouri-Rolla to explore the influence that a cantilever overhang has on the web crippling capacity of a cold-formed steel member. The Metal Building Manufacturer's Association financially supported this study.

The test specimens consisted of metal building industry standard C-and Z-shaped cross sections. The AISI Specification defines such members as having a single unreinforced web. The test specimens were fabricated to have a defined overhang length. The overhang length was varied from 0.5h to 1.5h. The test results indicated that the web crippling capacity was a function of two key parameters, the overhang length and the web slenderness ratio, h/t. The h/t ratio was varied from 67 to 154.

Based on the evaluation of the test results, a design recommendation was prepared to enable the determination of the web crippling capacity of a C- or Z-shaped section having an overhang length less than or equal to 1.5h. For overhang lengths greater than 1.5h, design guidance is also provided.

This report is based on the thesis presented to the Faculty of the Graduate School of the University of Missouri-Rolla in partial fulfillment of the requirements for the degree Masters of Science in Civil Engineering.

This investigation was sponsored by the Metal Building Manufacturer’s Association and their financial support is gratefully acknowledged. The MBMA task group of Lee Shoemaker, Maury Golovin, and Joe Nunnery provided valuable technical guidance. Special thanks are also extended to Dr. Shoemaker, Director, Research & Engineering for the Metal Building Manufacturer’s Association for his assistance throughout the research study.

Appreciation is also expressed to technical staff of the Civil Engineering Department for their assistance in the preparation, fabrication, and performance of the test program.

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1. INTRODUCTION

1.1. GENERAL

In an effort to explore more economical and environmentally benefiting construction materials, contractors, both residential and commercial, have turned to cold-formed steel. Cold-formed steel construction offers many benefits above other construction materials, including: increased feasibility, ease of construction, recyclablility, high strength-to-weight ratio, and non-combustibility.

The design of cold-formed members is governed by the American Iron and Steel Institutes' (AISI) publication *Specification for the Design of Cold-Formed Steel Structural Members (1996).* This document was first introduced in 1946 and has subsequently undergone revisions and reprinting as a result of ongoing research.

Since 1967, the University of Missouri Rolla has aided in research endeavors by investigating the behavior of cold-formed steel structural members and connections. This investigation is a continuation of those efforts, designed to enhance the AISI Specification.

1.2. PURPOSE OF INVESTIGATION

This research investigation served to fulfill two purposes:

i. Enhancement of the AISI Specification (2001)

ii. Increase the feasibility of cold-formed steel construction

1.2.1. Enhancement of the AISI Specification (2001). The primary focus of this investigation involved the study of web-crippling behavior in single web cold-formed steel structural members. The study focused on an end-one-flange loading condition with the specimen subjected to a web crippling failure. Design recommendations were developed based on the load carrying capacity of specimens subjected to a web crippling limit state under end-one-flange loading. Figure 1.1 illustrates the necessary conditions for end-one-flange loading as prescribed by AISI (2001).

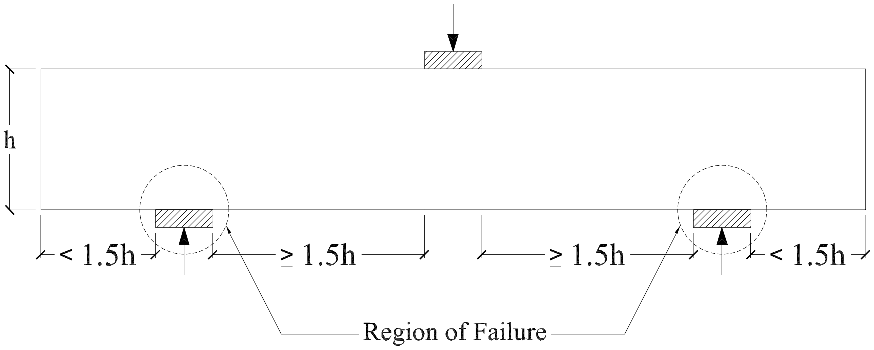

Figure 1.1. AISI End-One-Flange Loading Condition Criteria

The current loading condition guidelines do not allow for any increase in web crippling capacity based on an overhang between 0.0h and 1.5h. The findings of this investigation serve to enhance these guidelines by providing a modification to the end-one-flange web crippling design equation to account for this intermittent increase.

1.2.2. Impacts on Cold-Formed Steel Construction. Web crippling is a critical design limit state for cold-formed steel structural members. Current industry applications include the facilitating of cold-formed steel members in applications such as: studs, headers, trusses and roof purlins. An impact of the results of this investigation includes the economy of cold-formed steel members used in roof purlin applications. Often a roof purlin rests on a structural frame or wall with a segment of the purlin cantilevered over the endwall. The length of the cantilever is often less than 1.5 times the depth of the section (e.g. 1.5h) and the web crippling capacity must be designed according to a conservative end-one-flange loading condition as prescribed by AISI (2001).

1.3. SCOPE OF INVESTIGATION

The criteria for the scope of this investigation involved the following two elements:

i. Loading condition

ii. Section web crippling parameters

The characteristics of these two elements determined the limits of the resulting design recommendations.

1.3.1. Loading Condition.

This investigation focused on end-one-flange loading only. Criteria for this condition are illustrated in Figure 1.1. This investigation was limited to end-one-flange loading with stiffened or partially stiffened flanges fastened to the support. Figure 1.2 illustrates a typical specimen as studied by this investigation.

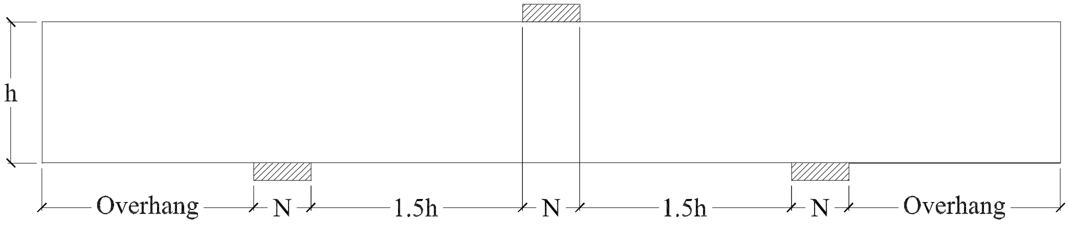

Figure 1.2. Typical Test Specimen Parameters

1.3.2. Section Parameters. Specimens tested in this investigation were C-shaped and Z-shaped sections with edge stiffened flanges. These sections were selected due to their high volume of use in the metal building industry. Figures 1.3 and 1.4 illustrate typical C- and Z-cross sections used in this study. Table 1.1 lists the cross section parameters for the specimens tested in this study.

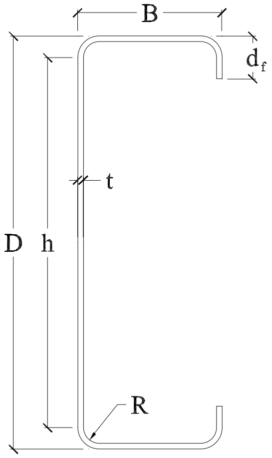

Figure 1.3. Typical C-Section Parameters

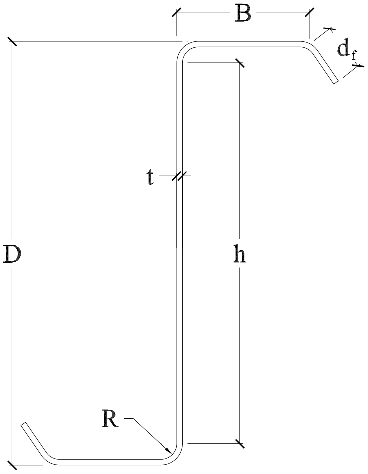

Figure 1.4. Typical Z-Section Parameters

Table 1.1. Cross Sectional Properties for the Test Specimens

Specimen	D (in.)	h (in.)	t (in.)	B (in.)	d_f (in.)	F_y (ksi)	N (in.)	h/t	R/t	N/t	N/h
8Z058	8.0	7.250	0.0625	2.481	1.089	64.18	3.25	116	5.00	52.00	0.45
8Z113	8.0	7.188	0.1069	2.698	1.108	55.08	3.25	67	2.80	30.40	0.45
8C058	8.0	7.375	0.0618	2.940	0.905	68.08	3.25	119	4.06	52.59	0.44
8C113	8.0	7.250	0.1032	3.039	0.953	58.72	3.25	70	2.63	31.49	0.45
10Z065	10.0	9.250	0.0600	2.458	0.955	71.30	3.25	154	5.25	54.17	0.35
10Z109	10.0	9.250	0.1041	2.472	0.935	67.62	3.25	89	2.60	31.22	0.35
12C068	12.0	11.41	0.1060	2.215	0.914	45.25	3.25	108	1.78	30.66	0.28

Table 1.2. Test Specimen Parameter Ranges

	D (in.)	h (in.)	t (in.)	B (in.)	d_f (in.)	F_y (ksi)	N (in.)	h/t	R/t	N/t	N/h
Maximum	12.0	11.41	0.1069	3.039	1.108	71.30	3.25	154	5.00	54.17	0.45
Minimum	8.0	7.188	0.0600	2.215	0.905	45.25	3.25	67	1.78	30.66	0.28

2. REVIEW OF LITERATURE

2.1. GENERAL

For the investigation of the web crippling strength for end-one-flange loaded specimens, the following information was considered:

1. Theoretical analysis of the web crippling capacity of cold-formed steel flexural members.
2. Previous research on web crippling of cold-formed steel structural members.
3. Development of the AISI specification.

2.2. THEORETICAL ANALYSIS OF WEB CRIPPLING CAPACITY OF COLD-FORMED STEEL FLEXURAL MEMBERS

The theoretical analysis of web crippling behavior in cold-formed steel flexural members is performed by analyzing the member ideally as a simply-supported thin plate. These thin plates are subjected to in-plane compressive forces distributed along the edge. The failure load can be calculated based on the critical elastic buckling stress of the plate. However, some elements develop post-buckling strength and do not fail at stress levels equal to the critical elastic buckling stress. For this reason, web crippling of cold-formed steel members becomes a complicated issue and is best documented by Yu (2000):

> ..the theoretical analysis of web crippling for cold-formed steel flexural members is rather complicated because it involves the following factors:
>
> 1. Nonuniform stress distribution under the applied load and adjacent portions of the web.
> 2. Elastic and inelastic stability of the web element.

3. Local yielding in the immediate region of load application.
4. Bending produced by eccentric load (or reaction) when it is applied of the bearing flange at a distance beyond the curved transition of the web.
5. Initial out-of-plane imperfection of plate elements.
6. Various edge restraints provided by beam flanges and interaction between flange and web elements.
7. Inclined webs for decks and panels.

For the investigation of idealized thin plates subjected to elastic buckling behavior, the following analysis approaches are reviewed.

2.2.1. Euler (Timoshenko and Gere, 1961). Based on Euler's equation, the critical elastic buckling load for thin plates can be represented as:

$$P_{cr} = \frac{k\pi^2 D}{h} \tag{2.1}$$

Where:

$D = \text{Flexural rigidity} \left(\frac{Et^3}{12(1-\nu^2)} \right)$

$E = \text{Young's modulus of elasticity}$

$h = \text{Depth of the plate}$

$k = \text{Buckling coefficient of the plate}$

$P_{cr} = \text{Critical elastic buckling load}$

$t = \text{Plate thickness}$

$\nu = \text{Poisson's ratio}$

Equation 2.1 applies to a simply supported plate subjected to a uniformly distributed load as shown in Figure 2.1. When l/h = 1 (square plate), the plate buckling coefficient is equal to 4. The coefficient then varies as a function of l/h as indicated by Figure 2.2.

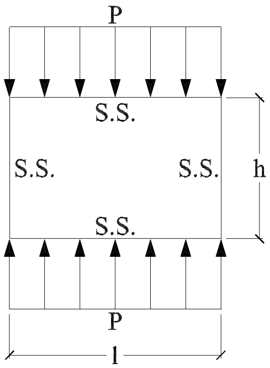

Figure 2.1. Simply Supported Plate Under Uniform Load P

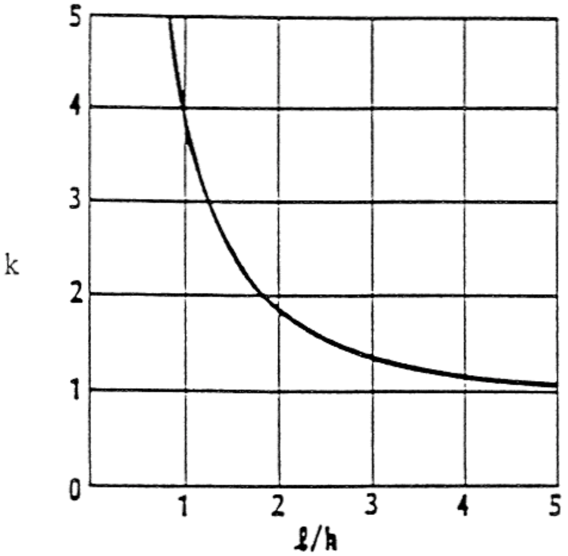

Figure 2.2. Buckling Coefficient k vs. l/h for Equation 2.1

2.2.2. Timoshenko and Gere (1961). For plates subjected to two equal and opposite concentrated edge forces, Timoshenko evaluated the critical elastic buckling model (Equation 2.1) with equal and opposite concentrated loads:

$$P_{cr} = \frac{k\pi^2 D}{h} \tag{2.2}$$

Where:

$D = \text{Flexural rigidity}\left(\frac{Et^3}{12\left(1-\nu^2\right)}\right)$

$E = \text{Young's modulus of elasticity}$

$h = \text{Depth of the plate}$

$k = \text{Buckling coefficient of the plate}$

$P_{cr} = \text{Critical elastic buckling load}$

$t = \text{Plate thickness}$

$\nu = \text{Poisson's ratio}$

Equation 2.2 applies to a simply supported plate subjected to two equal and opposite concentrated forces along the edges. The loading condition for Equation 2.2 is illustrated in Figure 2.3. The variation of the plate buckling coefficient, k vs. l/h as studied by Yamaki (1953) is shown in Figure 2.4.

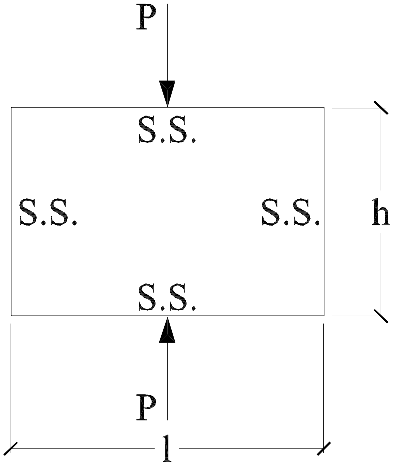

Figure 2.3. Simply Supported Plate Under Concentrated Load P

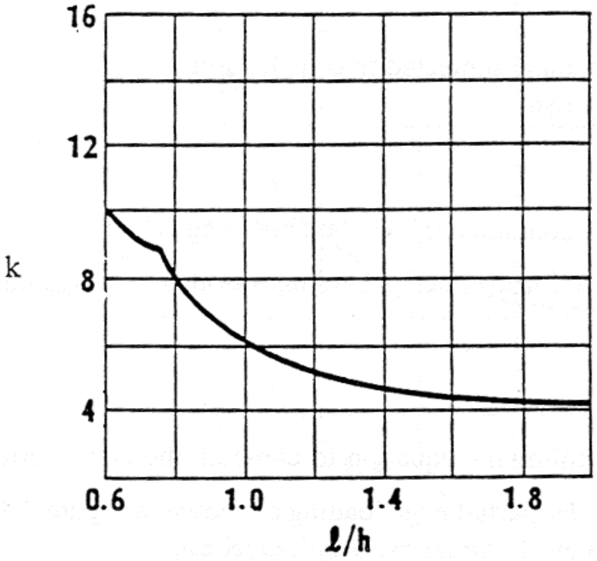

Figure 2.4. Buckling Coefficient k vs. l/h for Equation 2.2

2.2.3. Walker (1975). Walker studied a critical elastic buckling load equation accounting for simply supported plates subjected to partially distributed edge forces:

$$P_{cr} = \frac{k\pi^2 E t^3}{12(1-\upsilon^2)h} \tag{2.3}$$

Where:

E = Young's modulus of elasticity

h = Depth of the plate

k = Buckling coefficient of the plate

P_{cr} = Critical elastic buckling load

t = Plate thickness

ν = Poisson's ratio

Figure 2.5 illustrates the condition investigated by Walker (1975). The buckling coefficient k, now accounts for bearing length, N, and is shown in Figure 2.6 as a function of l/h.

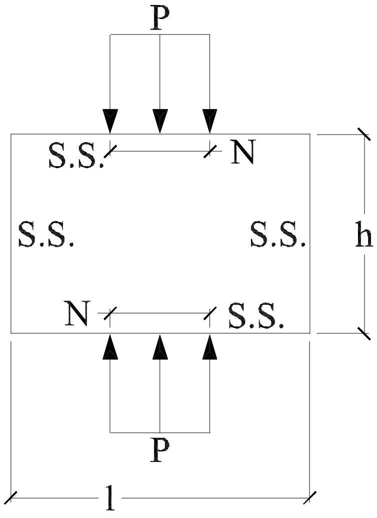

Figure 2.5. Simply Supported Plate Under Partially Distributed Load P

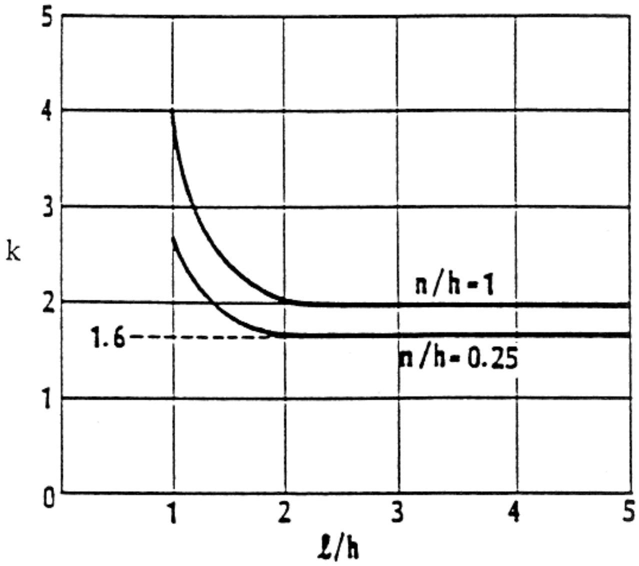

Figure 2.6. Buckling Coefficient k vs. l/h for Equation 2.3

2.3. PREVIOUS RESEARCH ON WEB CRIPPLING OF COLD-FORMED STEEL STRUCTURAL MEMBERS

There exists no research study that focused on the web crippling behavior of end-one-flange specimens with varying lengths of overhang. However, there exists a vast amount of research exploring the web crippling behavior of cold-formed steel flexural members loaded under varying conditions. The following section reviews previous research as it pertains to this investigation.

2.3.1. Winter and Pian (1946). During the 1940's, Winter and Pian investigated the web crippling behavior of cold-formed steel members at Cornell University. Their investigation covered four load cases:

1. End-One-Flange (EOF)
2. End-Two-Flange (ETF)

3. Interior-One-Flange (IOF)
4. Interior-Two-Flange (ITF)

A total of 136 I-sections were tested with the flanges not bolted to the bearing surfaces. Based on their studies, the following formulations were derived for I-sections and other sections providing web rotational restraint:

For End-One-Flange loading (EOF)

$$P_{ult} = F_y t^2 \left(10 + 1.25\sqrt{\frac{N}{t}} \right) \tag{2.4}$$

For Interior-One-Flange loading (IOF)

$$P_{ult} = F_y t^2 \left(15 + 3.25\sqrt{\frac{N}{t}} \right) \tag{2.5}$$

Where:

P_{ult} = Ultimate web crippling load per web

F_y = Yield strength

h = Flat dimension of the web measured in the plane of the web

N = Bearing length

t = Web thickness

2.3.2. Hetrakul and Yu (1978). In 1978, Hetrakul and Yu investigated the web crippling behavior of cold-formed steel members having single unreinforced webs. Based on 140 tests performed at the University of Missouri – Rolla and 96 tests performed at Cornell University, new expressions were derived for the web crippling capacity. These expressions are given as follows:

i. Interior-One-Flange loading (IOF) for stiffened and unstiffened flanges

$$P_{ult} = \frac{F_y t^2}{10^3} C_1 C_2 \left(16317 - 22.52\frac{h}{t}\right)\left(1 + 0.0069\frac{N}{t}\right) \tag{2.6}$$

If $\frac{N}{t} > 60$ then $\left(1 + 0.0069\frac{N}{t}\right)$ may be increased to $\left(0.748 + 0.0111\frac{N}{t}\right)$

ii. End-One-Flange loading (EOF)

For unstiffened flanges:

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 \left(6570 - 8.51\frac{h}{t}\right)\left(1 + 0.0099\frac{N}{t}\right) \tag{2.7}$$

If $\frac{N}{t} > 60$ then $\left(1 + 0.0099\frac{N}{t}\right)$ may be increased to $\left(0.706 + 0.0148\frac{N}{t}\right)$

For stiffened flanges:

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 \left(10018 - 18.24\frac{h}{t}\right)\left(1 + 0.0102\frac{N}{t}\right) \tag{2.8}$$

If $\frac{N}{t} > 60$ then $\left(1 + 0.0102\frac{N}{t}\right)$ may be increased to $\left(0.922 + 0.0115\frac{N}{t}\right)$

iii. Interior-Two-Flange loading (ITF) for stiffened and unstiffened flanges

$$P_{ult} = \frac{F_y t^2}{10^3} C_1 C_2 \left(23356 - 68.64 \frac{h}{t} \right) \left(1 + 0.0013 \frac{N}{t} \right) \qquad (2.9)$$

iv. End-Two-Flange loading (ETF) for stiffened and unstiffened flanges

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 \left(7411 - 17.28 \frac{h}{t} \right) \left(1 + 0.0099 \frac{N}{t} \right) \qquad (2.10)$$

Where:

P_{ult} = Web crippling load per web (ultimate)

$C_1 = (1.22 - 0.22k)$

$C_2 = \left(1.06 - 0.06 \frac{R}{t} \right)$

$C_3 = (1.33 - 0.33k)$

$C_4 = (1.15 - 0.15k)$

F_y = Yield strength

h = Flat dimension of the web measured in the plane of the web

$k = \frac{F_y\,(\text{ksi})}{33}$

N = Bearing length

R = Inside bend radius

t = Web thickness

2.3.3. Bhakta, LaBoube and Yu (1992). In 1992, Bhakta, LaBoube and Yu studied the influence of flange restraint on the web crippling capacity of web elements in flexure. A total of 52 specimens were studied including: channels, I-sections, Z-sections, floor decks and long span roof decks. From their research they observed that a

30% increase exists for the end-one-flange web crippling strength for Z-sections with end support flanges bolted to the supporting member. Based on this conclusion, the 1996 edition of the AISI design specification allows for a Z-section with end supports bolted to the supporting members, Eq. C3.4-1, to be multiplied by 1.3 provided the following provisions are met:

1. $h/t \leq 150$
2. $R/t \leq 4$
3. Cross-section base metal thickness ≥ 0.060 inches
4. Support member thickness ≥ 3/16 inches

2.3.4. Prabakaran (1993). In 1993, Prabakaran performed an extensive statistical analysis on the existing data for web crippling of cold-formed steel flexural members at the University of Waterloo. His objective was to develop a unified equation for the web crippling capacity of cold-formed steel sections. Based on his studies, he formulated the following expression:

$$P_n = Ct^2 F_y \sin\theta \left(1 - C_R\sqrt{\frac{r}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \tag{2.11}$$

Where:

P_n = Nominal web crippling strength

C = Coefficient

C_h = Web slenderness coefficient

C_N = Bearing length coefficient

C_R = Inside bend radius coefficient

F_y = Yield strength

h = Flat dimension of the web measured in the plane of the web

N = Bearing length

R = Inside bend radius

t = Web thickness

θ = Angle between the plane of the web and the plane of the bearing surface

For Equation 2.11, the following parameter limits exist: For I-sections and single web shapes, $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 1$ and $R/t \leq 4$; for multi-web sections, $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 2$ and $R/t \leq 10$. Equation 2.11 is used currently in the Canadian Standard (1994).

2.3.5. Schuster and Beshara (1999). Schuster and Beshara (1999) conducted a preliminary and investigative web crippling study at the University of Waterloo. They examined all existing web crippling data and tested 72 specimens not previously included within the data. The objective of the study was to formulate a better set of coefficients for the expression (Equation 2.11) derived by Prabakaran (1992). The resulting coefficients for the specimens tested in their program (C-sections and Z-sections) are shown in Table 2.1. The newly developed coefficients and safety factors served as the basis for the modification of the AISI specification (2001).

Table 2.1. Recommended Coefficients for Single Web Sections, Schuster and Beshara (1999)

Support and Flange Conditions		Load Cases		Section Type	C	C_R	C_N	C_h	S136		AISI		Max. Limits		
									Ω	Φ	Ω	Φ	h/t	R/t	N/t
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	C & Z	4	0.14	0.35	0.02	1.88	0.75	1.73	0.88	222	9.0	78.0
			Interior	C	13	0.23	0.14	0.01	1.77	0.80	1.65	0.92	249	5.0	121
		Two-Flange Loading or Reaction	End	C	7.5	0.08	0.12	0.048	1.86	0.77	1.70	0.89	195	12.0	70.0
				Z	9	0.05	0.16	0.052	1.93	0.74	1.76	0.86	195	12.0	70.0
			Interior	C	20	0.10	0.08	0.031	1.93	0.74	1.76	0.86	195	12.0	87.0
				Z	24	0.07	0.07	0.04	2.07	0.69	1.86	0.82	195	12.0	87.0
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	C	4	0.14	0.35	0.02	2.03	0.70	1.84	0.83	253	5.0	141
				Z	5	0.09	0.02	0.001	1.93	0.74	1.77	0.86	150	5.0	44.0
			Interior	C	13	0.23	0.14	0.01	1.77	0.80	1.65	0.92	249	5.0	121
		Two-Flange Loading or Reaction	End	C	13	0.32	0.05	0.04	1.78	0.80	1.66	0.92	255	3.0	64.0
			Interior	C	24	0.52	0.15	0.001	2.12	0.67	1.90	0.80	253	3.0	64.0
	Unstiffened Flanges	One-Flange Loading or Reaction	End	C	4	0.40	0.60	0.03	1.96	0.72	1.79	0.85	193	2.0	140
			Interior	C	13	0.32	0.10	0.01	1.99	0.71	1.81	0.84	192	1.0	61.0
		Two-Flange Loading or Reaction	End	C	2	0.11	0.37	0.01	2.18	0.65	1.94	0.78	193	1.0	62.0
			Interior	C	13	0.47	0.25	0.04	2.15	0.66	1.93	0.79	194	1.0	61.0

2.4. AISI Specification

The AISI Cold-Formed Steel Specification (1996), Section C3.4.1, Web Crippling Strength of Webs Without Holes, has been modified to reflect a new approach as developed by Prabakaran (1982), and further modified by Schuster and Beshara (1999). Both specifications are discussed below, however, it should be noted that the recommendations of this study are based on the later specification (AISI, 2001).

2.4.1. 1996 Specification. The American Iron and Steel Institute Specification was first published in 1946. The design specification was primarily based on studies performed at Cornell University. Since the original publication, the design specification has been revised by the AISI Committee in 1956, 1960, 1962, 1968, 1980, 1986 and 1996. The revisions have been the result of technical developments through continuing research.

The provisions for web crippling strength are primarily based on extensive research conducted at Cornell University by Winter and Pian (1946), by Zetlin (1955) and at the University of Missouri Rolla by Hetrakul and Yu (1978). For these experimental investigations the web crippling studies were conducted under the following four load conditions:

1. End-One-Flange (EOF) loading
2. Interior-One-Flange (IOF) loading
3. End-Two-Flange (ETF) loading
4. Interior-Two-Flange (ITF) loading

The four loading conditions are illustrated in Figures 2.7, 2.8, 2.9 and 2.10. In Figures 2.7 and 2.8 the bearing locations are spaced at a distance greater than 1.5 times the depth of the web to avoid two-flange loading.

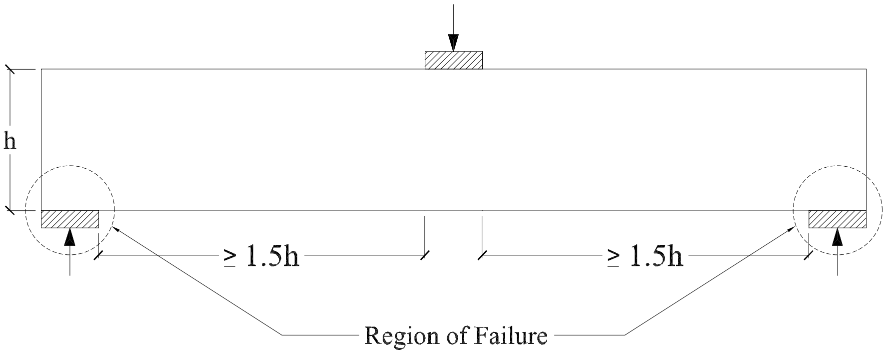

Figure 2.7. End-One-Flange Loading Condition

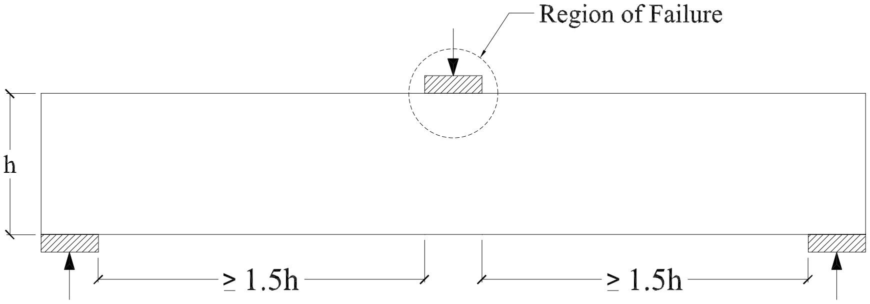

Figure 2.8. Interior-One-Flange Loading Condition

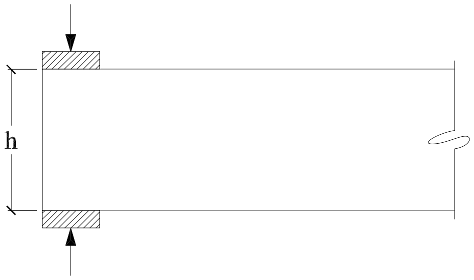

Figure 2.9. End-Two-Flange Loading Condition

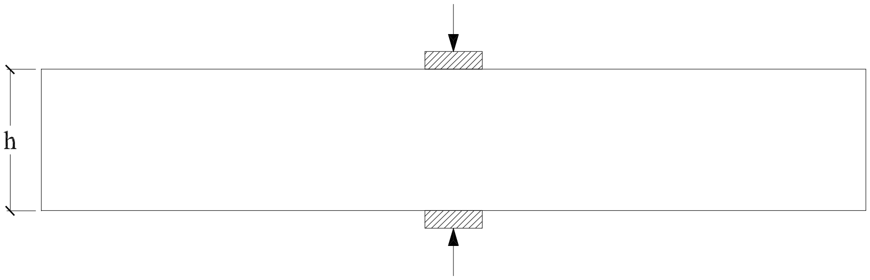

Figure 2.10. Interior-Two-Flange Loading Condition

With four specific loading conditions and various behaviorally unique section geometries, the specification computes the nominal web crippling strength as illustrated in Table 2.2:

Table 2.2. Equation Numbers for Nominal Strength of Webs, P_n, kips (N)

		Shapes Having Single Webs		I-Sections or Similar Sections
		Stiffened or Partially Stiffened Flanges	Unstiffened Flanges	Stiffened, Partially Stiffened and Unstiffened Flanges
Opposing Loads Spaced > 1.5h	End Reaction	Eq. (2.12)	Eq. (2.13)	Eq. (2.14)
	Interior Reaction	Eq. (2.15)	Eq. (2.15)	Eq. (2.16)
Opposing Loads Spaced ≤ 1.5h	End Reaction	Eq. (2.17)	Eq. (2.17)	Eq. (2.18)
	Interior Reaction	Eq. (2.19)	Eq. (2.19)	Eq. (2.20)

Equations for Table 2.2:

$$t^2kC_3C_4C_9C_\theta[331-0.61(h/t)][1+0.01(N/t)] \qquad (2.12)$$

$$t^2kC_3C_4C_9C_\theta[217-0.28(h/t)][1+0.01(N/t)] \qquad (2.13)$$

When N/t > 60, the factor [1+0.01(N/t)] may be increased to [0.71+0.015(N/t)]

$$t^2F_yC_6\left(10.0+1.25\sqrt{N/t}\right) \tag{2.14}$$

$$t^2kC_1C_2C_9C_\theta\left[538-0.74(h/t)\right]\left[1+0.007(N/t)\right] \tag{2.15}$$

When N/t > 60, the factor [1+0.007(N/t)] may be increased to [0.75+0.011(N/t)]

$$t^2F_yC_5(0.88+0.12m)\left(15.0+3.25\sqrt{N/t}\right) \tag{2.16}$$

$$t^2kC_3C_4C_9C_\theta\left[244-0.57(h/t)\right]\left[1+0.01(N/t)\right] \tag{2.17}$$

$$t^2F_yC_8(0.64+0.31m)\left(10.0+1.25\sqrt{N/t}\right) \tag{2.18}$$

$$t^2kC_1C_2C_9C_\theta\left[771-2.26(h/t)\right]\left[1+0.0013(N/t)\right] \tag{2.19}$$

$$t^2F_yC_7(0.82+0.15m)\left(15.0+3.25\sqrt{N/t}\right) \tag{2.20}$$

Where:

P_n = Nominal stength for concentrated load or reaction per web, kips (N)

$C_1 = 1.22 - 0.22k$

$C_2 = 1.06 - 0.06R/t \leq 1.0$

$C_3 = 1.33 - 0.33k$

$C_4 = 1.15 - 0.15R/t \leq 1.0$ but not less than 0.50

$C_5 = 1.49 - 0.53k \geq 0.6$

$$C_6 = 1 + \left(\frac{h/t}{750}\right) \text{when } h/t \leq 150$$

$$= 1.20 \text{ when } h/t > 150$$

$$C_7 = 1/k \text{ when } h/t \leq 66.5$$

$$= \left[1.10 - \frac{h/t}{665}\right]\frac{1}{k}, \text{ when } h/t > 66.5$$

$$C_8 = \left[0.98 - \frac{h/t}{865}\right]\frac{1}{k}$$

$C_9 = 1.0$ for U.S. customary units, kips and in

$= 6.9$ for metric units, N and mm

$C_\theta = 0.7 + 0.3(\theta/90)^2$

F_y = Design yield stress of the web

h = Depth of the flat portion of the web measured along the plane of the web, in. (mm)

$k = 894F_y / E$

m = t/0.075, when t is in inches

m = t/1.91, when t is in mm

t = Web thickness, in. (mm)

N = Actual length of bearing, in. (mm)

R = Inside bend radius

θ = Angle between the plane of the web and the plane of the bearing surface

The equations in Table 2.2 apply to beams when $R/t \leq 6$ and to decks when $R/t \leq 7$, $N/t \leq 210$ and $N/h \leq 3.5$. For Z-sections with flanges bolted to the end support member, Equation 2.12 may be multiplied by 1.3 for sections meeting limitations as specified previously in Section 2.3.3.

Equations 2.12, 2.13 and 2.17 were developed based on testing involving specimens with F_y less than 55 ksi. For beams with $F_y \geq$ 65 ksi, the value of kC_3 shall be taken as 1.34.

2.4.2. 2001 Specification. In an effort to modify the 1996 AISI expression for web crippling strength, the following expression was introduced:

$$P_n = Ct^2F_y \sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \tag{2.21}$$

Where:

P_n = Nominal web crippling strength

C = Coefficient

C_h = Web slenderness coefficient

C_N = Bearing length coefficient

C_R = Inside bend radius coefficient

h = Flat dimension of the web measured in the plane of the web

N = Bearing length

R = Inside bend radius

t = Web thickness

θ = Angle between the plane of the web and the plane of the bearing surface, $(45° < \theta \leq 90°)$

P_n represents the nominal web crippling strength for a load or reaction of one solid web element connecting top and bottom flanges. For webs consisting of two or more sheets, P_n shall be calculated for each sheet and added together to obtain the nominal load or reaction for the section. For the above-referenced coefficients the values are assembled in Tables 2.3, 2.4, 2.5, 2.6 and 2.7. The coefficients in these tables are assembled based on an accumulation of over sixty years of web crippling test data. The results and recommendations of this study are based in reference to Equation 2.21 for web crippling strength.

Table 2.3. Multi-Web Deck Sections

Support Conditions	Load Cases		C	C_R	C_N	C_h	Ω_W	Φ_W	Limits
Fastened to Support	One-Flange Loading or Reaction	End	3	0.08	0.70	0.055	2.25	0.65	$R/t \leq 7$
		Interior	8	0.10	0.17	0.004	1.75	0.85	$R/t \leq 10$
	Two-Flange Loading or Reaction	End	9	0.12	0.14	0.040	1.80	0.85	$R/t \leq 10$
		Interior	10	0.11	0.21	0.020	1.75	0.85	
Unfastened	One-Flange Loading or Reaction	End	3	0.08	0.70	0.055	2.25	0.65	$R/t \leq 7$
		Interior	8	0.10	0.17	0.004	1.75	0.85	
	Two-Flange Loading or Reaction	End	6	0.16	0.15	0.050	1.65	0.90	$R/t \leq 5$
		Interior	17	0.10	0.10	0.046	1.65	0.90	

The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 3$, $45° < \theta \leq 90°$.

Table 2.4. Single Web Z-Sections

Support Conditions		Load Cases		C	C_R	C_N	C_h	Ω_W	Φ_W	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	$R/t \leq 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	$R/t \leq 5$
		Two-Flange Loading or Reaction	End	9	0.05	0.16	0.052	1.75	0.85	$R/t \leq 12$
			Interior	24	0.07	0.07	0.04	1.85	0.80	$R/t \leq 12$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	5	0.09	0.02	0.001	1.80	0.85	$R/t \leq 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	$R/t \leq 3$
			Interior	24	0.52	0.15	0.001	1.90	0.80	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.80	0.85	$R/t \leq 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	$R/t \leq 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	$R/t \leq 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	

The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$ and $\theta = 90°$.

Table 2.5. Single Hat Sections

Support Conditions	Load Cases		C	C_R	C_N	C_h	Ω_W	Φ_W	Limits
Fastened to Support	One-Flange Loading or Reaction	End	5	0.25	0.68	0.04	2.00	0.75	$R/t \leq 5$
		Interior	17	0.13	0.13	0.04	1.90	0.80	$R/t \leq 10$
	Two-Flange Loading or Reaction	End	9	0.10	0.07	0.03	1.75	0.85	$R/t \leq 10$
		Interior	10	0.14	0.22	0.02	1.80	0.85	
Unfastened	One-Flange Loading or Reaction	End	4	0.25	0.68	0.04	2.00	0.75	$R/t \leq 4$
		Interior	17	0.13	0.13	0.04	1.70	0.90	$R/t \leq 4$

The above coefficients apply when $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 2$ and $\theta = 90°$.

Table 2.6. Built-Up Sections

Support Conditions		Load Cases		C	C_R	C_N	C_h	Ω_W	Φ_W	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	2.00	0.75	$R/t \leq 5$
			Interior	20	0.15	0.05	0.003	1.65	0.90	$R/t \leq 5$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	2.00	0.75	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	$R/t \leq 3$
		Two-Flange Loading or Reaction	End	15.5	0.09	0.08	0.04	2.00	0.75	$R/t \leq 3$
			Interior	36	0.14	0.08	0.04	2.00	0.75	
	Unstiffened Flanges	Two-Flange Loading or Reaction	End	10	0.14	0.28	0.001	2.00	0.75	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	1.75	0.85	$R/t \leq 3$

This table applies to I-beams made from two channels connected back to back.

The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 1.0$ and $\theta = 90°$.

Table 2.7. Single Web Channel and C-Sections

Support Conditions		Load Cases		C	C_R	C_N	C_h	Ω_W	Φ_W	Limits
Fastened to Support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.75	0.85	$R/t \leq 9$
			Interior	13	0.23	0.14	0.01	1.65	0.90	$R/t \leq 5$
		Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	1.75	0.85	$R/t \leq 12$
			Interior	20	0.10	0.08	0.031	1.75	0.85	$R/t \leq 12$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	1.85	0.80	$R/t \leq 5$
			Interior	13	0.23	0.14	0.01	1.65	0.90	
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	1.65	0.90	$R/t \leq 3$
			Interior	24	0.52	0.15	0.001	1.90	0.80	
	Unstiffened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	1.80	0.85	$R/t \leq 2$
			Interior	13	0.32	0.10	0.01	1.80	0.85	$R/t \leq 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	2.00	0.75	$R/t \leq 1$
			Interior	13	0.47	0.25	0.04	1.90	0.80	

The above coefficients apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$ and $\theta = 90°$.

3. END-ONE-FLANGE OVERHANG STUDY

3.1. INTRODUCTION

The web crippling provisions of the design specification (AISI 2001) for cold-formed steel flexural members is based on over forty years of research covering various conditions of loading and support placement. The recommendations of this study focused on an end-one-flange loading condition for single web elements bolted to the supports. Further limitations for the recommendations are noted in subsequent sections.

3.2. EXPERIMENTAL INVESTIGATION

An experimental study performed at the University of Missouri – Rolla focused on the web crippling capacity of a single web section loaded under an end-one-flange condition with a variance in overhang length beyond the end support. The purpose of the investigation was to develop a modification to the design specification that accounts for an increase in the web crippling capacity between an end-one-flange and an interior-one-flange loading condition. The web crippling provisions (AISI 2001) do not account for an increase in the web crippling capacity in this region.

3.2.1. Test Specimens. The test specimens were fabricated at the University of Missouri – Rolla Civil Engineering Structural Laboratory. The specimens consisted of edge-stiffened C- and Z-sections. The cross sectional parameters for each section are summarized in Table 1.1

Since it is difficult to load a single C-section or single Z-section, the sections were fabricated into box specimens to develop lateral and torsional stability. The sections were fastened together with aluminum angles located at the ends and 1/3 of the length using self-drilling screws as shown in Figure 3.1. Specimen parameters were based on geometry, depth, thickness and overhang length. Table 3.1 documents the specimens prepared and the corresponding parameters.

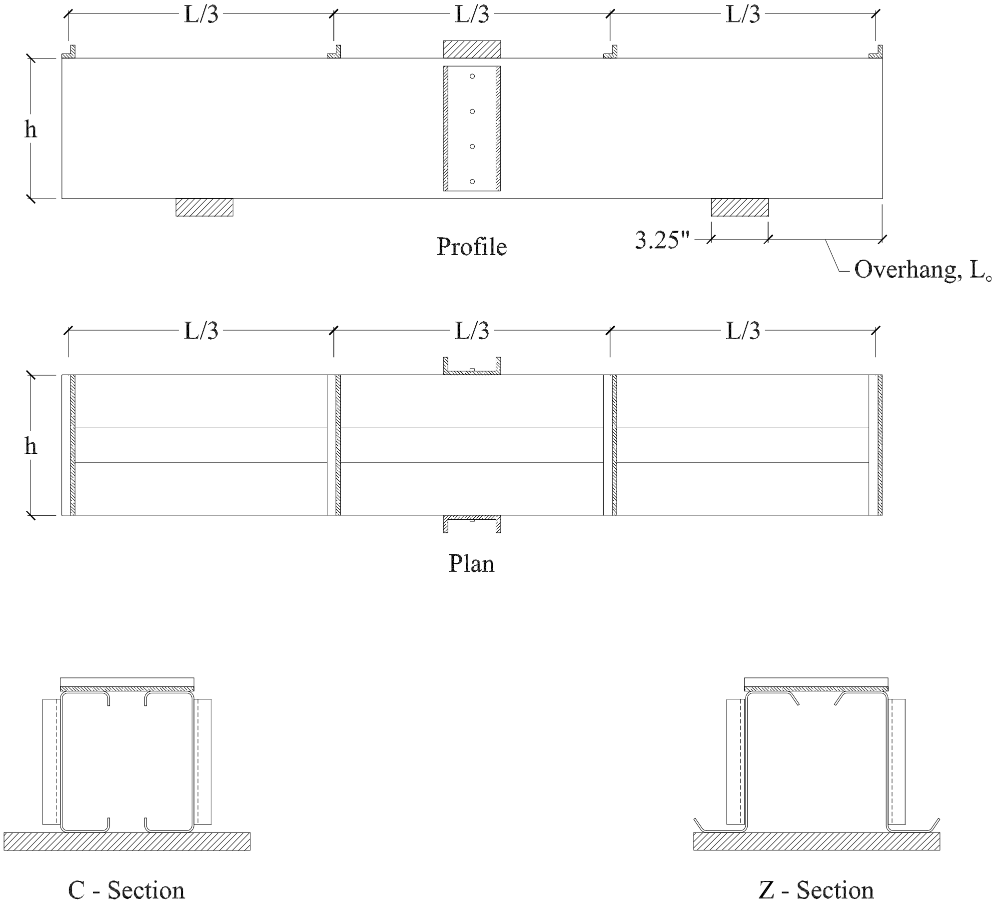

Figure 3.1. Specimen Fabrication Layout

A total of 29 specimens were fabricated and tested. Each specimen was coded based on cross-sectional parameters. An explanation of the specimen coding is given in Figure 3.2.

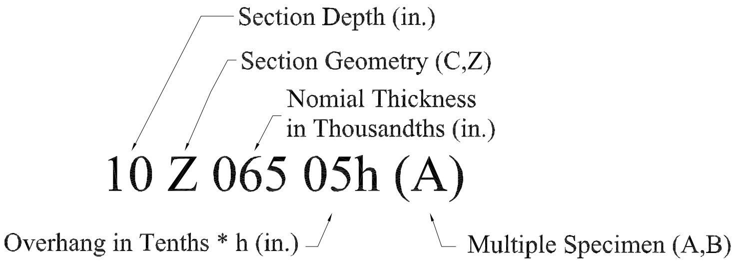

Figure 3.2. Specimen Coding Definition

Table 3.1. Specimen Parameters

Section	Specimen	Thickness (in.)	Depth (in.)	Length (L) (in.)	Overhang (in.)
C	8C05805h	0.0618	8.0	41.75	4.0
	8C05810h	0.0618	8.0	49.75	8.0
	8C05815h	0.0618	8.0	57.75	12.0
	8C11305h	0.1032	8.0	41.75	4.0
	8C11310h	0.1032	8.0	49.75	8.0
	8C11315h	0.1032	8.0	57.75	12.0
	12C06805h	0.0724	12.0	57.75	6.0
Z	8Z05805h	0.0625	8.0	41.75	4.0
	8Z05810h	0.0625	8.0	49.75	8.0
	8Z05815h	0.0625	8.0	57.75	12.0
	8Z11305h	0.1069	8.0	41.75	4.0
	8Z11310h	0.1069	8.0	49.75	8.0
	8Z11315h	0.1069	8.0	57.75	12.0
	10Z06505h	0.0600	10.0	49.75	5.0
	10Z06510h	0.0600	10.0	54.75	10.0
	10Z06515h	0.0600	10.0	59.75	15.0
	10Z10905h	0.1041	10.0	49.75	5.0
	10Z10910h	0.1041	10.0	54.75	10.0
	10Z10915h	0.1041	10.0	59.75	15.0

3.2.2. Mechanical Properties. For each section tested, four tensile coupons were taken from the webs for mechanical testing. The coupons were tested according to the American Society of Testing and Materials (ASTM A370, 1992). For galvanized specimens, the galvanizing was removed using a hydrochloric solution to allow for measurement of the base metal thickness. To measure the thickness of the painted specimens, the coupons were wire brushed to remove exterior coating. Table 3.2 summarizes the mechanical properties for the specimens tested.

Table 3.2. Mechanical Properties of Test Specimens

Specimen	t (in.)	Fy (ksi)	Fu (ksi)	% Elongation[*]
8C058	0.0618	68.08	80.84	16.36
8C113	0.1032	58.72	73.92	11.96
8Z058	0.0625	64.18	77.53	29.52
8Z113	0.1069	55.08	72.68	14.54
10Z065	0.0600	71.31	79.79	13.74
10Z119	0.1041	67.62	77.53	14.13
12C068	0.1060	45.25	64.03	14.77

[*] Based on 1 in. gauge length

3.2.3. Test Setup. To develop failure at the support locations, stiffeners were attached to the center of each specimen directly beneath the load point (Figure 3.1). The stiffeners consisted of channels attached to the webs of the specimens using self-drilling screws.

Each specimen was tested using an MTS machine as shown in Figure 3.3. The applied load and support restraints were transferred through I-sections having a flange width of 3¼ inches. Thus, the bearing length for all tests was 3¼ inches. Each specimen was carefully positioned and aligned with the platens of the testing machine.

Figure 3.3. Typical Specimen Testing Configuration

3.2.4. Test Procedure. The loading of the test specimens was controlled through the digital control panel of the MTS loading machine. Each specimen was loaded at a rate of 500 lbs/min to maintain consistency. The deflection of each specimen was recorded using the stroke of the platens. The recording of the load vs. deflection data was performed using the control panel of the MTS machine and data recording software.

Each specimen was loaded to failure to observe the behavior of the failure mechanism. For some of the specimens, failure was determined to occur at a point of excessive deformation.

3.3. TEST RESULTS

The per web failure load for each specimen is denoted as P_t and was calculated by dividing the resulting specimen failure load by four. Table 3.3 summarizes the recorded failure information for each specimen tested.

3.3.1. Behavioral Characteristics of Failure.

As previously noted, failure was determined as the maximum sustained load, or the load at which excessive deformation occurred. As each specimen failed, the behavior, or mechanism by which failure occurred, was observed. The specimens were studied for reoccurring patterns in failure behavior.

Two significant mechanisms of failure were observed during the course of the investigation. Each specimen failed either by a yielding mechanism in the area directly above the support, or by a buckling mechanism in which the entire specimen would buckle through a twisting motion along the length. The buckling mechanism occurred at the onset of a diagonal web buckle extending from the center of the specimen to the area above the support. The transition from a yielding to a buckling failure mechanism correlated to the h/t ratio as was evident in the resulting data patterns. Figure 3.4 illustrates the onset of a local yielding occurring in the area directly above the support reaction.

Table 3.3. End-One-Flange Test Results

							AISI (2001)		Overhang Coeff.	
Specimen	t (in.)	Fy (ksi)	h/t	N/t	R/t	P_t (kips/web)	P_n (kips/web)	P_t/P_n	P_c (kips/web)	P_t/P_c (kips/web)
8C05805h(A)	0.0618	68.08	119	52.59	4.06	2.077	2.065	1.006	2.065	1.006
8C05805h(B)	0.0618	68.08	119	52.59	4.06	1.990	2.065	0.964	2.065	0.964
8C05810h(A)	0.0618	68.08	119	52.59	4.06	2.475	2.065	1.199	2.065	1.199
8C05810h(B)	0.0618	68.08	119	52.59	4.06	2.185	2.065	1.058	2.065	1.058
8C05815h(A)	0.0618	68.08	119	52.59	4.06	3.351	2.065	1.623	2.238	1.497
8C05815h(B)	0.0618	68.08	119	52.59	4.06	2.800	2.065	1.356	2.238	1.251
8C11305h(A)	0.1032	58.72	70	31.49	2.63	6.519	4.773	1.366	5.730	1.138
8C11305h(B)	0.1032	58.72	70	31.49	2.63	6.348	4.773	1.330	5.730	1.108
8C11310h(A)	0.1032	58.72	70	31.49	2.63	8.004	4.773	1.677	6.862	1.166
8C11310h(B)	0.1032	58.72	70	31.49	2.63	7.536	4.773	1.579	6.862	1.098
12C06805h(A)	0.1060	45.25	108	30.66	1.78	3.790	3.850	0.984	3.850	0.984

Table 3.3. End-One-Flange Test Results (Continued)

							AISI (2001)		Overhang Coeff.	
Specimen	t (in.)	Fy (ksi)	h/t	N/t	R/t	P_t (kips/web)	P_n (kips/web)	P_t/P_n	P_c (kips/web)	P_t/P_c (kips/web)
8Z05805h(A)	0.0625	64.18	116	52.00	5.00	1.893	1.907	0.993	1.907	0.993
8Z05805h(B)	0.0625	64.18	116	52.00	5.00	1.720	1.907	0.902	1.907	0.902
8Z05810h(A)	0.0625	64.18	116	52.00	5.00	2.300	1.907	1.206	1.907	1.206
8Z05810h(B)	0.0625	64.18	116	52.00	5.00	2.356	1.907	1.236	1.907	1.236
8Z05815h(A)	0.0625	64.18	116	52.00	5.00	2.406	1.907	1.262	2.114	1.138
8Z05815h(B)	0.0625	64.18	116	52.00	5.00	2.347	1.907	1.231	2.114	1.110
8Z11305h(A)	0.1069	55.08	67	30.40	2.80	6.582	4.721	1.394	5.836	1.128
8Z11305h(B)	0.1069	55.08	67	30.40	2.80	5.906	4.721	1.251	5.836	1.012
8Z11310h(A)	0.1069	55.08	67	30.40	2.80	7.008	4.721	1.484	6.989	1.003
8Z11310h(B)	0.1069	55.08	67	30.40	2.80	7.055	4.721	1.494	6.989	1.010
8Z11315h(A)	0.1069	55.08	67	30.40	2.80	6.895	4.721	1.460	7.766	0.888

Table 3.3. End-One-Flange Test Results (Continued)

							AISI (2001)		Overhang Coeff.	
Specimen	t (in.)	Fy (ksi)	h/t	N/t	R/t	P_t (kips/web)	P_n (kips/web)	P_t/P_n	P_c (kips/web)	P_t/P_c (kips/web)
8Z11315h(B)	0.1069	55.08	67	30.40	2.80	7.193	4.721	1.524	7.766	0.926
10Z06505h(A)	0.0600	71.30	154	54.17	5.25	1.203	1.872	0.643	1.872	0.643
10Z06510h(A)	0.0600	71.30	154	54.17	5.25	1.735	1.872	0.927	1.872	0.927
10Z06515h(A)	0.0600	71.30	154	54.17	5.25	1.870	1.872	0.999	1.872	0.999
10Z10905h(A)	0.1041	67.62	89	31.22	2.60	4.670	5.446	0.857	5.543	0.843
10Z10910h(B)	0.1041	67.62	89	31.22	2.60	5.491	5.446	1.008	6.637	0.827
10Z10915h(A)	0.1041	67.62	89	31.22	2.60	6.424	5.446	1.180	7.375	0.871

Mean	1.039
Std. Dev.	0.164
Coef. Of Variation	0.158

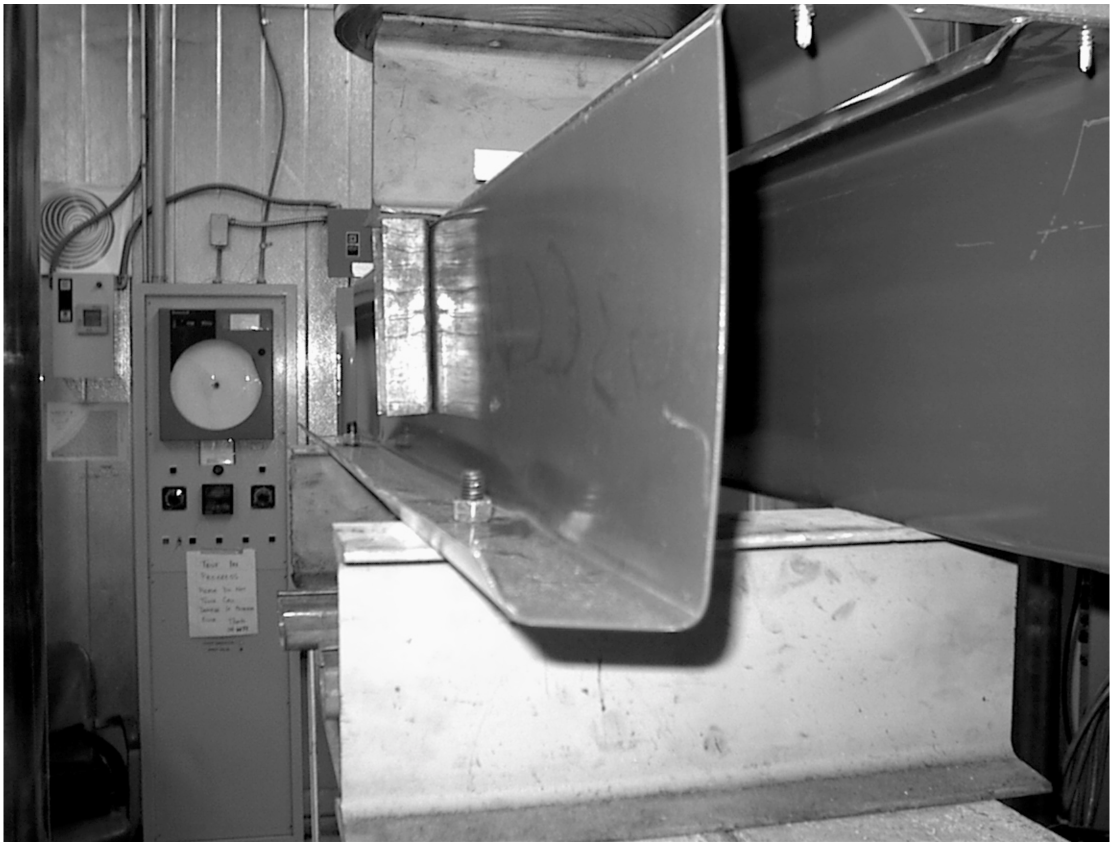

Figure 3.4. Local Yielding of Test Specimen

3.3.2. Deformations at Failure. The deflection of each specimen followed an elastic load vs. deformation relationship until either a yielding or buckling failure occurred. For specimens that failed in a yielding mechanism, the load vs. deflection relationship leveled at a yielding load and then sustained another small increment of load. Figure 3.5 illustrates the load vs. deflection relationship resulted from a yielding failure.

For specimens that failed due to a buckling type of mechanism, the load vs. deflection relationship initially followed the behavior of a yielding mechanism. However, once the yielding region was reached, the overall stability of the specimen failed through buckling. Figure 3.6 exhibits a typical load vs. deflection relationship for a specimen failed through a buckling mechanism

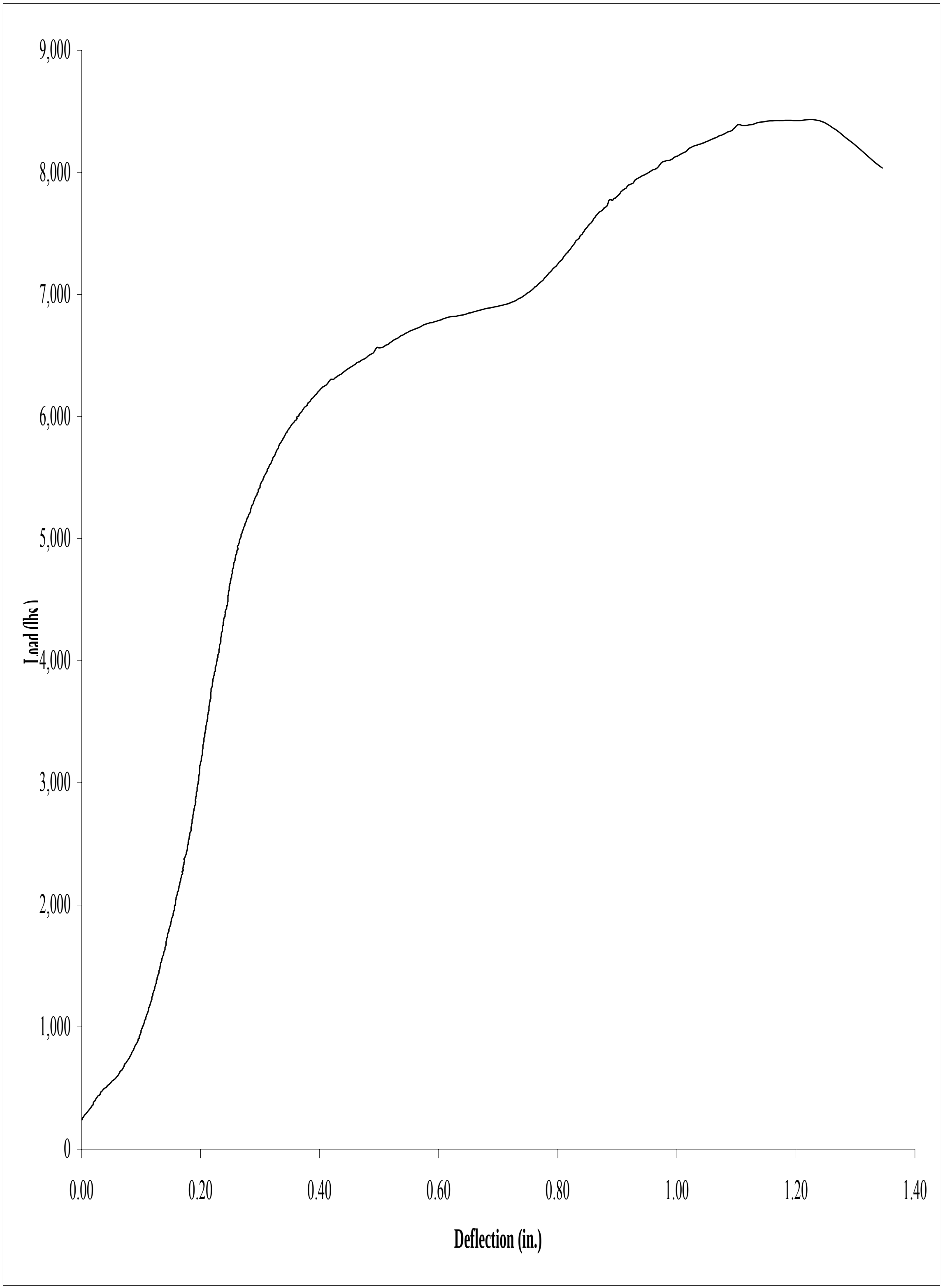

Figure 3.5. Typical Yielding Failure

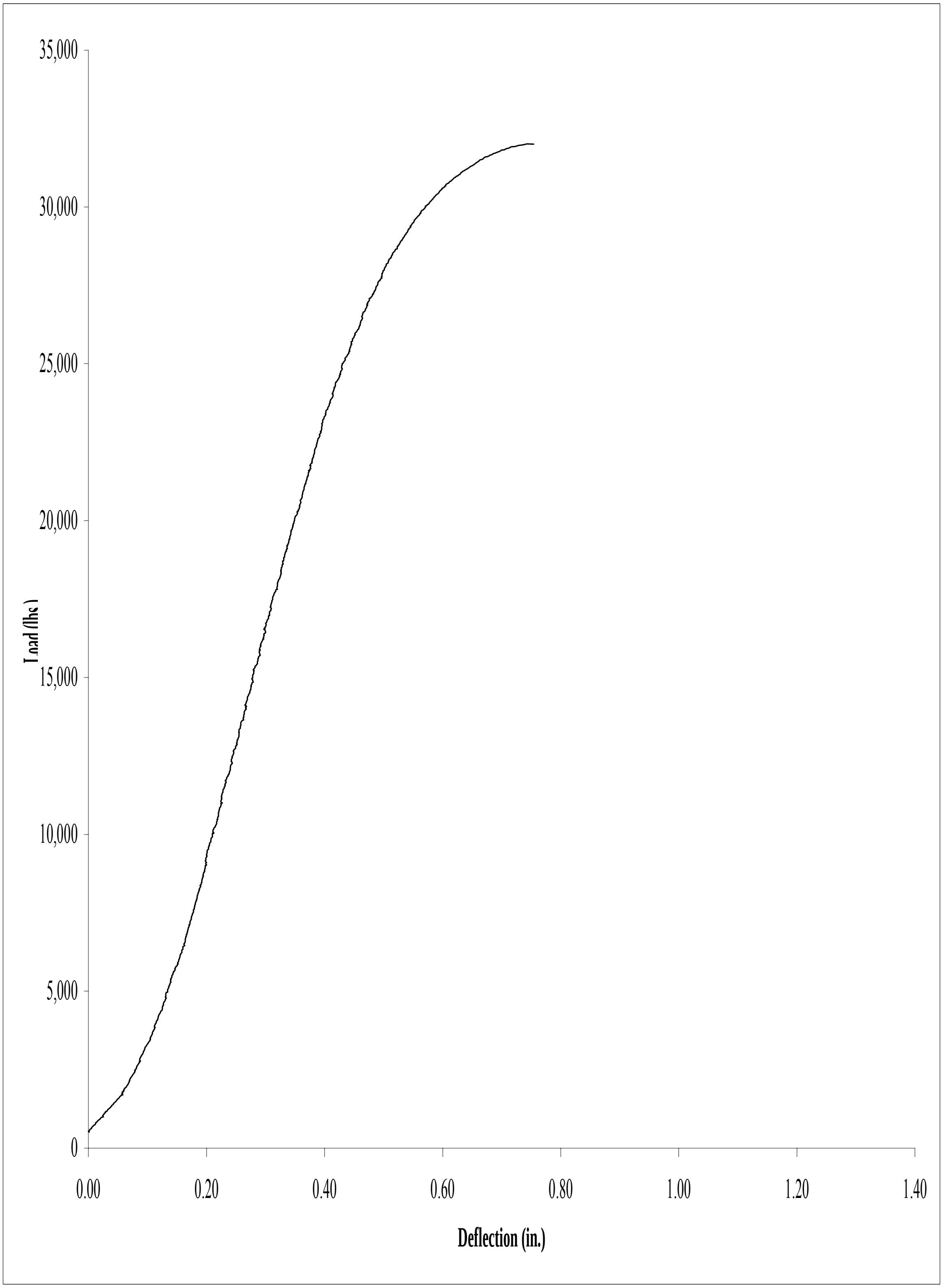

Figure 3.6. Typical Buckling Failure

3.4. EVALUATION OF TEST RESULTS

3.4.1. General. For the specimens tested in this study, the web crippling strength of the web was considered without degradation of the web strength due to bending stress. For the recommendations of this study, the recorded failure load per web for each specimen, P_t, was normalized by division of the corresponding design strength, P_n (EOF), as prescribed by the AISI Specification (Equation 2.21). The ratio of P_t/P_n (EOF) was used for analysis of the test data.

For the data gathered from this study, two factors, h/t ratio and overhang length were considered for analysis. Based on the correlation of h/t ratio and overhang length, design recommendations were formulated in the form of an adjustment factor equation. The effects of h/t ratio and overhang length are discussed further in Section 3.4.2 and Section 3.4.3.

3.4.2. Effect of Overhang on Web Crippling Capacity. Figure 3.7 presents the ratio P_t/P_n as a function of overhang length. Initial inspection of the data suggests a nonlinear relationship between increased web crippling capacity and overhang length. The data demonstrates a significant increase between 0.5h and 1.0h. The rate of increase then levels off as the overhang approaches 1.5h. Because of no further observed increase in capacity, the test program was limited to an upper overhang limit of 1.5h.

3.4.3. Effect of h/t Ratio on Web Crippling Capacity. As expected from plate buckling theory, as the h/t ratio of the specimen increased, the web crippling capacity decreased. Figure 3.7 indicates a behavioral difference between test specimens having h/t > 80 vs. h/t < 80. Figure 3.8 illustrates the P_t/P_n ratio as a function of the specimen h/t ratio. Inspection of Figure 3.8 suggests a linear degradation of the excess web crippling capacity as the h/t ratio increases.

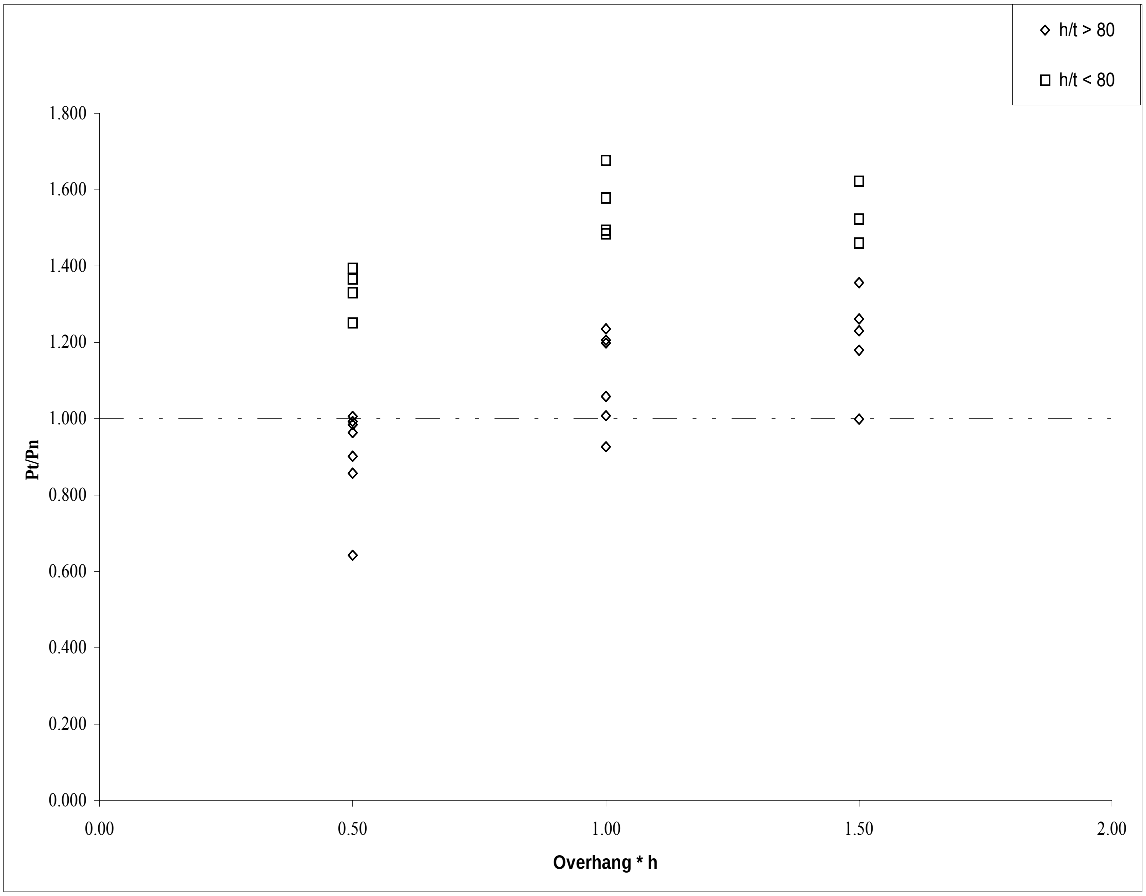

Figure 3.7. End-One-Flange Test Data vs. Overhang

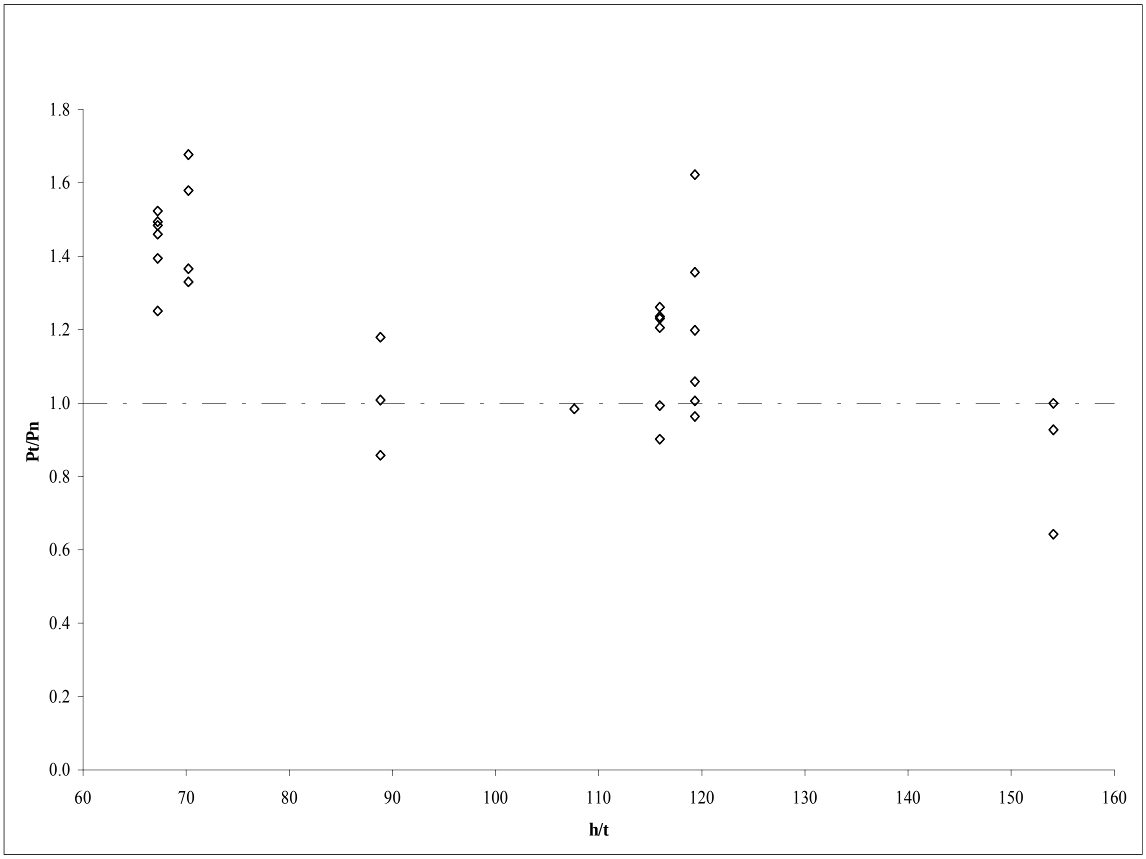

Figure 3.8. End-One-Flange Test Data vs. h/t

3.5. DEVELOPMENT OF A MODIFICATION FACTOR EQUATION

3.5.1. Design Formulation. Further inspection of Figure 3.7 reveals a trend involving two distinct groups of specimens. Investigation of the h/t ratios of the specimens in Figure 3.7 shows a distinct separation based on the h/t ratio of the specimen. Figure 3.9 shows the separation of these two groups of specimens. The separation of the data set suggests two behavioral patterns based on the two previously discussed failure mechanisms, section yielding and specimen buckling.

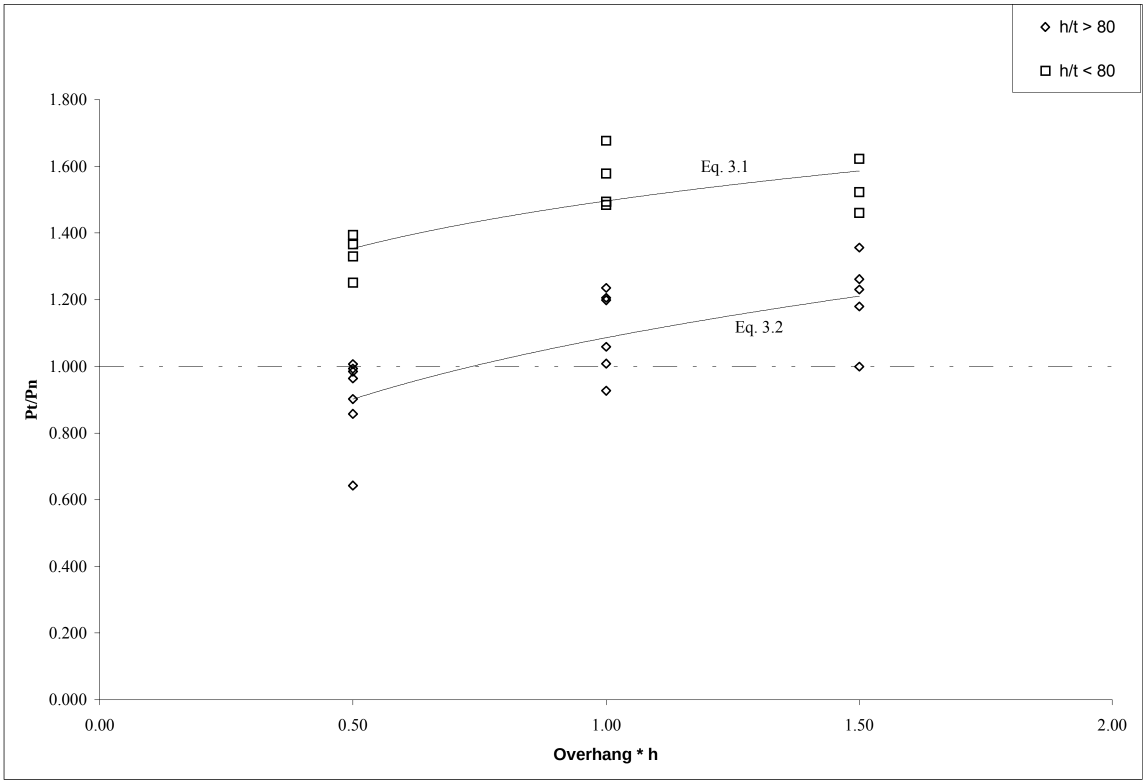

Figure 3.9. Specimen Grouping Based on Behavior

In order to provide an accurate approximation of the web crippling capacity strength increase, the data shown in Figure 3.9 must be normalized by a common function of the h/t ratio. This function was obtained by first correlating the web crippling strength increase of the two groups separately. The data sets were correlated using a power function to reflect the reduction in the rate of increase as the overhang approached 1.5h. Equations 3.1 and 3.2 provide a mathematical correlation for the two data sets.

$$1.50(\mathrm{OH})^{0.14} \tag{3.1}$$

$$1.09(\mathrm{OH})^{0.27} \tag{3.2}$$

Where:

OH = Overhang length expressed in terms of h (e.g. 0.5h)

To reflect the influence of h/t, the values of Equations 3.1 and 3.2 were evaluated at each overhang increment to provide a relative difference in the web crippling capacity increase between the two specimen groups. This influence was examined against the average h/t ratio for each group at the appropriate overhang location. Figure 3.10 demonstrates the web crippling adjustment vs. h/t ratio for each of the three overhang increments.

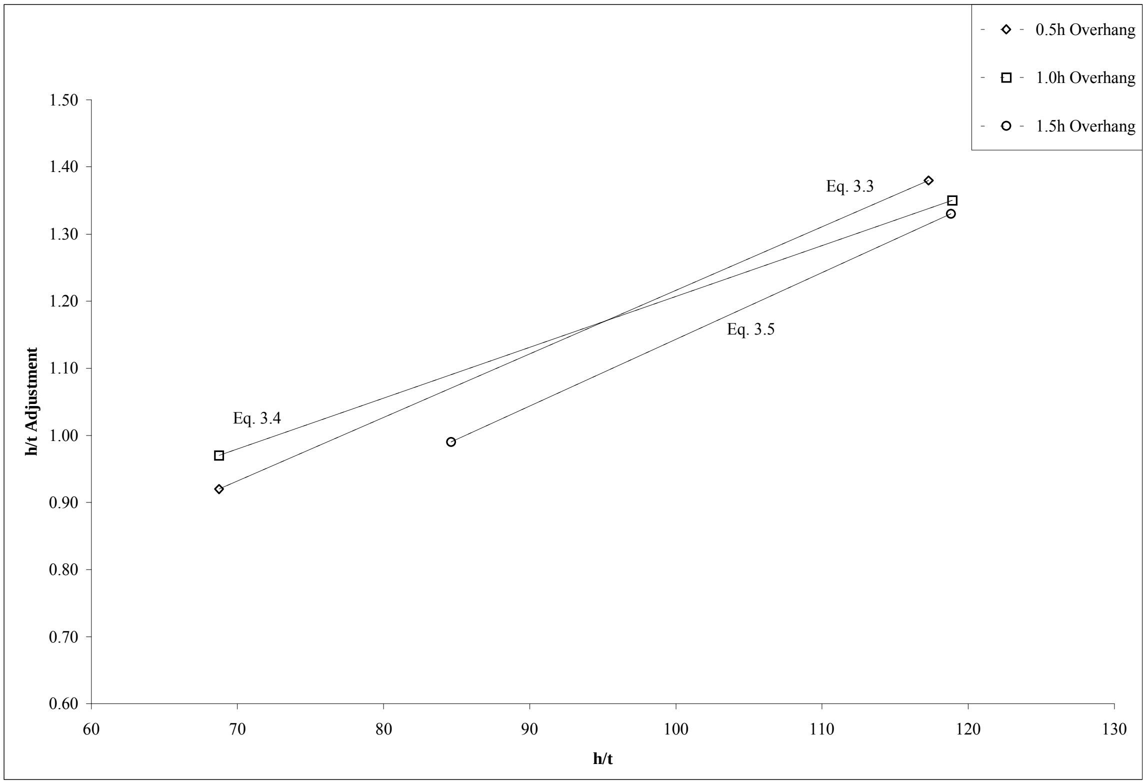

Figure 3.10. Adjustment in Web Crippling Capacity vs. h/t

The linear approximations of the relationships shown in Figure 3.10 are given by Equations 3.3, 3.4 and 3.5.

$$0.010\left(h/t\right)+0.27 \tag{3.3}$$

$$0.008\left(h/t\right)+0.45 \tag{3.4}$$

$$0.010\left(h/t\right)+0.15 \tag{3.5}$$

To provide an approximate normalization of the data set for each increment of overhang, a linear regression of the data shown by Figure 3.10 was used for a final formulation. The final equation representing the influence of specimen h/t ratio is given as Equation 3.6.

$$0.009\left(h/t\right)+0.30 \tag{3.6}$$

Multiplying the data represented in Figure 3.9 by the h/t modification factor given by Equation 3.6 yields the adjusted representation of the end-one-flange data set shown in Figure 3.11, where P_t’ = P_t multiplied by Equation 3.6.

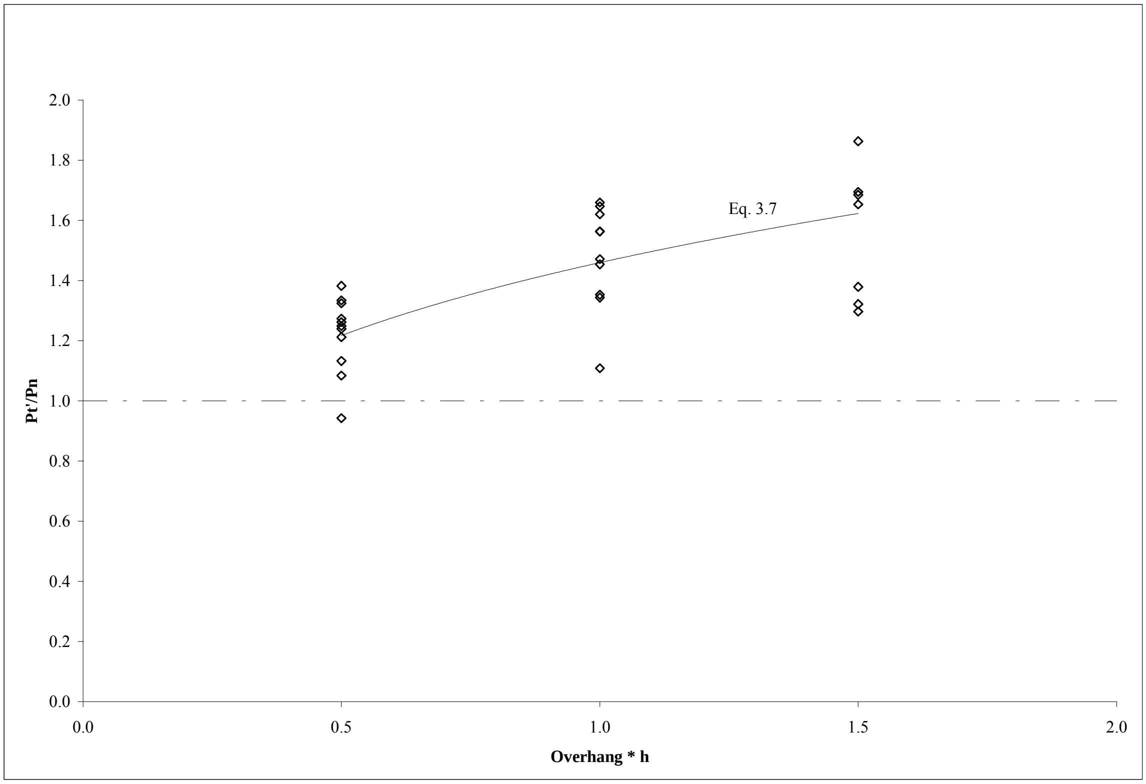

Figure 3.11. Adjusted End-One-Flange Test Data, P_t'/P_n vs. Overhang x h

The nonlinear mathematical model of the data set shown in Figure 3.11 is provided as a power function. Equation 3.7 provides a mathematical correlation for the adjusted data set.

$$1.34(\mathrm{OH})^{0.26} \tag{3.7}$$

Where:

OH = Overhang length expressed in terms of h (e.g. 0.5h)

3.5.2. End-One-Flange Modification Factor. The original data set shown in Figure 3.7 represents the test data in the form of:

$$\frac{P_t}{P_n} \tag{3.8}$$

Where:

P_t = The tested web crippling strength per web

P_n = The nominal end-one-flange web crippling strength per web (AISI 2001)

The adjusted data set shown in Figure 3.11 represents the test data in the adjusted form of:

$$C_{h/t}\left(\frac{P_t}{P_n}\right) \tag{3.9}$$

Where:

P_t = The tested web crippling strength per web

P_n = The nominal end-one-flange web crippling strength per web (AISI 2001)

$C_{h/t}$ = The h/t ratio modification factor given in Equation 3.6

Equation 3.7 expresses the relationship for the modified data set and now assumes the form given as:

$$C_{h/t}\left(\frac{P_t}{P_n}\right) = C_{OH} \tag{3.10}$$

Where:

P_t = The tested web crippling strength per web

P_n = The nominal end-one-flange web crippling strength per web (AISI 2001)

$C_{h/t}$ = The h/t ratio modification factor given in Equation 3.6

C_{OH} = The overhang modification factor given in Equation 3.7

Rearranging the terms of the expression given in Equation 3.10 yields the mathematical model for the computed web crippling capacity of single web elements. Equation 3.11 expresses the web crippling model.

$$P_c = P_n\left(\frac{C_{OH}}{C_{h/t}}\right) \tag{3.11}$$

Where:

P_c = The computed web crippling strength per web

P_n = The nominal end-one-flange web crippling strength per web (AISI 2001)

$C_{h/t}$ = The h/t ratio modification factor given in Equation 3.6

C_{OH} = The overhang modification factor given in Equation 3.7

3.6. DESIGN RECOMMENDATIONS

Substituting Equations 3.6 and 3.7 into the expression given in Equation 3.11 results in the final expression for the computed web crippling strength for single web sections:

$$P_c = P_n\alpha \tag{3.12}$$

Where:

P_c = The computed web crippling strength per web

P_n = The nominal end-one-flange web crippling strength per web (AISI 2001)

$$\alpha = \left[\frac{1.34 \left(L_o / h \right)^{0.26}}{0.009 \left(h / t \right) + 0.30} \right] \geq 1.0 \qquad (3.13)$$

L_o = Overhang length, Figure 3.1 $\left(\frac{L_o}{h} \leq 1.5 \right)$

3.6.1. Evaluation of End-One-Flange Modification Factor. The evaluation of the expression recommended in Equation 3.12 is demonstrated in Figure 3.12 as a comparison between P_c and P_t for the UMR test specimens.

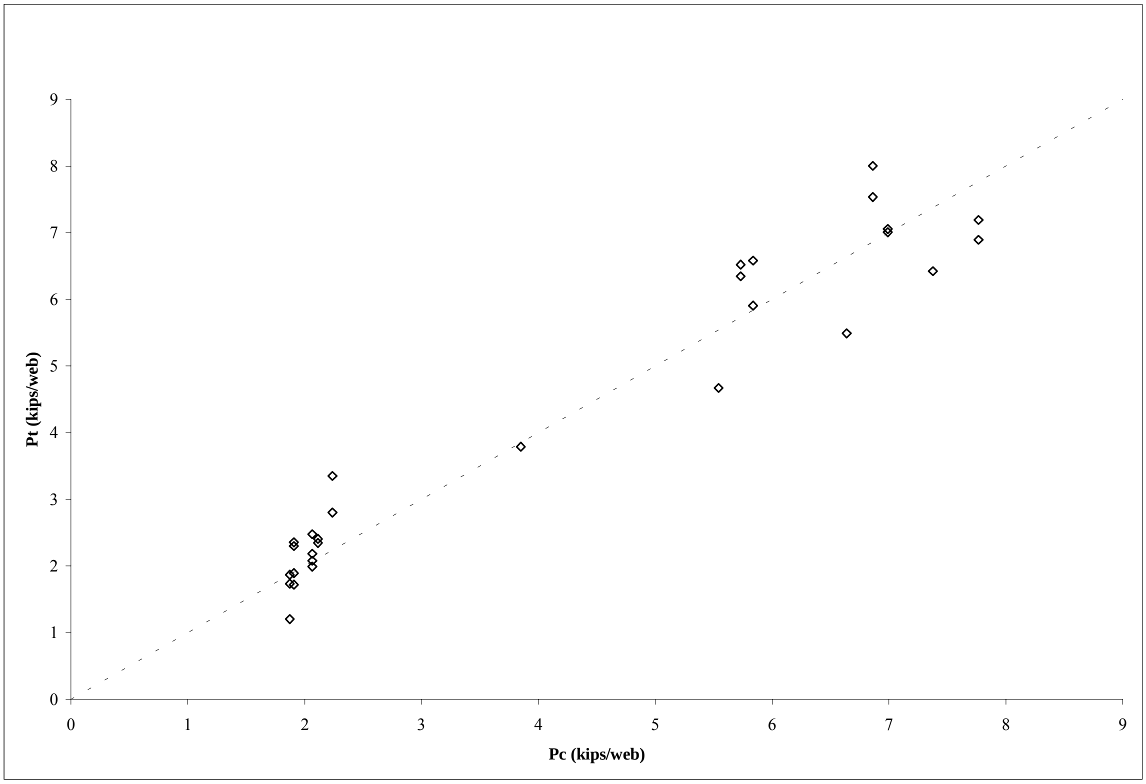

Figure 3.12. EOF Modification Factor Evaluation

A statistical analysis of the recommended modification factor as show in Figure 3.12 yields a mean of 1.039 and a 0.158 coefficient of variation in the accuracy of the recommended equation (Table 3.3). The appropriate factor of safety and phi factor for United States design applications are 1.79 and 0.86, as computed by the AISI specification.

3.6.2. Limitations. The recommended expression given by Equation 3.12 reflects a mathematical approximation of the behavior and resultant failure loads from the end-one-flange investigation performed at the University of Missouri – Rolla. The factors of the recommended expression should be maintained within the limits of the investigation. The governing limits of the recommended web crippling strength modification equation are given in Table 3.4. Parameters not listed in Table 3.4 must conform to the limits of Equation 2.21.

Table 3.4. End-One-Flange Web Crippling Strength Modification Factor Limitations

	$\frac{L_o}{h}$	h/t
Minimum	0.5	67
Maximum	1.5	154

4. CONCLUSIONS

A total of 29 End-One-Flange specimens were tested for web crippling capacity of C- and Z-sections with incremental increases in overhang length. An analysis of the resulting data produced a modification equation (Equation 3.12) to allow for the increase in web crippling strength as the overhang length increased beyond 0.5h. A comparison of Equation 3.12 with AISI 2001 and the tested specimens showed Equation 3.12 to provide a more accurate model of the behavior of the web crippling strength of single web sections with varying lengths of overhang.

Equation 3.12 can be easily applied in practice resulting in a more economical solution where web crippling is the limiting factor of design. Limitations have been provided to allow Equation 3.12 to only be applied to applications where the behavior of the section has been studied.

5. RECOMENDATIONS FOR FUTURE RESEARCH

The results of this research study are a continuation of over forty years of web crippling investigations. With each investigation, the understanding and knowledge concerning the web crippling limit state increases. This investigation focused on a confined area of End-One-Flange single web specimens with varying lengths of overhang. Possible areas of interest for future studies include:

i. Different cross-sections from those in this investigation

ii. Sections with parameters beyond the limitations included in this investigation

According to the AISI Specification (2001), the loading condition of Interior-One-Flange controls the web crippling design for specimens whose overhang is equal to or greater than 1.5h. For the specimens tested in this investigation the web crippling strength for an interior load was evaluated. Figure 5.1 illustrates the comparison between P_t and P_n (IOF) as calculated per AISI 2001. The web crippling strength for an Interior-One-Flange loading condition was not reached at 1.5h by the specimens of this study. An evaluation of the AISI specification concerning the controlling distance of 1.5h would serve as a beneficial future investigation for other load applications.

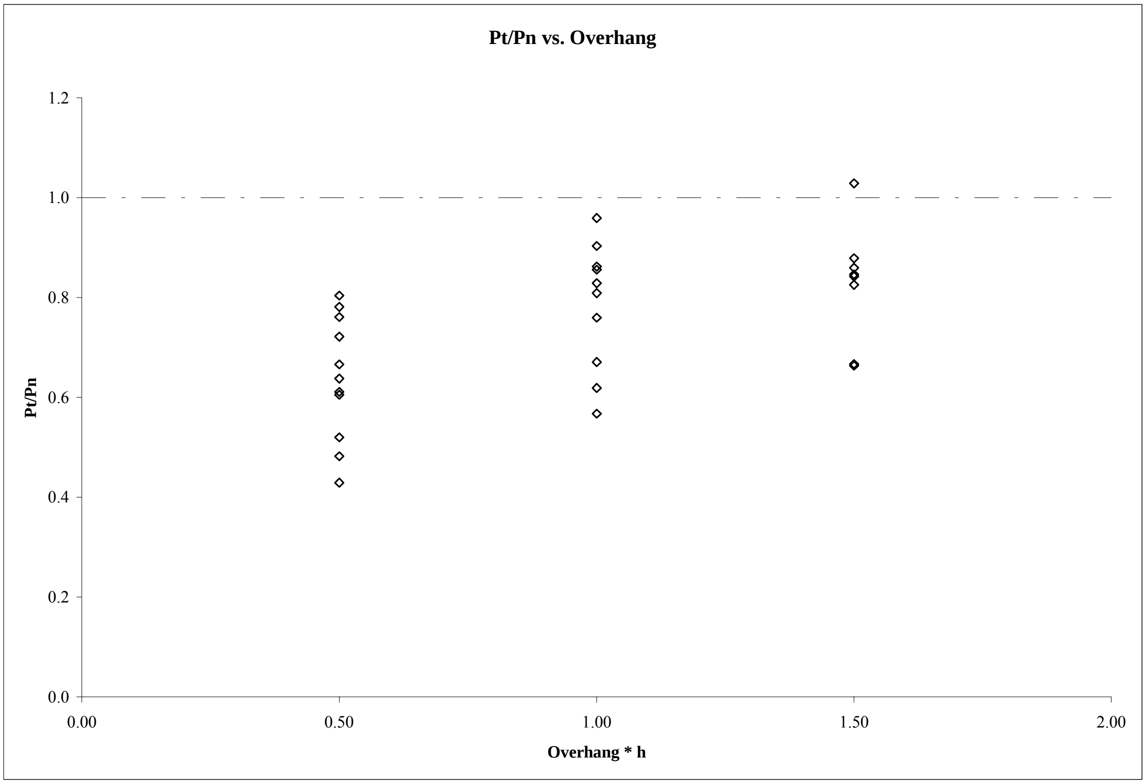

Figure 5.1. Interior-One-Flange Analysis of the Test Data

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