

**WEB-CRIPPLING STRENGTH OF MULTI-WEB COLD-FORMED
STEEL DECK SECTIONS SUBJECTED TO END ONE FLANGE
(EOF) LOADING**

By:
Onur Avci

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Approved:

W. Samuel Easterling, Chair

Thomas M. Murray

Raymond H. Plaut

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Blacksburg, VA 24061

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Committee Chairman: W. Samuel Easterling

Via Department of Civil and Environmental Engineering

(ABSTRACT)

The AISI (1996) Specification for the Design of Cold-Formed Steel Structural Members provisions for web-crippling are believed to be conservative for multi-web deck sections. They are based on unfastened specimens and are limited to the use of decks with certain geometric parameters. The unified web crippling equation of the North American (2002) Specification for the Design of Cold-Formed Steel Structural Members (adopted from Canadian S136-94 Specification) is also limited to certain geometric parameters. Although it has new web crippling coefficients for different load cases and different end conditions, in the End One Flange (EOF) loading case, coefficients for the unfastened configuration were used as a conservative solution for the fastened case because there was no directly applicable test data available in the literature.

This thesis presents the results of an experimental study on web-crippling strength of multiple-web cold-formed steel deck sections subjected to End One Flange (EOF) loading. Seventy-eight tests were conducted at Virginia Tech. Test specimens lying inside and outside of certain geometric parameters of the specifications were tested with both unrestrained and restrained end conditions. Test specimens lying inside the specification parameters have revealed conservative results in the prediction of web crippling capacity using both AISI (1996) and North American (2002) equations. Using the unified web-crippling equation of North American Specification, a nonlinear regression analysis was performed to update the unfastened case coefficients and derive new fastened case coefficients. Also, the calibration of these coefficients is done for both Canadian S136 (1994) and AISI (1996) specifications.

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LIST OF IMPORTANT SYMBOLS

C	Coefficient depending on the section type
C_h	Web slenderness coefficient
C_N	Bearing length coefficient
C_R	Inside bend radius coefficient
C.O.V.	Coefficient of variation
D	Total depth of the deck
E	Young's modulus of steel
EOF	End One Flange Loading
ETF	End Two Flange Loading
F_y	Yield strength of steel
h	Flat dimension of web measured in plane of web
IOF	Interior One Flange Loading
ITF	Interior Two Flange Loading
N	Bearing length
p	Pitch length
P_m	Mean
P_n	Computed web crippling strength
P_t	Web crippling strength in the test
R	Inside bend radius
t	Thickness of the web
V_p	Coefficient of variation
β	Reliability index
θ	Angle between the plane of the web and plane of bearing surface
Ω	Factor of safety
ϕ	Resistance factor
σ	Standard deviation

CHAPTER 1: INTRODUCTION

1.1 General

Cold-formed steel and hot-rolled steel are the two main steel material types that are used in the steel industry. Although hot-rolled steel is more familiar to structural engineers, the use and importance of cold-formed steel is growing in building construction.

Starting from the 1950's cold-formed steel was used as cladding for walls and as decking for floors and roofs. Advances in manufacturing technology made the production of heavier gauge cold-formed steel sections possible. Subsequently, cold-formed steel started to be used as an alternative to hot-rolled steel and timber structural members due to its versatility, high strength-to-weight ratio and economical considerations. Today, cold-formed steel is being used in roof and floor decks, roof trusses and primary structural members in residential and commercial applications.

Unlike hot-rolled steel sections, cold-formed steel sections are produced by cold forming operations: "press braking" and "roll forming." Sections with inclined webs and different types of intermediate or edge stiffeners can be formed with these production methods (Bakker 1992). Curved transition between the webs, flanges and stiffeners are the results of the cold forming operations (Bakker 1992). (Fig. 1.1)

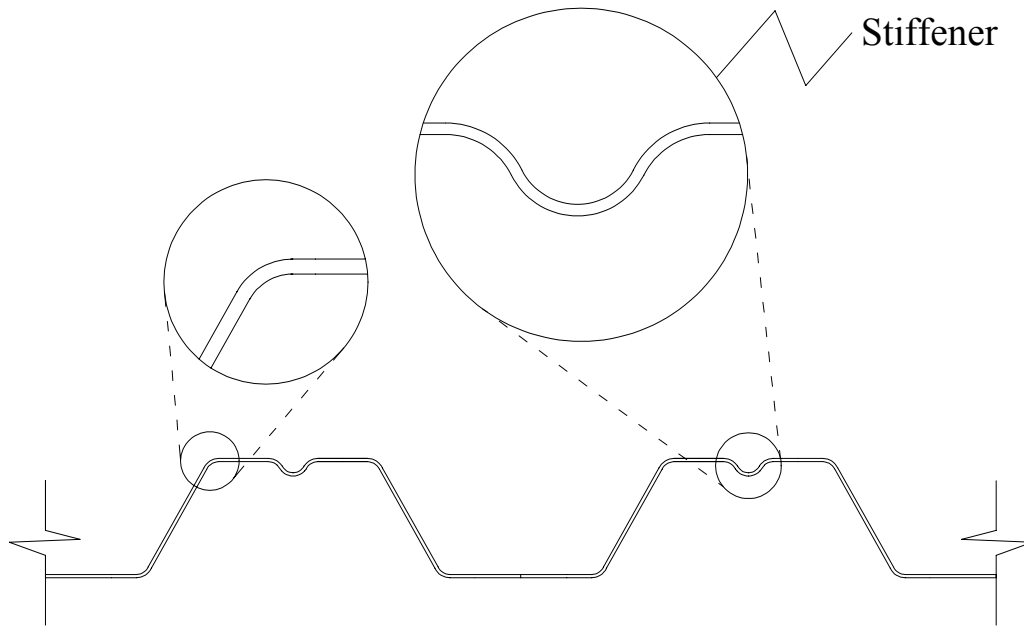


Figure 1.1 Curved transition between the webs, flanges and stiffeners

Width-to-thickness ratios of cold-formed steel sections are relatively high compared to hot-rolled steel sections. This property of the cold-formed steel sections causes local buckling at stress levels lower than the actual yield stress of the steel. However, it is the redistribution of the stresses that allows the member to continue to carry loads after local buckling. The ability of the section to carry loads after local buckling is called post-buckling behavior.

Web crippling is one of the failure modes that must be taken into consideration in cold-formed steel design. Cold-formed steel members may experience web-crippling failure due to the high local intensity of loads and/or reactions.

Investigation of web crippling behavior of cold formed steel members started in 1939 at Cornell University. Based on the research under the direction of George Winter, the first American Iron and Steel Institute design specification was published (AISI, 1946). The first codes for cold formed steel design in Canada were issued in 1963, while it was the 1970's when the first European cold formed steel codes were published (CSA, 1963). Based on the results of experimental research, the design provisions of AISI were

revised in 1956, 1960, 1962, 1968, 1980, 1986, 1991 and 1996, while the Canadian standards were updated in 1974, 1984, 1989 and 1994.

The web crippling strength of cold-formed steel sections is a function of many variables. Design equations in the specifications have always been empirical formulas developed by curve fitting of experimental data. While AISI (1996) has different design expressions for different types of sections and loading cases, the Canadian Standard (S136-94) has one “Unified Design Expression” with different coefficients for different section types and loading. In both of the standards the web crippling calculations are based on unfastened specimens and are limited to the use of decks with certain geometric parameters.

Updated coefficients were developed for the unified web crippling design expression in the North American Specification for the Design of Cold Formed Steel Structural Members (2002). Also, different coefficients were derived for fastened and unfastened end conditions.

1.2 Web Crippling Strength

Web crippling of a cold-formed steel section depends on many factors. Section type, cross sectional parameters, bearing length and loading conditions are the major factors that affect web crippling strength.

1.2.1 Section Type

There are many cold-formed steel section types being used in building construction. Although web crippling occurs in the webs of the members, the interaction of the web element with the flanges plays an important role in web crippling strength. The rotation of the web is directly proportional to the degree of the restraint of the web provided by the flanges as illustrated in Fig. 1.2. Because web-flange interaction is one of the major influences in the web crippling strength of a section, different types of cross sections show different behavior in web crippling failure. I-sections, Hat sections, Z-sections, C-sections and multi-web sections, as illustrated in Fig. 1.3, are the most common cross section types being used in the cold-formed steel industry.

AISI (1996) classifies cold-formed steel sections into two categories for web crippling calculations: “Shapes Having Single Webs” and “I-Sections or Similar Sections”. In the Canadian (S136-94) and North American (North American 2002) Specifications, the unified web crippling expression has different coefficients for different cross sections. Additionally, all of the above specifications classify some cross sections into stiffened or unstiffened categories.

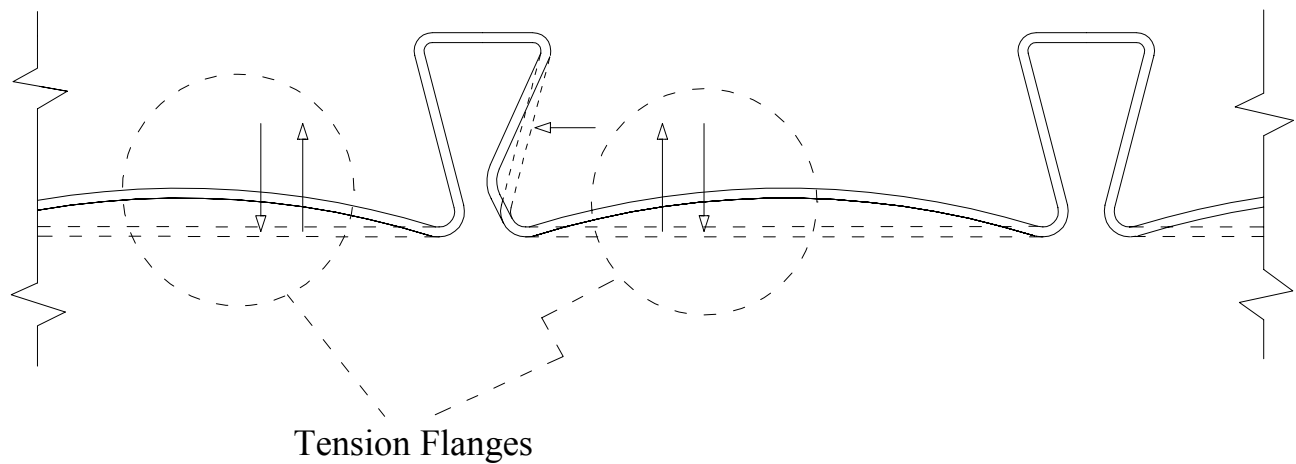
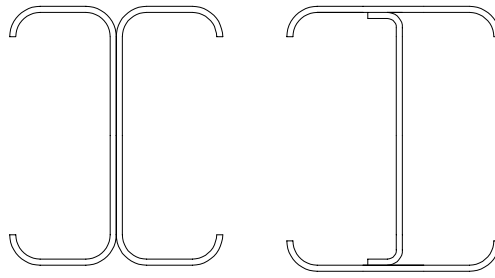
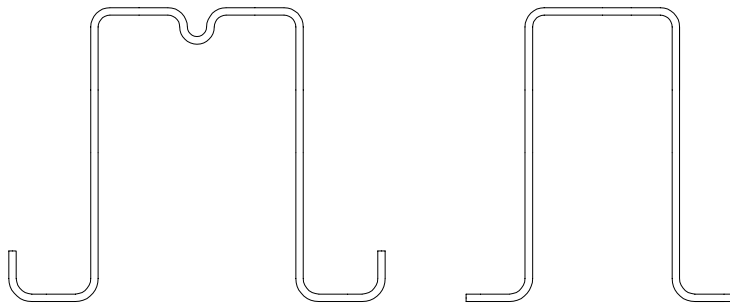


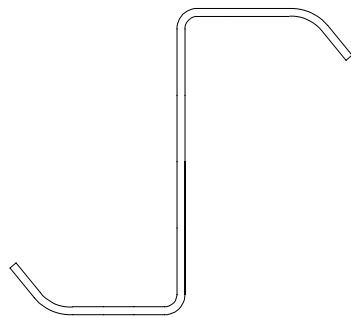
Figure 1.2 Tension Flanges Restrain the Movement of the Web



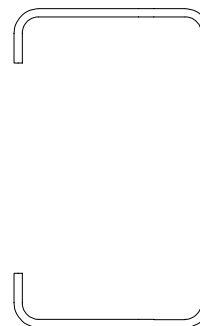
I-Sections



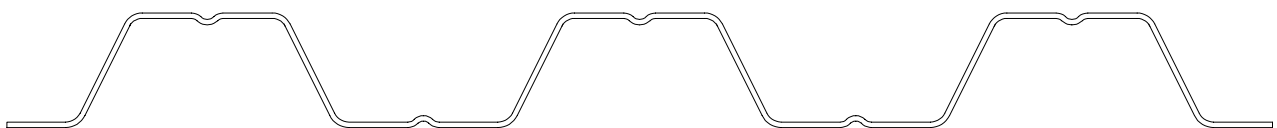
Hat Sections



Z-Section



C-Section



Multi-Web Deck Section

Figure 1.3 Common Cold Formed Steel Cross Sections

1.2.2 Cross Sectional Parameters and Bearing Length

There are six major parameters used in web crippling capacity calculations: thickness of the web (t), yield strength of the material (F_y), inside bend radius to thickness ratio (R/t), flat portion of the web to thickness ratio (h/t), bearing length to thickness ratio (N/t) and the inclination of the web element (θ). Both American (AISI, 1996) and Canadian (CSA, S136-94) web crippling equations are functions of the above parameters. North American Specification (North American 2002) which has been adopted from Canadian Specification (CSA, S136-94) has the same web crippling equation as the Canadian Specification.

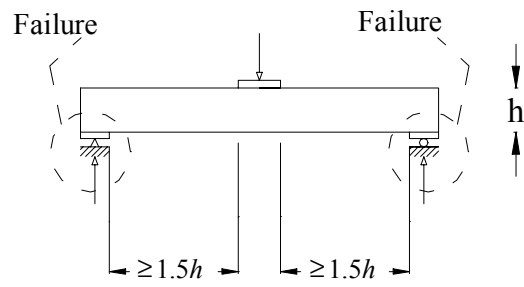
Fastening of the specimens to the supports has been accepted as a factor affecting the web crippling capacity (Beshara 2000); however, existing specifications do not include it as a parameter. The North American Specification for the Design of Cold Formed Steel Structural Members (North American 2002) does recognize the influence of fastening for some cross sections and loading cases. The unfastened coefficients are used for both fastened and unfastened cases for some members because there are not enough data available to generate separate coefficients.

1.2.3 Loading Conditions

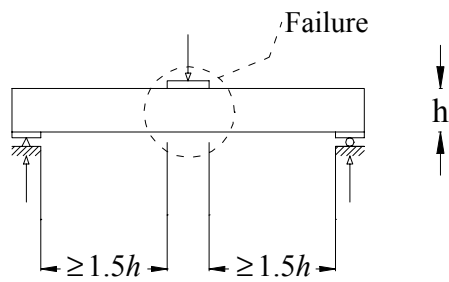
There are four different loading cases for web crippling. Both AISI (1996) and CSA (1994) define these cases according to the number of flanges under loading (One Flange Loading or Two Flange Loading) and location of the load (Interior Loading or End Loading):

- a) End One Flange Loading
- b) Interior One Flange Loading
- c) End Two Flange Loading
- d) Interior Two Flange Loading

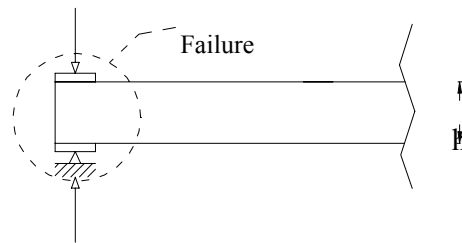
The four loading cases are illustrated in Fig. 1.4.



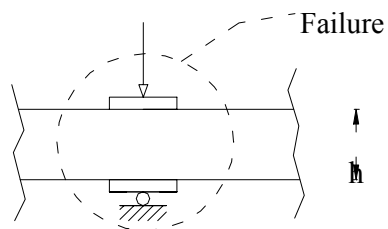
End One Flange Loading



Interior One Flange Loading



End Two Flange Loading



Interior Two Flange Loading

Figure 1.4 Web Crippling Load Classifications

1.3 Objective and Scope of Research

The North American Specification (North American 2002) has new web crippling coefficients for different load cases and different end conditions. However, in the End One Flange (EOF) loading case of multi-web deck sections the coefficients for the unfastened configuration were used as a conservative solution for the fastened case. This was because there was no directly applicable test data available in the literature. For that reason, seventy-eight tests were conducted in the Structures and Materials Research Laboratory at Virginia Polytechnic Institute and State University. The web crippling strength of multiple-web cold-formed steel deck sections subjected to End One Flange loading was investigated. The test results were compared with different strength prediction approaches. The study resulted in development of new coefficients for unfastened and fastened multi-web deck sections subjected to End One Flange (EOF) Loading.

This thesis is organized in the following manner. Chapter 1 is an introduction containing background information. Chapter 2 is a literature review of the material related to the research. Chapter 3 describes the experimental investigation including testing procedures and test results. Chapter 4 focuses on the analytical investigation. It presents a comparison of experimental and analytical results. A statistical analysis is performed in Chapter 5 and the new coefficients are derived and calibrated. Chapter 6 contains the summary, conclusions and recommendations for further investigations. Tensile coupon test results and sample calculations are presented in the appendices.

CHAPTER 2: LITERATURE REVIEW

2.1 Existing Research

Research on web crippling strength of cold-formed steel members was started in 1939 at Cornell University. Winter and Pian (1946) carried out web crippling tests on I-sections and developed the following web crippling equations for I-sections:

- i) For end one flange loading (EOF)

$$P_{ult} = F_y t^2 \left(10 + 1.25 \sqrt{\frac{N}{t}} \right) \quad (2.1)$$

- ii) For interior one flange loading (IOF)

$$P_{ult} = F_y t^2 \left(15 + 3.25 \sqrt{\frac{N}{t}} \right) \quad (2.2)$$

where:

P_{ult} = ultimate web crippling load per web

F_y = yield strength of steel

h = flat dimension of web measured in plane of web

N = bearing length of load

t = thickness of the web

The ranges of parameters in this study were:

$$30 < h/t < 175$$

$$7 < N/t < 77$$

$$30 < F_y < 39 \text{ ksi}$$

Fig. 2.1 shows the cross sections used by Winter and Pian (1946).

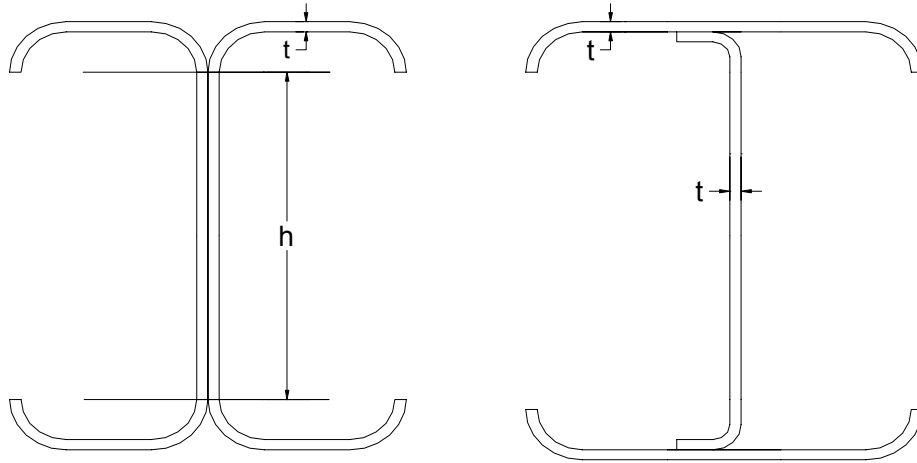


Figure 2.1 Cross Sections Used by Winter and Pian

During the 1950's many tests were conducted at Cornell University on cold-formed beams that have single unreinforced webs (Hat and U-sections). Fig. 2.2 shows the hat sections used. After these studies it was realized that the web crippling resistance of cold-formed steel members is a function of h/t , R/t , N/t and F_y . The following equations were derived for cold-formed steel sections with unreinforced webs (Cornell 1953).

- i) For end reactions and for concentrated loads on outer ends of cantilevers:

For $R/t \leq 1$

$$P_{ult} = \frac{F_y t^2}{10^3} \left((1.33 - 0.33k)(5450 + 235 \left(\frac{N}{t} \right) - 1.2 \left(\frac{N}{t} \right) H - 0.6H) \right) \quad (2.3)$$

For $1 < R/t \leq 4$

$$(P_{ult})_1 = \left(1.15 - 0.015 \left(\frac{R}{t} \right) \right) (P_{ult}) \quad (2.4)$$

- ii) For reactions at interior supports or for concentrated loads:

For $R/t \leq 1$

$$P_{ult} = \frac{F_y t^2}{10^3} (1.22 - 0.22k) \left(17000 + 125 \left(\frac{N}{t} \right) - 0.5 \left(\frac{N}{t} \right) H - 30H \right) \quad (2.5)$$

For $1 < R/t \leq 4$

$$(P_{ult})_1 = \left(1.06 - 0.06 \left(\frac{R}{t} \right) \right) (P_{ult}) \quad (2.6)$$

where:

P_{ult} = ultimate computed web crippling load per web

F_y = yield strength

t = thickness of the web

k = $F_y(\text{ksi}) / 33$; $F_y(\text{N/mm}^2) / 228$

N = bearing length of load

h = flat dimension of web measured in plane of web

H = web slenderness ratio, h / t

R = inside bend radius

The webs were perpendicular to the flanges in the tests mentioned above, so the web inclination was not considered in the above equations. Because there was not any other study conducted related to the web inclination before 1968, the above equations were used in the 1968 AISI Specification.

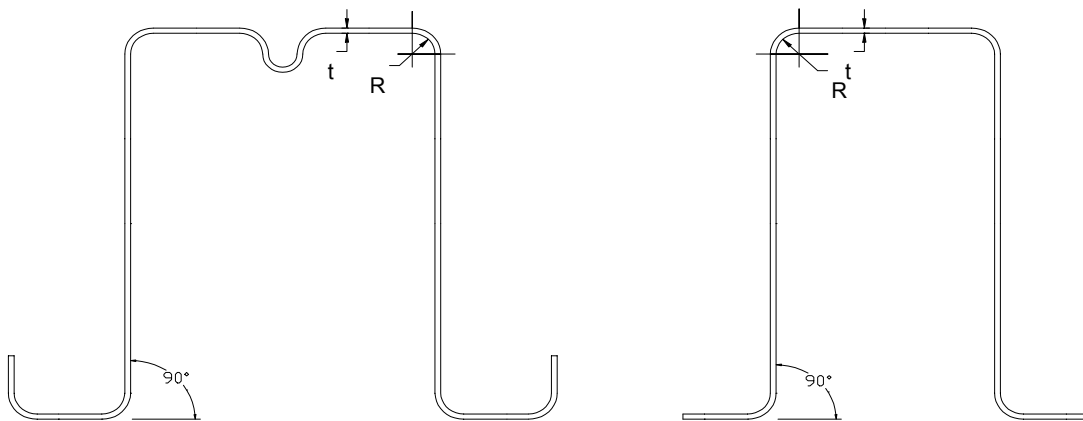


Figure 2.2 Hat Sections Used in Cornell Study

The development and use of different geometrical configurations of cold formed steel sections made the web crippling strength calculations more difficult and brought about the need for additional research and investigation. Therefore, many web-crippling studies were conducted in the United States and other countries.

Baehre (1975) tested unreinforced multi web hat sections under interior one flange loading at the Royal Institute of Technology, Sweden. He found the web inclination, θ , to be an important factor that influences web-crippling strength. He developed the following relationship for the ultimate load at intermediate supports:

$$P_{ult} = 1.8F_y t^2 (2.8 - 0.8k) \left(1 - 0.1 \sqrt{\frac{R}{t}} \right) \left(1 + 0.01 \left(\frac{N}{t} \right) \right) \left(2.4 + \frac{\theta}{90} \right)^2 \quad (2.7)$$

where:

P_{ult} = computed ultimate web crippling load per web

F_y = yield strength

t = thickness of the web

h = clear distance between flanges measured in the plane of the web

H = web slenderness ratio, h/t

k = $F_y(\text{ksi}) / 49.3$

N = bearing length of load

R = inside bend radius

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

The ranges of parameters in this study were:

$$h/t < 170$$

$$R/t < 10$$

$$50^\circ < \theta < 90^\circ$$

Baehre (1975) also stated that for end supports, one half of the ultimate load applicable to the intermediate support should be a value on the safe side.

Starting in 1973, an experimental study was carried out by Hetrakul and Yu at the University of Missouri at Rolla (UMR). Based on the Cornell test data and the tests conducted at UMR, modified web crippling design equations were proposed by Hetrakul and Yu (1978):

- i) For interior one flange loading, IOF (for stiffened and unstiffened flanges)

$$P_{ult} = \frac{F_y t^2}{10^3} C_1 C_2 (16317 - 22.52H) \left(1 + 0.0069 \left(\frac{N}{t} \right) \right) \quad (2.8)$$

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

- ii) For end one flange loading, EOF

For stiffened flanges:

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 (10018 - 18.24H) \left(1 + 0.0102 \left(\frac{N}{t} \right) \right) \quad (2.9)$$

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

For unstiffened flanges:

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 (6570 - 8.51H) \left(1 + 0.0099 \left(\frac{N}{t} \right) \right) \quad (2.10)$$

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

- iii) For interior two flange loading, ITF (for stiffened and unstiffened flanges)

$$P_{ult} = \frac{F_y t^2}{10^3} C_1 C_2 (23356 - 68.64H) \left(1 + 0.0013 \left(\frac{N}{t} \right) \right) \quad (2.11)$$

- iv) For exterior two flange loading, ETF (for stiffened and unstiffened flanges)

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 (7441 - 17.28H) \left(1 + 0.0099 \left(\frac{N}{t} \right) \right) \quad (2.12)$$

where:

P_{ult} = computed ultimate computed web crippling load per web

F_y = yield strength of steel

t = thickness of the web

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

C_1 = $(1.22 - 0.22k)$

C_2 = $(1.06 - 0.06R/t)$

$$C_3 = (1.33-0.33k)$$

$$C_4 = (1.15-0.15k)$$

h = clear distance between flanges measured in the plane of the web

H = web slenderness ratio, h/t

N = bearing length of load

R = inside bend radius

The ranges of parameters in this study were:

$$\theta = 90^\circ$$

$$33 < F_y < 54 \text{ ksi}$$

$$45 < h/t < 258$$

$$1 < R/t < 3$$

$$11 < N/t < 140$$

Because the modified web crippling equations based on Cornell and UMR test data were limited by vertical webs, and by small R/t and N/t ratios, the suitability of these equations was not certain for every cross section. For this reason, another experimental study was conducted at UMR from 1979 to 1981. Multi-web deck sections were tested under different loading conditions and the validity of AISI (1980) web crippling equations was investigated. At the end of the study, AISI (1980) equations were found to be conservative for multi-web deck sections.

Wing (1981) carried out an extensive study on web crippling and the combination of web crippling and bending of multi-web cold-formed sections at the University of Waterloo. All of the members were fastened to the support locations. He derived new web crippling equations for all loading cases except end one flange loading. These are:

i) Interior one flange loading, IOF

$$P_w = 16.6t^2F_y(\sin\theta)(1 - 0.000985H)\left(1 + 0.00526\left(\frac{N}{t}\right)\right)\left(1 - 0.074\sqrt{\frac{R}{t}}\right)(1 - 0.107k) \quad (2.13)$$

ii) Interior two flange loading, ITF

$$P_w = 18t^2F_y(\sin\theta)(1 - 0.00139H)\left(1 + 0.00948\left(\frac{N}{t}\right)\right)\left(1 - 0.0306\sqrt{\frac{R}{t}}\right)(1 - 0.22k) \quad (2.14)$$

iii) End two flange loading, ETF

$$P_w = 10.9t^2 F_y (\sin \theta) (1 - 0.00206H) \left(1 + 0.00887 \left(\frac{N}{t} \right) \right) \left(1 - 0.111 \sqrt{\frac{R}{t}} \right) (1 - 0.0777k) \quad (2.15)$$

where:

P_w = computed ultimate computed web crippling load per web

F_y = yield strength of steel

t = thickness of the web

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

h = clear distance between flanges measured in the plane of the web

H = web slenderness ratio, h / t

N = bearing length of load

R = inside bend radius

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

The ranges of parameters in Wing's study were:

$$h / t < 200$$

$$R / t < 10$$

Studnicka (1990) conducted an extensive experimental study on web crippling resistance of multi-web cold-formed steel sections at Czech Technical University, Prague, Czechoslovakia. For interior loading conditions, satisfactory conformity was obtained with the Canadian 1984 expressions. Tests with end support conditions did not compare favorably with the Canadian Standard (CSA 1984) or American (AISI 1986) expressions (Studnicka 1990).

The effect of the flange restraint was investigated in an experimental study by Bhakta, La Boubé and Yu (1992). Z-sections, multi-web roof and floor deck sections, channel sections and I-sections were tested under end one flange and interior one flange loading. When the flanges were fastened to the support locations, there was an average increase of 37% in the web crippling resistance of long span roof decks while the increase in web crippling resistance of floor decks was around 20% under one flange loading. On the other hand, there was almost no increase in web crippling strength of channel and I-sections when they are subjected to either end one flange or interior one flange loading. The web crippling strength of Z-sections fastened to supports was

increased 30% under end one flange loading and 3% under interior one flange loading (Bhakta, La Boube and Yu 1992).

An extensive statistically based study on web crippling of cold-formed steel members was completed at the University of Waterloo by Parabakaran (1993). The available experimental data in the literature were used to derive one expression to calculate web-crippling capacity of cold-formed steel sections:

$$P_n = Ct^2F_y(\sin\theta)\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_H\sqrt{\frac{h}{t}}\right) \quad (2.16)$$

where:

P_n = nominal computed ultimate computed web crippling load or reaction per web

F_y = yield strength of steel

t = thickness of the web

C = coefficient from tables

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

C_R = inside bend radius coefficient

C_N = bearing length coefficient

C_H = web slenderness coefficient

R = inside bend radius

N = bearing length of load

h = clear distance between flanges measured in the plane of the web

The ranges of parameters in Parabakaran's study were:

For I-sections and sections having single webs:

$$h/t \leq 200$$

$$N/t \leq 200$$

$$R/t \leq 4$$

$$N/h \leq 1$$

For multi-web sections:

$$h/t \leq 200$$

$$N/t \leq 200$$

$$R / t \leq 10$$

$$N / h \leq 2$$

Equation (2.16) is the unified equation for web crippling strength with different coefficients for single web, I- and multi-web sections. It is still being used in the Canadian Standard (CSA 1994).

Cain, La Boubé and Yu (1995) conducted an experimental study on Z-sections under end one flange loading and I-sections under interior one flange loading. Based on these tests it was found that AISI (1986) expressions were conservative for the web crippling capacity of unfastened Z-sections under end one flange loading, and also for fastened and unfastened I-sections under interior one flange loading.

In an experimental study at the University of Waterloo, Gerges (1997) developed new parameter coefficients for Parabakaran's expression for C-sections subjected to end one flange loading:

$$C = 4.70$$

$$C_R = 0.0521 \text{ (inside bend radius coefficient)}$$

$$C_N = 0.165 \text{ (bearing length coefficient)}$$

$$C_H = 0.0221 \text{ (web slenderness coefficient)}$$

The specimens were fastened to the supports in this study.

Young and Hancock (1998) investigated web-crippling behavior of cold formed steel unlipped channel sections at the University of Sydney. The specimens were tested under four different load conditions of web crippling: End One Flange (EOF), Interior One Flange (IOF), End Two Flange (ETF) and Interior Two Flange (ITF). Based on the test results, the AISI-1996 web-crippling capacity equations were found to be unconservative for the unlipped channel cross sections and a new equation was proposed using a simple plastic mechanism approach.

2.2 AISI (1996) Specification

AISI (1996) specification provisions are primarily based on the research conducted at Cornell University and UMR that has been reviewed. The equations are based on unfastened test specimens and are limited to the use of decks with certain

geometric parameters. Two classifications are used for web crippling in the AISI Specification (1996). These are “shapes having single webs” and “I- sections or similar sections”. For four different loading conditions the nominal web crippling strength, P_n can be determined according to the following table.

Table 2.1 Equation Numbers for Nominal Strength of webs, P_n , kips (N) at a Concentrated Load or Reaction.

		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.(2.17)	Eq.(2.18)	Eq.(2.19)
	Interior Reaction	Eq.(2.20)	Eq.(2.20)	Eq.(2.21)
Opposing Loads Spaced $< 1.5h$	End Reaction	Eq.(2.22)	Eq.(2.22)	Eq.(2.23)
	Interior Reaction	Eq.(2.24)	Eq.(2.24)	Eq.(2.25)

$$t^2 k C_3 C_4 C_9 C_\theta [331 - 0.61(h/t)][1 + 0.01(N/t)] \quad (2.17)$$

$$t^2 k C_3 C_4 C_9 C_\theta [217 - 0.28(h/t)][1 + 0.01(N/t)] \quad (2.18)$$

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

When $F_y \geq 66.5$ ksi (459 Mpa), the value of kC_3 shall be taken as 1.34

$$t^2 F_y C_6 (10.0 + 1.25\sqrt{N/t}) \quad (2.19)$$

$$t^2 k C_1 C_2 C_9 C_\theta [538 - 0.74(h/t)][1 + 0.007(N/t)] \quad (2.20)$$

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

$$t^2 F_y C_5 (0.88 + 0.12m)(15.0 + 3.25\sqrt{N/t}) \quad (2.21)$$

$$t^2 k C_3 C_4 C_9 C_\theta [244 - 0.57(h/t)][1 + 0.01(N/t)] \quad (2.22)$$

When $F_y \geq 66.5$ ksi (459 Mpa), the value of kC_3 shall be taken as 1.34

$$t^2 F_y C_8 (0.64 + 0.31m)(10.0 + 1.25\sqrt{N/t}) \quad (2.23)$$

$$t^2 k C_1 C_2 C_9 C_\theta [771 - 2.26(h/t)][1 + 0.0013(N/t)] \quad (2.24)$$

$$t^2 F_y C_7 (0.82 + 0.15m)(15.0 + 3.25\sqrt{N/t}) \quad (2.25)$$

where:

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

$$C_1 = 1.22 - 0.22k \quad (2.26)$$

$$C_2 = 1.06 - 0.06R/t \leq 1.0 \quad (2.27)$$

$$C_3 = 1.33 - 0.33k \quad (2.28)$$

$$C_4 = 1.15 - 0.15R/t \leq 1.0 \text{ but no less than } 0.50 \quad (2.29)$$

$$C_5 = 1.49 - 0.53k \geq 0.6 \quad (2.30)$$

$$C_6 = 1 + \left(\frac{h/t}{750} \right) \text{ when } h/t \leq 150 \quad (2.31)$$

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

$$C_7 = 1/k \text{ when } h/t \leq 66.5 \quad (2.32)$$

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

$$C_8 = \left[0.98 - \frac{h/t}{865} \right] \frac{1}{k} \quad (2.33)$$

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

$= 6.9$ for metric units, N and mm

$$C_\theta = 0.7 + 0.3(\theta/90)^2 \quad (2.34)$$

F_y = Design yield stress of the web

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

$$k = 894 F_y / E \quad (2.35)$$

$$m = t/0.075, \text{ when } t \text{ is in inches} \quad (2.36)$$

$$m = t/1.91, \text{ when } t \text{ is in mm} \quad (2.37)$$

t = Web thickness, in. (mm)

N = Actual length of bearing, in. (mm). For the case of two equal and opposite concentrated loads distributed over unequal bearing lengths, the smaller value of N shall be taken.

R = Inside bend radius

θ = Angle between the plane of the web and the plane of the bearing surface $\geq 45^\circ$, but not more than 90°

The equations in Table 2.1 can be applied to beams when $R/t \leq 6$ and to decks when $R/t \leq 7$, $N/t \leq 210$ and $N/h \leq 3.5$. P_n represents the nominal strength for concentrated load or reaction for one solid web connecting top and bottom flanges. For two or more webs, P_n shall be computed for each individual web and the results added to obtain the nominal load or reaction for the multiple web (AISI 1996).

In AISI (1996) it is noted that “when $F_y \geq 66.5$ ksi (459 Mpa), the value of kC_3 shall be taken as 1.34” in the equations (2.17), (2.18) and (2.22). Due to the consideration of higher yield strengths of the specimens, this section was revised in Supplement No.1 (July 30, 1999) and the factor C_3 was replaced by C_1 in the equations (2.17), (2.18) and (2.22). Because the web crippling strength is directly proportional to the yield strength of the material, the actual behavior is reflected better by the factor kC_1 than the factor kC_3 . These relationships are illustrated in Fig. 2.3

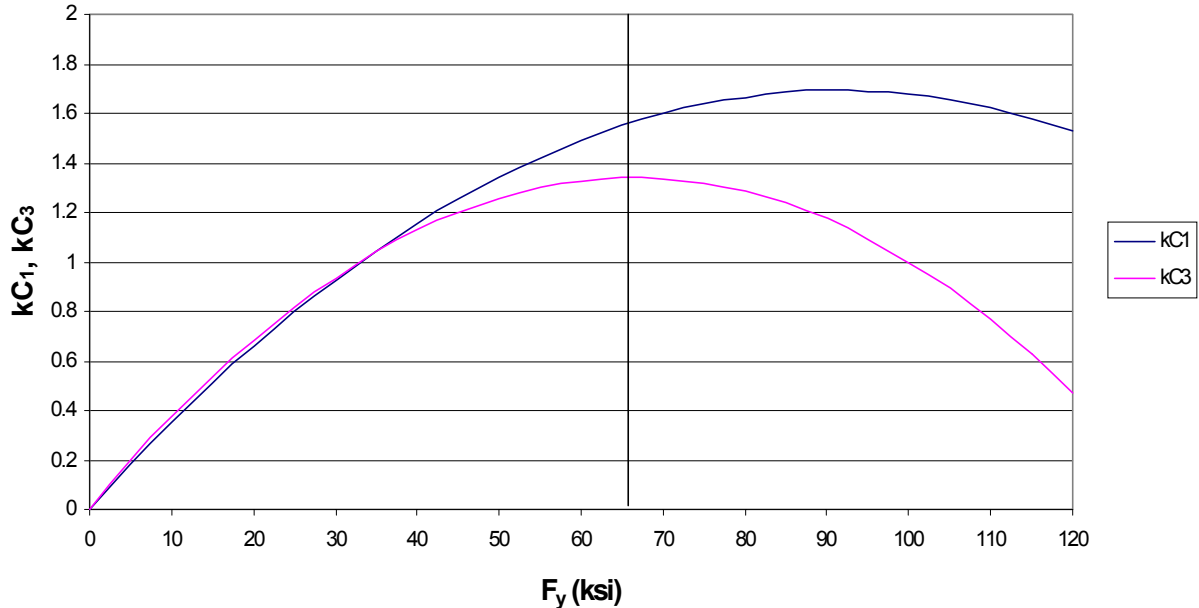


Figure 2.3 Variation of kC_1 and kC_3 with respect to F_y

2.3 Canadian Specification (S136-94)

The Canadian Specification (S136-94) is based on the unified web crippling expression derived by Parabakaran (1993) at the University of Waterloo. The unified expression has different coefficients that depend on the cross-section and load case. Beshara (2000) performed an extensive statistical analysis of all available web crippling experimental data in the literature and improved these coefficients. The support conditions are taken into consideration and different coefficients were derived for fastened and unfastened specimens. These new coefficients were approved by AISI committee in the North American Specification for the Design of Cold-Formed Steel Structural Members (North American 2002). The equation and coefficients are given by:

$$P_n = Ct^2 F_y \sin \theta \left(1 - C_R \sqrt{\frac{R}{t}} \right) \left(1 + C_N \sqrt{\frac{N}{t}} \right) \left(1 - C_H \sqrt{\frac{h}{t}} \right) \quad (2.38)$$

where:

P_n = nominal web crippling strength

C = coefficient from Table 2.2, 2.3, 2.4, 2.5 or 2.6

t = thickness of the web

F_y = yield strength of steel

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

C_R = inside bend radius coefficient from Table 2.2, 2.3, 2.4, 2.5 or 2.6

C_N = bearing length coefficient from Table 2.2, 2.3, 2.4, 2.5 or 2.6

C_H = web slenderness coefficient from Table 2.2, 2.3, 2.4, 2.5 or 2.6

R = inside bend radius

N = bearing length [$\frac{3}{4}$ in. (19mm) minimum]

h = flat dimension of web measured in plane of web

Table 2.2 Built-up Sections when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 1.0$ and $\theta=90^\circ$

Support and Flange 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	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	$R/t \leq 3$

Table 2.3 Single Web Channel and C- Sections when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$ and $\theta = 90^\circ$

Support and Flange 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	

Table 2.4 Single Web Z- Sections when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 2.0$ and $\theta = 90^\circ$

Support and Flange 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	

Table 2.5 Single Hat Sections when $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 2$ and $\theta = 90^\circ$

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	4	0.25	0.68	0.04	2.00	0.75	$R/t \leq 4$
		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	$R/t \leq 4$

Table 2.6 Multiple Web Deck Sections when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 3$ and $45^\circ < \theta \leq 90^\circ$

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	

Although the North American Specification for the Design of Cold-Formed Steel Structural Members (North American 2002) has new web crippling coefficients for different load cases and different end conditions, in the End One Flange loading case the coefficients for the “unfastened configuration” were used as a conservative solution for the “fastened case” for multi-web deck sections. This was because there were no directly applicable test data available in the literature.

Because of the lack of data for the EOF fastened configuration, seventy-eight tests were conducted in the Structures and Materials Research Laboratory at Virginia Polytechnic Institute and State University. From these tests, the web-crippling strength of multiple-web cold-formed steel deck sections subjected to EOF loading was determined for both fastened and unfastened end conditions. The test results were then compared to results from several strength prediction approaches. Because of the scatter in the results, new coefficients for unfastened and fastened multi-web deck sections subjected to EOF loading were developed.

CHAPTER 3: EXPERIMENTAL STUDY

3.1 General

Before Beshara (2000) improved the coefficients of the unified web crippling equation and derived new coefficients for different support conditions (fastened or unfastened), the restraining effect of the fasteners was not considered in the S136 (1994) or AISI (1996) specifications. The new coefficients were approved by the AISI committee in the North American Specification for the Design of Cold Formed Steel Structural Members (North American 2002). However, for multi-web deck sections subjected to end one flange loading, coefficients for the unfastened configuration were used as a conservative solution for the fastened case. This was because there were no directly applicable test data available in the literature.

For that reason, seventy-eight tests were conducted in the Structures and Materials Research Laboratory at Virginia Polytechnic Institute and State University. The web crippling strength of multiple-web cold-formed steel deck sections subjected to end one flange loading was investigated. In addition, the behavior of cross sections that did not fall into the range of AISI (1996) or CSA (1994) parameters were investigated.

3.2 Description of Test Specimens

Test specimens lying inside and outside of certain geometric parameter ranges of the specifications were tested under end one flange loading. The deck specimens were provided by Consolidated Systems, Inc. (CSI) and Vulcraft.

A two-phase experimental study was followed. Five different types of decks, including CSI designations B, HD, EHD, Versa Deck and S deck types, as illustrated in Fig. 3.1, were tested in the first phase. With unreinforced webs and unstiffened flanges, each type of CSI deck varied in thickness (t), yield strength (F_y), inside bend radius to

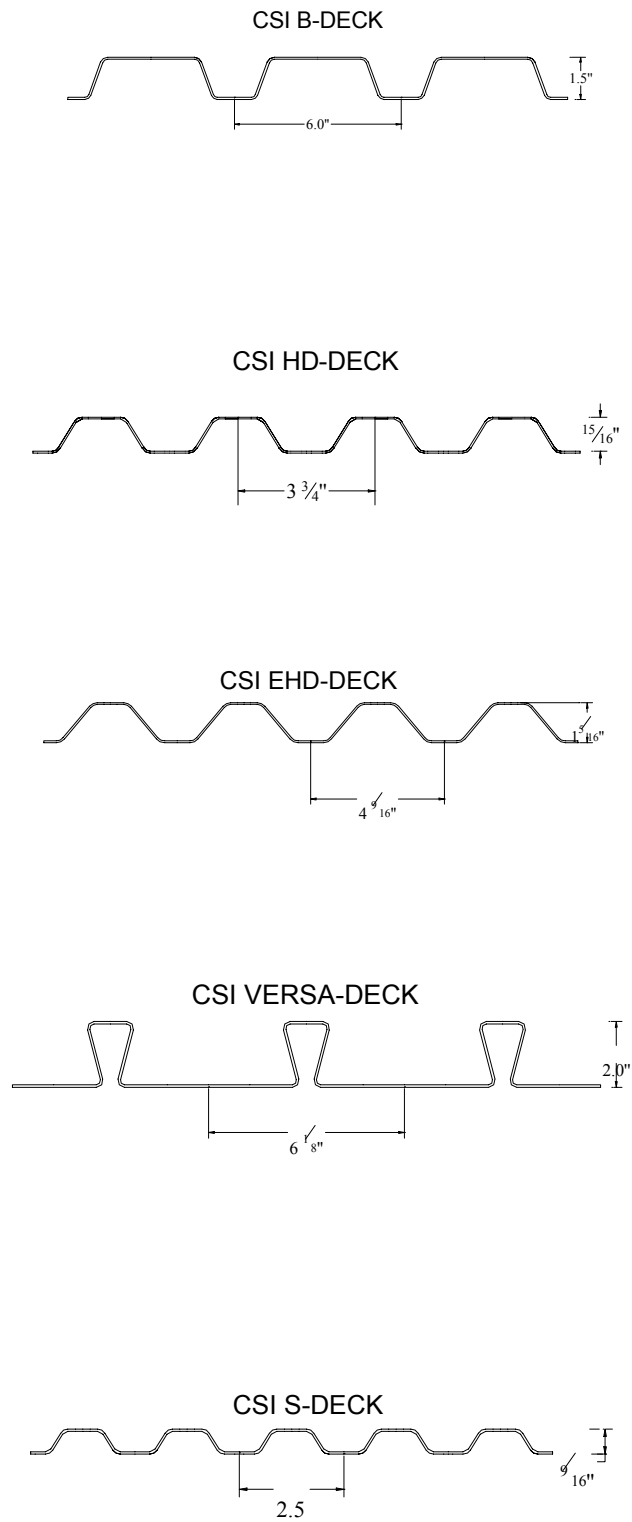


Figure 3.1 Deck Cross Sections Used in the Study

thickness ratio (R/t) and web slenderness ratio (h/t). Tests were conducted with both unrestrained and restrained end conditions.

In the second phase of the experimental study, Vulcraft 2VLI and 3VLI decks as illustrated in Fig. 3.2 were tested. Tests for four different gauges (16, 18, 20 and 22) of 2VLI and 3VLI decks were conducted with both unrestrained and restrained end conditions. Different gage types of VLI decks varied in web slenderness ratio (h/t) while the radius to thickness (R/t) ratios were the same. Unlike CSI decks, the webs of Vulcraft decks were reinforced with embossments. Also, both tension and compression flanges were stiffened, as illustrated in Fig. 3.3. Details of the deck profiles are shown in Fig. 3.4 and Table 3.1.

Each specimen is given a designation based on the deck type, gage number and support condition. The test designation is as follows:

“s-m-g-i”

“s” represents the support condition at the supports: *Restrained by fastening (R) or Unrestrained (U)*.

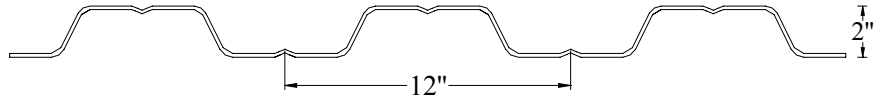
“m” indicates the member type: *B, HD, EHD, Versa Deck (V), S, 2VLI or 3VLI*.

“g” designates the gage number of the steel: *16, 18, 20, 22, 26 or 28*.

“i” shows the order of the test (each test is repeated 3 times).

Tensile coupon tests were performed according to ASTM E8-00b standards. Tensile yield properties were determined in accordance with ASTM 370 standards. Coupons were tested using an Instron-4468 testing machine with 10 kips (50kN) load capacity. Appendix A shows the first yield portion of the stress strain curves of the coupons. The tensile coupon test results are summarized in Table 3.2.

VULCRAFT 2VLI DECK



VULCRAFT 3VLI DECK

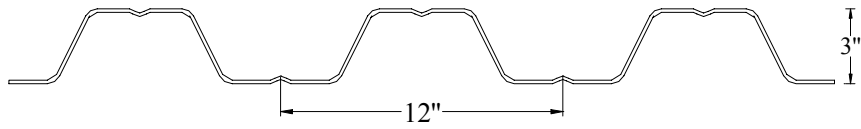


Figure 3.2 Deck Cross Sections Used in the Study

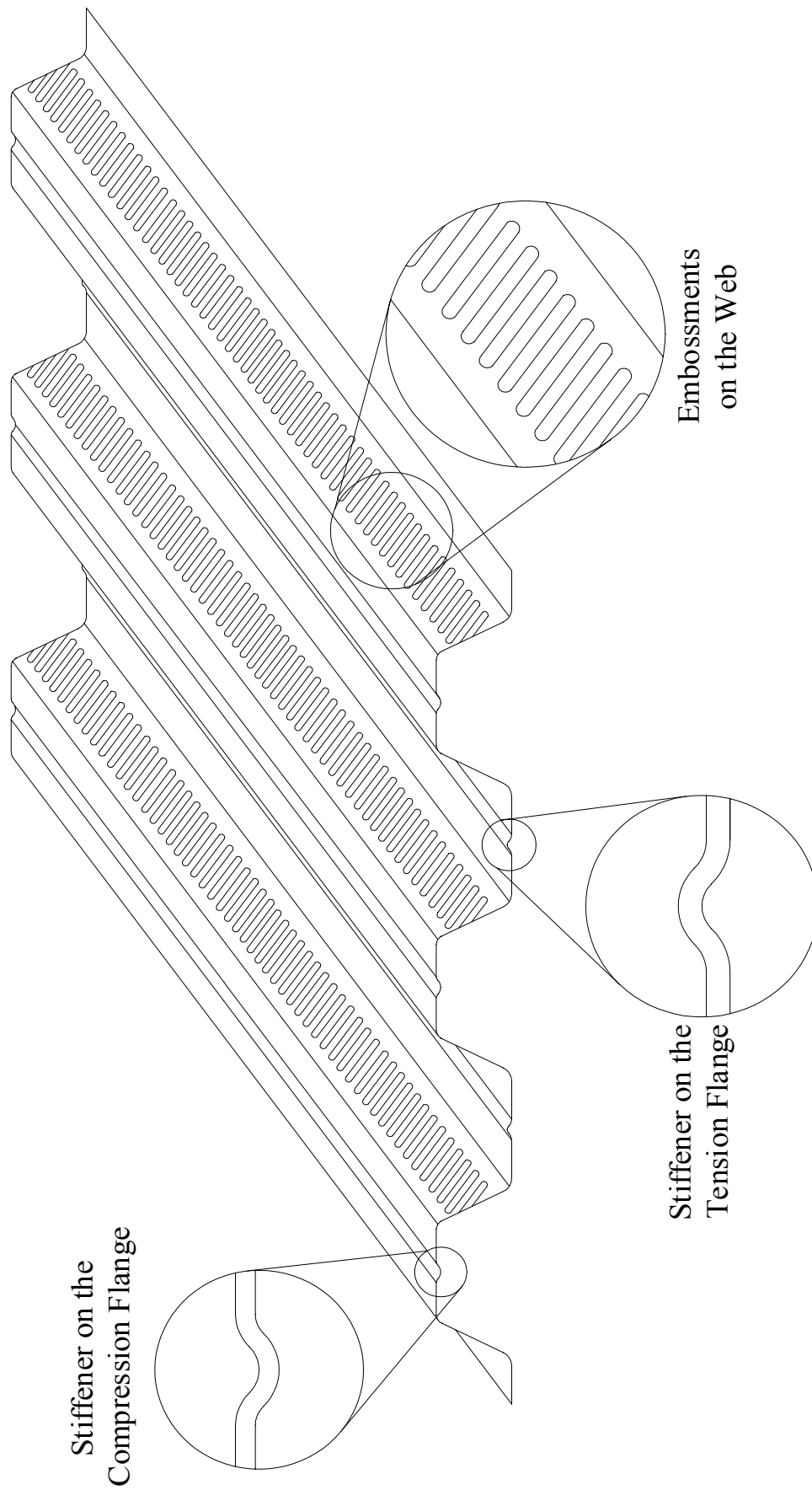


Figure 3.3 Vulcraft Composite Deck

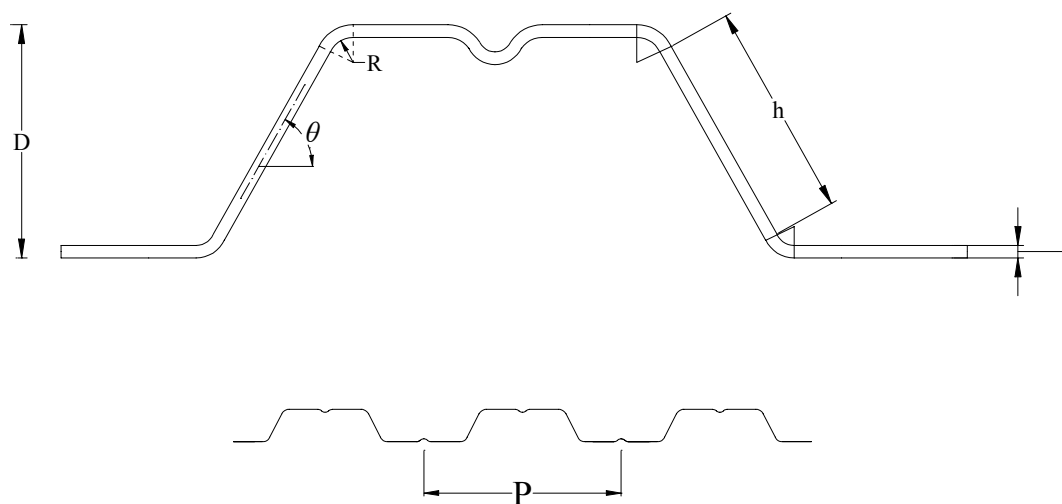


Figure 3.4 Details of the Deck Profiles

Table 3.1 Deck Profile Properties

TYPE OF DECK	Gage No	F _{y-catalog} (ksi)	Thickness at Web		Inside Bent Radius, R (in)	Web inclination,θ (deg)	Total depth of the deck, D (in)	Pitch length, P (in)
			t _{-catalog} (in)	t _{-measured} (in)				
B DECK	22	33	0.0295	0.0295	13/64	70	1 1/2	6
HD DECK	26	80	0.0179	0.0182	17/64	58	15/16	3 3/4
EHD DECK	26	80	0.0179	0.0183	5/16	50	1 5/16	4 9/16
VERSA- DECK	22	40	0.0295	0.0300	13/64	75.5	2	6 1/8
S DECK	28	80	0.0149	0.0153	11/64	58	9/16	2 1/2
2VLI	16	50	0.0598	0.0598	3/16	63	2	12
	18	50	0.0474	0.0474		63		
	20	50	0.0358	0.0358		63		
	22	50	0.0295	0.0295		63		
3VLI	16	36	0.0598	0.0598		67	3	
	18	50	0.0474	0.0474		67		
	20	50	0.0358	0.0358		67		
	22	50	0.0295	0.0295		67		

Table 3.2 Tensile Coupon Test Results

TYPE OF DECK	Gage No	F _{y-catalog} (ksi)	F _{y-measured} (ksi)
B DECK	22	33	45.8
HD DECK	26	80	95.4
EHD DECK	26	80	103.9
VERSA- DECK	22	40	48.0
S DECK	28	80	105.2
2VLI	16	50	46.5
	18	50	49.5
	20	50	52.0
	22	50	54.0
3VLI	16	36	35.0
	18	50	48.0
	20	50	53.5
	22	50	52.5

3.3 Test Setup

Each deck specimen was prepared in a similar manner and simulated a simple beam in the entire experimental study. Deck specimens were cut such that they had three ribs and six webs parallel to the beam line. The load applied by the ram was simulated as a point load at the midspan location. The test setup used for the tests is shown in Figs. 3.5 and 3.6.



Figure 3.5 Test Setup- View 1

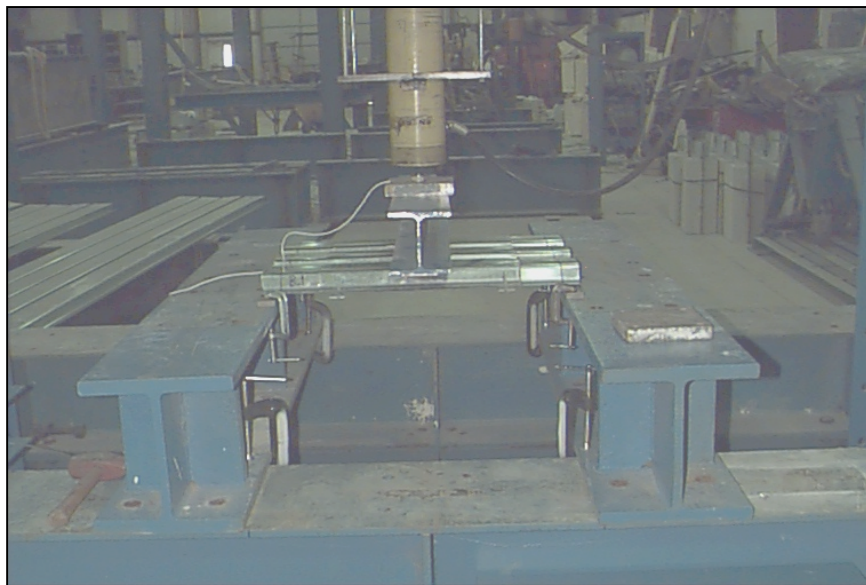


Figure 3.6 Test setup- View 2

The midspan region of the test specimens was strengthened by pieces of the same deck type to prevent a flexural failure. As a result, web-crippling failures occurred at the exterior flanges instead of a bending failure at midspan. The End One Flange (EOF) loading condition, as defined in the cold-formed steel specifications, is shown in Figs. 3.7 and 3.8. At the supports, a bearing length of 1.5in. was used.

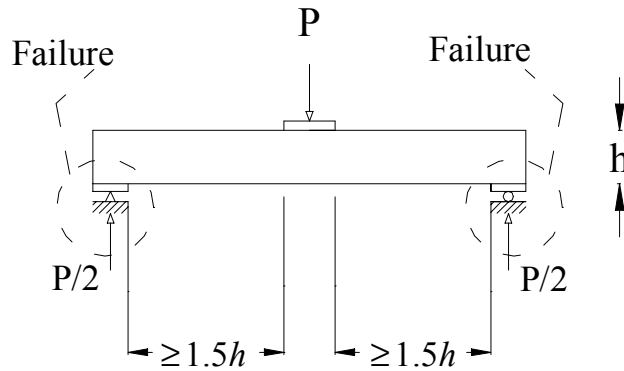


Figure 3.7 End One Flange Loading

The deck specimens were tied with straps to prevent spreading during loading. The deck pieces and tie straps were connected with $\frac{1}{4}$ -14x1" self-drilling screws. The screws not only connected the deck pieces together but also prevented the sliding of deck pieces with respect to each other.

An H-shape was used as a spreader beam to distribute the point load applied by the ram to the entire deck, as illustrated in Fig. 3.9. The load cell was placed between the ram and the spreader beam. Before the application of the load, the instrumentation was zeroed. A manually operated hydraulic jack was used to load the specimens and a strain indicator was used to monitor the load applied.

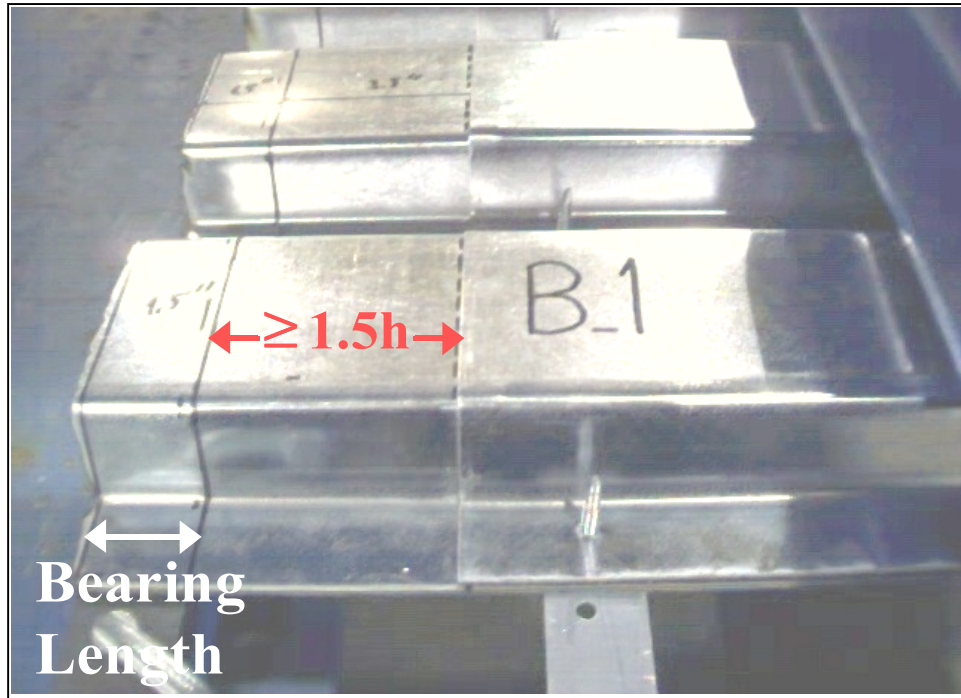


Figure 3.8 End One Flange Loading

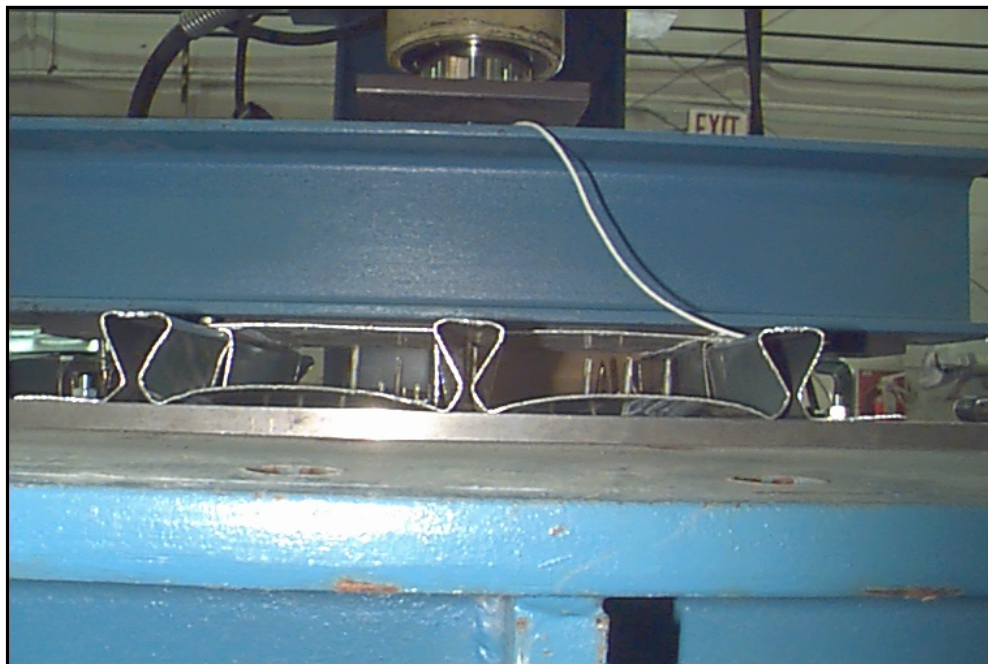


Figure 3.9 Spreader Beam Distributed the Applied Point Load to the Entire Deck

3.4 Test Procedure

A two-phase loading was applied. In the first phase, the deck specimens were loaded continuously until the allowable design load is reached. The allowable design load is calculated by dividing the smaller nominal web-crippling value of AISI (1996) and North American (2002) approaches by a factor of safety. In the second phase, the load was increased monotonically by adding 20% of the allowable design load to the previous load. The loading was continued after five minute waiting periods until the web crippling failure was observed at exterior end flanges. The maximum load was recorded as the web crippling strength of the specimen under end one flange loading. One half of the recorded load was the load transferred to each support. The load carried by each support is divided by the number of webs at each support (“six” for all of the specimens in this study) to find the web crippling strength per web. Figs. 3.10 to 3.16 show web-crippling failure for different types of decks.

The above procedure was the same for both unfastened and fastened tests. In the fastened tests the ends of the specimens were bolted to the supports through the tension flanges at every 12 in. (Fig. 3.17). The restraining effect of the fastening increased the web crippling capacity in all types of decks.

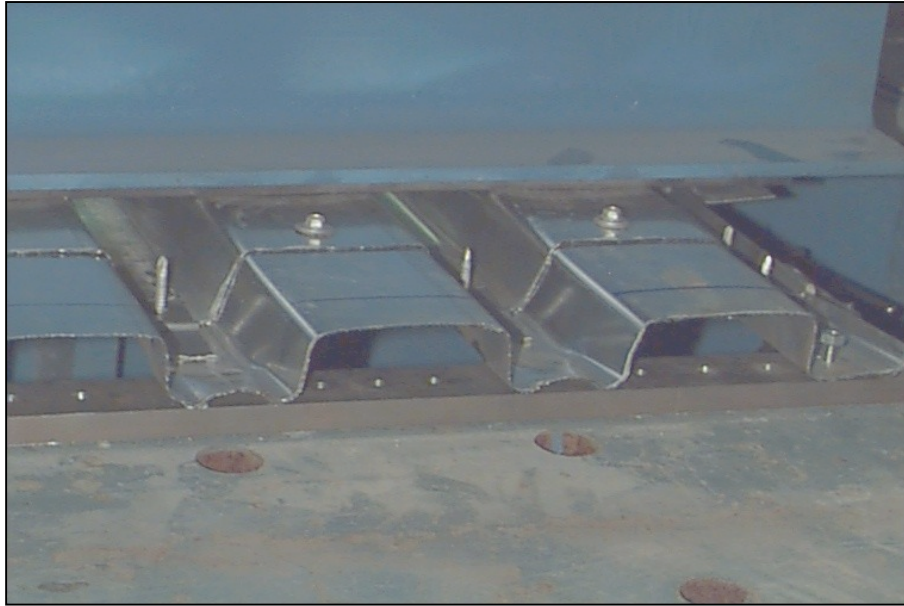


Figure 3.10 Crippled B-Deck



Figure 3.11 Crippled HD-Deck

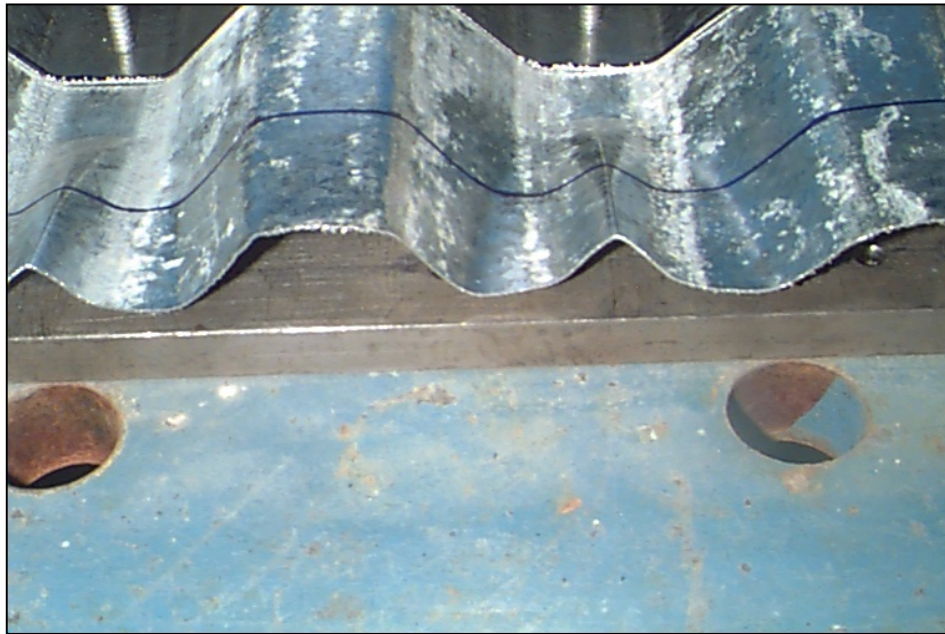


Figure 3.12 Crippled EHD-Deck

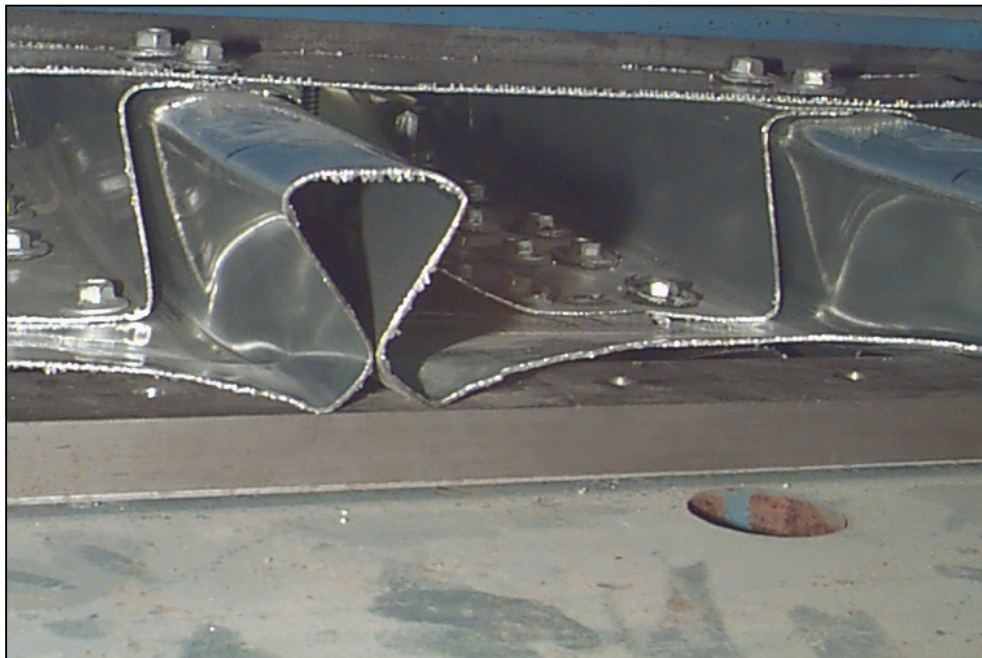


Figure 3.13 Crippled Versa Deck

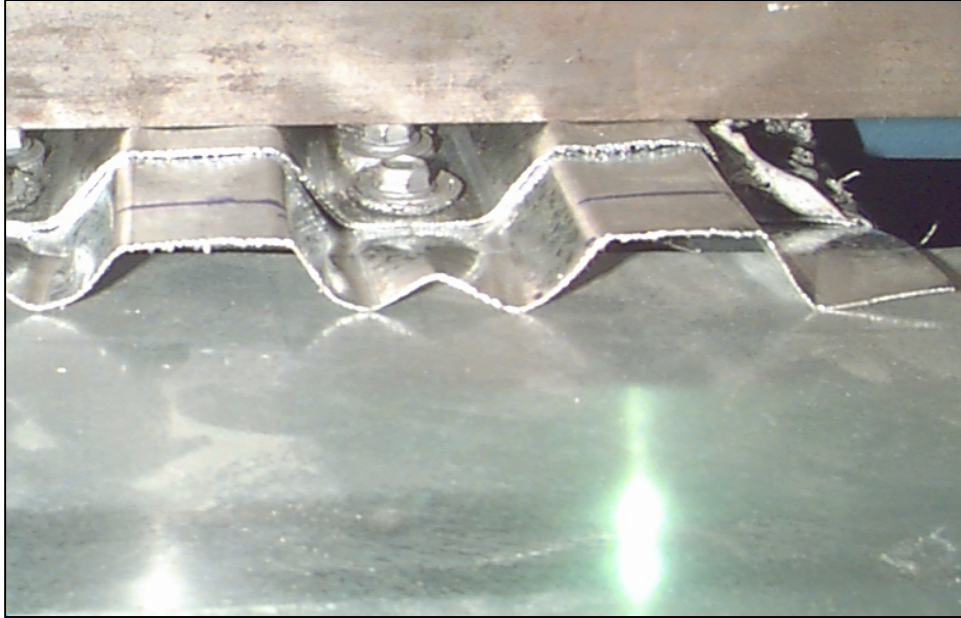


Figure 3.14 Crippled S-Deck



Figure 3.15 Crippled 3VLI-Deck

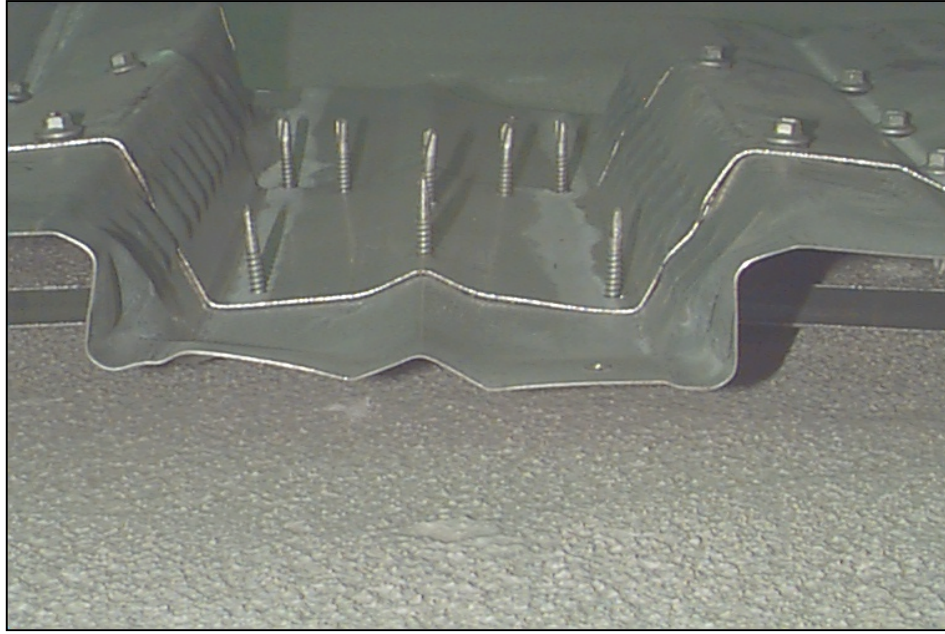


Figure 3.16 Crippled 2VLI-Deck

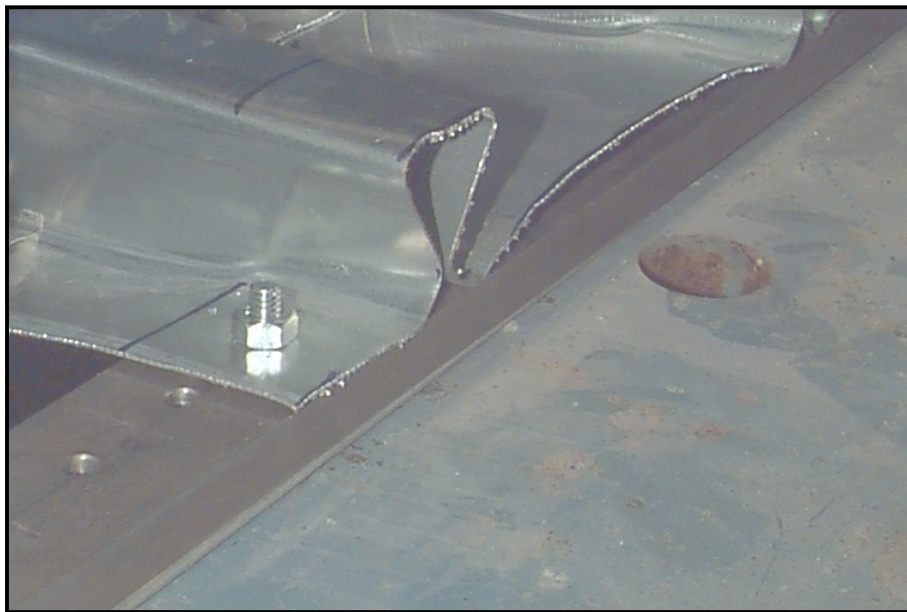


Figure 3.17 Fastened Tests: Ends of the Specimens Were Bolted to the Supports

3.5 Test Results

The additional short steel deck pieces attached to the central portion of the specimens made the web crippling failure occur at both ends. Otherwise a premature bending failure at the center of the beam was unavoidable.

The progression of crippling on the webs of the specimens initiated at an interior web followed by the outer webs as the load increased. The crippling of the webs caused deformation on the tension flanges of the specimens and moved the tension flanges upwards. (Yu (1981) also observed this type of behavior.) The redistribution of the forces enabled the deck specimens to carry load after the web crippling failure of the interior webs until all webs experienced the failure.

The maximum load carried by each specimen was recorded as the web crippling strength of the specimen. The amount of resistance provided by the outer webs to the inner webs was higher in fastened cases than unfastened ones. The results of the 78 tests are shown in Tables 3.3 to 3.5.

Observation of the tests revealed that there is an increase in web crippling strength of specimens when the ends of the specimens are fastened to the supports. It is observed that the specimens tended to fail in the central portions unless the central portions were not reinforced by additional deck pieces of the same type.

Table 3.3 Specimen Parameters and Test Results of CSI Steel Specimens

Specimen	t (in)	F _y (ksi)	h/t	R/t	N/t	θ	No. of webs	P _t per web (kips)
U-B-22-1	0.0295	45.8	42.7	6.88	50.85	70	6	0.344
U-B-22-2	0.0295	45.8	42.7	6.88	50.85	70	6	0.341
U-B-22-3	0.0295	45.8	42.7	6.88	50.85	70	6	0.346
R-B-22-1	0.0295	45.8	42.7	6.88	50.85	70	6	0.373
R-B-22-2	0.0295	45.8	42.7	6.88	50.85	70	6	0.380
R-B-22-3	0.0295	45.8	42.7	6.88	50.85	70	6	0.371
U-HD-26-1	0.0182	95.4	42.8	14.59	82.42	58	6	0.181
U-HD-26-2	0.0182	95.4	42.8	14.59	82.42	58	6	0.188
U-HD-26-3	0.0182	95.4	42.8	14.59	82.42	58	6	0.183
R-HD-26-1	0.0182	95.4	42.8	14.59	82.42	58	6	0.203
R-HD-26-2	0.0182	95.4	42.8	14.59	82.42	58	6	0.208
R-HD-26-3	0.0182	95.4	42.8	14.59	82.42	58	6	0.202
U-EHD-26-1	0.0183	103.9	75.9	17.08	81.97	50	6	0.161
U-EHD-26-2	0.0183	103.9	75.9	17.08	81.97	50	6	0.158
U-EHD-26-3	0.0183	103.9	75.9	17.08	81.97	50	6	0.168
R-EHD-26-1	0.0183	103.9	75.9	17.08	81.97	50	6	0.178
R-EHD-26-2	0.0183	103.9	75.9	17.08	81.97	50	6	0.183
R-EHD-26-3	0.0183	103.9	75.9	17.08	81.97	50	6	0.173
U-V-22-1	0.0300	48.0	56.6	6.77	50.00	75.5	6	0.386
U-V-22-2	0.0300	48.0	56.6	6.77	50.00	75.5	6	0.392
U-V-22-3	0.0300	48.0	56.6	6.77	50.00	75.5	6	0.393
R-V-22-1	0.0300	48.0	56.6	6.77	50.00	75.5	6	0.425
R-V-22-2	0.0300	48.0	56.6	6.77	50.00	75.5	6	0.422
R-V-22-3	0.0300	48.0	56.6	6.77	50.00	75.5	6	0.431
U-S-28-1	0.0153	105.2	29.2	11.23	98.04	58	6	0.203
U-S-28-2	0.0153	105.2	29.2	11.23	98.04	58	6	0.203
U-S-28-3	0.0153	105.2	29.2	11.23	98.04	58	6	0.200
R-S-28-1	0.0153	105.2	29.2	11.23	98.04	58	6	0.220
R-S-28-2	0.0153	105.2	29.2	11.23	98.04	58	6	0.223
R-S-28-3	0.0153	105.2	29.2	11.23	98.04	58	6	0.229

Table 3.4 Specimen Parameters and Test Results of Vulcraft 2VLI Specimens

Specimen	t (in)	F _y (ksi)	h/t	R/t	N/t	θ	No. of webs	P _t per web (kips)
U-2VLI-16-1	0.0598	46.5	30.3	3.14	25.08	63	6	1.374
U-2VLI-16-2	0.0598	46.5	30.3	3.14	25.08	63	6	1.322
U-2VLI-16-3	0.0598	46.5	30.3	3.14	25.08	63	6	1.390
R-2VLI-16-1	0.0598	46.5	30.3	3.14	25.08	63	6	1.590
R-2VLI-16-2	0.0598	46.5	30.3	3.14	25.08	63	6	1.622
R-2VLI-16-3	0.0598	46.5	30.3	3.14	25.08	63	6	1.580
U-2VLI-18-1	0.0474	49.5	38.7	3.96	31.65	63	6	1.000
U-2VLI-18-2	0.0474	49.5	38.7	3.96	31.65	63	6	0.956
U-2VLI-18-3	0.0474	49.5	38.7	3.96	31.65	63	6	1.011
R-2VLI-18-1	0.0474	49.5	38.7	3.96	31.65	63	6	1.179
R-2VLI-18-2	0.0474	49.5	38.7	3.96	31.65	63	6	1.233
R-2VLI-18-3	0.0474	49.5	38.7	3.96	31.65	63	6	1.244
U-2VLI-20-1	0.0358	52.0	51.8	5.24	41.90	63	6	0.629
U-2VLI-20-2	0.0358	52.0	51.8	5.24	41.90	63	6	0.611
U-2VLI-20-3	0.0358	52.0	51.8	5.24	41.90	63	6	0.584
R-2VLI-20-1	0.0358	52.0	51.8	5.24	41.90	63	6	0.778
R-2VLI-20-2	0.0358	52.0	51.8	5.24	41.90	63	6	0.745
R-2VLI-20-3	0.0358	52.0	51.8	5.24	41.90	63	6	0.753
U-2VLI-22-1	0.0295	54.0	63.3	6.36	50.85	63	6	0.417
U-2VLI-22-2	0.0295	54.0	63.3	6.36	50.85	63	6	0.456
U-2VLI-22-3	0.0295	54.0	63.3	6.36	50.85	63	6	0.444
R-2VLI-22-1	0.0295	54.0	63.3	6.36	50.85	63	6	0.585
R-2VLI-22-2	0.0295	54.0	63.3	6.36	50.85	63	6	0.574
R-2VLI-22-3	0.0295	54.0	63.3	6.36	50.85	63	6	0.565

Table 3.5 Specimen Parameters and Test Results of Vulcraft 3VLI Specimens

Specimen	t (in)	F _y (ksi)	h/t	R/t	N/t	θ	No. of webs	P _t per web (kips)
U-3VLI-16-1	0.0598	35.0	48.4	3.14	25.08	67	6	1.100
U-3VLI-16-2	0.0598	35.0	48.4	3.14	25.08	67	6	1.121
U-3VLI-16-3	0.0598	35.0	48.4	3.14	25.08	67	6	1.025
R-3VLI-16-1	0.0598	35.0	48.4	3.14	25.08	67	6	1.457
R-3VLI-16-2	0.0598	35.0	48.4	3.14	25.08	67	6	1.467
R-3VLI-16-3	0.0598	35.0	48.4	3.14	25.08	67	6	1.485
U-3VLI-18-1	0.0474	48.0	61.5	3.96	31.65	67	6	0.983
U-3VLI-18-2	0.0474	48.0	61.5	3.96	31.65	67	6	0.957
U-3VLI-18-3	0.0474	48.0	61.5	3.96	31.65	67	6	0.967
R-3VLI-18-1	0.0474	48.0	61.5	3.96	31.65	67	6	1.311
R-3VLI-18-2	0.0474	48.0	61.5	3.96	31.65	67	6	1.337
R-3VLI-18-3	0.0474	48.0	61.5	3.96	31.65	67	6	1.333
U-3VLI-20-1	0.0358	53.5	82.1	5.24	41.90	67	6	0.650
U-3VLI-20-2	0.0358	53.5	82.1	5.24	41.90	67	6	0.630
U-3VLI-20-3	0.0358	53.5	82.1	5.24	41.90	67	6	0.634
R-3VLI-20-1	0.0358	53.5	82.1	5.24	41.90	67	6	0.878
R-3VLI-20-2	0.0358	53.5	82.1	5.24	41.90	67	6	0.854
R-3VLI-20-3	0.0358	53.5	82.1	5.24	41.90	67	6	0.860
U-3VLI-22-1	0.0295	52.5	100.0	6.36	50.85	67	6	0.390
U-3VLI-22-2	0.0295	52.5	100.0	6.36	50.85	67	6	0.364
U-3VLI-22-3	0.0295	52.5	100.0	6.36	50.85	67	6	0.378
R-3VLI-22-1	0.0295	52.5	100.0	6.36	50.85	67	6	0.490
R-3VLI-22-2	0.0295	52.5	100.0	6.36	50.85	67	6	0.484
R-3VLI-22-3	0.0295	52.5	100.0	6.36	50.85	67	6	0.468

CHAPTER 4: ANALYTICAL STUDY

4.1 Web-Crippling Strength Calculations

For each specimen the web crippling strength of multiple-web cold-formed steel deck sections subjected to End One Flange (EOF) Loading was calculated using the AISI (1996) and S136 (1994) specifications. AISI (1996) and North American (2002) web crippling equations are not applicable to the decks whose inside bend radius-to-thickness ratios (R/t) are greater than 7.0. Therefore, web crippling strength calculations were not possible for HD, EHD and S decks of Consolidated Systems. However, the test results of these specimens are reported herein for comparison to the predicted values of AISI (1996) and North American (2002) specifications. Note that the web crippling equations are the same for unfastened and fastened multi-web specimens in both specifications. In AISI (1996) fastening of the flanges at end conditions were not considered to affect the web-crippling strength in EOF loading. In the North American (2002) specification the unfastened coefficients are used for fastened cases in a conservative approach. Moreover, the embossments on the webs of composite decks are not considered to affect web-crippling strength in any of the specifications.

The following equation is used in AISI (1996) to calculate the web-crippling strength of multi-web deck sections subjected to end one flange loading:

$$P_n = t^2 k C_1 C_4 C_9 C_\theta [331 - 0.61(h/t)][1 + 0.01(N/t)] \quad (4.1)$$

where:

P_n = nominal strength for concentrated load or reaction per web, kips

$$C_1 = 1.22 - 0.22k \quad (4.2)$$

$$C_4 = 1.15 - 0.15R/t \leq 1.0 \text{ but no less than } 0.50 \quad (4.3)$$

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

$$C_\theta = 0.7 + 0.3(\theta/90)^2 \quad (4.4)$$

F_y = design yield stress of the web

h = depth of flat portion of the web measured along the plane of the web, in.

$$k = 894F_y/E \quad (4.5)$$

t = web thickness, in.

N = actual length of bearing, in.

R = inside bend radius

θ = angle between the plane of the web and the plane of the bearing surface $\geq 45^\circ$, but not more than 90°

Equation 4.1 can be applied to decks when $R/t \leq 7$, $N/t \leq 210$ and $N/h \leq 3.5$. P_n represents the nominal strength for one solid web connecting top and bottom flanges. For two or more webs, P_n is computed for each individual web and the results added to obtain the nominal load or reaction for the multiple web (AISI 1996). The results of the analyses are illustrated in Table 4.1. An example analysis for B-Deck is shown in Appendix B.

Web crippling strength is calculated by the following unified expression in the North American Specification (2002), which was adopted from the Canadian S136 (1994) standards:

$$P_n = Ct^2F_y \sin \theta \left(1 - C_R \sqrt{\frac{R}{t}} \right) \left(1 + C_N \sqrt{\frac{N}{t}} \right) \left(1 - C_H \sqrt{\frac{h}{t}} \right) \quad (2.38)$$

where:

P_n = nominal web crippling strength, kips

C = coefficient from Table 4.2

C_R = inside bend radius coefficient from Table 4.2

C_N = bearing length coefficient from Table 4.2

C_H = web slenderness coefficient from Table 4.2

F_y = yield strength of steel

h = depth of flat portion of the web measured along the plane of the web, in.

t = thickness of the web, in.

N = actual length of bearing, in.

R = inside bend radius

θ = angle between the plane of the web and the plane of the bearing surface $\geq 45^\circ$, but not more than 90°

The equation can be applied to decks when $R/t \leq 7$, $h/t \leq 200$, $N/t \leq 210$ and $N/h \leq 3.0$. P_n represents the nominal strength for one solid web connecting top and bottom flanges. For two or more webs, P_n is computed for each web and the results are added to find the web-crippling strength for the multiple webs. The results of the analyses are summarized in Table 4.3. An example analysis for B-Deck is shown in Appendix B.

Table 4.1 Web Crippling Strength Calculations with AISI (1996) Specification

$$P_n = t^2 k C_1 C_4 C_5 C_\theta [331 - 0.61(h/t)][1 + 0.01(N/t)]$$

Type of deck	t (in)	F _y (ksi)	R/t	θ (deg)	k=894F _y /E	C ₁ = 1.22-0.22k	C ₄ =(1.15-0.15R/t) >0.5	C ₅	C _θ = 0.7+0.3(θ/90) ²	N/t	Total depth of deck (in)	h, flat portion of the web (in)	h/t	N/h	P _n per web (kips)
B-G22	0.0295	45.8	6.9	70	1.39	0.91	0.50	1.00	0.88	50.85	1.50	1.260	42.71	1.19	0.224
HD-G26	0.0182	95.4	14.6	58	2.89	0.58	0.50	1.00	0.82	82.42	0.94	0.779	42.83	1.93	0.128*
EHD-G26	0.0183	103.9	17.1	50	3.15	0.53	0.50	1.00	0.79	81.97	1.31	1.389	75.93	1.08	0.114*
V-G22	0.0300	48.0	6.8	76	1.45	0.90	0.50	1.00	0.91	50.00	2.00	1.697	56.57	0.88	0.238
S-G28	0.0153	105.2	11.2	58	3.19	0.52	0.50	1.00	0.82	98.04	0.56	0.446	29.16	3.36	0.098*
2VLI-G16	0.0598	46.5	3.1	63	1.41	0.91	0.68	1.00	0.85	25.08	2.00	1.901	31.78	0.79	1.031
2VLI-G18	0.0474	49.5	4.0	63	1.50	0.89	0.56	1.00	0.85	31.65	2.00	1.922	40.55	0.78	0.572
2VLI-G20	0.0358	52.0	5.2	63	1.58	0.87	0.50	1.00	0.85	41.90	2.00	1.942	54.25	0.77	0.317
2VLI-G22	0.0295	54.0	6.4	63	1.64	0.86	0.50	1.00	0.85	50.85	2.00	1.953	66.21	0.77	0.228
3VLI-G16	0.0598	35.0	3.1	67	1.06	0.99	0.68	1.00	0.87	25.08	3.00	2.895	48.42	0.52	0.833
3VLI-G18	0.0474	48.0	4.0	67	1.45	0.90	0.56	1.00	0.87	31.65	3.00	2.917	61.54	0.51	0.549
3VLI-G20	0.0358	53.5	5.2	67	1.62	0.86	0.50	1.00	0.87	41.90	3.00	2.937	82.05	0.51	0.310
3VLI-G22	0.0295	52.5	6.4	67	1.59	0.87	0.50	1.00	0.87	50.85	3.00	2.948	99.94	0.51	0.213

* Equation 4.1 is not applicable.

Table 4.2 Multiple Web Deck Sections when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 3$ and $45^\circ < \theta \leq 90^\circ$

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	

Table 4.3 Web Crippling Strength Calculations with North American (2001) Specification

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

Type of deck	t (in)	F _y (ksi)	θ (deg)	Total depth of deck (in)	h, flat portion of the web (in)	R/t	N/t	h/t	N/h	C	C _R	C _N	C _H	P _n per web (kips)
B-G22	0.0295	45.8	70	1.50	1.260	6.9	50.85	42.71	1.19	3.00	0.08	0.70	0.055	0.341
HD-G26	0.0182	95.4	58	0.94	0.779	14.6	82.42	42.83	1.92	3.00	0.08	0.70	0.055	0.263*
EHD-G26	0.0183	103.9	50	1.31	1.389	17.1	81.97	75.93	1.08	3.00	0.08	0.70	0.055	0.204*
V-G22	0.0300	48.0	76	2.00	1.697	6.8	50.00	56.57	0.88	3.00	0.08	0.70	0.055	0.347
S-G28	0.0153	105.2	58	0.56	0.446	11.2	98.04	29.16	3.36	3.00	0.08	0.70	0.055	0.256*
2VLI-G16	0.0598	46.5	63	2.00	1.901	3.1	25.08	31.78	0.79	3.00	0.08	0.70	0.055	1.191
2VLI-G18	0.0474	49.5	63	2.00	1.922	4.0	31.65	40.55	0.78	3.00	0.08	0.70	0.055	0.805
2VLI-G20	0.0358	52.0	63	2.00	1.942	5.2	41.90	54.25	0.77	3.00	0.08	0.70	0.055	0.481
2VLI-G22	0.0295	54.0	63	2.00	1.953	6.4	50.85	66.21	0.77	3.00	0.08	0.70	0.055	0.333
3VLI-G16	0.0598	35.0	67	3.00	2.895	3.1	25.08	48.42	0.52	3.00	0.08	0.70	0.055	0.827
3VLI-G18	0.0474	48.0	67	3.00	2.917	4.0	31.65	61.54	0.51	3.00	0.08	0.70	0.055	0.705
3VLI-G20	0.0358	53.5	67	3.00	2.937	5.2	41.90	82.05	0.51	3.00	0.08	0.70	0.055	0.430
3VLI-G22	0.0295	52.5	67	3.00	2.948	6.4	50.85	99.94	0.51	3.00	0.08	0.70	0.055	0.272

* Equation 2.38 is not applicable

4.2 Comparison of Analytical Results with the Test Results

In Tables C.1 and C.2 in Appendix C the test results are compared with the predicted values using the P_t/P_n ratios for both the unfastened and fastened cases. This comparison is also illustrated in Figs. C.1 and C.2. All 78 test specimens resulted in P_t/P_n values greater than unity by the AISI (1996) method. North American Specification method resulted in P_t/P_n values greater than unity for most of the specimens, meaning that the tested web-crippling values are greater than the predicted web-crippling values. This makes the analytical approaches conservative. For the North American (2002) method, P_t/P_n values which were found to be less than unity belonged to specimens with R/t ratios greater than 7.0.

AISI (1996) values are more conservative than North American Specification values for most of the specimens. When unfastened and fastened cases are compared, it is realized that P_t/P_n values for the fastened case are more conservative than for the unfastened cases. The degree of conservativeness for the North American Specification is more than 50% in unfastened cases and more than 100% in fastened cases for the specimens that have the maximum P_t/P_n ratios. On the other hand, the degree of conservativeness for AISI (1996) is more than 100% in unfastened cases and more than 175% in fastened cases for the specimens that have the maximum P_t/P_n ratios.

CHAPTER 5: DERIVATION AND CALIBRATION OF NEW COEFFICIENTS

5.1 General

In this section, new coefficients are derived for the End One Flange loading of multi-web cold-formed steel sections. Available experimental data in the literature reported by Beshara (2000) and the results of this particular study are used for that purpose. Although fastening of specimens was believed to affect the web-crippling capacity, it was only recently that this influence was incorporated into the AISI (North American Specification 2002). Also, the unified web crippling equation (Eq. 2.38) of the Canadian Standards (S136-94) was accepted by the AISI in the 2001 draft:

$$P_n = Ct^2 F_y \sin \theta \left(1 - C_R \sqrt{\frac{R}{t}} \right) \left(1 + C_N \sqrt{\frac{N}{t}} \right) \left(1 - C_H \sqrt{\frac{h}{t}} \right) \quad (2.38)$$

5.2 Web Crippling Tests (EOF Loading) in the Literature

The coefficients for the unfastened cases were used for fastened cases in the study by Beshara (2000) because not enough data were available to determine coefficients for the fastened case. The fastened tests conducted at Virginia Tech made the development of coefficients for the fastened cases possible. Also, the coefficients for the unfastened cases were improved by the results of unfastened tests at Virginia Tech. Table 5.1 shows the experimental studies used in the development of the new coefficients. Tables A.1 to A.4 show the results of the experimental studies mentioned in Table 5.1. The cross sectional parameters of the specimens and test results of the studies by Yu (1981), Bhakta (1992) and Wu (1997) were reported by Beshara (2000).

Table 5.1 Experimental Studies on EOF Loading of Deck Sections

Support Condition	Name	University	Number of Data Points
Unfastened	Yu, 1981	University of Missouri- Rolla	18
	Bhakta, 1992	University of Missouri- Rolla	2
	Wu, 1997	University of Missouri- Rolla	16
	Avci, 2001	Virginia Tech	39
Fastened	Bhakta, 1992	University of Missouri- Rolla	2
	Avci, 2001	Virginia Tech	39

5.3 Derivation of New Coefficients

A nonlinear regression analysis was performed by using the unified web crippling expression to update the unfastened case coefficients and predict fastened case coefficients for multi-web deck cross sections subjected to EOF loading. For the regression analysis, the results of studies illustrated in Fig. 5.1 were analyzed using SigmaPlot 2000 computer software. A total number of 75 data points was used for the derivation of unfastened case coefficients. The number of data points used for the derivation of the fastened case coefficients was 41. The program was executed several times and different coefficient combinations were compared to obtain as large adjusted R^2 values as possible where satisfying Normality and Constant Variance tests at the same time. The new coefficients (C , C_R , C_N and C_h) for Eq. 2.38 are proposed for the unfastened and fastened cases in Table 5.2.

Table 5.2 New Coefficients for Multi-web Deck Cross Sections (EOF Loading)

Support Condition	C	C_R	C_N	C_h
Unfastened	4.49	0.05	0.42	0.05
Fastened	5.11	0.20	0.99	0.05

For both unfastened and fastened cases, the old coefficients were $C=3.00$, $C_R=0.08$, $C_N=0.70$ and $C_h=0.055$. The comparison of the test loads and predicted loads calculated by old and new coefficients are presented in Tables C.1 to C.4 and illustrated in Figs. C.3 to C.5. For the unfastened cases, the proposed coefficients resulted in better predictions than the old coefficients for normal strength steels as shown in Fig. C.3. However, for high strength steels neither old nor new coefficients resulted in satisfactory predictions. With the new coefficients, the web crippling capacity was overestimated by more than 50% in some cases, while old coefficients resulted in both conservative and unconservative predictions for high strength steels, as illustrated in Fig. C.4. In Fig. C.6, the ratio of test loads to the predicted loads (P_t/P_n) is shown with respect to varying yield strength values. It is clear that for high strength steels neither the new nor the old coefficients are satisfactory. For the fastened cases, the proposed coefficients resulted in better predictions than the old coefficients, as illustrated in Fig. C.5. The mean (P_m), standard deviation (σ) and coefficient of variation (COV or V_p) results of the P_t/P_n values calculated by the new coefficients are presented in Table 5.3.

Table 5.3 Statistical Results of the Regression Analysis for P_t/P_n Values

	Unfastened Case	Fastened Case
Mean, P_m	0.925	0.991
Standard Deviation, σ	0.237	0.181
Coefficient of Variation, COV or V_p	0.257	0.183

5.4 Calibration of New Coefficients

Uncertainties in the designs are overcome by resistance factors (ϕ) in the Load and Resistance Factor Design (LRFD) method and by factors of safety (Ω) in the Allowable Stress Design (ASD) Method. Both AISI (1996) and S136 (1994) use the Reliability index, β , as given in Eq. 5.1, to introduce a measure of reliability of a structural component or an entire structure. The reliability index value depends on the

type of load action and type of resistance. The recommended lower bound β value for *members* in the AISI (1996) specification is 2.5, while it is 3.0 in S136 (1994). New coefficients were calibrated for AISI (1996) and Canadian S136 (1994) using both ASD and LRFD methods based on the procedures introduced by Hsiao (1988), Supornsilaphachai (1979) and Gerges (1997):

$$\beta = \frac{\ln(R_m/Q_m)}{\sqrt{V_R^2 + V_Q^2}} \quad (5.1)$$

$$e^{\beta\sqrt{V_R^2 + V_Q^2}} = R_m/Q_m \quad (5.2)$$

where R_m is the mean value of resistance and Q_m is the mean value of the load effect. V_R is the coefficient of variation for the resistance and V_Q is the coefficient of variation for the load effect. R_m , the mean value of resistance, can be determined from

$$R_m = R_n M_m F_m P_m \quad (5.3)$$

where R_n is the nominal resistance, and M_m , F_m and P_m are the mean values of the dimensionless random variables reflecting the uncertainties in the material properties, the geometry of the cross section and the prediction of the ultimate resistance, respectively (Beshara 2000).

Q_m , the mean value for the load effect can be determined from

$$Q_m = C(D_m + L_m) \quad (5.4)$$

where C is a deterministic influence coefficient which transforms the dead and live load intensities to load effects, and D_m and L_m are the mean values of the dead load and the live load intensities, respectively (Beshara 2000).

V_R , the coefficient of variation for the resistance and V_Q , the coefficient of variation for the load effect, can be determined from

$$V_R = \sqrt{V_M^2 + V_F^2 + V_P^2} \quad (5.5)$$

$$V_Q = \frac{\sqrt{D_m^2 V_D^2 + L_m^2 V_L^2}}{D_m + L_m} \quad (5.6)$$

where V_M , V_F and V_P are the coefficients of variation of the dimensionless random variables. V_M reflects the uncertainties in the material properties, whereas V_F and V_P reflect the uncertainties in the geometry of the cross section and the prediction of the

ultimate resistance, respectively. V_D and V_L are the coefficients of variation for the dead and live loads. Dividing both the numerator and denominator by L_m in Eq. 5.6,

$$V_Q = \frac{\sqrt{(D_m/L_m)^2 V_D^2 + V_L^2}}{(D_m/L_m) + 1.0} \quad (5.7)$$

Based on the statistical analysis and past experience (experimental investigations and cross sectional measurements), the following values are recommended by Hsiao (1988):

$$\begin{aligned} M_m &= 1.10 \\ F_m &= 1.00 \\ V_m &= 0.10 \\ V_F &= 0.05 \\ D_m &= 1.05 D_n \\ V_D &= 0.10 \\ V_L &= 0.25 \\ L_m &= L_n \end{aligned} \quad (5.8)$$

D_n is the nominal value for the dead load while L_n is the nominal value for the live load intensity. When the values in Eq. 5.8 are substituted into Eq. 5.3, Eq. 5.9 is obtained. Eq. 5.10 is obtained when the values in Eq. 5.8 are substituted into Eq. 5.5:

$$R_m = 1.10 P_m R_n \quad (5.9)$$

$$V_R = \sqrt{0.0125 + V_P^2} \quad (5.10)$$

Substituting the values of Eq. 5.8 into Eq. 5.7, Eq. 5.11 is obtained:

$$V_Q = \frac{\sqrt{(1.05 D_n / L_n)^2 (0.1)^2 + 0.0625}}{(1.05 D_n / L_n) + 1.0} \quad (5.11)$$

$$\frac{D_n}{L_n} = \frac{1}{3} \quad \text{in S136 (1994)} \quad (5.12)$$

$$\frac{D_n}{L_n} = \frac{1}{5} \quad \text{in AISI (1996)} \quad (5.13)$$

Substituting the values of Eq. 5.12 and Eq. 5.13 into Eq. 5.11, Eq. 5.14 and Eq. 5.15 are obtained:

$$\text{For S136 (1994), } V_Q = 0.187 \quad (5.14)$$

$$\text{For AISI (1996), } V_Q = 0.207 \quad (5.15)$$

5.4.1 Derivation of Factor of Safety (Ω) for Allowable Stress Design

Allowable stress design strength is calculated by dividing the nominal strength (R_n) by a factor of safety (Ω).

$$(\text{Allowable Strength} = R_n / \Omega) \geq (\Sigma Q_i = \text{Load Effects}) \quad (5.16)$$

$$\text{Nominal Strength, } R_n = \Omega C(D_n + L_n) \quad (5.17)$$

Substituting Eq. 5.2, Eq. 5.3 and Eq. 5.4 into Eq. 5.17, the following three equations can be obtained (Beshara 2000).

$$\Omega = \left(\frac{R_m}{M_m F_m P_m} \right) \frac{1}{C(D_n + L_n)} \quad (5.18)$$

$$\Omega = \left(\frac{R_m}{M_m F_m P_m} \right) \left(\frac{D_m + L_m}{D_n + L_n} \right) \frac{1}{Q_m} \quad (5.19)$$

$$\Omega = \frac{e^{\beta \sqrt{V_R^2 + V_Q^2}}}{M_m F_m P_m} \left(\frac{1.05 D_n / L_n + 1}{D_n / L_n + 1} \right) \quad (5.20)$$

5.4.1.1 Unfastened Case

Substituting Eq. 5.8, Eq. 5.10, Eq. 5.12, Eq. 5.13, Eq. 5.14, Eq. 5.15, mean ($P_m=0.925$) and coefficient of variation ($COV=V_P=0.257$) from Table 5.7 into Eq. 5.20, one obtains:

For S136 (1994), ($\beta=3.0$, recommended as a lower bound value for members),

$$\Omega = \frac{e^{3.0 \sqrt{0.0475 + V_P^2}}}{1.0864 P_m} = 2.73 \quad (5.21)$$

For AISI (1996), ($\beta=2.5$, recommended as a lower bound value for members),

$$\Omega = \frac{e^{2.5 \sqrt{0.0554 + V_P^2}}}{1.0909 P_m} = 2.37 \quad (5.22)$$

5.4.1.2 Fastened Case

Substituting Eq. 5.8, Eq. 5.10, Eq. 5.12, Eq. 5.13, Eq. 5.14, Eq. 5.15, mean ($P_m=0.991$) and coefficient of variation ($COV=V_P=0.183$) from Table 5.7 into Eq. 5.20, one obtains:

For S136 (1994), ($\beta=3.0$, recommended as a lower bound value for members),

$$\Omega = \frac{e^{3.0\sqrt{0.0475+V_P^2}}}{1.0864P_m} = 2.18 \quad (5.23)$$

For AISI (1996), ($\beta=2.5$, recommended as a lower bound value for members),

$$\Omega = \frac{e^{2.5\sqrt{0.0554+V_P^2}}}{1.0909P_m} = 2.13 \quad (5.24)$$

5.4.2 Derivation of Resistance Factor (ϕ) for Load and Resistance Factor Design

In the LRFD approach, the nominal strength (R_n) multiplied by the resistance factor (ϕ) has to be greater than or equal to the load effects ($\Sigma\gamma_i Q_i$) for any member or the structure.

$$(\text{Design Strength} = \phi R_n) \geq (\Sigma\gamma_i Q_i = \text{Load Effects}) \quad (5.25)$$

$$\text{Nominal Strength, } \phi R_n = C(\alpha_D D_n + \alpha_L L_n) \quad (5.26)$$

Dividing both the numerator and denominator by L_n in Eq. 5.26,

$$\phi R_n = CL_n \left(\alpha_D \frac{D_n}{L_n} + \alpha_L \right) \quad (5.27)$$

$$\text{and } \phi = \frac{CL_n}{R_n} \left(\alpha_D \frac{D_n}{L_n} + \alpha_L \right) \quad (5.28)$$

Substituting Eq. 5.2, Eq. 5.3 and Eq. 5.4 into Eq. 5.28, the following three equations can be obtained:

$$\phi = \frac{M_m F_m P_m Q_m}{R_m \left(\frac{1.05 D_n}{L_n} + 1.0 \right)} \left(\alpha_D \frac{D_n}{L_n} + \alpha_L \right) \quad (5.29)$$

$$\phi = \frac{M_m F_m P_m}{e^{\beta \sqrt{V_R^2 + V_Q^2}} \left(\frac{1.05 D_n}{L_n} + 1.0 \right)} \left(\alpha_D \frac{D_n}{L_n} + \alpha_L \right) \quad (5.30)$$

$$\frac{D_n}{L_n} = \frac{1}{3}, \alpha_D = 1.25, \alpha_L = 1.50 \text{ in S136 (1994).} \quad (5.31)$$

$$\frac{D_n}{L_n} = \frac{1}{5}, \alpha_D = 1.20, \alpha_L = 1.60 \text{ in AISI (1996).} \quad (5.32)$$

5.4.2.1 Unfastened Case

Substituting Eq. 5.8, Eq. 5.10, Eq. 5.14, Eq. 5.15, Eq. 5.31, Eq. 5.32, mean ($P_m=0.925$) and coefficient of variation ($COV=V_P=0.257$) from Table 5.7 into Eq. 5.30, one obtains:

For S136 (1994), ($\beta=3.0$, recommended as a lower bound value for members),

$$\phi = \frac{1.562P_m}{e^{3.0\sqrt{0.0475+V_P^2}}} = 0.53$$

For AISI (1996), ($\beta=2.5$, recommended as a lower bound value for members),

$$\phi = \frac{1.673P_m}{e^{2.5\sqrt{0.0554+V_P^2}}} = 0.65$$

5.4.2.2 Fastened Case

Substituting Eq. 5.8, Eq. 5.10, Eq. 5.14, Eq. 5.15, Eq. 5.31, Eq. 5.32, mean ($P_m=0.991$) and coefficient of variation ($COV=V_P=0.183$) from Table 5.7 into Eq. 5.30, one obtains:

For S136 (1994), ($\beta=3.0$, recommended as a lower bound value for members),

$$\phi = \frac{1.562P_m}{e^{3.0\sqrt{0.0475+V_P^2}}} = 0.66$$

For AISI (1996), ($\beta=2.5$, recommended as a lower bound value for members),

$$\phi = \frac{1.673P_m}{e^{2.5\sqrt{0.0554+V_P^2}}} = 0.79$$

Table 5.8 presents the resistance factors (ϕ) and factors of safety (Ω) derived by calibrating the ratios of test web crippling loads (EOF) to the predicted web-crippling loads (P_t/P_n).

Table 5.4 Results of the Calibration for Multi-web Sections Under EOF Loading

	S136 (1994)		AISI (1996)	
	Ω	ϕ	Ω	ϕ
Unfastened Case	2.73	0.53	2.37	0.65
Fastened Case	2.18	0.66	2.13	0.79

CHAPTER 6: SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

The objective of this study was to evaluate the web-crippling strength of cold-formed steel deck sections subjected to end one flange (EOF) loading. The comparison of the test results with different strength prediction approaches and derivation of new coefficients were the main objectives. The study also focused on the effect of fastening through the supports of the members.

The complicated nature of web-crippling behavior makes an experimental investigation the best way to determine the web crippling resistance of a cross-section. The extensive statistical analysis conducted by Beshara (2000) resulted in identical coefficients for both unfastened and fastened specimens because there were not enough data points for fastened cases. He recommended the use of unfastened coefficients as a conservative approach for the fastened case. However, after the experimental study presented in this report, the derivation of separate coefficients for the fastened cases became possible. The derivation of the coefficients for the fastened support conditions was important because field practice can be represented better with the fastened test specimens than with the unfastened ones.

A total of 78 tests was conducted. The specimens were constructed and tested at the Structures and Materials Research Laboratory at Virginia Tech. Each specimen type was tested both fastened and unfastened to the supports to determine the effect of fastening. Some of the decks had web embossments, whereas the others did not. The experimental study and the analytical study are presented in Chapter 3 and Chapter 4, respectively. Considering the experimental data cited by Beshara (2000) and the data points of this study, the new coefficients for the unified expression of the Canadian Code were derived statistically in Chapter 5. The derivation of the coefficients was done by a

nonlinear regression analysis using SigmaPlot software. Also in Chapter 5, the coefficients were calibrated in accordance with Canadian S136-94 and the American AISI (1996) specification. Tensile coupon tests and sample calculations are presented in the appendices.

6.2 Conclusions

From the test results, the Canadian S136-94 and the American AISI (1996) web crippling equations were found to be conservative under end one flange loading for multi-web deck cross sections. The new coefficients for the unified web crippling equation resulted in a better prediction of web crippling loads because an increased number of data points were used in the regression analysis.

For the unfastened case, there are some data points for which the old coefficients resulted in better approximations. However, the majority of the data points were approximated better with the new coefficients than the old coefficients.

The test value (P_t) and the predicted value (P_n) for the web crippling capacity are close for most of the fastened specimens. In other words, the ratio, P_t/P_n is close to 1.0 for most of the fastened data points. However, for high strength steels, the P_t/P_n ratio was as high as 1.41 for some fastened specimens. For the unfastened cases, the ratio, P_t/P_n was as high as 1.3 and as low as 0.44 for some specimens. The low P_t/P_n values for the unfastened specimens were observed especially on the specimens made of high strength steels, where F_y values exceed 90 ksi. That means for high strength steels the coefficients are not as satisfactory as they are for normal strength steels in the analysis of both unfastened and fastened specimens.

6.3 Recommendations for Further Research

In the experimental study, some of the deck cross-sections had embossments on their webs, while others did not. An experimental investigation should be conducted to determine the effect of the embossments on the web crippling strength.

Because the field practice can be represented better by the fastened specimens, more experimental study should be conducted to improve the coefficients for that case.

Also, additional tests should be carried out for the unfastened cases to improve the accuracy of the coefficients.

Because the derived coefficients for high strength steels did not give as good results as for the normal strength steels, the web crippling strength of the high strength steels need to be investigated further. Conducting more tests, hence deriving more accurate coefficients, may eliminate very high and very low P_t/P_n values for both unfastened and fastened support conditions.

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APPENDIX-A TENSILE COUPON TESTS

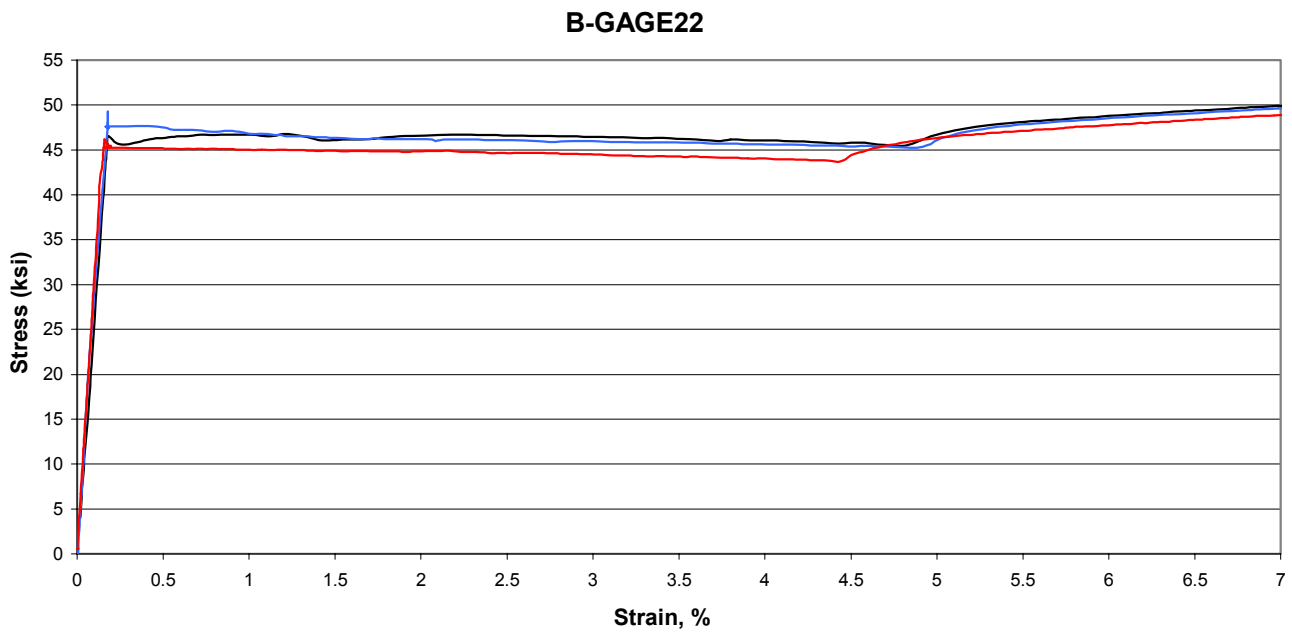


Figure A.1 Tensile Coupon Tests of B-Deck

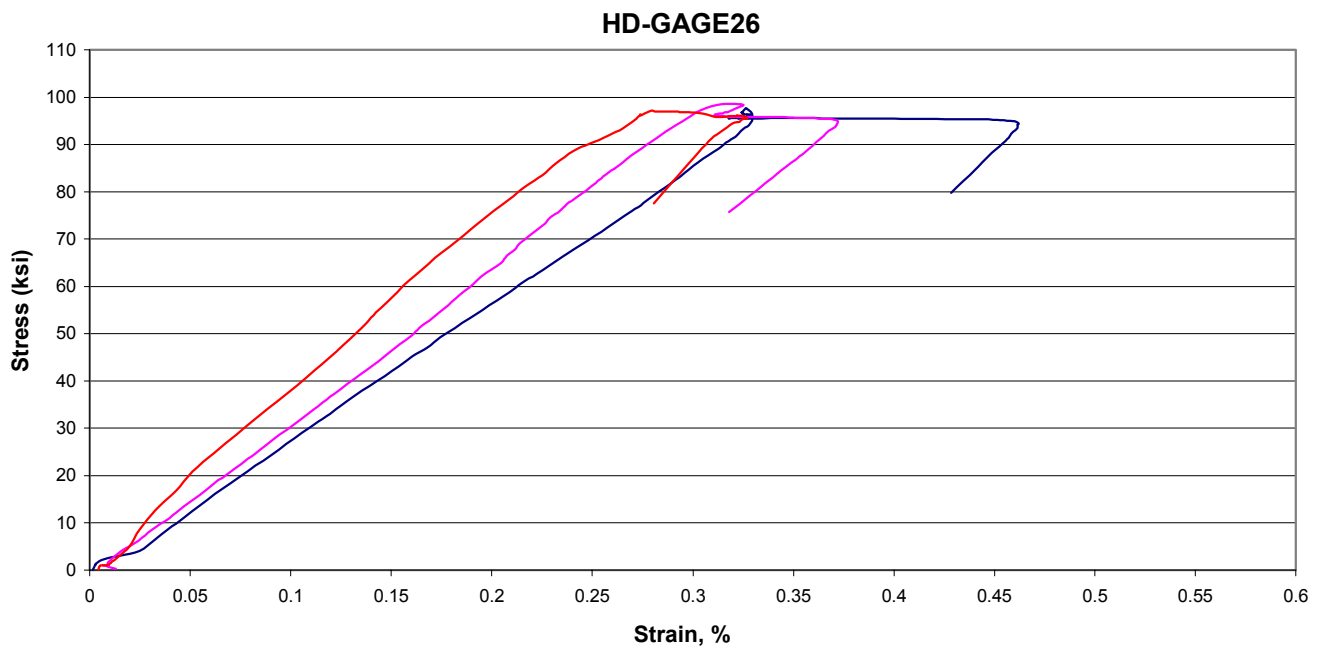


Figure A.2 Tensile Coupon Tests of HD-Deck

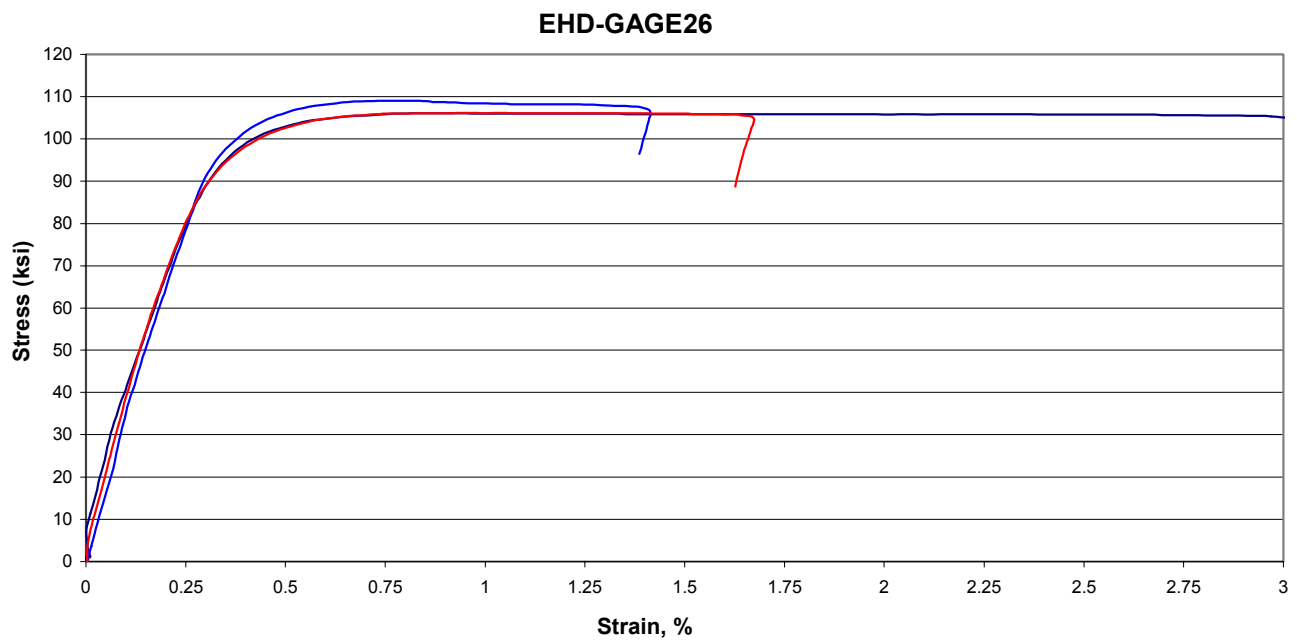


Figure A.3 Tensile Coupon Tests of EHD-Deck

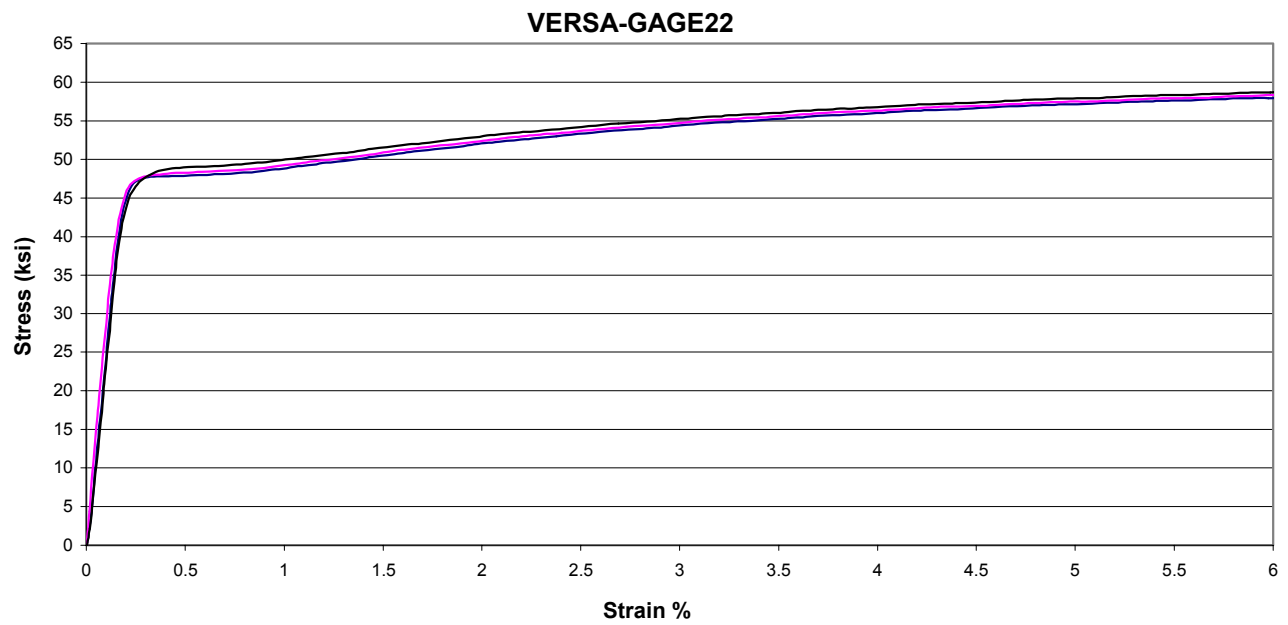


Figure A.4 Tensile Coupon Tests of Versa-Deck

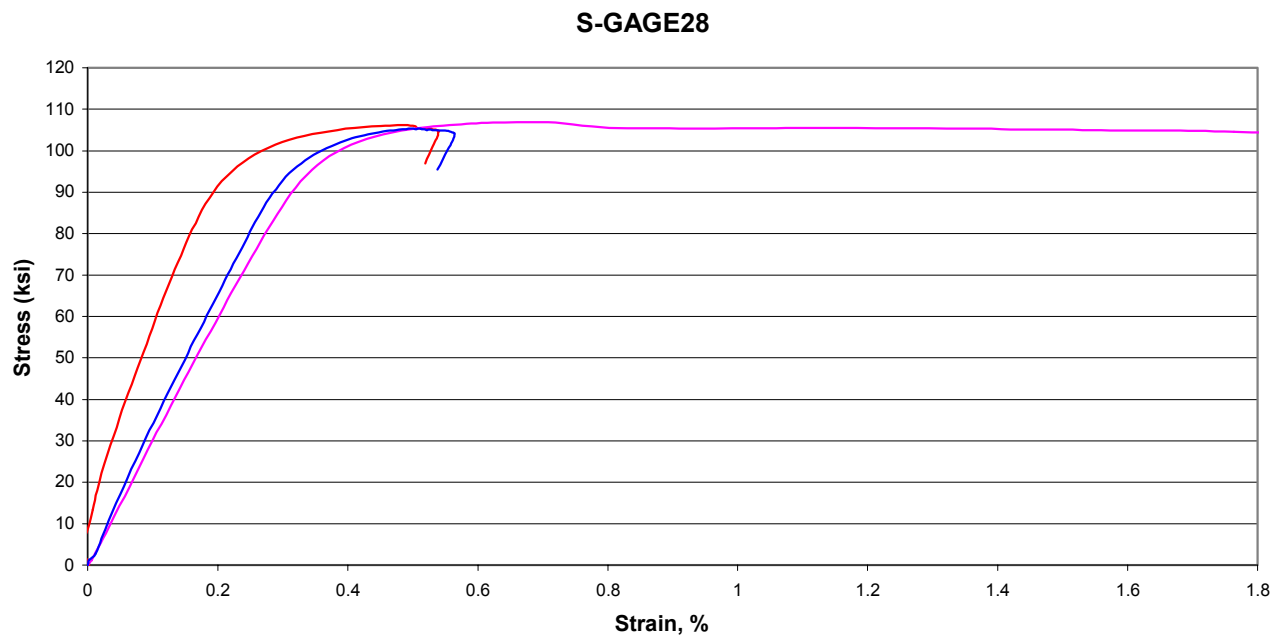


Figure A.5 Tensile Coupon Tests of S-Deck

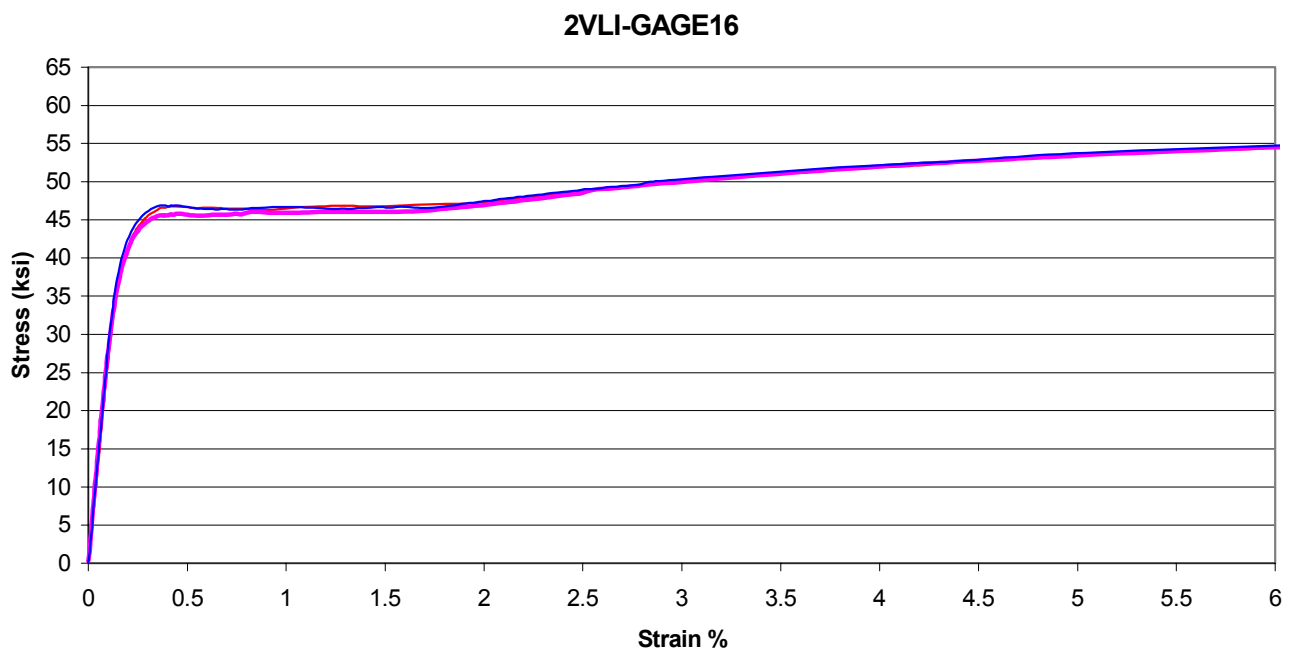


Figure A.6 Tensile Coupon Tests of 2VLI(Gage16)-Deck

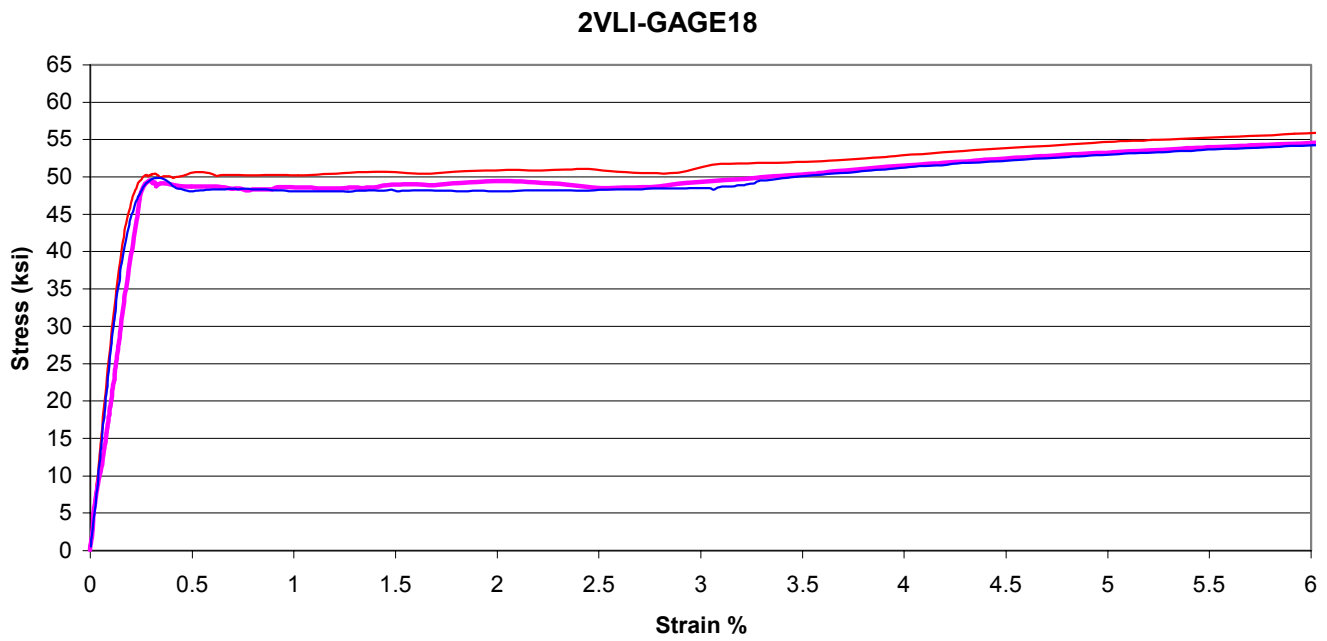


Figure A.7 Tensile Coupon Tests of 2VLI(Gage18)-Deck

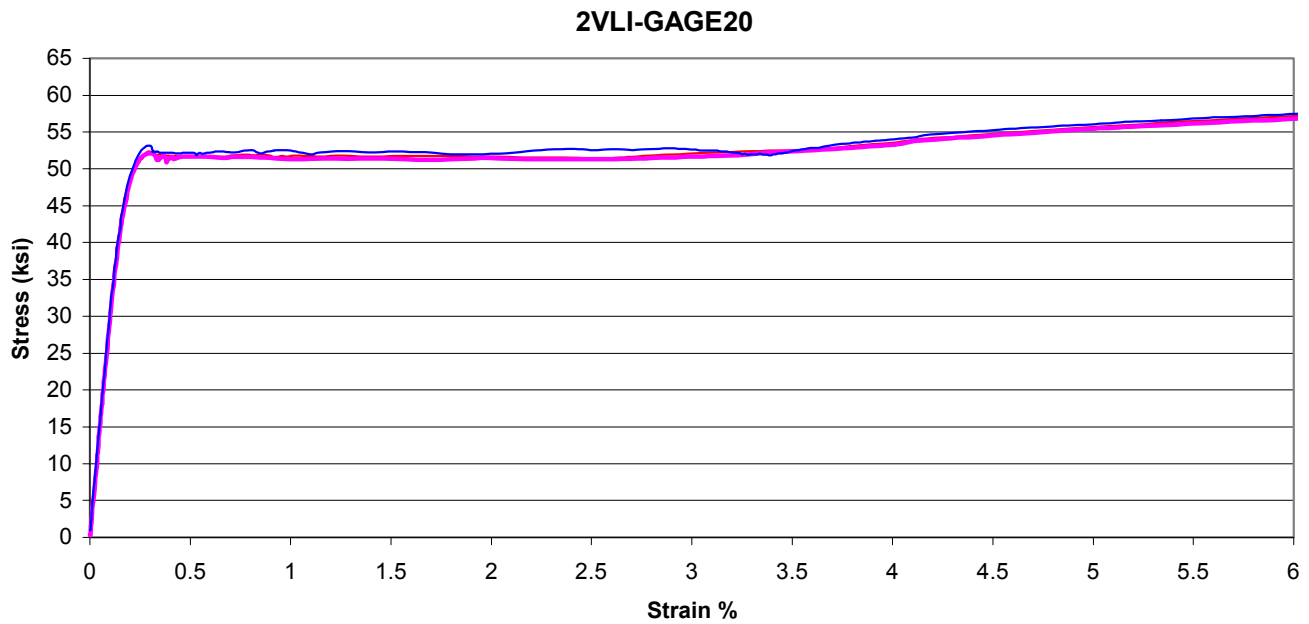


Figure A.8 Tensile Coupon Tests of 2VLI(Gage20)-Deck

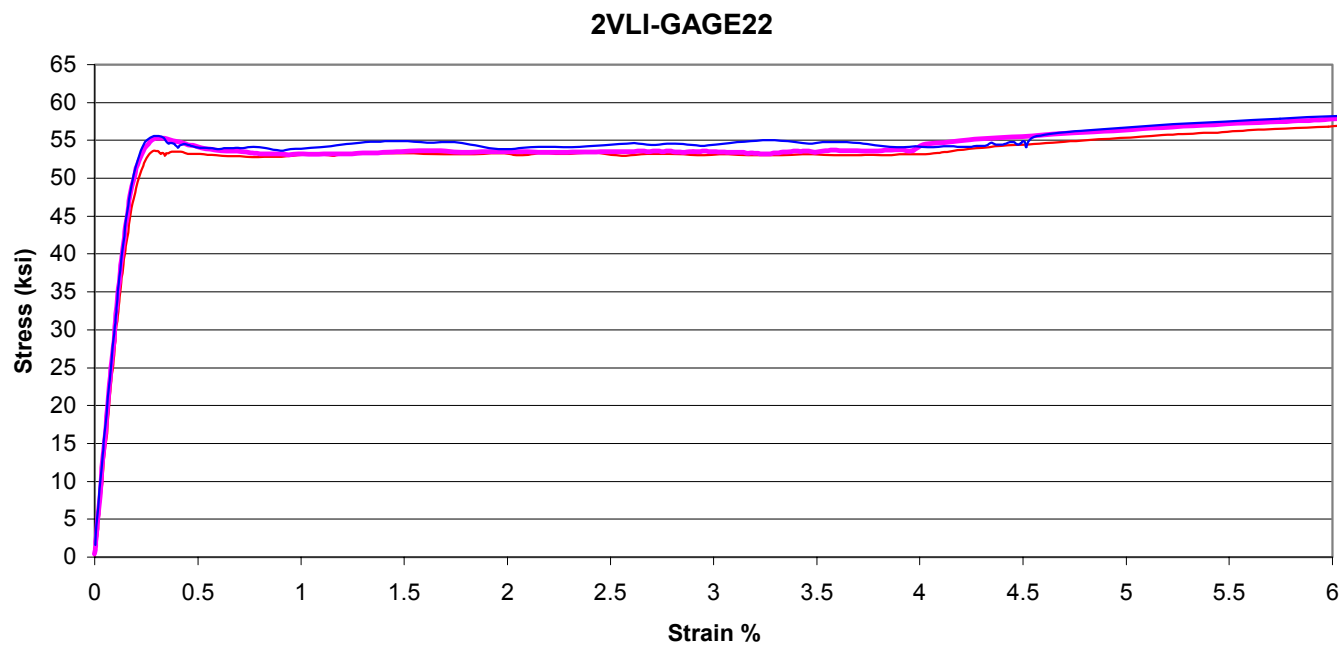


Figure A.9 Tensile Coupon Tests of 2VLI(Gage22)-Deck

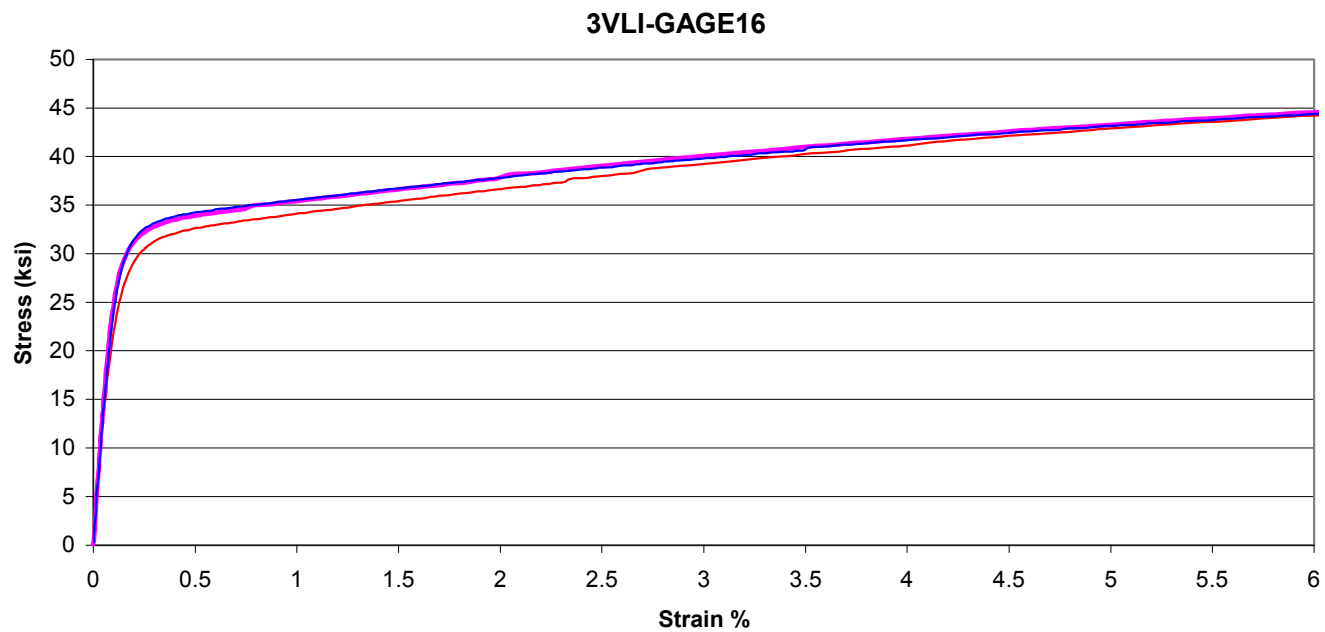


Figure A.10 Tensile Coupon Tests of 3VLI(Gage16)-Deck

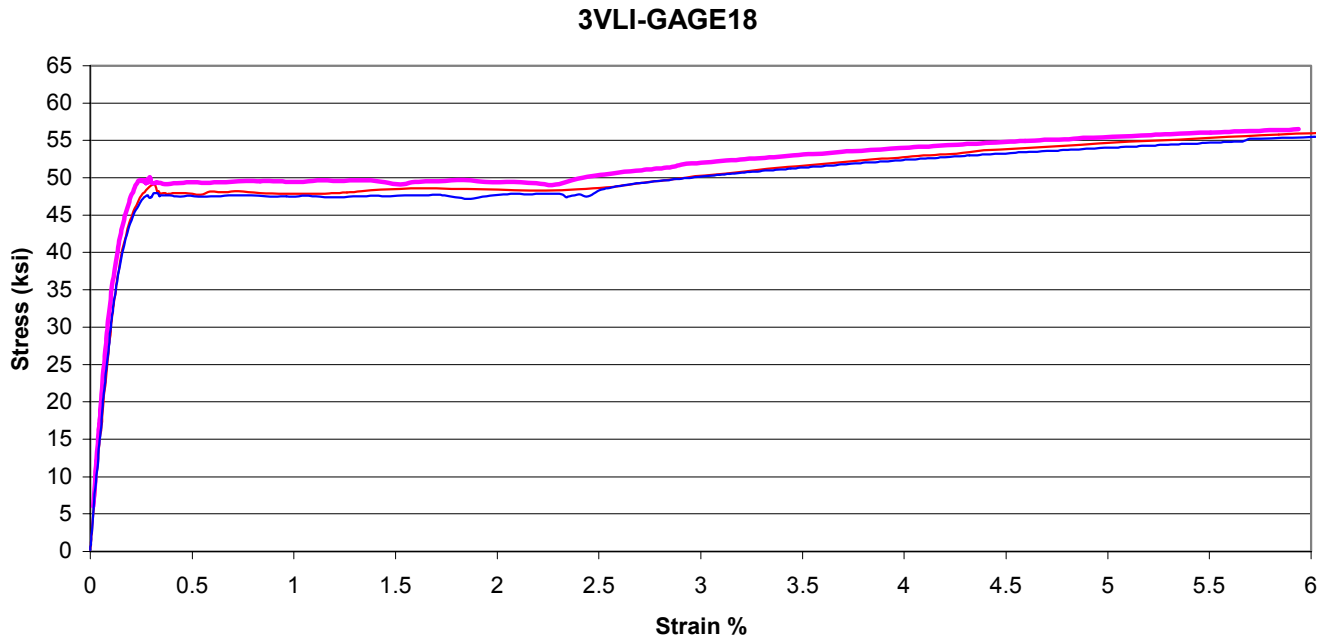


Figure A.11 Tensile Coupon Tests of 3VLI(Gage18)-Deck

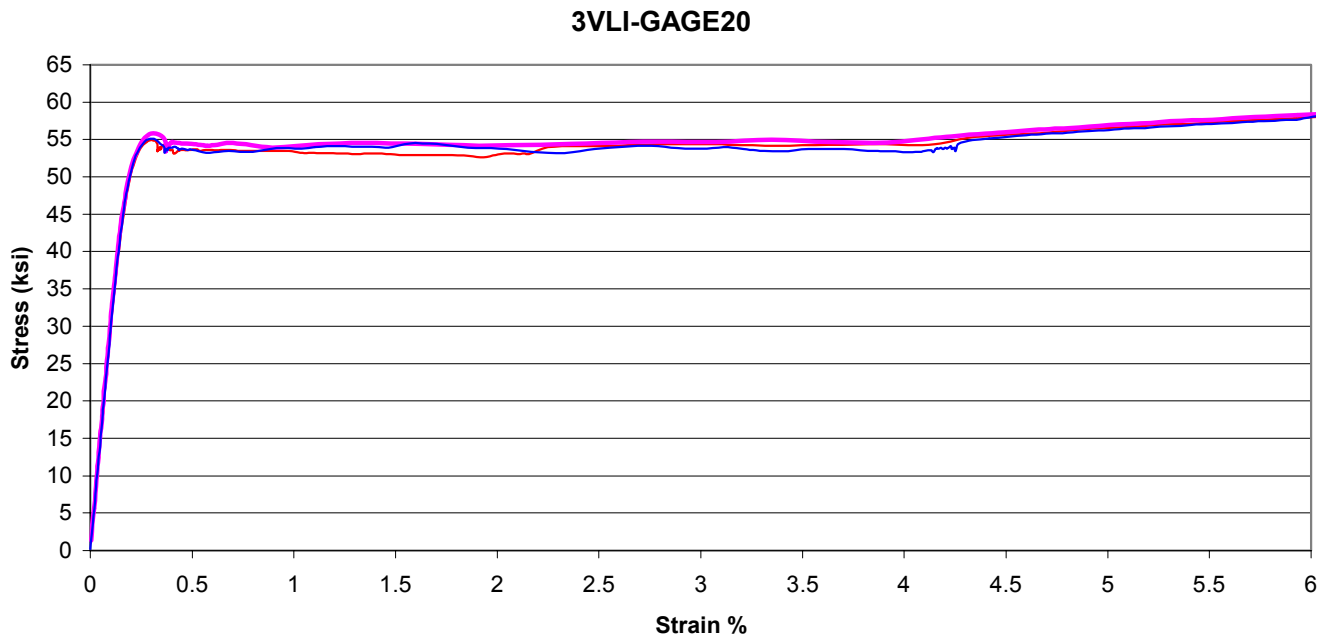


Figure A.12 Tensile Coupon Tests of 3VLI(Gage20)-Deck

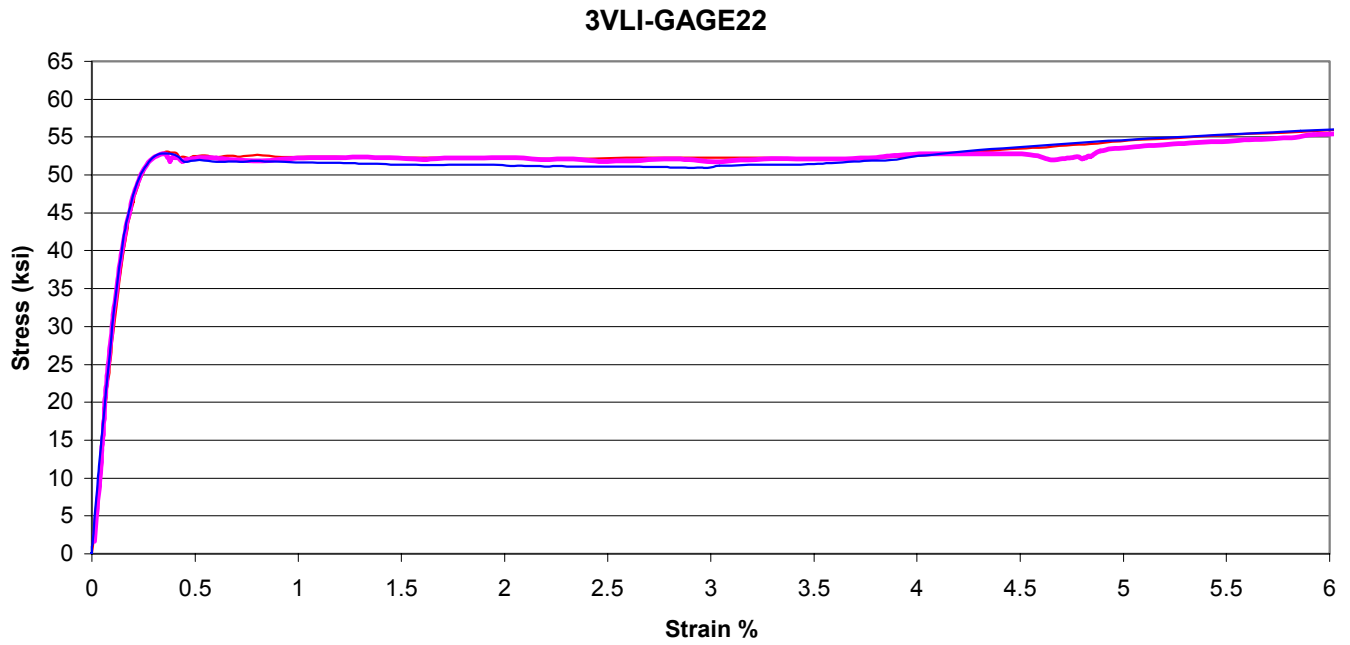


Figure A.13 Tensile Coupon Tests of 3VLI(Gage22)-Deck

APPENDIX-B WEB CRIPPLING STRENGTH CALCULATION EXAMPLE

B.1 Cross-Sectional Parameters of B-Deck

The web crippling capacity of the unfastened CSI B-deck section is presented here. The procedure is the same for the other deck cross sections. Determination of the flat portion of the web (h) is the major concern in web crippling calculations. Using the web inclination (θ), the total depth of the deck (D), the thickness (t) and the inside bent radius (R), the flat portion of the web (h) can be calculated. The cross sectional detail of B-deck is shown in Fig. B.1.

$$\theta = 70^\circ$$

$$t = 0.0295 \text{ in}$$

$$D = 1.50 \text{ in}$$

$$R = 13/64 \text{ in and } R/t = 6.89$$

From geometry,

$$h_{flat} = \frac{1.031}{\sin 70^\circ} + 2 \left(\frac{0.218}{\tan 70^\circ} \right) = 1.26 \text{ in.}$$

$$\text{Then, } h/t = 42.71$$

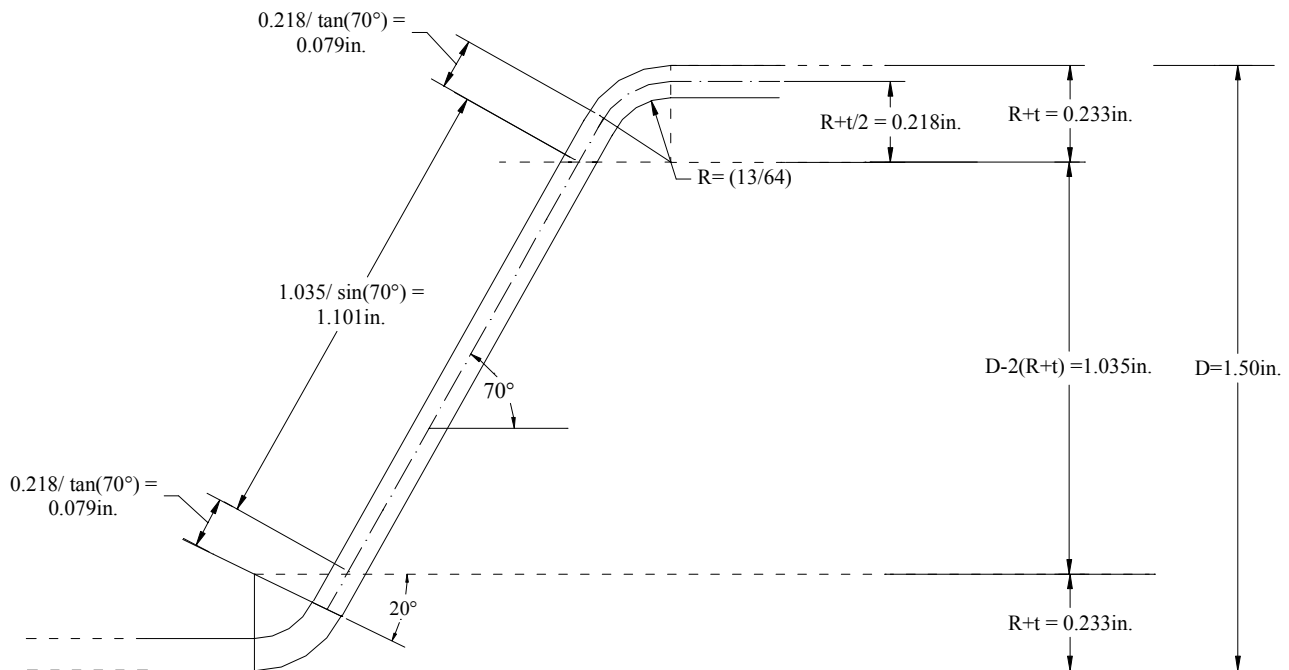


Figure B.1 Cross Sectional Detail of B-Deck

The bearing length (N) is 1.5in for B-deck and for the rest of the specimens. Hence, $N/t=50.85$ for B-deck.

B.2 Web Crippling Calculations for B-Deck

B.2.1 American Iron and Steel Institute Design Specification (1996) Approach

The following equation is used in order to calculate the web-crippling capacity of multi-web deck sections subjected to end one flange loading:

$$P_n = t^2 k C_1 C_4 C_9 C_\theta [331 - 0.61(h/t)][1 + 0.01(N/t)]$$

where,

P_n = Nominal strength for concentrated load or reaction per web, kips

$$k = \frac{894 F_y}{E} = \frac{(894)(45.8)}{29500} = 1.39$$

$$C_1 = 1.22 - 0.22k = 0.91$$

$$C_4 = 1.15 - 0.15 \left(\frac{R}{t} \right) = 0.115 < 0.5 \quad \text{so, } C_4 = 0.5$$

$C_9 = 1.0$ for U.S. customary units.

$$C_\theta = 0.7 + 0.3 \left(\frac{\theta}{90} \right)^2 = 0.88$$

Substituting,

$$P_n = 0.244 \text{ kips}$$

B.2.2 North American Specification (September 2001 Draft) Approach

The following equation is used in order to calculate web-crippling capacity of multi-web deck sections subjected to end one flange loading:

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

where

P_n = Nominal web crippling strength, kips

With old coefficients,

$$C = 3.00$$

$$C_R = 0.08$$

$$C_N = 0.70$$

$$C_h = 0.055$$

Substituting,

$$P_n = 0.341 \text{ kips}$$

With new coefficients,

$$C = 4.49$$

$$C_R = 0.05$$

$$C_N = 0.42$$

$$C_h = 0.05$$

Substituting,

$$P_n = 0.393 \text{ kips}$$

APPENDIX-C TEST RESULTS AND COMPARISONS

Table C.1 Multi-web Deck Sections, EOF Loading, Unfastened Tests

Specimen Name	Study	No	t (in)	F _y (ksi)	θ (deg)	R/t	N/t	h/t	AISI (1996)			North Ame. (2002)		New Coefficients	
									P _t (kips)	P _n (kips)	P _t /P _n	P _n (kips)	P _t /P _n	P _n (kips)	P _t /P _n
U-B-22-1		1	0.0295	45.8	70.0	6.9	50.8	42.7	0.344	0.22	1.53	0.34	1.01	0.39	0.88
U-B-22-2		2	0.0295	45.8	70.0	6.9	50.8	42.7	0.341	0.22	1.52	0.34	1.00	0.39	0.87
U-B-22-3		3	0.0295	45.8	70.0	6.9	50.8	42.7	0.346	0.22	1.54	0.34	1.01	0.39	0.88
U-HD-26-1		4	0.0182	95.4	58.0	14.6	82.4	42.8	0.181	0.13	1.41	0.26	0.69	0.31	0.58
U-HD-26-2		5	0.0182	95.4	58.0	14.6	82.4	42.8	0.188	0.13	1.47	0.26	0.71	0.31	0.60
U-HD-26-3		6	0.0182	95.4	58.0	14.6	82.4	42.8	0.183	0.13	1.43	0.26	0.70	0.31	0.58
U-EHD-26-1		7	0.0183	103.9	50.0	17.1	82.0	75.9	0.161	0.11	1.41	0.20	0.79	0.26	0.63
U-EHD-26-2		8	0.0183	103.9	50.0	17.1	82.0	75.9	0.158	0.11	1.39	0.20	0.77	0.26	0.61
U-EHD-26-3		9	0.0183	103.9	50.0	17.1	82.0	75.9	0.168	0.11	1.47	0.20	0.82	0.26	0.65
U-V-22-1		10	0.0300	48.0	75.5	6.8	50.0	56.6	0.386	0.24	1.62	0.35	1.11	0.40	0.95
U-V-22-2		11	0.0300	48.0	75.5	6.8	50.0	56.6	0.392	0.24	1.64	0.35	1.13	0.40	0.97
U-V-22-3		12	0.0300	48.0	75.5	6.8	50.0	56.6	0.393	0.24	1.65	0.35	1.13	0.40	0.97
U-S-28-1		13	0.0153	105.2	58.0	11.2	98.0	29.2	0.203	0.10	2.07	0.26	0.79	0.29	0.69
U-S-28-2		14	0.0153	105.2	58.0	11.2	98.0	29.2	0.203	0.10	2.07	0.26	0.79	0.29	0.69
U-S-28-3		15	0.0153	105.2	58.0	11.2	98.0	29.2	0.200	0.10	2.04	0.26	0.78	0.29	0.68
U-2VLI-16-1	Avei, 2001 (VT)	16	0.0598	46.5	63.0	3.1	25.1	31.8	1.374	1.03	1.33	1.19	1.15	1.35	1.02
U-2VLI-16-2		17	0.0598	46.5	63.0	3.1	25.1	31.8	1.322	1.03	1.28	1.19	1.11	1.35	0.98
U-2VLI-16-3		18	0.0598	46.5	63.0	3.1	25.1	31.8	1.390	1.03	1.35	1.19	1.17	1.35	1.03
U-2VLI-18-1		19	0.0474	49.5	63.0	4.0	31.6	40.6	1.000	0.57	1.75	0.81	1.24	0.92	1.09
U-2VLI-18-2		20	0.0474	49.5	63.0	4.0	31.6	40.6	0.956	0.57	1.67	0.81	1.19	0.92	1.04
U-2VLI-18-3		21	0.0474	49.5	63.0	4.0	31.6	40.6	1.011	0.57	1.77	0.81	1.26	0.92	1.10
U-2VLI-20-1		22	0.0358	52.0	63.0	5.2	41.9	54.3	0.629	0.32	1.99	0.48	1.31	0.55	1.13
U-2VLI-20-2		23	0.0358	52.0	63.0	5.2	41.9	54.3	0.611	0.32	1.93	0.48	1.27	0.55	1.10
U-2VLI-20-3		24	0.0358	52.0	63.0	5.2	41.9	54.3	0.584	0.32	1.85	0.48	1.22	0.55	1.05
U-2VLI-22-1		25	0.0295	54.0	63.0	6.4	50.8	66.2	0.417	0.23	1.83	0.33	1.25	0.39	1.07
U-2VLI-22-2		26	0.0295	54.0	63.0	6.4	50.8	66.2	0.456	0.23	2.00	0.33	1.37	0.39	1.17
U-2VLI-22-3		27	0.0295	54.0	63.0	6.4	50.8	66.2	0.444	0.23	1.95	0.33	1.33	0.39	1.14
U-3VLI-16-1		28	0.0598	35.0	67.0	3.1	25.1	48.4	1.100	0.83	1.32	0.83	1.33	0.95	1.15
U-3VLI-16-2		29	0.0598	35.0	67.0	3.1	25.1	48.4	1.121	0.83	1.35	0.83	1.36	0.95	1.18
U-3VLI-16-3		30	0.0598	35.0	67.0	3.1	25.1	48.4	1.025	0.83	1.23	0.83	1.24	0.95	1.08
U-3VLI-18-1		31	0.0474	48.0	67.0	4.0	31.6	61.5	0.983	0.55	1.79	0.70	1.39	0.82	1.20
U-3VLI-18-2		32	0.0474	48.0	67.0	4.0	31.6	61.5	0.957	0.55	1.74	0.70	1.36	0.82	1.17
U-3VLI-18-3		33	0.0474	48.0	67.0	4.0	31.6	61.5	0.967	0.55	1.76	0.70	1.37	0.82	1.18
U-3VLI-20-1		34	0.0358	53.5	67.0	5.2	41.9	82.1	0.650	0.31	2.09	0.43	1.51	0.51	1.27
U-3VLI-20-2		35	0.0358	53.5	67.0	5.2	41.9	82.1	0.630	0.31	2.03	0.43	1.46	0.51	1.24
U-3VLI-20-3		36	0.0358	53.5	67.0	5.2	41.9	82.1	0.634	0.31	2.04	0.43	1.47	0.51	1.24
U-3VLI-22-1		37	0.0295	52.5	67.0	6.4	50.8	100.0	0.390	0.21	1.83	0.27	1.43	0.33	1.18
U-3VLI-22-2		38	0.0295	52.5	67.0	6.4	50.8	100.0	0.364	0.21	1.71	0.27	1.34	0.33	1.11
U-3VLI-22-3		39	0.0295	52.5	67.0	6.4	50.8	100.0	0.378	0.21	1.77	0.27	1.39	0.33	1.15

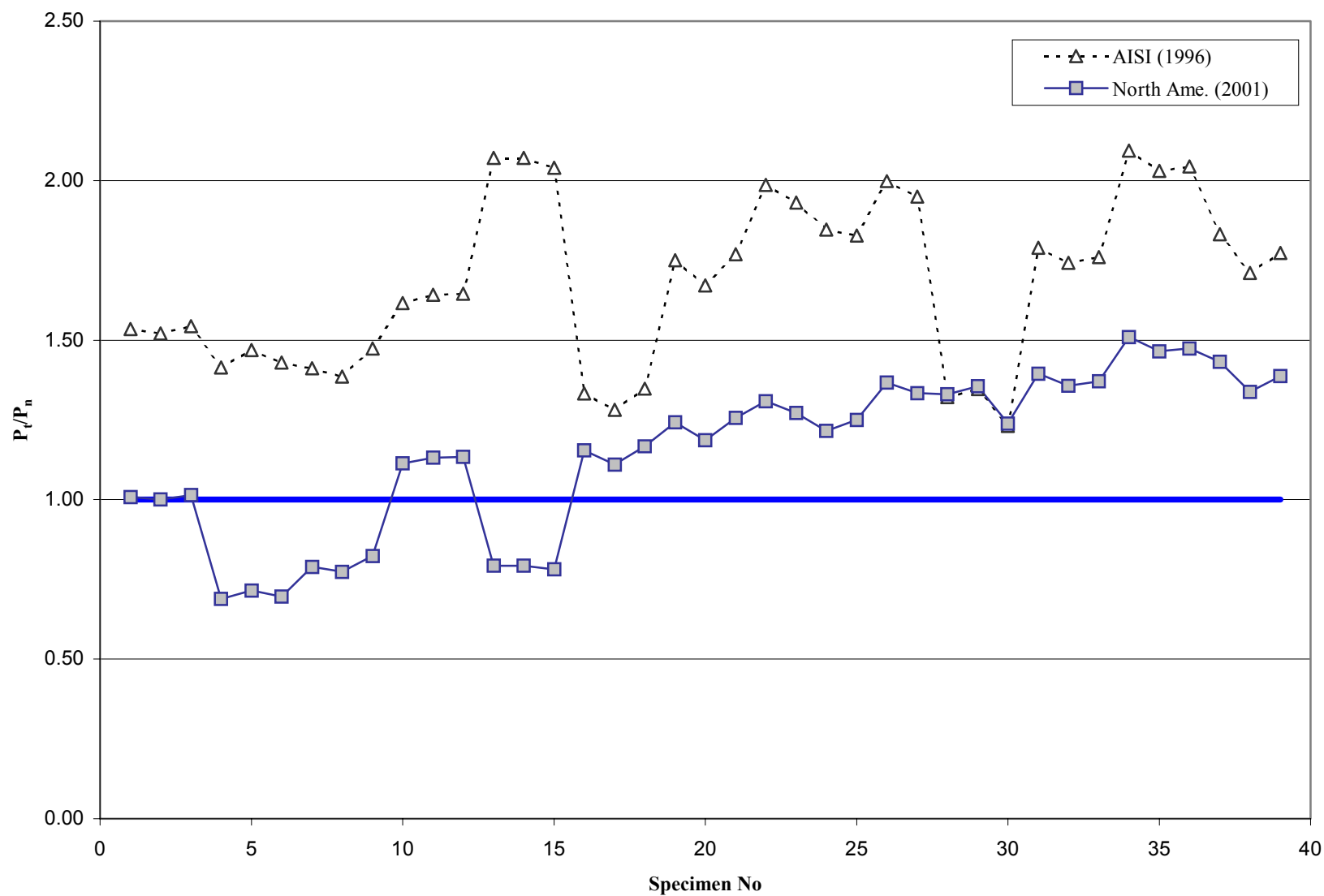


Figure C.1 P_t/P_n for Multi-web Deck Sections, EOF Loading, Unfastened Tests

Table C.2 Multi-web Deck Sections, EOF Loading, Fastened Tests

Specimen Name	Study	No	t (in)	F _y (ksi)	θ (deg)	R/t	N/t	h/t	AISI (1996)			North Ame. (2002)		New Coefficients	
									P _t (kips)	P _a (kips)	P _t /P _a	P _a (kips)	P _t /P _a	P _a (kips)	P _t /P _a
R-B-22-1		1	0.0295	45.8	70.0	6.9	50.8	42.7	0.373	0.22	1.66	0.34	1.09	0.49	0.76
R-B-22-2		2	0.0295	45.8	70.0	6.9	50.8	42.7	0.380	0.22	1.70	0.34	1.11	0.49	0.77
R-B-22-3		3	0.0295	45.8	70.0	6.9	50.8	42.7	0.371	0.22	1.66	0.34	1.09	0.49	0.76
R-HD-26-1		4	0.0182	95.4	58.0	14.6	82.4	42.8	0.203	0.13	1.59	0.26	0.77	0.21	0.95
R-HD-26-2		5	0.0182	95.4	58.0	14.6	82.4	42.8	0.208	0.13	1.63	0.26	0.79	0.21	0.98
R-HD-26-3		6	0.0182	95.4	58.0	14.6	82.4	42.8	0.202	0.13	1.58	0.26	0.77	0.21	0.95
R-EHD-26-1		7	0.0183	103.9	50.0	17.1	82.0	75.9	0.178	0.11	1.56	0.20	0.87	0.13	1.37
R-EHD-26-2		8	0.0183	103.9	50.0	17.1	82.0	75.9	0.183	0.11	1.61	0.20	0.90	0.13	1.41
R-EHD-26-3		9	0.0183	103.9	50.0	17.1	82.0	75.9	0.173	0.11	1.52	0.20	0.85	0.13	1.34
R-V-22-1		10	0.0300	48.0	75.5	6.8	50.0	56.6	0.425	0.24	1.78	0.35	1.23	0.51	0.84
R-V-22-2		11	0.0300	48.0	75.5	6.8	50.0	56.6	0.422	0.24	1.77	0.35	1.22	0.51	0.83
R-V-22-3		12	0.0300	48.0	75.5	6.8	50.0	56.6	0.431	0.24	1.81	0.35	1.24	0.51	0.85
R-S-28-1		13	0.0153	105.2	58.0	11.2	98.0	29.2	0.220	0.10	2.24	0.26	0.86	0.27	0.80
R-S-28-2		14	0.0153	105.2	58.0	11.2	98.0	29.2	0.223	0.10	2.28	0.26	0.87	0.27	0.81
R-S-28-3		15	0.0153	105.2	58.0	11.2	98.0	29.2	0.229	0.10	2.34	0.26	0.89	0.27	0.83
R-2VLI-16-1	Avei, 2001 (VT)	16	0.0598	46.5	63.0	3.1	25.1	31.8	1.590	1.03	1.54	1.19	1.34	2.08	0.76
R-2VLI-16-2		17	0.0598	46.5	63.0	3.1	25.1	31.8	1.622	1.03	1.57	1.19	1.36	2.08	0.78
R-2VLI-16-3		18	0.0598	46.5	63.0	3.1	25.1	31.8	1.580	1.03	1.53	1.19	1.33	2.08	0.76
R-2VLI-18-1		19	0.0474	49.5	63.0	4.0	31.6	40.6	1.179	0.57	2.06	0.81	1.46	1.36	0.87
R-2VLI-18-2		20	0.0474	49.5	63.0	4.0	31.6	40.6	1.233	0.57	2.16	0.81	1.53	1.36	0.91
R-2VLI-18-3		21	0.0474	49.5	63.0	4.0	31.6	40.6	1.244	0.57	2.18	0.81	1.55	1.36	0.92
R-2VLI-20-1		22	0.0358	52.0	63.0	5.2	41.9	54.3	0.778	0.32	2.46	0.48	1.62	0.77	1.02
R-2VLI-20-2		23	0.0358	52.0	63.0	5.2	41.9	54.3	0.745	0.32	2.35	0.48	1.55	0.77	0.97
R-2VLI-20-3		24	0.0358	52.0	63.0	5.2	41.9	54.3	0.753	0.32	2.38	0.48	1.57	0.77	0.98
R-2VLI-22-1		25	0.0295	54.0	63.0	6.4	50.8	66.2	0.585	0.23	2.57	0.33	1.76	0.50	1.16
R-2VLI-22-2		26	0.0295	54.0	63.0	6.4	50.8	66.2	0.574	0.23	2.52	0.33	1.72	0.50	1.14
R-2VLI-22-3		27	0.0295	54.0	63.0	6.4	50.8	66.2	0.565	0.23	2.48	0.33	1.70	0.50	1.12
R-3VLI-16-1		28	0.0598	35.0	67.0	3.1	25.1	48.4	1.457	0.83	1.75	0.83	1.76	1.47	0.99
R-3VLI-16-2		29	0.0598	35.0	67.0	3.1	25.1	48.4	1.467	0.83	1.76	0.83	1.77	1.47	1.00
R-3VLI-16-3		30	0.0598	35.0	67.0	3.1	25.1	48.4	1.485	0.83	1.78	0.83	1.80	1.47	1.01
R-3VLI-18-1		31	0.0474	48.0	67.0	4.0	31.6	61.5	1.311	0.55	2.39	0.70	1.86	1.21	1.08
R-3VLI-18-2		32	0.0474	48.0	67.0	4.0	31.6	61.5	1.337	0.55	2.43	0.70	1.90	1.21	1.10
R-3VLI-18-3		33	0.0474	48.0	67.0	4.0	31.6	61.5	1.333	0.55	2.43	0.70	1.89	1.21	1.10
R-3VLI-20-1		34	0.0358	53.5	67.0	5.2	41.9	82.1	0.878	0.31	2.83	0.43	2.04	0.70	1.25
R-3VLI-20-2		35	0.0358	53.5	67.0	5.2	41.9	82.1	0.854	0.31	2.75	0.43	1.98	0.70	1.21
R-3VLI-20-3		36	0.0358	53.5	67.0	5.2	41.9	82.1	0.860	0.31	2.77	0.43	2.00	0.70	1.22
R-3VLI-22-1		37	0.0295	52.5	67.0	6.4	50.8	100.0	0.490	0.21	2.30	0.27	1.80	0.43	1.15
R-3VLI-22-2		38	0.0295	52.5	67.0	6.4	50.8	100.0	0.484	0.21	2.27	0.27	1.78	0.43	1.14
R-3VLI-22-3		39	0.0295	52.5	67.0	6.4	50.8	100.0	0.468	0.21	2.20	0.27	1.72	0.43	1.10

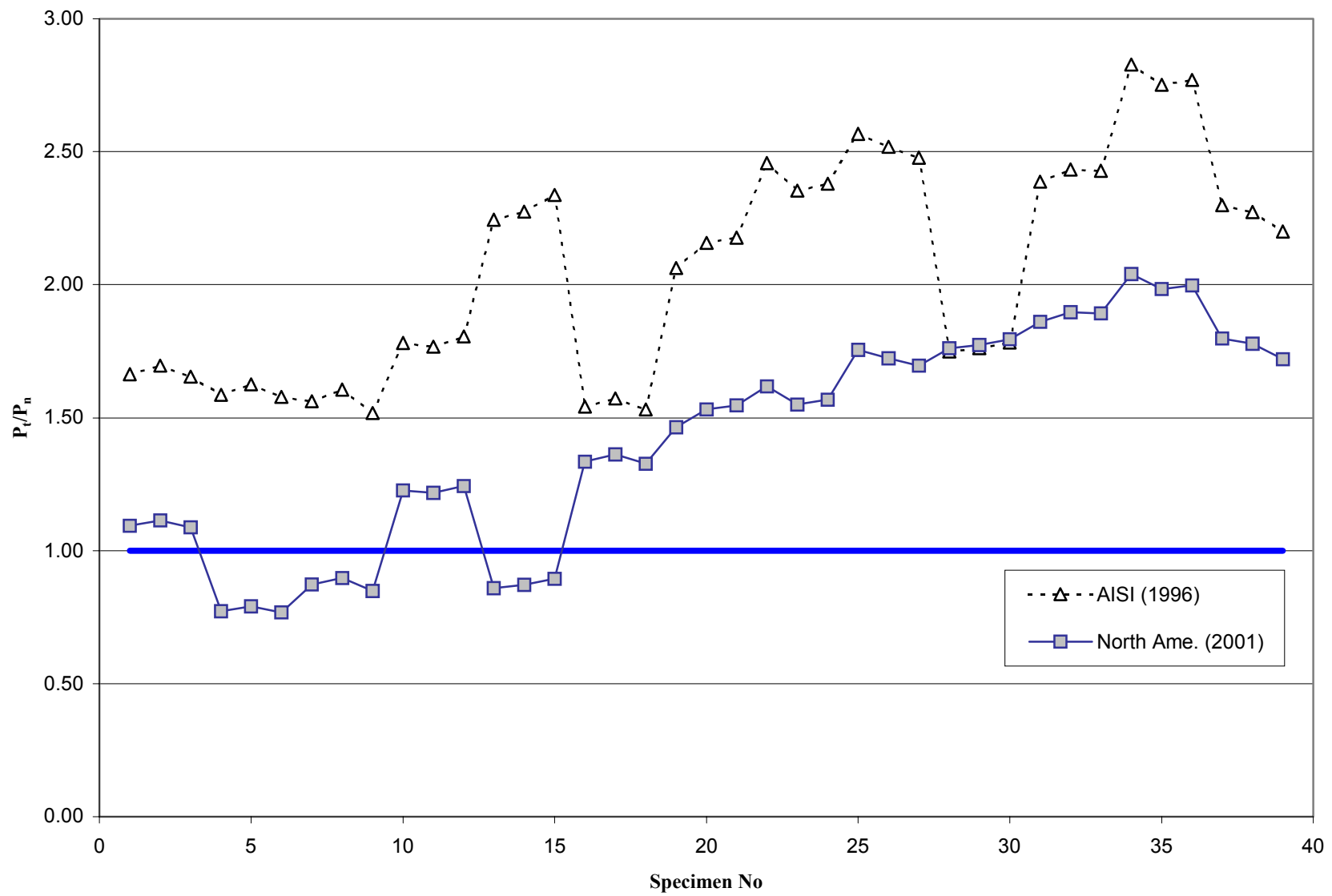


Figure C.2 P_t/P_n for Multi-web Deck Sections, EOF Loading, Fastened Tests

Table C.3 Experimental Studies on Multi-web Deck Sections, EOF Loading, Unfastened Tests

Specimen Name	Study	No	t (in)	F _y (ksi)	θ (deg)	R/t	N/t	h/t	P _t (kips)	North Ame. (2002)		New Coefficients	
										P _n (kips)	P _t /P _n	P _n (kips)	P _t /P _n
EOF-1A	Yu, 1981 (UMR)	40	0.0292	43.3	62.4	6.8	102.1	62.7	0.476	0.35	1.35	0.40	1.18
EOF-1B		41	0.0293	43.3	61.6	6.8	101.7	62.1	0.481	0.35	1.36	0.40	1.19
EOF-2A		42	0.0301	43.3	62.1	7.0	197.0	59.5	0.588	0.51	1.15	0.57	1.03
EOF-2B		43	0.0296	43.3	62.7	7.1	200.3	61.1	0.578	0.49	1.17	0.55	1.04
EOF-3A		44	0.0442	42.9	63.7	4.5	67.4	40.3	1.188	0.82	1.45	0.91	1.30
EOF-3B		45	0.0447	42.9	63.0	4.5	66.7	39.8	1.201	0.83	1.44	0.93	1.29
EOF-4A		46	0.0472	42.9	64.4	4.4	125.6	38.1	1.244	1.26	0.99	1.36	0.91
EOF-4B		47	0.0471	42.9	64.5	4.5	125.9	38.0	1.224	1.25	0.98	1.36	0.90
EOF-5A		48	0.0311	48.1	69.5	6.4	95.8	88.7	0.398	0.39	1.01	0.46	0.86
EOF-5B		49	0.0317	48.1	70.0	6.3	94.0	87.4	0.408	0.41	0.99	0.48	0.85
EOF-6A		50	0.0293	48.1	70.5	6.8	202.4	92.2	0.603	0.48	1.26	0.55	1.10
EOF-6B		51	0.0294	48.1	70.0	6.8	201.7	93.5	0.606	0.48	1.28	0.55	1.11
EOF-7A		52	0.0488	41.2	71.3	3.9	61.1	55.7	1.002	0.90	1.12	1.01	0.99
EOF-7B		53	0.0479	41.2	72.2	4.0	62.2	57.2	1.003	0.86	1.16	0.97	1.03
EOF-8A		54	0.0460	41.2	71.3	4.6	128.9	58.0	1.433	1.07	1.34	1.18	1.21
EOF-8B		55	0.0480	41.2	71.3	4.4	123.5	54.8	1.408	1.17	1.20	1.29	1.09
EOF-19A		56	0.0288	41.2	75.9	4.9	103.5	57.6	0.329	0.39	0.85	0.43	0.76
EOF-19B		57	0.0287	41.2	75.1	4.9	103.8	56.4	0.303	0.39	0.78	0.43	0.70
FD1	Bhakta, 1992	58	0.0260	57.5	71.0	6.6	101.0	102.7	0.340	0.31	1.09	0.37	0.92
FD2		59	0.0260	57.5	71.0	6.6	101.0	102.9	0.333	0.31	1.07	0.37	0.90
t26h0.75R3/32*60	Wu, 1997 (UMR)	60	0.0170	112.5	61.0	5.5	58.8	45.3	0.164	0.28	0.59	0.32	0.52
t26h0.75R3/64*60		61	0.0170	112.5	61.0	2.8	58.8	45.3	0.170	0.30	0.57	0.33	0.52
t26h1.5R3/32*60		62	0.0170	112.5	61.0	5.5	58.8	90.0	0.110	0.21	0.52	0.25	0.44
t26h1.5R3/64*60		63	0.0170	112.5	60.1	2.8	58.8	88.8	0.124	0.22	0.55	0.26	0.48
t22h0.75R5/64*60		64	0.0290	103.9	60.4	2.7	34.5	27.9	0.468	0.72	0.65	0.80	0.59
t22h0.75R1/16*60		65	0.0290	103.9	60.6	2.2	34.5	25.9	0.486	0.74	0.66	0.82	0.59
t22h1.5R5/64*60		66	0.0290	103.9	59.8	2.7	34.5	53.4	0.412	0.60	0.69	0.68	0.60
t22h1.5R1/16*60		67	0.0290	103.9	60.0	2.2	34.5	52.1	0.464	0.62	0.75	0.70	0.67
t22h2R5/64*60		68	0.0290	103.9	61.0	2.7	34.5	70.7	0.314	0.55	0.57	0.63	0.50
t22h2R1/16*60		69	0.0290	103.9	59.9	2.2	34.5	69.0	0.325	0.56	0.59	0.64	0.51
t22h3R5/64*60		70	0.0290	103.9	60.4	2.7	34.5	105.9	0.432	0.44	0.98	0.53	0.82
t22h3R1/16*60		71	0.0290	103.9	60.5	2.2	34.5	103.4	0.464	0.45	1.02	0.54	0.86
t22h4.5R5/64*60		72	0.0290	103.9	61.6	2.7	34.5	156.9	0.337	0.32	1.06	0.41	0.82
t22h4.5R1/16*60		73	0.0290	103.9	61.0	2.2	34.5	155.5	0.368	0.32	1.13	0.41	0.89
t22h6R5/64*60		74	0.0290	103.9	62.8	2.7	34.5	208.3	0.277	0.21	1.30	0.31	0.90
t22h6R1/16*60		75	0.0290	103.9	61.0	2.2	34.5	206.9	0.299	0.22	1.38	0.31	0.97

Table C.4 Experimental Studies on Multi-web Deck Sections, EOF Loading, Fastened Tests

Specimen Name	Study	No	t (in)	F _y (ksi)	θ (deg)	R/t	N/t	h/t	P _t (kips)	North Ame. (2002)		New Coefficients	
										P _n (kips)	P _t /P _n	P _n (kips)	P _t /P _n
FD3-F	Bhakta, 1992	40	0.0260	57.5	71.0	6.6	101.0	102.9	0.402	0.31	1.29	0.49	0.82
FD4-F	(UMR)	41	0.0260	57.5	71.0	6.6	101.0	102.8	0.415	0.31	1.33	0.49	0.85

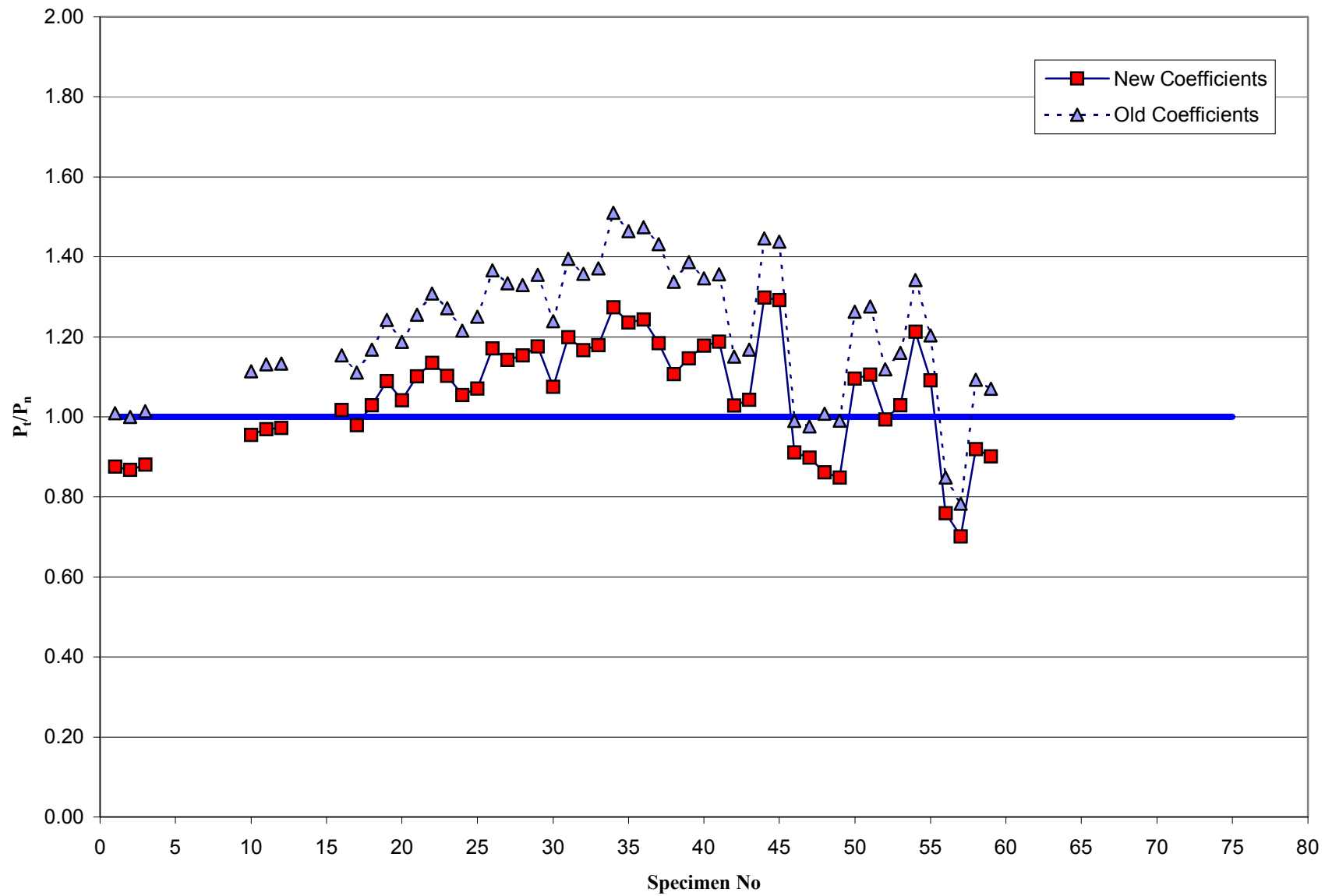


Figure C.3 P_t/P_n for Multi-web Deck Sections, EOF Loading, Unfastened Tests- Normal Strength Steel

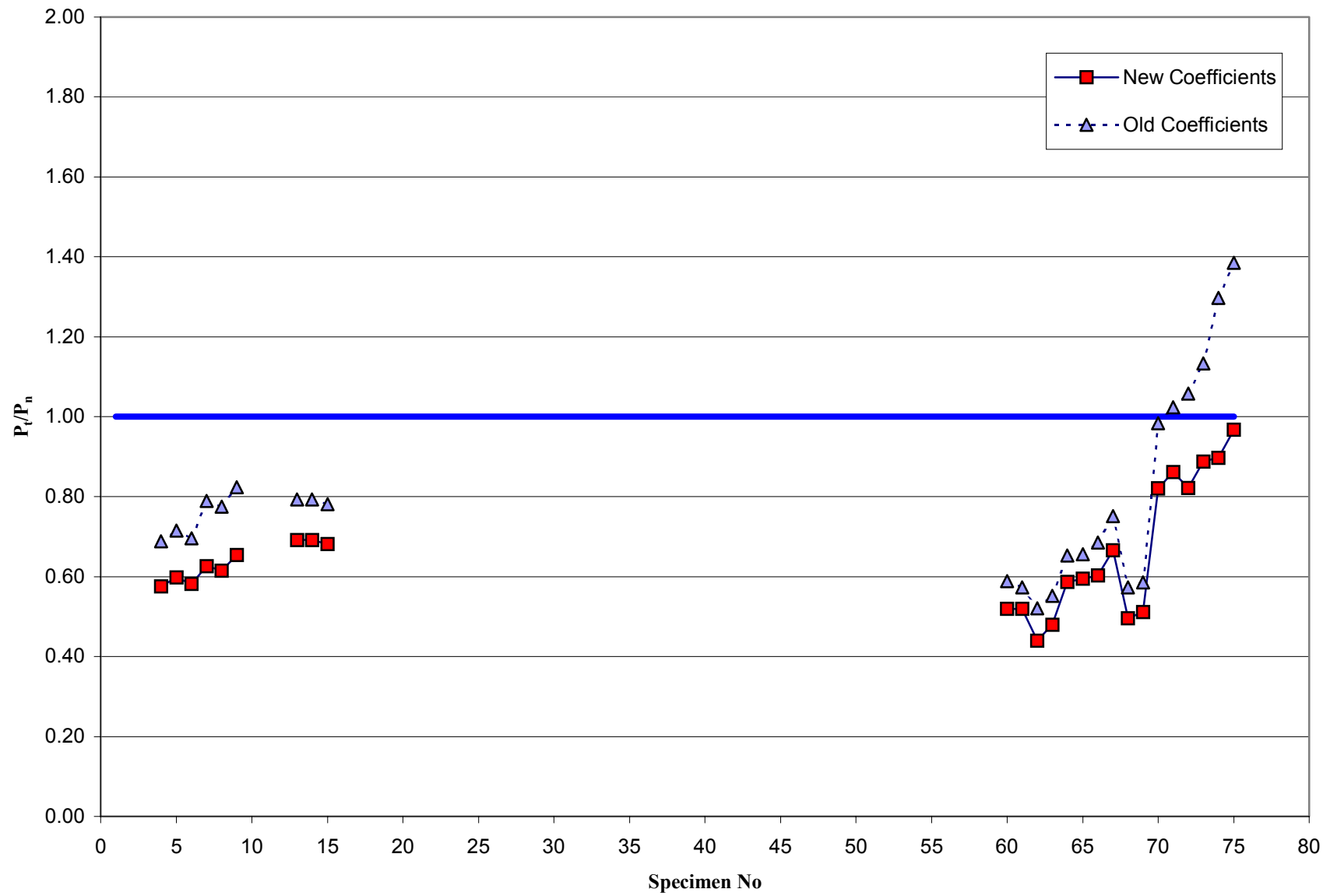


Figure C.4 P_t/P_n for Multi-web Deck Sections, EOF Loading, Unfastened Tests- High Strength Steel

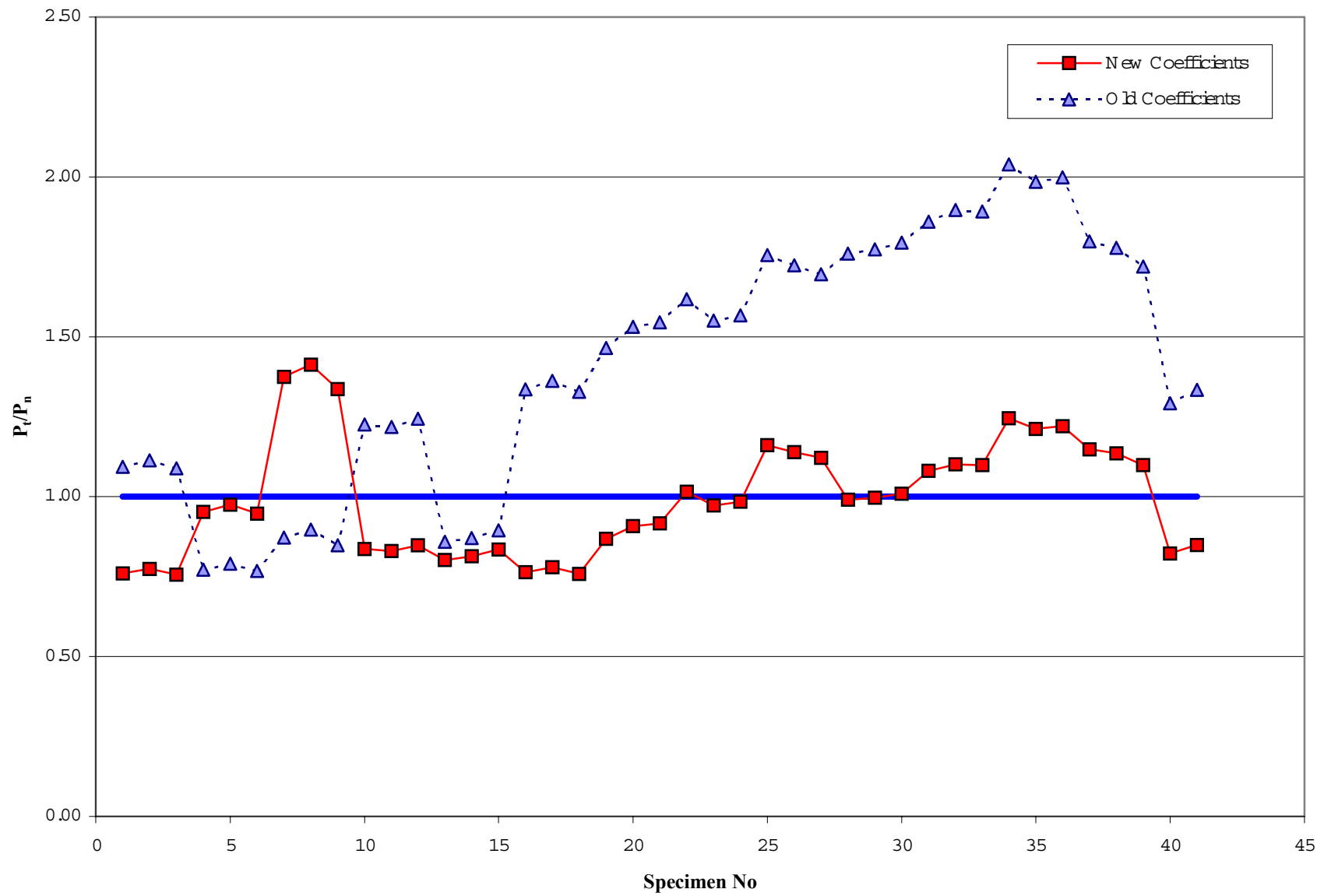


Figure C.5 P_t/P_n for Multi-web Deck Sections, EOF Loading, Fastened Tests

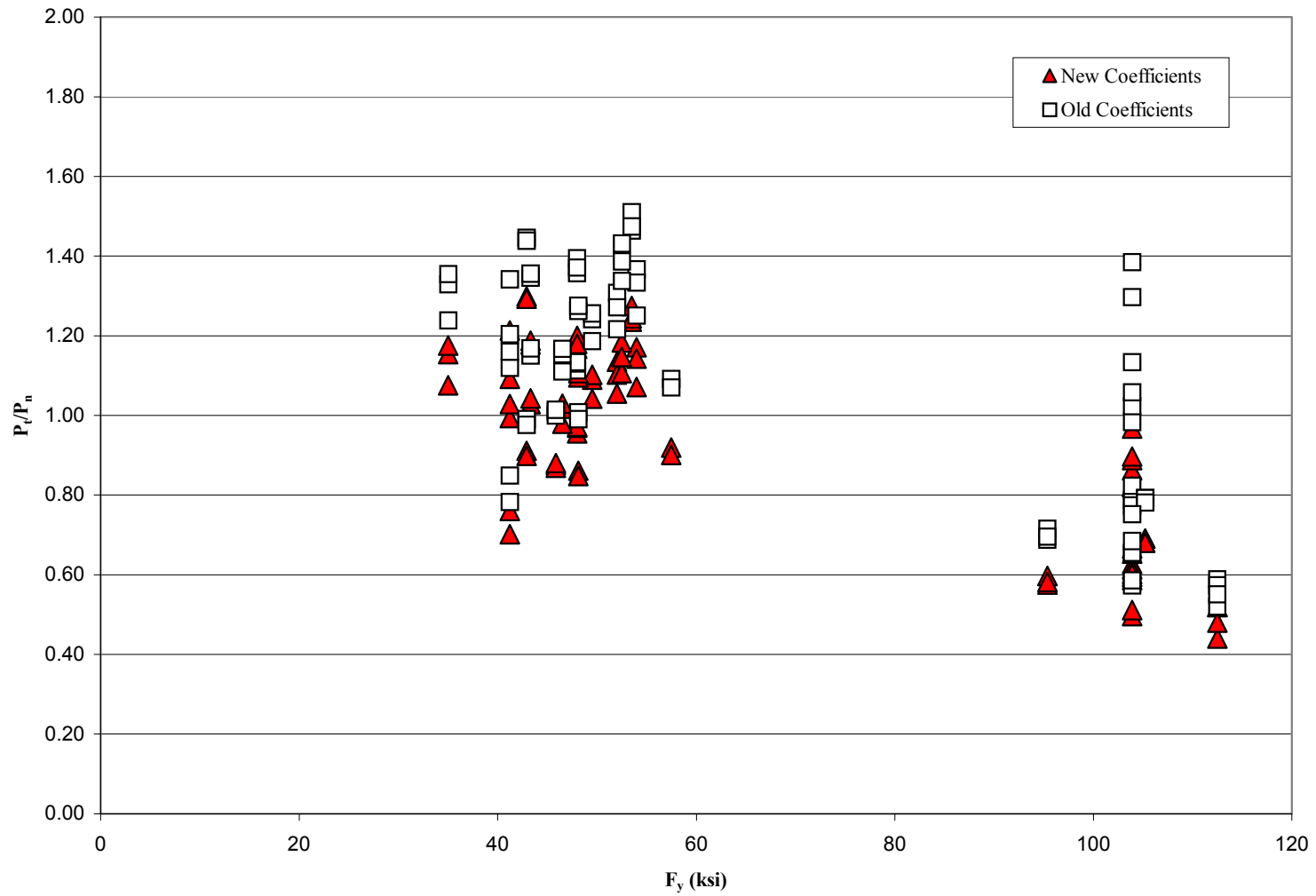


Figure C.6 Test Loads to the Predicted Loads Ratio (P_t/P_n) with Respect to Yield Strength Values

VITA

Onur Avci was born in Ankara, Turkey on September 11, 1979. He was raised in Ankara and graduated from Yukselis High School in 1996. He attended Middle East Technical University from 1996 to 2000 where he received a Bachelor of Science degree in Civil Engineering. In the fall of 2000, he enrolled at the Virginia Polytechnic Institute and State University to pursue a Master of Science degree in the Structural Engineering and Materials Program Area of the Via Department of Civil and Environmental Engineering.