1-1-2008

Experimental study on web crippling of lapped cold-formed steel channels subjected to interior two-flange loading

Quzzafi Rahman
Ryerson University

EXPERIMENTAL STUDY ON WEB CRIPPLING OF LAPPED COLD-FORMED STEEL CHANNELS SUBJECTED TO INTERIOR TWO-FLANGE LOADING

by

Quzzafi Rahman
(B.sc, Pakistan, Jan 2001)

A thesis
presented to Ryerson University

in partial fulfilment of the
requirement for the degree of
Master of Applied Science
In the program of
Civil Engineering.

Toronto, Ontario, Canada, 2008

AUTHOR'S DECLARATION

I hereby declare that I am the sole author of this thesis.

I authorize Ryerson University to lend this thesis to other institutions or individual for the purpose of scholarly research.

I further authorize Ryerson University to reproduce this thesis by photocopying or by other means, in total or in part, at the request of other institution or individuals for the purpose of scholarly research.

BORROWERS

Ryerson University requires the signature of all persons using or photocopying this thesis.

Please Sign below, and give address and date.

Name	Signature	Address	Date

EXPERIMENTAL STUDY ON WEB CRIPPLING OF LAPPED COLD-FORMED STEEL CHANNELS SUBJECTED TO INTERIOR TWO-FLANGE LOADING

By
Quzzafi Rahman
Master of Applied Sciences in Civil Engineering
Department of Civil Engineering
Ryerson University, Toronto
2008

ABSTRACT

The current North American Specifications for the Design of Cold-Formed Steel (CFS) Structural Members, AISI-S136-01, specifies expressions for web crippling strength of different joist geometries in case of exterior end and concentrated load locations. However, it does not permit an increase in web crippling capacity when lapped cold-formed steel channels are subjected to interior two-flange loading. Thus, the objective of this research in this thesis is to generate experimental data for CFS channels where both webs of channel members are lapped at the interior support location and being loaded simultaneously. This thesis summarizes the results of a parametric study to examine few parameters that affect web crippling strength of such lapped channels. These parameters include the unbraced length of channel member, the presence of corrugated steel deck that is fastened to the top flange at equal intervals using self-drilling screws, the level of flange restraint at the interior support location, channel size and load bearing length (i.e. lap length). Test specimens were loaded to failure and load history and the failure pattern were recorded. Based on experimental findings, a reliable and economical design expression was developed for web crippling strength of lapped CFS channels at interior support location when subjected to two-flange loading.

ACKNOWLEDEMENTS

My sincere thanks and gratitude are due to His Almighty ALLAH, who helped me and blessed my work during the days of my study and research.

I would like to express my sincere appreciation and gratitude to my supervisor, Dr. Khaled Sennah, for his continuous advice, guidance and encouragement throughout this research.

Also, I would like to thank Dr. Steve Fox, General Manager of Canadian Sheet Steel Building Institute (CSSBI) for his valuable guidance, encouragement and support.

Moreover, I am grateful to the Canadian Sheet Steel Building Institute (CSSBI) for providing test specimens for this research.

Finally, I wish to express my sincere thanks to my parents, my wife, my brothers and sister for their love, support and encouragement given to me during the course of this study.

TABLE OF CONTENTS

NOTATIONS

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 Channel
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_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

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1
INTRODUCTION

1.1 General

Available structural steel members are divided into two types. One is of hot rolled sections and the other is of cold formed sections. Cold formed steel is less familiar compared to hot rolled steel. The use of cold formed steel members in building construction began in 1850s in both the United States of America and Great Britain. However, in North America such steel members were not widely used in buildings until the publication of the first edition of the American Iron and Steel Institute (AISI) Specification in 1946. The first research was carried out by Professor George Winter at Cornell University in 1939. The design standard was primarily based on his research work. His research work has been published in numerous journals. In Canada, the first cold formed design standard was issued in 1963 (CSA 1963). Continuous research in this field have led to a number of editions over the years, resulting in the development of the 2001 edition of the Design Standard for Cold Formed Steel Members "CAN/CSA-S136-01" [1]. The use of cold formed steel members in buildings has been growing since early 1950's.

In its early use in the industry, cold formed steel was used as a cladding and decking material for roofs, floor and walls in residential construction. Later on, it was introduced to commercial and industrial buildings as an alternative to traditional structural elements (i-e. timber and hot rolled steel joists). In recent years, timber members such as studs, joists, and rafters in residential buildings have been replaced by cold formed "C" and "Z" sections as an alternative for cost-effective construction. In large commercial buildings, cold formed steel members are used as secondary member, while hot rolled steel members are used as primary members.

Cold formed steel members have high strength-to-weight ratios compared to other building materials like concrete, masonry, and timber. They are considerably thinner in comparison with hot rolled steel members. Sections can be fabricated in large quantities by roll-forming from steel coils. The thickness of steel sheets or strip generally used in cold formed steel structural members ranges from 0.4 mm (0.0149 in) to about 6.4 mm (0.25 in). Steel plates and bars as thick as 25 mm (1 in) can be cold formed successfully into structural shapes. Fabrication of cold formed steel sections can be subdivided into three basic processes as follows:

i) cold roll forming;

ii) press brake operation; and

iii) bending brake operation.

1.2 Advantages of Cold Formed Steel on Other Materials

Compared with other structural materials such as timber and concrete, the following qualities can be realized for cold formed steel structural members [2]:

i) lightness

ii) high strength and stiffness

iii) ease of prefabrication and mass production

iv) fast and easy erection and installation

v) substantial elimination of delays due to weather

vi) more accurate detailing

vii) no shrinking and no creeping at ambient temperature

viii) formwork unneeded

ix) termite-proof and rot proof

x) uniform quality

xi) economy in transportation and handling

xii) no combustibility

xiii) recyclable material

1.3 Disadvantages of Cold Formed Steel

i. It has low fire resistance

ii. Maximum section thickness up to 6.4mm

iii. It can't use as a primary member for a building

1.4 Web Crippling

The individual components of cold formed steel members are usually so thin with respect to their widths, these thin elements may buckle at stress levels less than yield point if they are subjected to compression, shear, bending, or bearing. Local buckling of such elements is therefore one of the major design considerations. It is well known that elements will continue to carry load after the onset of local elastic buckling, reaching loads that can be up to seven times the value at which elastic local buckling first occurs [2]. This increase in capacity beyond elastic buckling arises from the redistribution of stresses within the compression element. The post-buckling capacity of thin plate elements is a characteristic which makes cold formed steel design unique, in that the strength beyond the initial elastic buckling stress is included in determining the capacity of a section. [3]. Web crippling is one of many failure modes that must be considered in the design of cold formed steel member. ***"It is a localized failure of structural members caused by a concentrated load or reaction applied on a short length of the member"***. To obtain safe and economic design, an accurate predication of the web crippling resistance is required. In the North American Specification for the design of Cold formed steel structural members (CAN/CSA-S136-01), a unified approach has adopted with one design expression of different coefficients, to determine the web crippling resistance of the different types of section geometries and load cases.

1.5 Factors Affecting Web Crippling Resistance

Web crippling resistance depends on many factors such as section geometry, section parameters, load case and bearing length, as presented in the following subsections.

1.5.1 Member Geometry

Web crippling is affected by the degree of restraint of the web against rotation; each section has different behavior characteristics. Following are most common shapes used in the market for building construction, as detailed in Fig. 1.1:

- I-Sections
- C-Sections
- Z-Sections
- Single Hat Sections
- Multi-Web Sections (Decks)

Web crippling basically occurs in the web of member and the web-flange interaction affects web crippling strength of such members. Web crippling resistance also depends on whether the section flanges are stiffened or unstiffened. Sections having stiffened flange have higher crippling resistance than unstiffened sections. In this study, only stiffened flange C-section was investigated since C-section is more efficient in practical application.

1.5.2 Member Sizes

There are six key parameters that influence the web crippling strength of cold-formed steel members. These key parameters are:

- web thickness (t);
- yield strength of the web (F_y);
- Web slenderness ratio (h/t);
- Inside bend radius to thickness ratio (R/t);
- Length of bearing to thickness ratio (N/t); and
- Web inclination (θ).

In addition to these key parameters, an additional parameter that must be considered in a test program is the fastening of the flanges. Earlier web crippling tests were performed with the test specimens not being attached to their supporting flange elements. These tests have shown that the presence of flange attachment can have a significant influence on the web crippling strength. In order to include all of these parameters in the test program, a number of different tests must be carried out to adequately cover the range of these parameters.

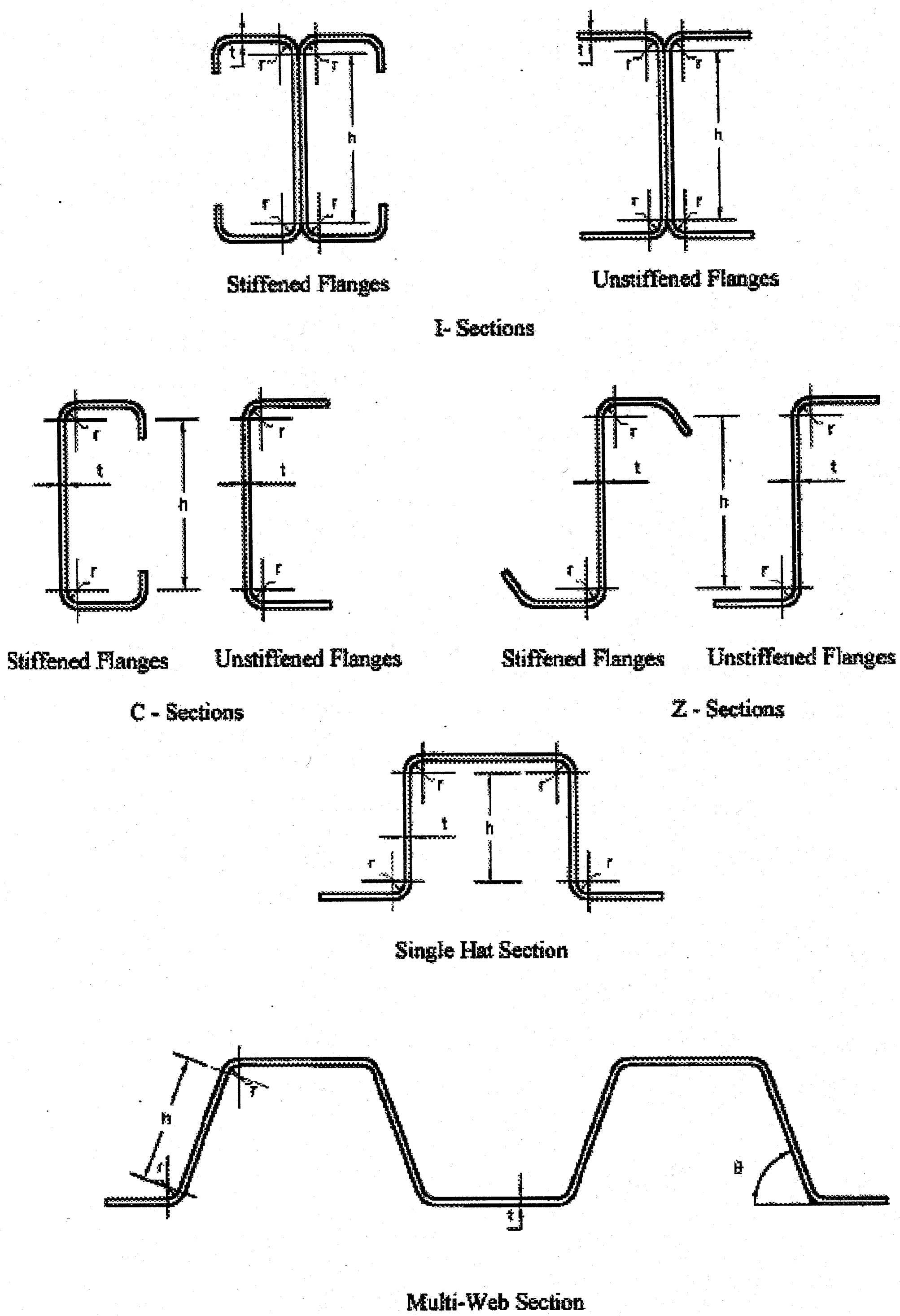


Fig. 1.1 Common Shapes of Cold Formed Steel Sections

1.5.3 Load Cases

As shown in Fig. 1.2, there are four different loading cases that can occur in CFS framing. The two main loading cases are ***One-Flange Loading and Two-Flange Loading***. In each loading case, ***END*** and ***INTERIOR*** concentrated load or reaction may be considered. Thus, a total of four different loading cases may occur for web crippling as follows.

For One-Flange Loading Case

1- End One-Flange Loading (EOF)

2- Interior One-Flange Loading (IOF)

For Two-Flange Loading Case

1- End Two-Flange Loading (ETF)

2- Interior Two-Flange Loading (ITF)

1.5.4 Bearing Length

Bearing length (n) has a greater influence on web crippling strength. The longer the bearing plate width, the greater web crippling resistance and vise versa.

Based on the literature review, it was observed that the current Canadian standard for cold formed steel structural members, CAN/CSA-S136-01, specifies imperial expression for web crippling strength of different CFS member geometries in case of exterior end and concentrated load locations. However, it does not permit an increase in web crippling capacity when lapped cold-formed steel channels, with lap splice are subjected to interior two-flange loading. This attributed to the lack of experiment data on web crippling strength at interior support locations. As such experimental testing to-collapse is required to generate database required to extend the applicability of the above-mentioned expression to CFS channels with lap splice at interior support location when subjected to two-flange loading. Figure 1.4 shows schematic diagram of floor system made with CFS channel lapped at interior support

1.6 Objectives of this Study

The main objectives of this study are:

(i) to generate experimental data for CFS channels where both webs of channel members are lapped at the interior support location and being loaded simultaneously;

(ii) Develop new coefficients for the CAN/CSA-S136-01 expression for web crippling strength applicable to such CFS channel configuration and loading condition.

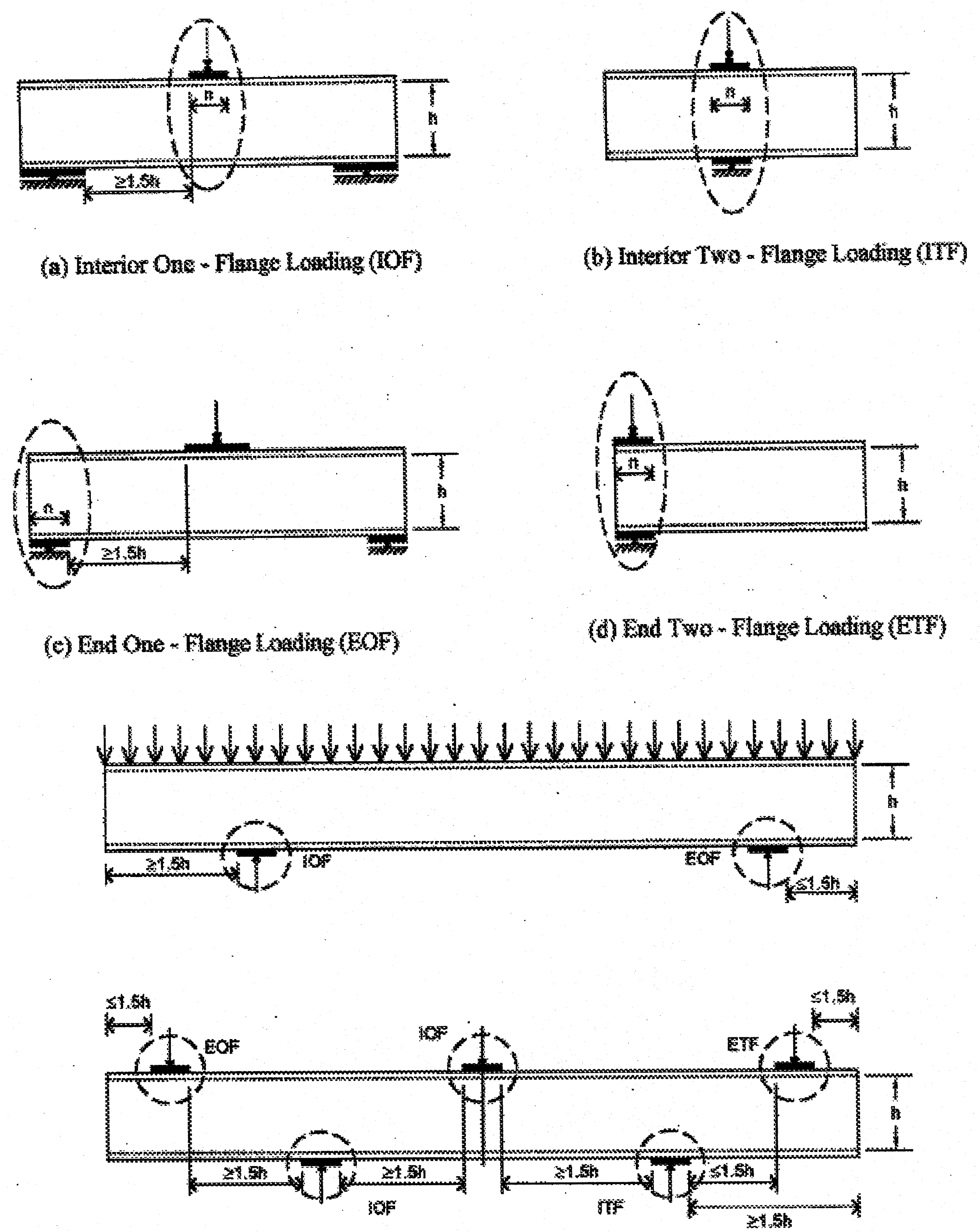


Fig. 1.2 Classification of Load Cases for Web Crippling [5]

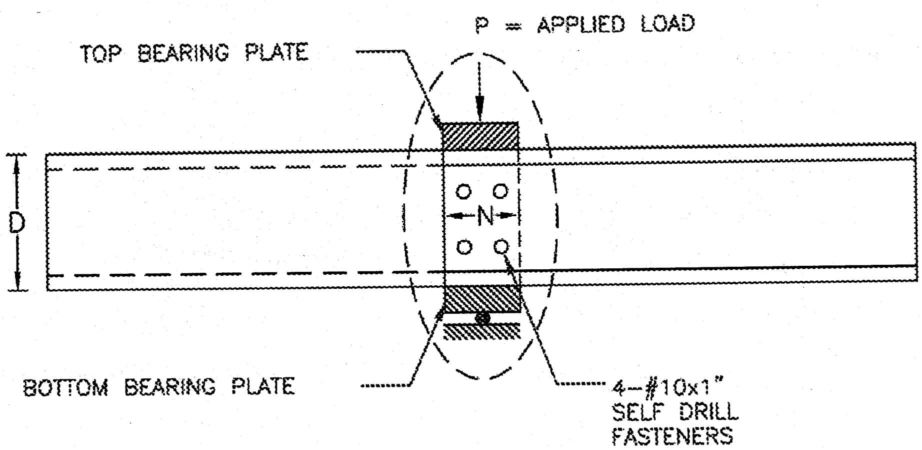


Fig. 1.3 Interior Two Flange Loading Condition Criteria

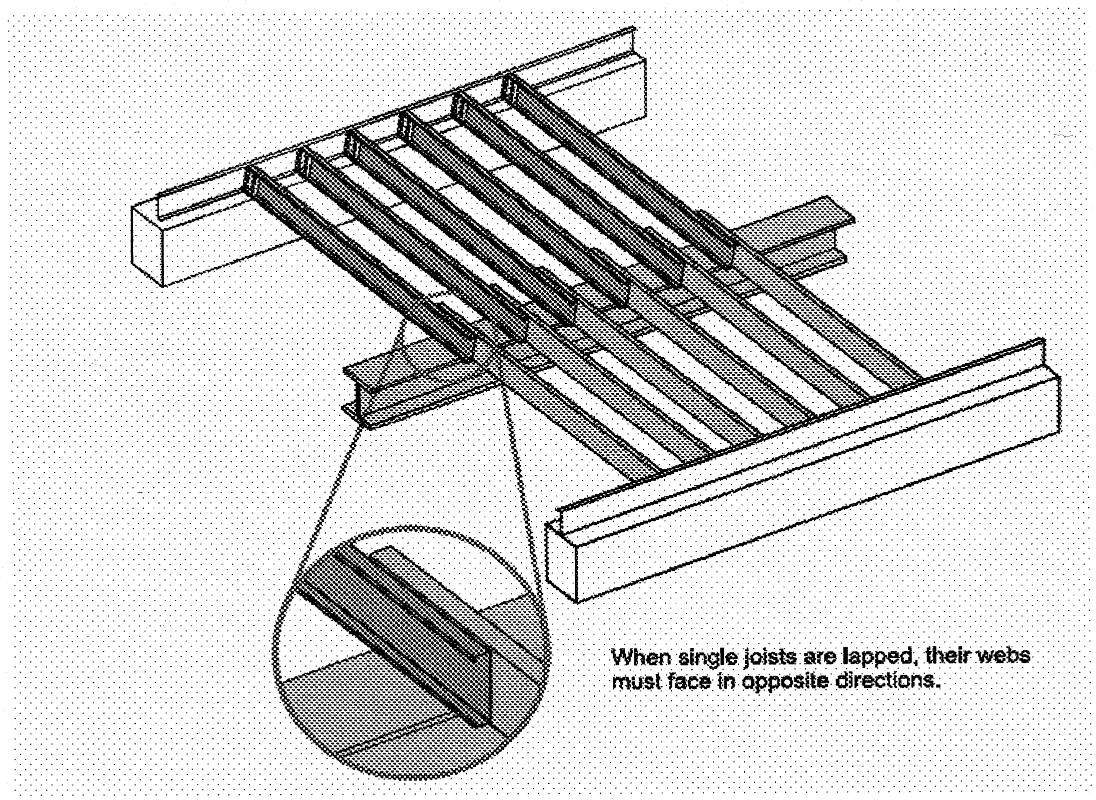


Fig. 1.4 Schematic diagram of floor system made with CFS channel lapped at interior support

1.7 Scope of Study

First of all, the available research reports and technical publications related to the web crippling loading were reviewed and organized. The current North American web crippling expressions and analytical studies of plate element subjected to in-plane edge loading were reviewed. A literature review is presented in Chapter 2. An experimental program was carried out on cold formed steel stiffened C-section subjected to ITF loading with lap splice as shown in Fig. 1.3. This was necessary in order to establish the behaviour and to quantify the web crippling resistance. Detailed descriptions of test specimens, test set up, and test procedures are presented in chapter 3. Chapter 4 describes the test results, while Chapter 5 summarizes the process for the development of new design coefficients to predict the ultimate web crippling resistance for the studies CFS member configuration and loading condition. A summary of the study, a long with concluding remarks and design recommendations, is presented in chapter 6.

CHAPTER 2
REVIEW OF LITERATURE

2.1 General

Various publication and research reports were carefully reviewed in the initial phase of this study. Although the main objective of this research was to develop an experimentally-based design expression for the web crippling resistance of single unreinforced web of channels with lap splice subjected to interior two flange loading, all available analytical solutions and numerical models for the web crippling problem have been reviewed. Both analytical solutions and numerical models are useful to understand the structural behaviour and to establish parameters that affect the web crippling resistance of cold formed steel member.

2.2 Experimental Studies in Web Crippling

Performing laboratory test on real CFS members is the most reliable approach to investigate the behaviour of any CFS member. The computation of the web crippling strength by means of theoretical analysis is quite complex, as it involves a large number of factors, such as the initial imperfection of web element, local yielding in the region of load application, instability of web element, and other factors [4]. The computation of web crippling strength obtained using empirical methods is rapid and safe within their range of applications. The literature review indicated that there is no research study exists that focused on the web crippling behaviour of lapped channel at interior support. However, there are few research papers available on the web crippling behaviour of cold-formed steel member under different loading conditions.

In 1940's, Winter and Pian [5] at Cornell University investigated the web crippling problem with cold formed steel flexural member. Four different load cases were considered in investigation, namely: (i) end one flange loading (EOF); (ii) interior one flange loading (IOF);

(iii) end two flange loading (ETF); and (iv) interior two flange loading (ITF). 136 tests were performed on I section, with flanges not fastened to the base plate to transmit the load from a standard testing machine to the web. Only the ultimate load was recorded at which the specimens were failed, from which expressions for computing the web crippling capacity of cold formed steel I sections were developed as follows:

a) For end one flange loading (EOF)

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

b) For interior one flange loading (IOF)

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

Where:

P_{ult} =ultimate web crippling load per web; F_y =yield strength of steel; h =clear distance between flanges measured in the plane of the web; N =bearing length; t =thickness of web. The parameters ranges for these tests were, 30 <h/t <175, 7 <N/t <77 and 207 <F_y <270 MPa.

In 1978, Hetrakul and Yu [6] investigated the web crippling strength of cold formed steel sections having single unreinforced webs. Empirical expressions were developed based on the results from 140 tests performed at the University of Missouri-Rolla and 96 tests performed at Cornell University. The following expressions were derived.

a) Interior one flange loading, IOF (for stiffened and unstiffened flanges)

$$P_{ult} = \frac{F_y t^2}{10^3} C_1 C_2 \left(16317 - 22.52\frac{h}{t}\right)\left(1 + 0.0069\frac{N}{t}\right) \tag{2.3}$$

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

b) End one flange loading, EOF

i. For unstiffened flanges

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 \left(6570 - 8.51\frac{h}{t}\right)\left(1 + 0.0099\frac{N}{t}\right) \tag{2.4}$$

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

ii. For stiffened flanges

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 \left(10018 - 18.24\frac{h}{t}\right)\left(1 + 0.0102\frac{N}{t}\right) \tag{2.5}$$

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

c) Interior two flange loading, ITF (for stiffened and unstiffened flanges)

$$P_{ult} = \frac{F_y t^2}{10^3} C_1 C_2 \left(23356 + 68.64\frac{h}{t}\right)\left(1 + 0.0013\frac{N}{t}\right) \tag{2.6}$$

d) End two flange loading, ETF (for stiffened and unstiffened flanges)

$$P_{ult} = \frac{F_y t^2}{10^3} C_3 C_4 \left(7411 - 17.28\frac{h}{t}\right)\left(1 + 0.0099\frac{N}{t}\right) \tag{2.7}$$

Where:

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

C_1 =(1.22 - 0.22k)

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

C_3 =(1.33 - 0.33k)

C_4 =(1.15 - 0.15k)

F_y =yield strength of steel

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

k $=F_y$ (ksi)/33

N =Bearing length

R =inside bend radius

t =web thickness

The parameter ranges for all tests were 45 $<h/t<$258, 11 $<N/t<$140, 1 $<R/t<$3 and 227 $<F_y<$ 372 MPa with a web inclination, θ, of 90° for all tests.

Bhakta, Laboube and Yu in 1992 [7] experimentally investigated the influence of the flange restraint on the web crippling capacity of beam web elements. 52 different tests were performed subjected to end one flange loading and interior one flange loading. Specimens considered in the tests were channels, I-Sections, Z-Sections, long span roof decks and floor decks. They concluded that Z-sections have 30% more strength with the flanges fastened to the support for end one flange loading conditions. While only 3% strength increase was noted for interior one flange loading. In 1993, Prabakaran [8] performed an extensive statistical analysis of the web crippling capacity of cold formed steel sections by using the existing experimental data. The objective of his study was to develop unified equation to calculate the web crippling capacity of cold formed steel section with different coefficients for the design of CFS I-sections, single web sections and multi web sections (deck), as follows:

$$P_n = Ct^2F_y \sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \tag{2.8}$$

Where:

P_n =Nominal web crippling strength

C =Coefficient

C_h =Web slenderness coefficient

C_N =Bearing length coefficient

C_R =Inside bend radius coefficient

F_y =Yield strength

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

N =Bearing length

R =Inside bend radius

t =Web thickness

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

The parameter limits for I-sections and single web shapes were $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 1$ and $R/t \leq 4$; for multi-web sections, $h/t \leq 200$, $N/t \leq 200$, $N/h \leq 2$ and $R/t \leq 10$. Equation 2.8 is currently specified in CAN/CSA S136-01.

In 1997, Gerges [9] examined the validity of the North American Design expression, Equation 2.8, to predict web crippling resistance of single web cold formed steel member subjected to end one flange loading (EOF) with inside bend radius to thickness ratio, R/t, up to 10. ($R/t \leq 10$). The main purpose of his investigation was to develop a new web crippling expression for end one flange loading that can be valid for sections having R/t values up to 10. Based on the results from 72 tests on C-section fastened to the supports, he developed new parameter coefficients in the Prabakaran's equation 2.8, applicable to the tested CFS configuration and loading conditions. The new parameter coefficients were: C =4.70; C_R = 0.0521; C_N =0.165 and C_h =0.0221.

In 1999, Schuster and Beshara [3] performed tests on CFS members with four different loading conditions to check the validity of the current design expression in both the Canadian standard and the American Specification for predicting web crippling capacity and then they developed other web crippling coefficients for Equation 2.8. They found that the current expression in both the Canadian Standard and the American Specification underestimate web crippling strength in some cases and overestimates it in others. They developed new web crippling coefficients valid for all tested sections and all load cases. Table 2.1 shows all webs crippling coefficient developed by "Schuster and Beshara".

Table 2.1 Recommended Design Coefficients for Web Crippling Strength of Single Web Sections [3]

Support and Flange Conditions		Load Cases		Section Type	C	C_R	C_N	C_H	S136 [8]		AISI [1]		Parameter Limits
									Ω	ϕ	Ω	ϕ	
FASTENED TO SUPPORT	**Stiffened or Partially Stiffened Flanges**	One - Flange Loading or Reaction	End	C & Z	4	0.14	0.35	0.02	1.91	0.75	1.75	0.88	$H \leq 222$, $R \leq 9.0$, $N \leq 78.0$
			Interior	C	13	0.23	0.14	0.01	1.77	0.80	1.65	0.92	$H \leq 249$, $R \leq 5.0$, $N \leq 121$
		Two - Flange Loading or Reaction	End	C	7.5	0.08	0.12	0.048	1.88	0.77	1.72	0.89	$H \leq 195$, $R \leq 12.0$, $N \leq 70.0$
				Z	9	0.05	0.16	0.052	1.95	0.74	1.78	0.86	$H \leq 195$, $R \leq 12.0$, $N \leq 70.0$
			Interior	C	20	0.10	0.08	0.031	1.96	0.74	1.78	0.86	$H \leq 195$, $R \leq 12.0$, $N \leq 87.0$
				Z	24	0.07	0.07	0.04	2.09	0.69	1.88	0.82	$H \leq 195$, $R \leq 12.0$, $N \leq 87.0$
UNFASTENED	**Stiffened or Partially Stiffened Flanges**	One - Flange Loading or Reaction	End	C	4	0.14	0.35	0.02	2.06	0.70	1.86	0.83	$H \leq 253$, $R \leq 5.0$, $N \leq 141$
				Z	5	0.09	0.02	0.001	1.96	0.74	1.78	0.86	$H \leq 150$, $R \leq 5.0$, $N \leq 44.0$
			Interior	C	13	0.23	0.14	0.01	1.79	0.80	1.66	0.92	$H \leq 249$, $R \leq 5.0$, $N \leq 121$
		Two - Flange Loading or Reaction	End	C	13	0.32	0.05	0.04	1.80	0.80	1.67	0.92	$H \leq 253$, $R \leq 3.0$, $N \leq 64.0$
			Interior	C	24	0.52	0.15	0.001	2.14	0.67	1.92	0.80	$H \leq 253$, $R \leq 3.0$, $N \leq 64.0$
	Unstiffened Flanges	One - Flange Loading or Reaction	End	C	4	0.40	0.60	0.03	1.99	0.72	1.80	0.85	$H \leq 193$, $R \leq 2.0$, $N \leq 140$
			Interior	C	13	0.32	0.10	0.01	2.01	0.71	1.82	0.84	$H \leq 192$, $R \leq 1.0$, $N \leq 61.0$
		Two - Flange Loading or Reaction	End	C	2	0.11	0.37	0.01	2.21	0.65	1.96	0.78	$H \leq 193$, $R \leq 1.0$, $N \leq 62.0$
			Interior	C	13	0.47	0.25	0.04	2.18	0.66	1.94	0.79	$H \leq 194$, $R \leq 1.0$, $N \leq 61.0$

In 2001, Ben Young and Gregory [4] performed a number of tests on cold-formed unlipped channels. The web slenderness values of the channels ranged from 15.3 to 45. The tests were conducted under four loading conditions (i.e. End-one-Flange, Interior-One-Flange, End-Two-Flange, and Interior-Two-Flange). The test strengths were compared with the design strength obtained from Australian/New Zealand Standard. It was noted that the design strengths predicted by this Standard were generally unconservative for unlipped channels. The test strengths as low as 43% of the design strength were noted. They developed the following design equation through a combination of theoretical analyses and experimental findings for unlipped channels. It is observed that the web crippling strength obtained by the proposed design equations was generally conservative for unlipped channels with web slenderness values less than or equal to 45.

$$P_{Pm} = \frac{M_P N_m}{r}\left[1.44 - 0.0133\left(\frac{h}{t}\right)\right] \tag{2.9}$$

Where

$$M_p = \frac{f_y t^2}{4}$$

$$r = r_i + \frac{t}{2}$$

$$N_m = \begin{cases} N + id & \text{for interior loading} \\ N + \dfrac{ed}{2} & \text{for end loading} \end{cases}$$

P_{pm} =web crippling strength predicated by using the plastic mechanism model

M_p =plastic moment per unit length

r and r_i =centerline and inside corner radii, respectively

h =depth of the flat portion of the web measured along the plane of the web

t =thickness of the web

F_y =yield stress

d =overall depth of the web

N =length of the bearing

N_m =assumed mechanism length, or length of plastic hinge at the mid-depth of the web

i and e = correction factors for interior loading and end loading, respectively i =1.3 and 1.4 for IOF and ITF, respectively

e =1.0 and 0.6 for EOF and ETF, respectively

In 2003, Holesapple and LaBoube [10] investigated the effect of overhang length on the web crippling capacity of cold-formed steel members. A total of 29 specimens were tested to determine the web crippling capacity of "C" and "Z" sections with overhangs or cantilever extensions. An analysis of the resulting data produced a modification equation that is applied to the end-one-flange loading condition. The modification equation enables computation of the increase in web crippling strength as a function of the overhang length and the slenderness of the web. Analysis of the test data also revealed that the current design practice of assuming an interior-one-flange loading condition for overhang lengths of 1.5h or greater is not conservative. The following proposed design equation was presented.

$$P_c = P_n \alpha \qquad (2.10)$$

Where

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

P_c =the computed web crippling strength per web

P_n =the nominal end-one-flange web crippling strength per web (Eq 2.8)

L_o =overhang length

h =flat width of the web element

t =base steel thickness

In 2004, Young and Hancock [11] performed a series of web crippling tests on Cold-formed unlipped channels with flanges restrained (fastened) as well as channels with flanges unrestrained (unfastened). The tests were performed under end and interior two-flange loading conditions. The flanges of the channels were either bolted to one or two bearing plates for the specimens with flanges restrained. The web crippling strength were compared with the design strengths obtained from the North American Specification, Australian/New Zealand Standard and American Iron and Steel Institute Specification for Cold-formed steel structures. It was noted that the design strength by North American Specification using the fastened design rules as well as predicted by the Australian/ New Zealand Standard and AISI Specification are unconservative for both the ETF and ITF loading conditions, even when the flanges of the specimens had restraint. However, the design strength predicted by the North American Specification using the unfastened design rules are generally conservative for channels with flanges restrained and unrestrained for the ETF and ITF loading conditions. It was recommended that the web crippling unfastened design rules for ETF and ITF loading conditions in the North American Specification be used for the design of cold-formed unlipped channels irrespective of whether the flanges restrained and unrestrained.

In 2007, Zhou and Young [12] studied the web crippling strength of cold-formed high strength stainless steel tubular sections. A series of test was performed on cold-formed high steel strength stainless steel square and rectangular hollow sections. The type of stainless steel investigated in study was high strength austenitic and duplex material. The test was carried out under four loading conditions, namely: end-one-flange, interior-one-flange, end-two-flange, and interior two flange loading conditions. The web crippling strengths were compared with the design strength obtained using American, Australian/New Zealand, and European Specifications for stainless steel structures. The North American Specification for cold-formed carbon steel structural members was also used to predict the web crippling strengths and compared with test results. It is observed that the design strength predicted by the specifications is either unconservative or very conservative. New coefficients for cold-formed high strength stainless steel square and rectangular hollow were proposed.

In 2002, Young and Hancock [13] performed an experimental study on cold-formed channels subjected to combined bending and web crippling. A series of test was performed on unlipped channel rolled from high strength structural steel sheets. The specimens were tested at various length using the interior-one flange loading condition specified in the AS/NZS of 1996 [14] and AISI of 1996 [1] specifications, and the test strengths were compared with the design strength obtained from these specifications. It was noted that the combined bending and web crippling design strength predicted by the specifications were generally conservative for unlipped channels with web slenderness values less than or equal to 45.

In 2007, Zhou and Young [15] performed an experimental study on cold-formed stainless steel tubular section subjected to combined bending and web crippling. A series of test was performed on square and rectangular hollow sections fabricated by cold-rolling from high strength material of duplex and high strength austenitic stainless sheets. The flanges of hollow section were not fastened to bearing plate during testing. The specimens were tested using the interior-one-flange loading condition specified in the American specification and Australian/New Zealand standard. Different specimen lengths were test to obtained the inter action relationship between moment and concentrated load. The combined bending and web crippling test results were compared with the design strength obtained using the current American specification and AS/NZ Standard for cold formed stainless steel structures, and it was shown that the American Specification and AS/NZ standard conservatively predicted the strengths of cold-formed high strength stainless steel square and rectangular hollow section subjected to combined bending and web crippling.

In 2003, Stephens and LaBoube [16] presented the results of an experimental study on CFS beam headers to establish the web crippling strength of both box- and I-beam headers for an interior-one-flange (IOF) loading condition. The beam header specimens were tested as a system consisting of two C-sections together with top and bottom track sections. It was found that web crippling strength was greater that that for two independent, single web C-sections. Based on the experimental results, design recommendations were proposed for such CFS section configurations. In 2002, Serrette [17] presented the results from a pilot study on the web crippling performance of two common exterior load-bearing wall conditions (rim joist and rim track with a framing joist) in light framed CFS construction. In 1999, LaBoube et al. [18] studied experimentally the effect of web openings on the web crippling strength of CFS members. In 1994, LaBoube et al. [19] conducted testing to-collapse on nested Z-sections utilized in building construction for continuous span roof purlin systems. In 1995, Korvink et al. [20] conducted tests on lipped channels sections to verify theoretical prediction of their web crippling strength.

2.3 Theoretical Studies in Web Crippling

The theoretical analysis of web crippling behaviour will be reviewed in this section. Webs of cold formed steel members can be idealized as simply supported thin plate. These thin plates are subjected to in-plane compressive forces distributed along the edge. The failure load can be calculated based on the critical elastic buckling stress of the plate. Some elements develop post-buckling strength by means of redistribution of stresses and do not fail at stress levels equal to the critical elastic buckling stress. The computation of the post buckling strength is complex. The connection between flange and web element makes it further complex. The theoretical analysis of web crippling for cold formed steel flexural members is complex because it involves the following factors [2]:

i. Non-uniform stress distribution under the applied load and adjacent portions of the web.
ii. Elastic and inelastic stability of the web element.
iii. Local yielding in the immediate region of load application.
iv. Bending produced by eccentric load (or reaction) when it is applied on the bearing flange at a distance beyond the curved transition of the web.
v. Initial out-of-plane imperfection of plate elements.
vi. Various edge restraints provided by beam flanges and interaction between flange and web elements.
vii. Inclined webs for decks and panels.

Following are the few reviewed of the elastic buckling behaviour of idealized thin plates.

The elastic critical load of thin plate based on Euler's equation [21] can be represented as:

$$P_{cr} = \frac{k\pi^2 D}{h} \tag{2.11}$$

Where:

D =flexural rigidity $\left(\frac{Et^3}{12(1-v^2)}\right)$

E = Young's modulus of elasticity

h =depth of plate

k =buckling coefficient of plate subjected to edge loading

P_{cr} =elastic critical buckling load

t =plate thickness

ν = Poisson's ratio

Equation (2.11) applies to simply supported rectangular plates subjected to a uniformly distributed load (Fig 2.1a). For a square plate, i.e. when $h = l$, the plate buckling coefficient, k, is equal to 4 and for a long plate, the plate buckling coefficient then varies as function of l/h as shown in Fig 2.1b.

For rectangular simply supported plate subjected to two equal and opposite concentrated edge load. The critical elastic buckling model is evaluated by Timoshenko [21] as follows:

$$P_{cr} = \frac{k\pi^2 D}{h} \tag{2.12}$$

Where:

D =flexural rigidity $\left(\dfrac{Et^3}{12(1-\nu^2)}\right)$

E = Young's modulus of elasticity

h =depth of plate

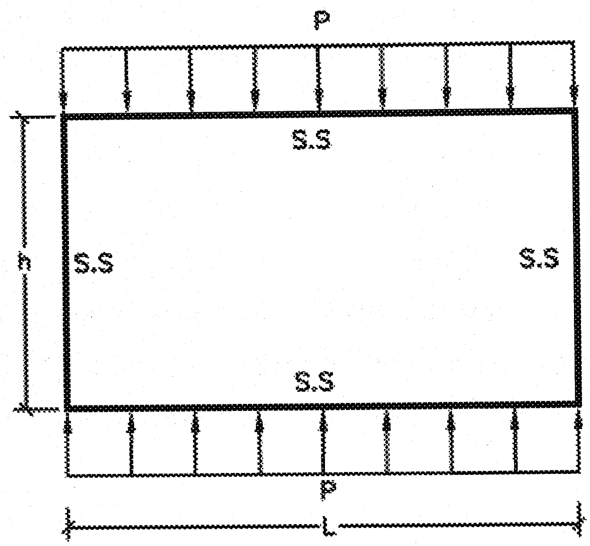
k =buckling coefficient of plate subjected to edge loading

P_{cr} =elastic critical buckling load

t =plate thickness

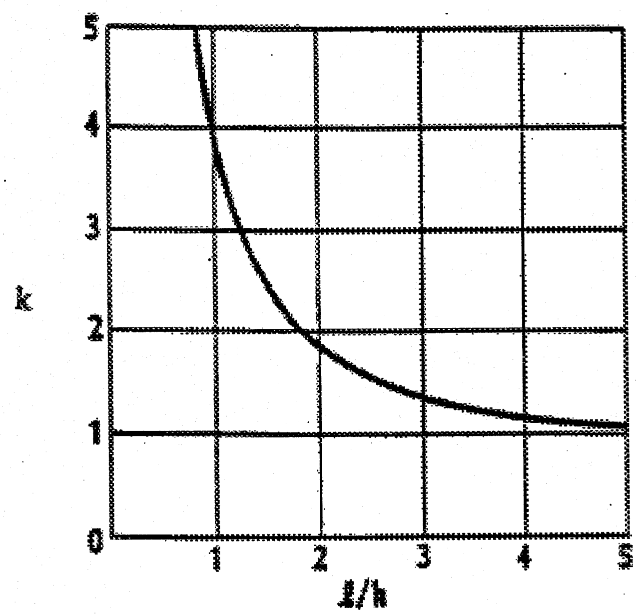
ν = Poisson's ratio

The above equation (2.12) applies to a simply supported plate subjected to two equal and opposite concentrated forces along the edge. The boundary condition for equation (2.12) is shown in Fig 2.2a. Yamaki [22] shows the plate buckling coefficient, k, as in Fig. 2.2b with the change of l / h, where l is the length of the plate and h is the depth of the plate.



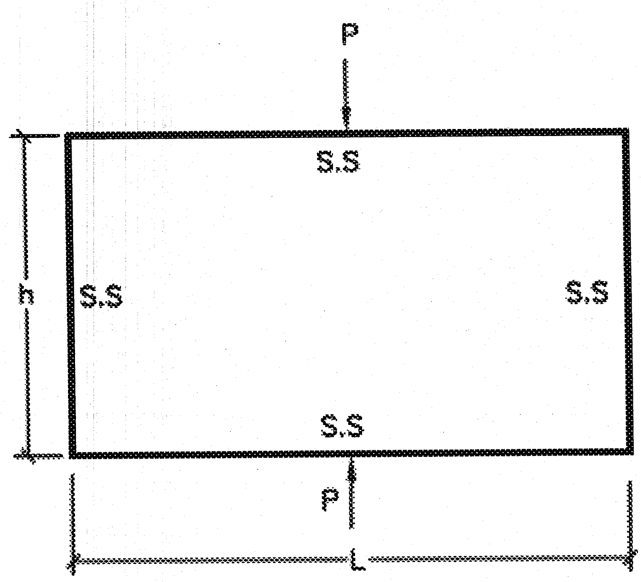
S.S =Simply Supported

(a) Simply supported plate Under Uniform Load P

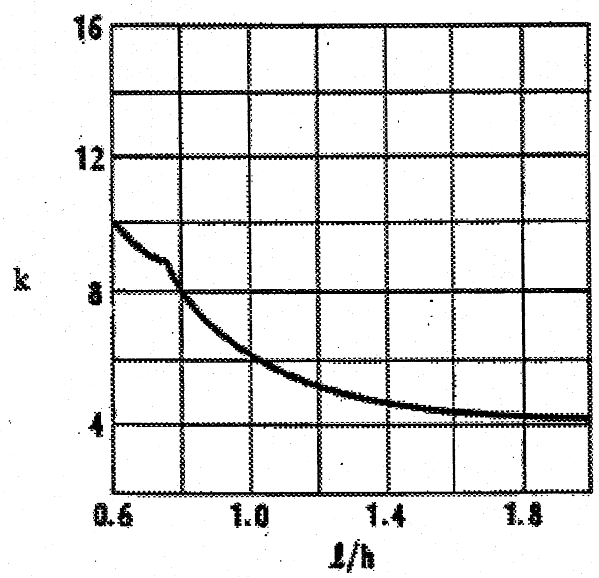


(b) Buckling Coefficient k vs. l/h

Fig. 2.1 Rectangular Plate Subjected to In-Plane Uniformly Distributed Loading [10]

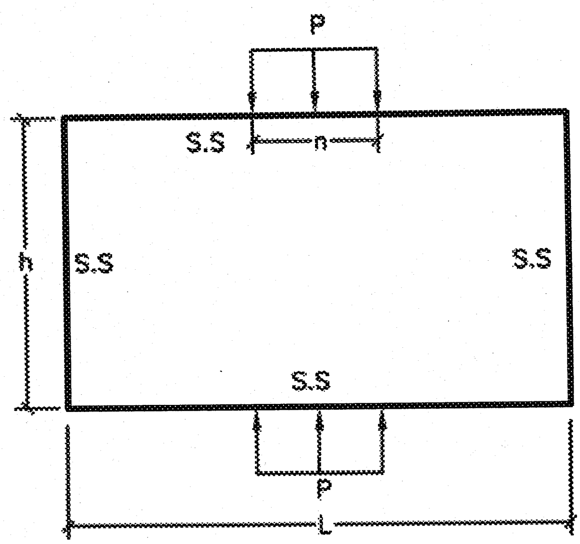


(a) Simply Supported Plate Loading Under Load P

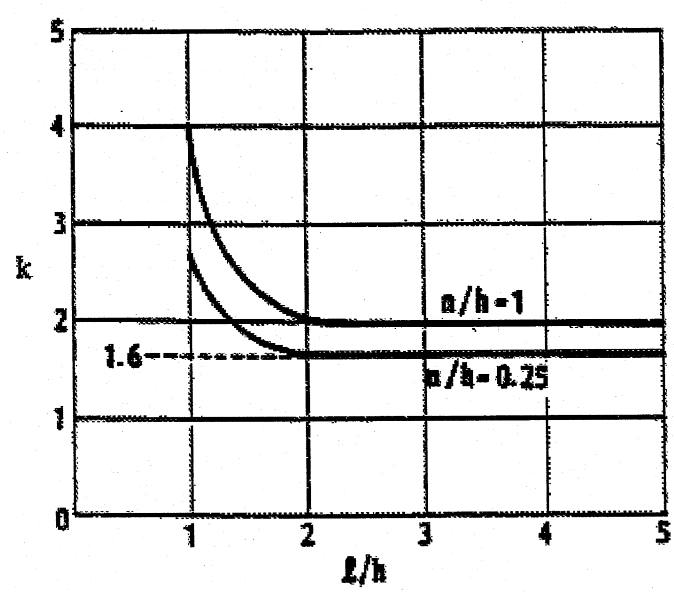


(b) Plate Buckling Coefficient k vs l / h ratio

Fig 2.2 Simply Supported Plate Subjected to Two opposite Concentrated Loads [12]



(a) Plate Under Partially Distributed Load P



(b) Buckling Coefficient k vs l / h ratio

Fig. 2.3 Rectangular Plate Subjected to In-Plan Partially Distributed Loading [14]

In 1975, Walker [23] developed the following critical elastic buckling load equation for a simply supported rectangular plate under compression due to equal and opposite partially distributed force as shown in Fig 2.3a.

$$P_{cr} = \frac{k\pi^2 E t^3}{12(1-v^2)h} \quad (2.13)$$

Where:

E = Young's modulus of elasticity

h =depth of plate

k =buckling coefficient of plate subjected to edge loading

P_{cr} =elastic critical buckling load

t =plate thickness

ν = Poisson's ratio

The bearing plate length "n" is considered in the plate buckling coefficient "k" as shown in Fig 2.3b.

In 2006, Ren et al. [24] carried out a nonlinear finite-element analysis on a series of laboratory tests on cold-formed steel channels subjected to web crippling under end-one-flange and interior-one-flange loading conditions. Geometric and material nonlinearities were included in the finite-element analysis. The finite-element results demonstrated that the ultimate load-carrying capacity, web crippling failure modes, and web deformation curves agreed well with the test results. The verified finite-element models were then used to conduct an extensive parametric study on different channel configurations. It was found that the design strength calculated from the North American Specification were generally unconservative for channels sections with unstiffened flanges having web slenderness ranging from 7.8 to 108.5 subjected to web crippling under the end-one-flange and interior-one-flange loading conditions. Therefore, updated coefficients of the design formula in the North American Specification and new design formula were proposed.

In 2006, Ren et al. [25] presented accurate finite element models to predict the behaviour and ultimate strengths of cold-formed steel channels subjected to pure bending as well as combined bending and web crippling. The nonlinear finite element models were verified against experimental results of cold-formed steel channels subjected to pure bending and web crippling.

The finite element analytical results show a good agreement with the experimental results. The channels strength predicted from the parametric study were compared with the design strengths calculated from the North American Specification for cold-formed steel structures. It was observed that the design rules in the North American Specification are generally conservative for channel section with unstiffened flanges having the web slenderness ranged from 7.8 to 108.5 subjected to combined bending and web crippling.

In 2008, Zhou and Young [26] conducted a total of 150 web crippling tests, under end-two-flange and interior-two-flange conditions. The test specimens were Aluminium tubular sections fabricated by extrusion using 6063-T5 and 6061-T6 heat treated aluminium alloys. A nonlinear finite element modelling was developed and verified against the test results. The geometric and material non-linearity was considered in the FEM. The FEM closely predicted the web crippling strengths and failure modes of the tested specimens. The specimen experimental strengths were also compared with the design strengths obtained using the unified web crippling equation as specified in the North American Specification for the cold-formed steel member. It was observed that the design strengths predicted by the aforementioned Specification were either quite conservative or un-conservative, based on the member configuration. As such, updated coefficients were proposed for the unified web crippling equation in the North American Specification [27].

In 2006, Zhou and Young [28] performed yield line mechanism analysis on web crippling of CFS square and rectangular hollow sections under two-flange loading. In 2004, Fox [29] conducted finite-element modeling of stiffened CFS C-sections that have bearing stiffeners installed between their flanges to examine the interaction between the stiffener and the C-section when the assembly was loaded to failure. Few authors conducted theoretical and/or experimental study on cold formed steel sheets and decks [30, 31, 32, 33, 34, 35, 36 and 37].

2.4. Standard Specification for Web Crippling Strength

In North America, cold formed steel members were not widely used in buildings until the publication of the first edition of the American Iron and steel institute Specification in 1946. The Specifications was primary based on research work done at Cornell University since 1939. It was revised subsequently by AISI Committees in 1956, 1960, 1962, 1968, 1980, and 1986 to reflect the technical development and the result of continuing research. AISI published the first edition of Load and Resistance Factor Design Specification for Cold Formed Steel Members in 1991. In 1996 both allowable stress design (ASD) and load and resistance factor design (LRFD) specification were combined into one single document. In Canada, the Canadian Standards Association (CSA) published its first edition of Design of Light Gauge Steel Members in 1963 based on the 1962 edition of the AISI Specification. Subsequent editions were published in 1974, 1984, 1989 and 1994. The Canadian standard for cold formed steel structural member (CSA, 1994) was based on Limit States Design (LSD) method.

The first edition of the unified North American Specification was prepared and issued in 2001 [1]. It is applicable to the Canada, United States, and Mexico for the design of cold formed steel members. In Canada it is approved by the Canadian Standards Association (CSA), in the United States by the American Iron and Steel Institute (AISI), and in Mexico by CANACERO. This edition of the specification was developed on the basis of the 1996 AISI Specification with the 1999 Supplement (AISI, 1996, 1999), the 1994 CSA Standard (CSA, 1994), and subsequent developments. In this new North American Specification, the ASD and LRFD methods are used in Mexico and United States, respectively, while the LSD method is used in Canada only. In North American Specification for the design of cold formed steel structural member 2001 edition, the factored web crippling resistance, P_n, of an unreinforced web of a member in bending is specified as follows:

$$P_n = Ct^2F_y \sin\theta\left(1 - C_R\sqrt{\frac{R}{t}}\right)\left(1 + C_N\sqrt{\frac{N}{t}}\right)\left(1 - C_h\sqrt{\frac{h}{t}}\right) \qquad (2.14)$$

Where:

P_n =Nominal web crippling strength

C =Coefficient

C_h =Web slenderness coefficient

C_N =Bearing length coefficient

C_R =Inside bend radius coefficient

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

N =Bearing length

R =Inside bend radius

t =Web thickness

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

P_n represents the nominal web crippling strength for a load or reaction of one solid web element connecting top and bottom flanges. For webs consisting of two or more sheets, P_n shall be calculated for each sheet and added together to obtain the nominal load or reaction for the section. For the above-referenced coefficients the values are given in Tables 2.2, 2.3, 2.4, 2.5, and 2.6

Table 2.2 Built-Up Sections (CAN/CSA-S136-01)

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	$Ø_W$	Limits
Fastened to support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	0.60	$R/t \leq 5$
			Interior	20	0.15	0.05	0.003	0.80	$R/t \leq 5$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	0.60	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	0.75	$R/t \leq 3$
		Two-Flange Loading or Reaction	End	15.5	0.09	0.08	0.04	0.65	$R/t \leq 3$
			Interior	36	0.14	0.08	0.04	0.65	$R/t \leq 3$
	Unfastened Flanges	One-Flange Loading or Reaction	End	10	0.14	0.28	0.001	0.60	$R/t \leq 5$
			Interior	20.5	0.17	0.11	0.001	0.75	$R/t \leq 3$

Notes: This table applies to I-beams made from two channels connected back to back. The above coefficient apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 1.0$ and $\theta = 90°$

Table 2.3 Single Web Channel and C-Section (CAN/CSA-S136-2001)

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h	$Ø_W$	Limits
Fastened to support	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	0.75	$R/t \leq 9$
			Interior	13	0.23	0.14	0.01	0.80	$R/t \leq 5$
		Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	0.75	$R/t \leq 12$
			Interior	20	0.10	0.08	0.031	0.75	$R/t \leq 12$
Unfastened	Stiffened or Partially Stiffened Flanges	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	0.70	$R/t \leq 5$
			Interior	13	0.23	0.14	0.01	0.80	$R/t \leq 5$
		Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	0.80	$R/t \leq 3$
			Interior	24	0.52	0.15	0.001	0.65	$R/t \leq 3$
	Unfastened Flanges	One-Flange Loading or Reaction	End	4	0.40	0.60	0.03	0.70	$R/t \leq 2$
			Interior	13	0.32	0.10	0.01	0.70	$R/t \leq 1$
		Two-Flange Loading or Reaction	End	2	0.11	0.37	0.01	0.65	$R/t \leq 1$
			Interior	13	0.47	0.25	0.04	0.65	$R/t \leq 1$

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

Table 2.4 Single Web Z-Section (CAN/CSA-S136-2001)

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

Notes: The above coefficient apply when $h/t \le 200$, $N/t \le 210$, $N/h \le 2.0$ and $\theta = 90°$

Table 2.5 Single Hat Section (CAN/CSA-S136-2001)

Support and Flange Conditions	Load Cases		C	C_R	C_N	C_h	$Ø_W$	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	0.75	R/t ≤ 9
		Interior	13	0.23	0.14	0.01	0.80	R/t ≤ 5
	Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	0.75	R/t≤ 12
		Interior	20	0.10	0.08	0.031	0.75	R/t 12
Unfastened to support	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	0.70	R/t ≤ 5
		Interior	13	0.23	0.14	0.01	0.80	R/t ≤ 5

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

Table 2.6 Multi Web Deck Sections (CAN/CSA-S136-2001)

Support and Flange Conditions	Load Cases		C	C_R	C_N	C_h	$Ø_W$	Limits
Fastened to Support	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	0.75	R/t ≤ 9
		Interior	13	0.23	0.14	0.01	0.80	R/t ≤ 5
	Two-Flange Loading or Reaction	End	7.5	0.08	0.12	0.048	0.75	R/t≤ 12
		Interior	20	0.10	0.08	0.031	0.75	R/t 12
Unfastened to support	One-Flange Loading or Reaction	End	4	0.14	0.35	0.02	0.70	R/t ≤ 5
		Interior	13	0.23	0.14	0.01	0.80	R/t ≤ 5
	Two-Flange Loading or Reaction	End	13	0.32	0.05	0.04	0.80	R/t ≤ 3
		Interior	24	0.52	0.15	0.001	0.65	R/t ≤ 3

Notes: (1) The above coefficient apply when $h/t \leq 200$, $N/t \leq 210$, $N/h \leq 3.0$

(2) $45° \leq \theta \leq = 90°$

CHAPTER 3
EXPERMENTAL PROGRAM

3.1 General

The current web crippling provision specified in North American Design Specification (CAN/CSA-S136-01) for cold formed steel members subjected to end two-flange loading, ETF, or interior two-flange loading, ITF, is based on channels without lap splice. It do not provide for an increase in web crippling when lap is present. This reported research study in this thesis focused on the case of interior two-flange loading condition for lapped C-channel fastened to supports, on which numerous tests were conducted at the structural laboratory of Ryerson University to investigate their web crippling strength. The purpose of this experimental study was to develop new coefficients for this type of CFS member configuration and loading condition for possible inclusion in the forthcoming edition of the North American CFS Design Specifications.

3.2 Test Specimens

The cold formed steel specimens used to perform the experimental program were of edge stiffened C-sections, as shown in Fig. 3.1, and were supplied by Canadian Sheet Steel Building Institute (CSSBI). The cross sectional parameters for each section are shown in Table 3.1 and parameter ranges are shown in Table 3.2.

The tested specimens were divided into four groups. The first group of channel sections had a galvanizing lining, a thickness of 1.719 mm (14Ga, 0.068"), total depth of 254 mm (10"), depth of the stiffened edge of 20 mm and fillet radius of 3.38 mm. The second group of channel sections had a thickness of 1.43 mm (16Ga; 0.056"), total depth of 254 mm (10"), depth of the stiffened edge of 18 mm and fillet radius of 2.27 mm. The third group of channel sections had a thickness of 1.43 mm (16Ga, 0.056"), total depth of 203 mm (8"), depth of the stiffened edge of 18 mm and fillet radius of 2.77 mm. The fourth group of channel sections had a thickness of 1.196 mm (18Ga, 0.0451"), total depth of 203 mm (8"), depth of the stiffened edge of 18 mm and fillet radius of 1.78 mm.

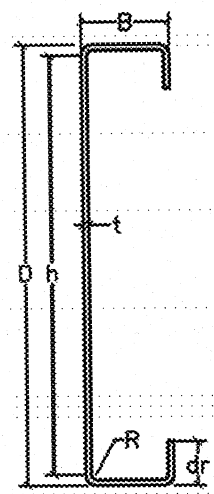


Fig. 3.1 Typical C-Section of the tested specimens

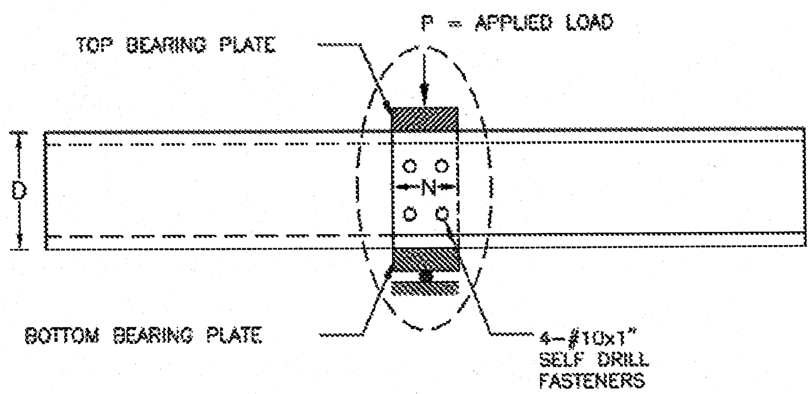


Fig. 3.2 Typical specimen and ITF loading configurations

Table 3.1 Test Specimens Cross Sectional Properties

Specimen	D (mm)	h (mm)	t (mm)	B (mm)	R (mm)	dr (mm)	Fy (Mpa)	N (mm)	h/t	R/t	N/t	N/h
A	254	242	1.719	41	3.38	20	357.14	63.50	140.70	1.96	36.94	0.26
	254	242	1.719	41	3.38	20	357.14	92.10	140.70	1.96	53.57	0.38
	254	242	1.719	41	3.38	20	357.14	152.40	140.70	1.96	88.65	0.63
B	254	246	1.430	41	2.77	18	377.14	63.50	172.0	1.93	44.40	0.26
	254	246	1.430	41	2.77	18	377.14	92.10	172.0	1.93	64.40	0.37
	254	246	1.430	41	2.77	18	377.14	152.40	172.0	1.93	106.57	0.62
C	203	194	1.403	41	2.77	18	366.57	63.50	138.30	1.97	45.26	0.33
	203	194	1.403	41	2.77	18	366.57	92.10	138.30	1.97	65.64	0.47
	203	194	1.403	41	2.77	18	366.57	152.40	138.30	1.97	108.62	0.78
D	203	197	1.196	41	1.78	18	389.55	63.50	164.71	1.48	53.09	0.32
	203	197	1.196	41	1.78	18	389.55	92.10	164.71	1.48	77.0	0.47
	203	197	1.196	41	1.78	18	389.55	152.40	164.71	1.48	127.42	0.77

Table 3.2 Test Specimens' Parameter Ranges

Parameter	Minimum Values	Maximum Values
Thickness (t), mm	1.196	1.719
Web slenderness ratio (h/t)	138.30	172.0
Inside bend ratio (R/t)	1.48	1.97
Bearing plate length ratio (N/t)	36.94	127.42
Yield strength (F_y), MPa	357.14	389.55

Each test specimen is made of two channels of 1.5 m long each (5') and lapped over a certain length. Table 3.1 shows that each test group consists of three specimens with varying lap length, as follows: 63.5 mm (2.5"), 92.1 mm (3.635") and 152.4 mm (6"). The lapped channels were fastened together using 4 self drilling screws #10x1" as shown in Fig. 3.2. It should be noted that the two rows of screws are located at the third point of the web depth. It should be noted that bearing plate over the top flanges and the base plate located under the bottom flanges at the interior support, shown in Fig. 3.2, had the same lap length specific above for each test.

3.3 Mechanical Properties of CFS Material

The material properties of the test specimens were determined by tensile coupon tests. For each C-section tested, three coupons were cut from the center of the web plate in the longitudinal direction of the undisturbed specimens. The tensile coupons were prepared and tested according to American Society of Testing and Materials Test Method "ASTM A370" of 2005 [38]. To measure the actual thickness of each coupon, the galvanized coating was removed by hydrochloric acid solution. Figure 3.3 shows dimensions of the coupon sample, while Fig. 3.4 shows view of the typical setup for tensile coupon testing. Figure 3.5 shows view of the specimens after failure in the tensile testing machine, while Fig. 3.6 shows view of the failure shape in one of the tested coupons. Figure 3.7 depicts a typical stress-strain relationship of the tested coupon sample Table 3.3 shows the material mechanical properties of tested specimens in the form of yield strength, ultimate tensile strength and percentage elongation at failure. It should be noted that the average values of the web thickness and the yield strength obtained from coupon tests were used to correlate the experimental findings and the results from the available CAN/CSA-S136 Standard equation for web crippling strength.

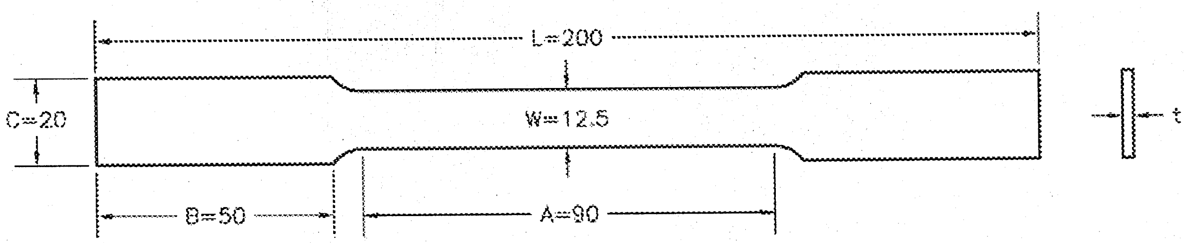


Fig. 3.3 Dimension of coupon sample per ASTM A370

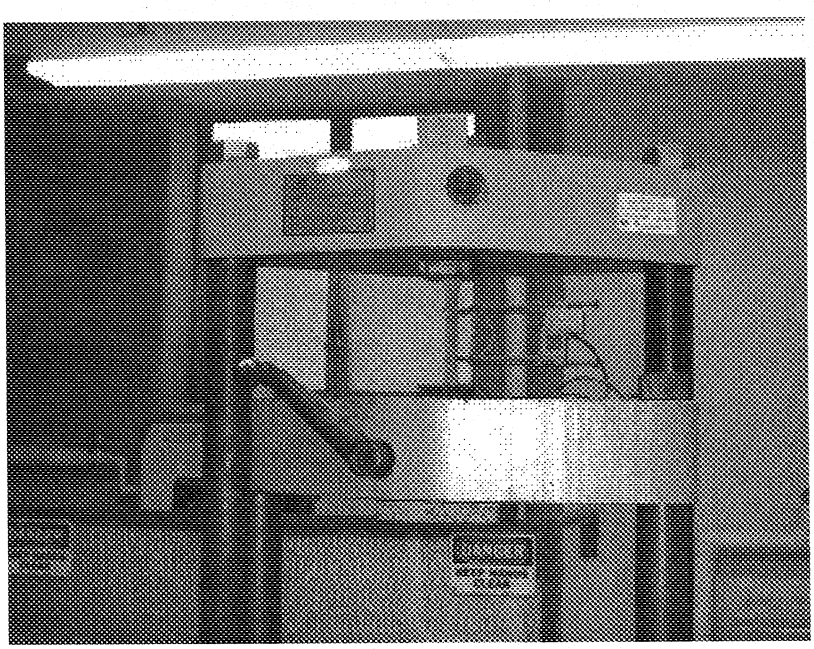


Fig. 3.4 View of a typical setup for tensile coupon testing

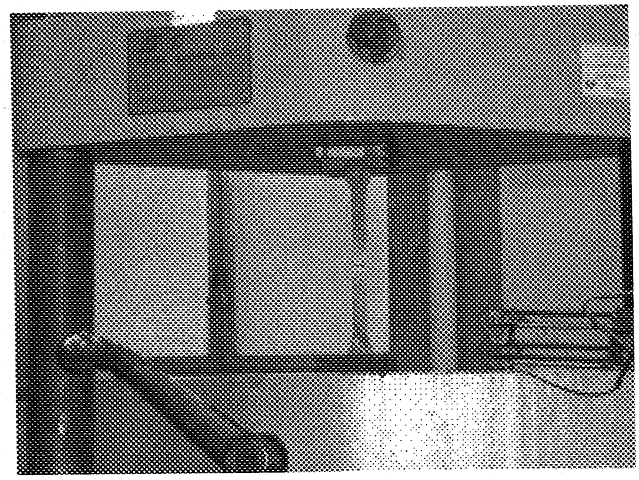


Fig. 3.5 View of the coupon sample after failure in the tensile testing machine

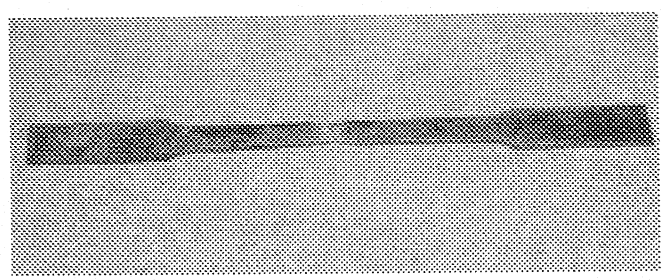


Fig. 3.6 View of the failure shape in one of the tested coupons

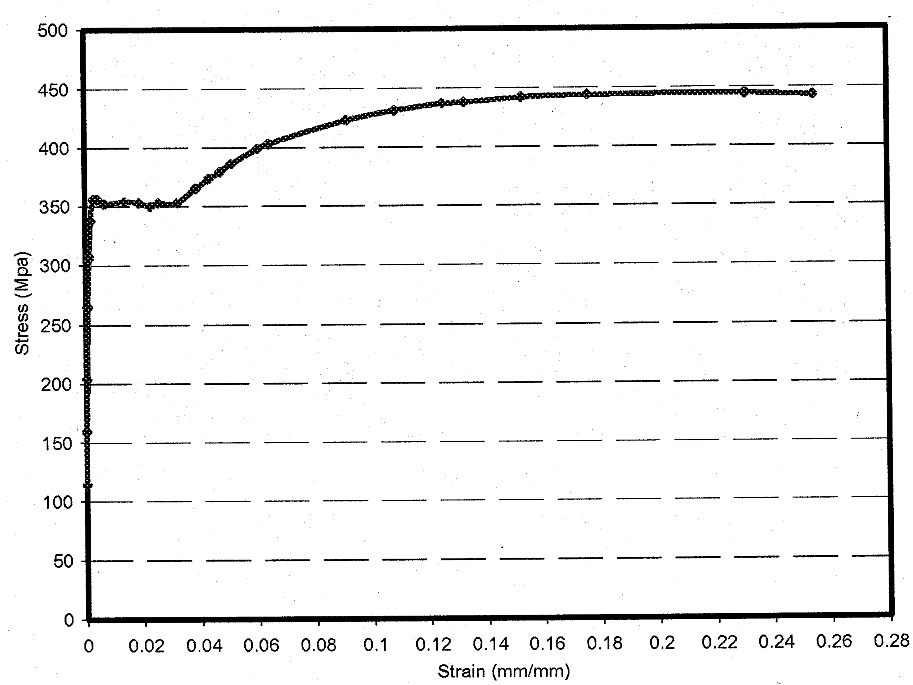


Fig. 3.7 Typical stress-strain relationship of a tested coupon sample

Table 3.3 Mechanical Properties

Specimen	Test #	t (mm)	Fy (Mpa)	Fu (Mpa)	% Elongation *
A	1	1.722	357.84	472.29	34.5
	2	1.717	359.90	475.74	34.5
	3	1.719	354.40	472.29	34.0
	Average	**1.719**	**357.14**	**473.67**	**34.3**
B	1	1.432	378.52	479.18	32.5
	2	1.427	374.38	483.32	33.0
	3	1.432	378.52	484.01	32.5
	Average	**1.430**	**377.14**	**481.94**	**32.7**
C	1	1.410	366.80	480.56	34
	2	1.397	366.80	482.63	34
	3	1.404	366.11	482.63	34
	Average	**1.403**	**366.57**	**481.94**	**34**
D	1	1.188	386.80	503.32	22
	2	1.211	391.62	504.70	22
	3	1.191	390.24	503.32	22
	Average	**1.196**	**389.55**	**503.78**	**22**

* Based on 50mm gauge length

3.4 Test Setup and Test Procedure

A number of tests were performed on single CFS stiffened channels with both flanges of channel members lapped at the interior support location and being loaded simultaneously as shown in Fig. 3.2. A test setup was prepared at the structures laboratory of Ryerson University as shown in Figs. 8, 9 and 10. To perform each test, two channels were first fastened together and laid over a base plate of 12.7 mm (0.5") thick and a length equal to the channel's lap length. Channels' sectional rotation at there ends were restrained by inserting their ends into a U-shape steel support system. In this case, the channel member is considered unbraced between this end rotational restraint and the interior support location. A 222 kN (50,000 Ib) capacity hydraulic jack was used to apply a compressive force to the test specimens over the interior support. The jacking load was applied on the channel flanges by means of bearing plates of 12.7 mm thickness, length equal to the lap length and width equal to the total width of the channel flanges. Web lateral deflection and vertical movement of the channel top flange were recorded using LVDT's. Each specimen was loaded incrementally till complete collapse. Failure was considered at the point at which the specimens could not accept no further load. Figure 3.11 shows views of the setup for each tested group.

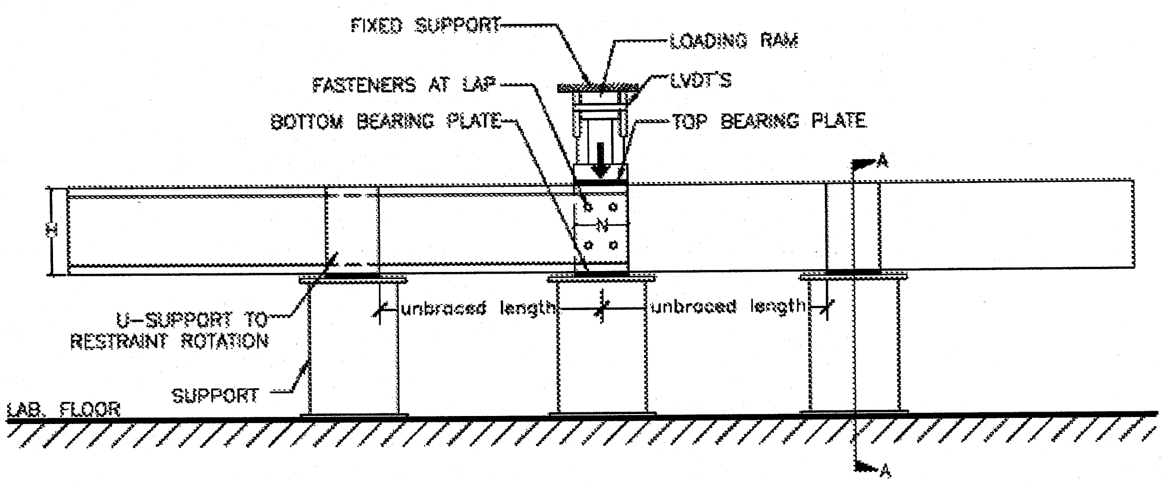


Fig. 3.8 Schematic diagram of the test setup

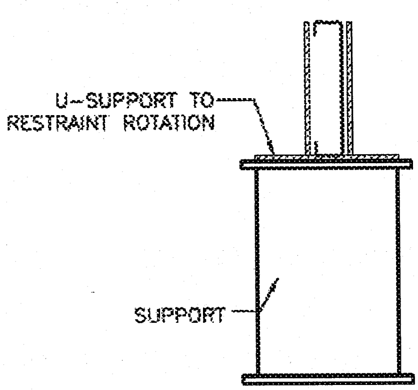


Fig. 3.9 Section at A-A

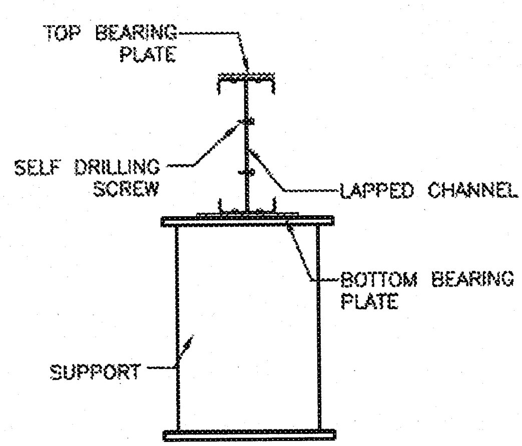
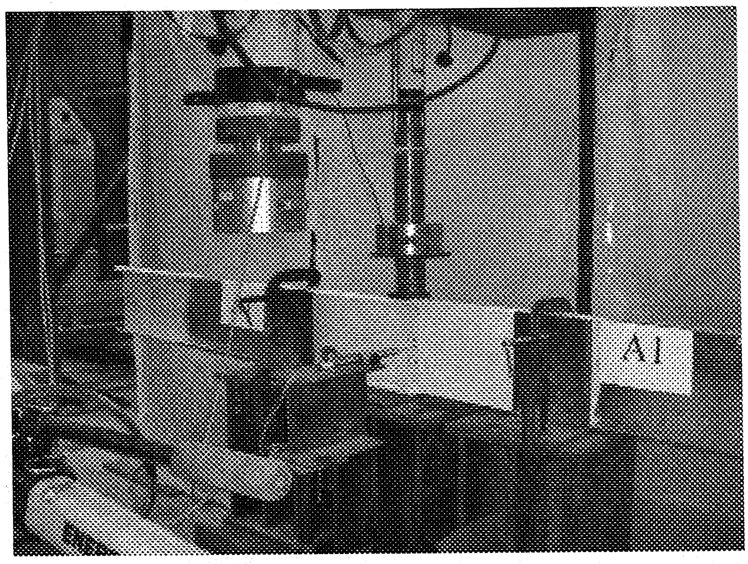
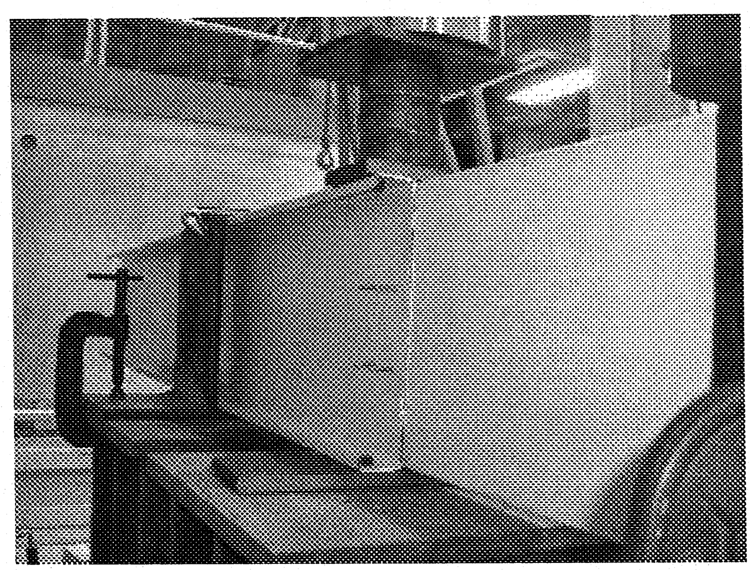


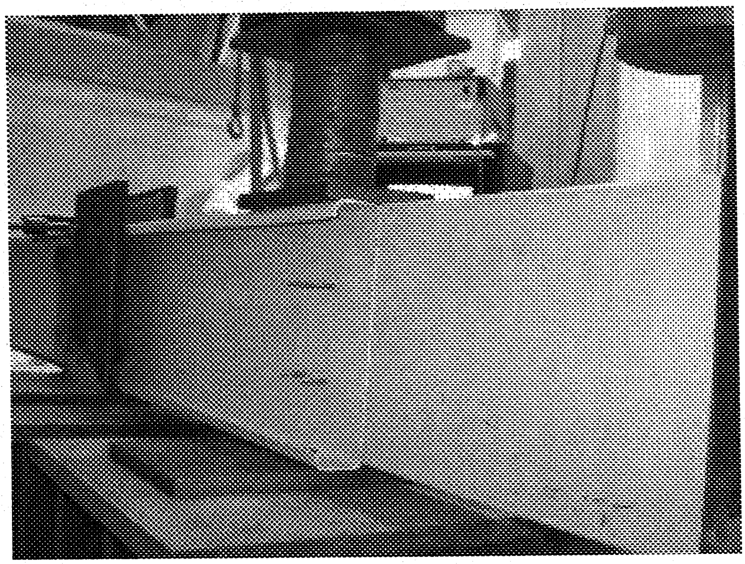
Fig. 3.10 Section at lap location



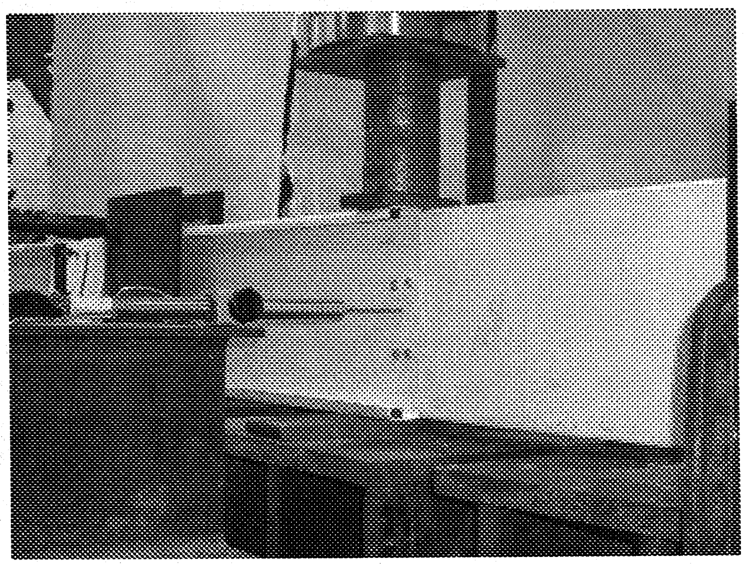
a) View of test setup for a specimen in Group A



b) View of test setup for a specimen in Group B



c) View of test setup for a specimen in Group C



d) View of test setup for a specimen in Group D

Fig. 3.11 Views of test setup for the tested groups

CHAPTER 4

EXPERMENTAL RESULTS

4.1 General

This chapter presents the experimental results of a parametric study to examine few parameters that affect web crippling strength of lapped channels at interior support location when subjected to two-flange loading. These parameters include the unbraced length of channel member, the presence of corrugated steel deck that is fastened to the top flange at equal intervals using self-drilling screws, the level of flange restraint at the interior support location, channel size and load bearing length (i.e. lap length). The specimen used in this parametric study is 254mm deep, 1.43mm thick, and lap length of 92.10mm. Test specimens were loaded to failure and load history and the failure pattern were recorded. A database of experimental findings was established so that it can be further used to develop a reliable and economical design expression for web crippling strength of lapped CFS channels at interior support location when subjected to two-flange loading.

4.2 Effect of unbraced length

The length of the channel member on each side of the interior support is believed to be an important parameter affecting the web crippling strength. To reflect the advantageous benefit of the CFS structural system, the test specimen configuration and bracing should replicate the in-place conditions. For interior two-flange loading, AISI standard test method [39] recommends a length of a single–web specimen of at least equal to five times the section depth and be positioned symmetrically in the test frame. In the same context, AISI stated that a length of three-times the member depth provides a conservative web crippling strength. The dilemma herein is that the very short length of the channel may result in global web bucking failure along the channel length, while the long length may result in global rotation of the channel member along the unbraced length, rather than localized web crippling. It should be noted that web crippling failure aimed at in this research may take the form of web local yielding or local buckling of the web just under the applied load location. As such, five tests were conducted on specimens with different channel lengths on each side of the interior support. The unbraced length between the interior support location and the rotationally-restrained end of each channel was taken as 2h, 3h, 4h, 5h and 6h, respectively, where h is the depth of the channel. Figs 4.1 4.2, 4.3, 4.4 and 4.5 show views of the tested specimens of unbraced lengths of 2h, 3h, 4h, 5h and 6h, respectively, after failure. Fig. 4.6 presented the jacking load-vertical displacement relationship for such specimens. Results show that specimens with shorter unbraced length (i.e. 2h) failed due to web crippling in the form of local web buckling just over the support with maximum lateral deflection occurred at the top third point of the web. However, the specimen with longer braced length (i.e. 4h, 5h and 6h), failed due to global twisting of channel along the unbraced length. It was observed that specimen with unbraced length of 3h failed due to combined web crippling and global torsional buckling along the unbraced length. In practice, twisting of single-web channels is prevented by the presence of the metal deck fastened to the top flange and/or by providing bridging in between channels. Figure 4.6 shows that web crippling strength decreases with an increase in the unbraced length of each channel. It can be concluded that unbraced length of 2h is sufficient to obtain reliable web crippling strength of single-web channels lapped at the interior support.

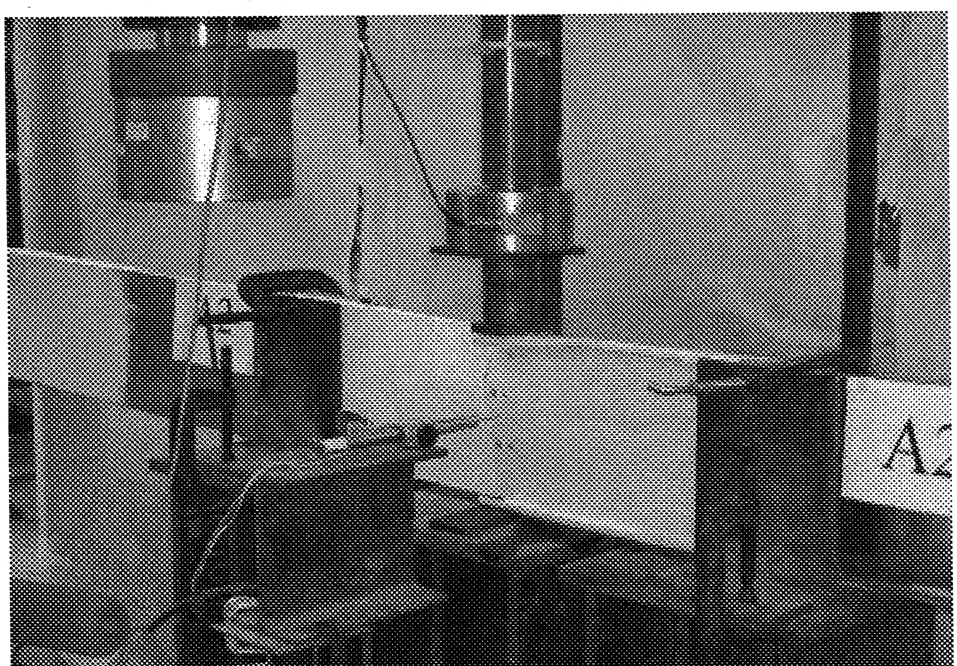


Fig. 4.1-a Web crippling failure in front face of a specimen with unbraced length of 2h

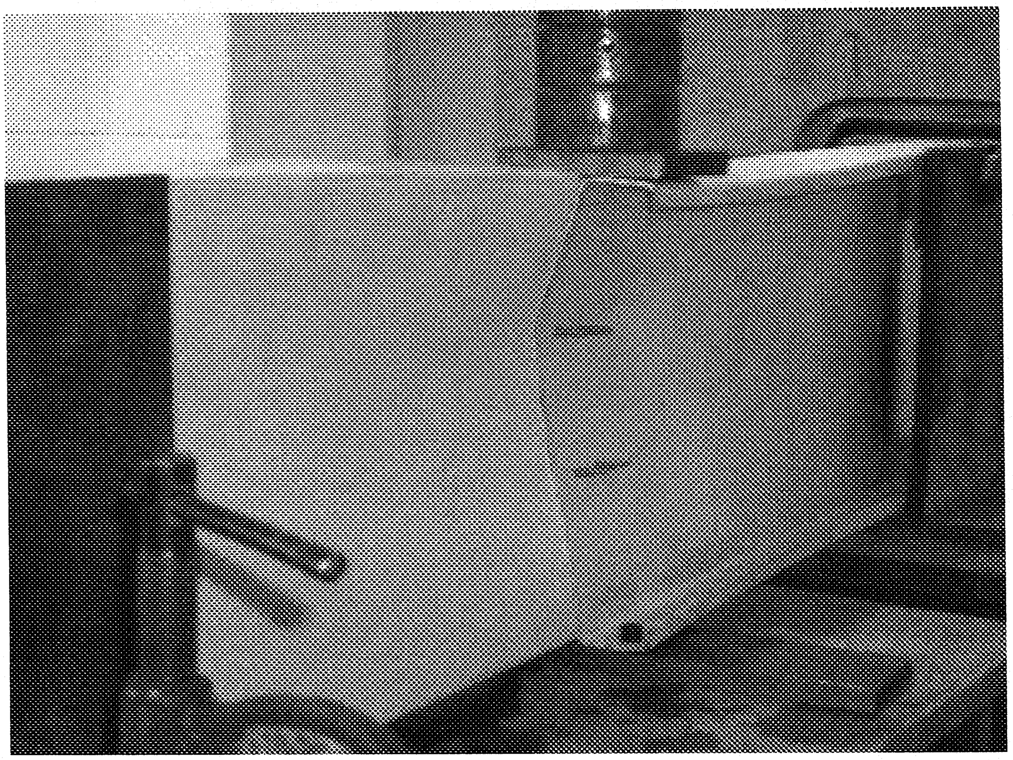


Fig. 4.1-b Web crippling failure in the back face of a specimen with unbraced length of 2h

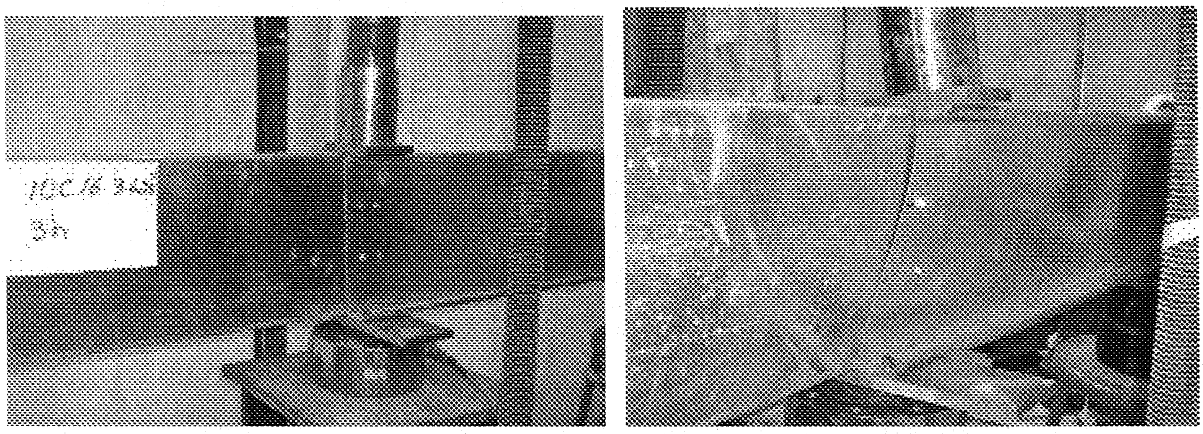


Fig. 4.2 Views of a specimen with unbraced length of 3h before and after failure

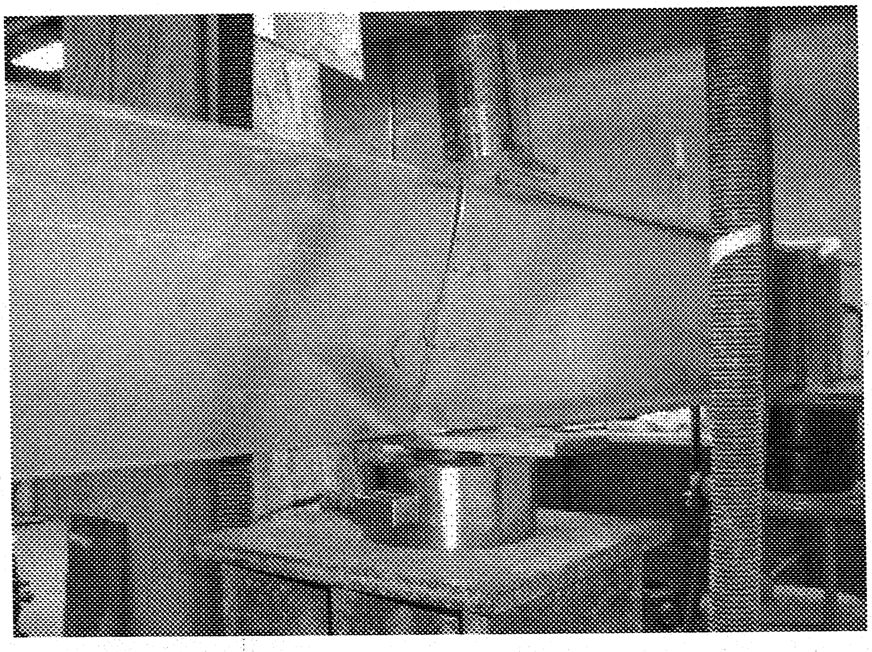


Fig. 4.3 Lateral tosional buckling failure of a specimen with unbraced length of 4h

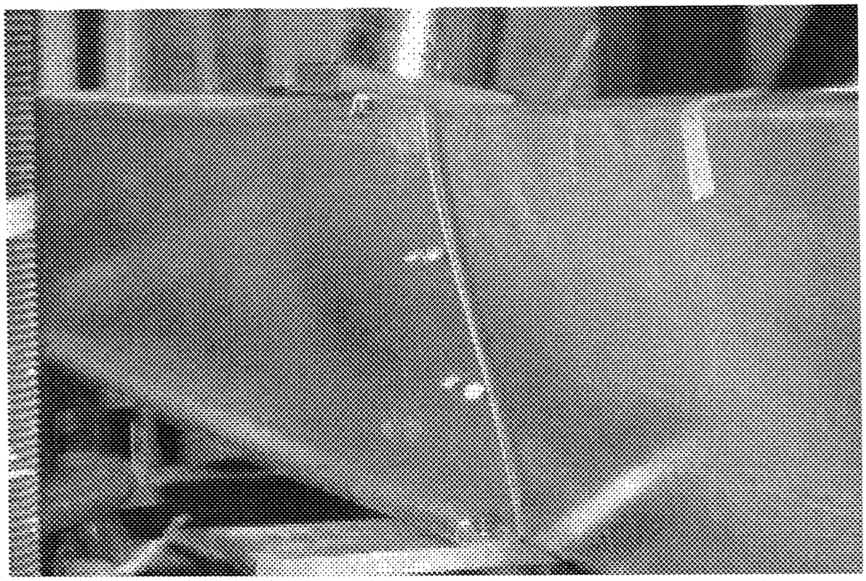


Fig. 4.4 Lateral torsional buckling failure of a specimen with unbraced length of 5h

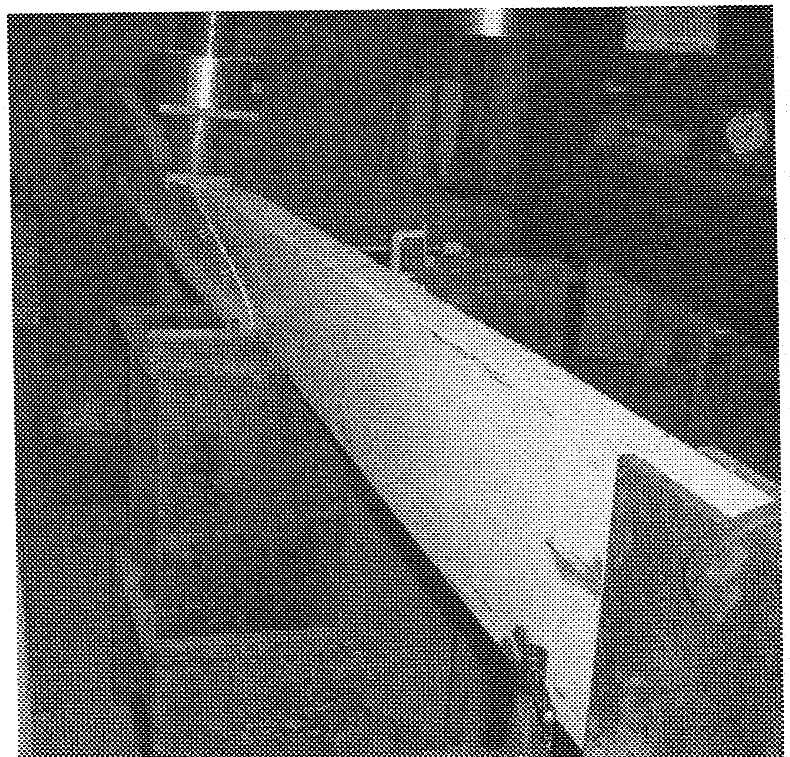


Fig. 4.5 Lateral torsional buckling of a specimen with unbraced length of 6h

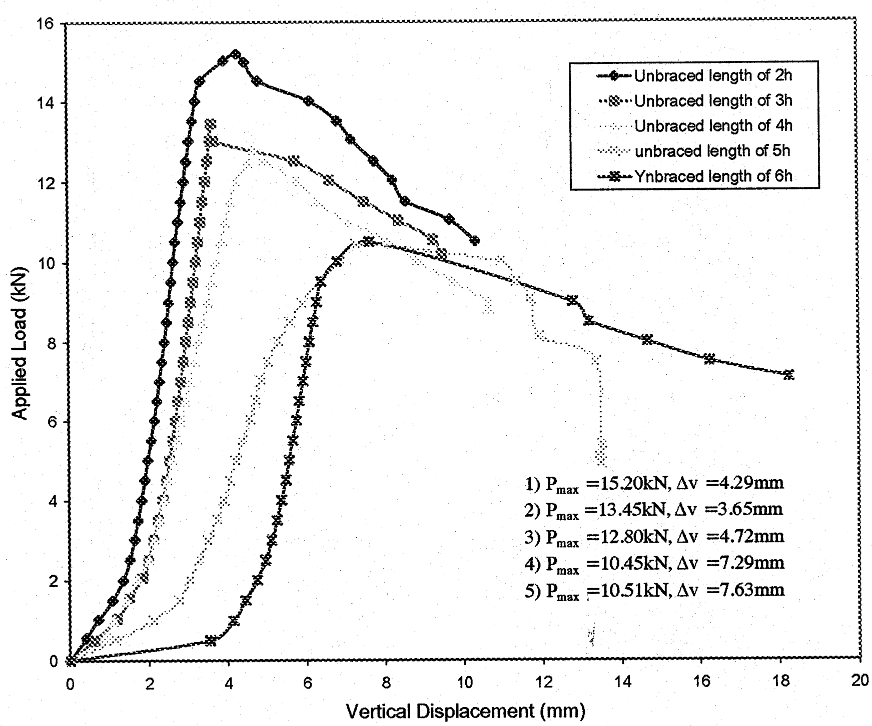


Fig. 4.6 Load-vertical deflection relationship for tested specimens with different unbraced length

4.3 Effect of the presence of metal deck

The second parameter considered in this sensitivity study was the presence of 1 ½" thick trapezoidal corrugated steel deck (Fig. 4.7) that is fastened to the top flange at equal intervals using self-drilling screws. This case is realistic in case of construction of floor and roofs with metal deck supported over CFS joists. Normally, the sheeting is longer than the spacing between joists and covers the plan area over the interior support location. Figure 4.7 shows view of two joists spaced at 408 mm (16") with the metal deck fastened to the top flange, in addition to the presence of torsion restraint support at distance 2h from the interior support. Figure 4.8 shows load-vertical deflection relationship for the tested specimens. It can be observed that the failure loads due to web crippling were 15.2, and 15.05 kN for specimens with and without the presence of metal deck, respectively. As such, it can be concluded that the unbraced length of 2h, without the presence of metal deck is sufficient to obtain a reliable web crippling strength of single-web channels lapped at the interior support.

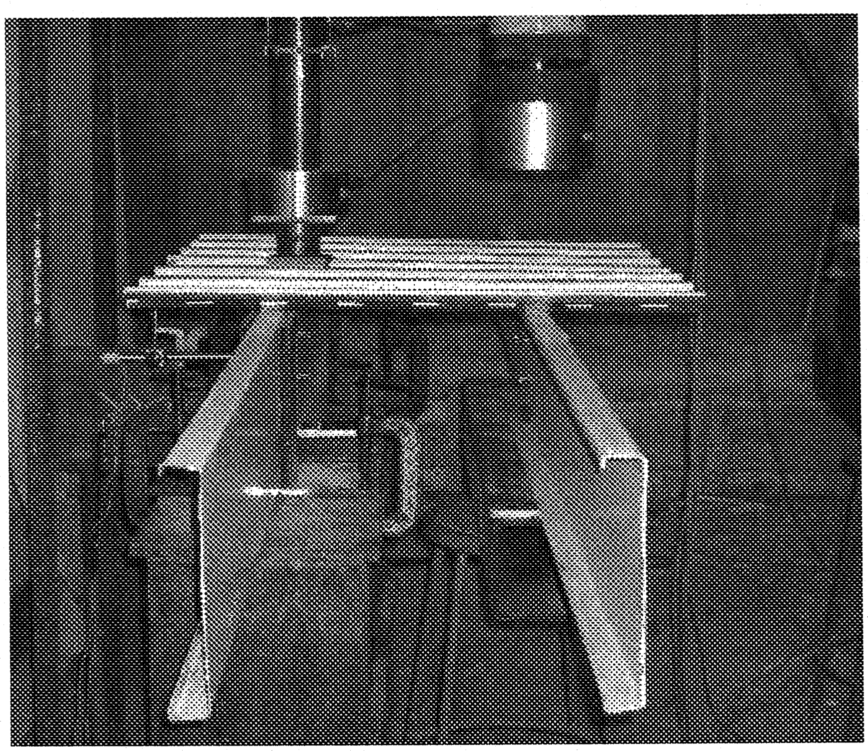


Fig. 4.7 Test setup for a specimen with metal deck

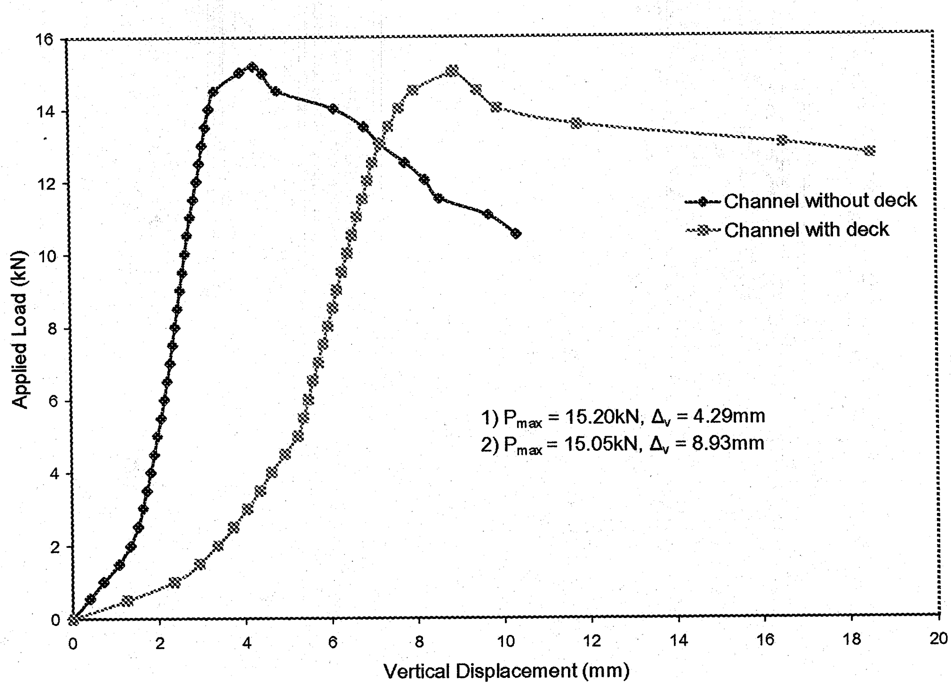


Fig. 4.8 Load-vertical deflection relationship for tested specimens with and without metal deck

4.4 Effect of Flange restraint condition

There are two different support restraint conditions that may be considered, i.e., (1) where specimen is not fastened to the support or bearing plate, and (2) where the specimen is fastened to the support or bearing plate during testing. Two specimens were tested to investigate the effect of this parameter on web crippling strength. Figure 4.9 shows view of the fastened top and bottom flanges of the channel sections to the bearing plate and the support, respectively. Fig. 4.10 shows view of the specimen with flanges free to rotate. One may observe that web crippling occurred in the web of the specimen that has restraint flanges, in the form of local bucking in the web as shown in Fig. 4.9. However, Fig. 4.10 shows instability of top flanges that triggered global failure of the specimen. Figure 4.11 depicts the jacking load-vertical displacement relationship for the two specimens. It can be observed that fastening the top and bottom flanges at load support location increased the failure load by 73% compared to the specimen with flanges free to rotate.

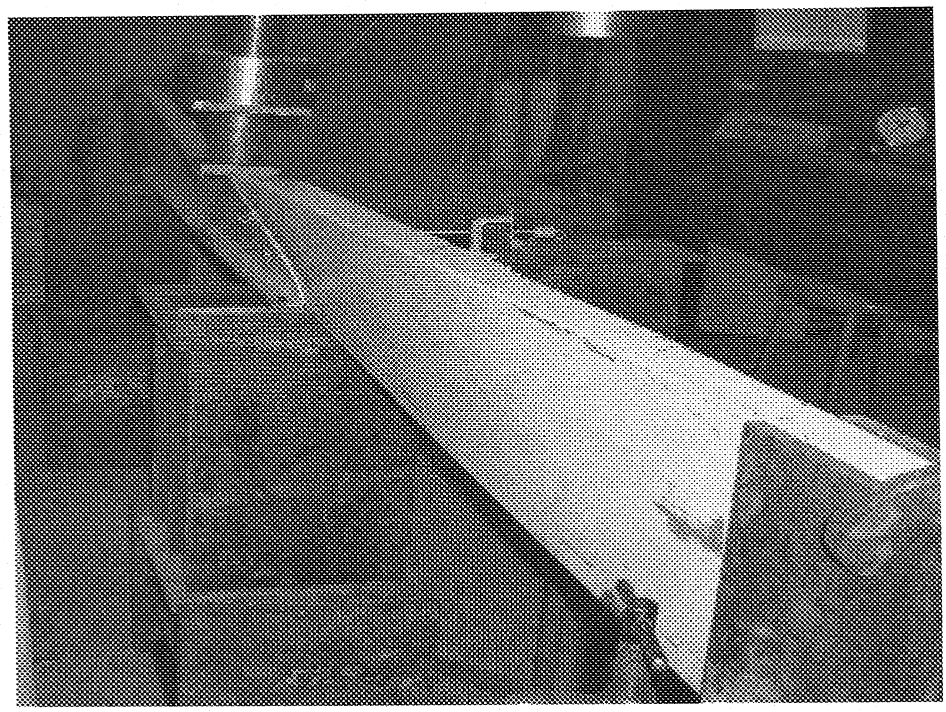


Fig. 4.9 Failure mode of specimen with restrained flanges

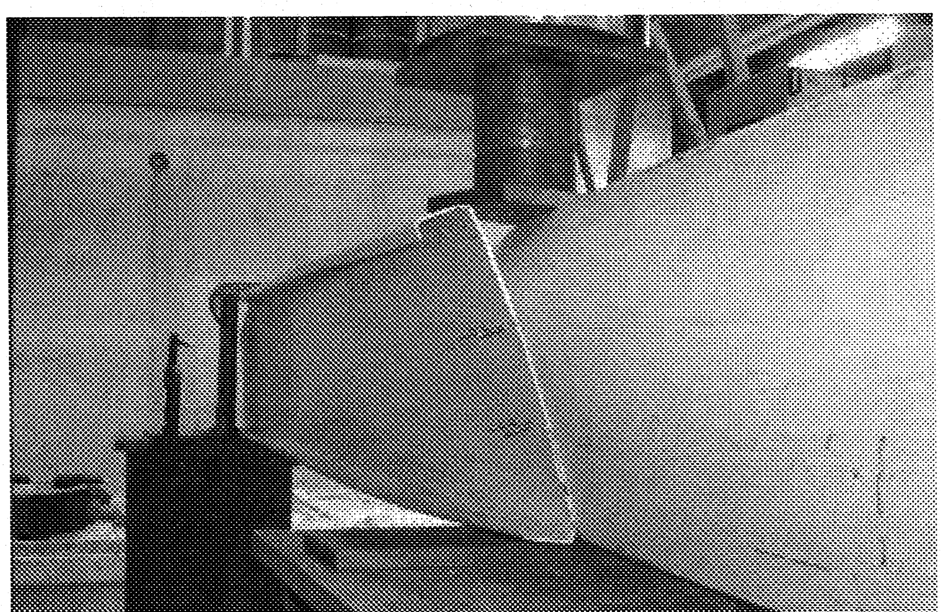


Fig. 4.10 Failure mode of specimen with unrestrained flanges

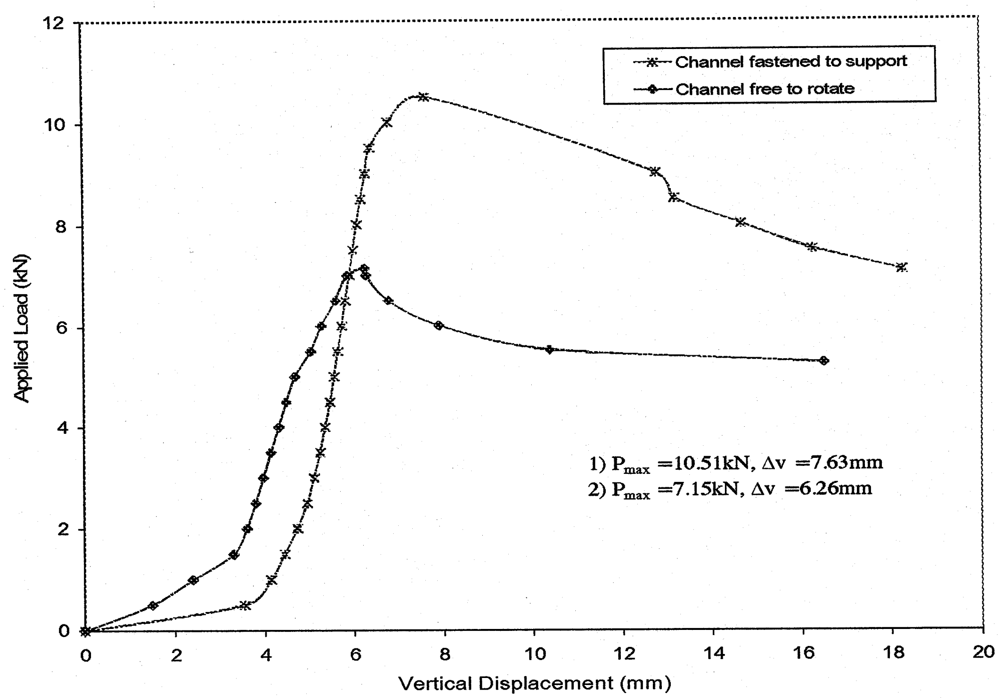


Fig. 4.11 Effect of flange fastening conditions on the load-vertical deflection relationship

4.5 Effect of joist lap length at interior support

Two sets of three identical specimens were tested to-failure to investigate the effect of lap length on web crippling strength. Three lap lengths of 63.5, 92.0, and 152.4 mm (2.5", 3.625" and 6") were considered. It should be noted that the bearing plate length were maintained equal to the lap length as it changes. Table 4.1 below shows the results for the tested specimens. It can be observed the experimental web crippling resistance increases by 26% and 73% when increasing the lap length to 92.0 and 152.4 mm, respectively.

Table 4.1 Effect of joist lap length on web crippling strength

Specimens No.	Web thick. (mm)	Web depth (mm)	Lap length (mm)	Web crippling resistance (kN)		
				Test No. 1	Test No. 2	Average
1	1.43	254	63.5	12.62	14.25	13.44
2	1.43	254	92.0	16.24	15.20	15.72
3	1.43	254	152.4	22.50	23.98	23.24

4.6 Experimental Database for Web Crippling Strength

The tested specimens were divided into four groups. Group A of channel sections had a galvanizing lining, a thickness of 1.719 mm (14Ga, 0.068"), total depth of 254 mm (10"), depth of the stiffened edge of 20 mm and fillet radius of 3.38 mm. Group B of channel sections had a thickness of 1.43 mm (16Ga; 0.056"), total depth of 254 mm (10"), depth of the stiffened edge of 18 mm and fillet radius of 2.27 mm. This means that the difference between specimens tested in groups A and B was mainly the plate thickness. Group C of channel sections had a thickness of 1.43 mm (16Ga, 0.056"), total depth of 203 mm (8"), depth of the stiffened edge of 18 mm and fillet radius of 2.77 mm. While group D of channel sections had a thickness of 1.196 mm (18Ga, 0.0451"), total depth of 203 mm (8"), depth of the stiffened edge of 18 mm and fillet radius of 1.78 mm. One may observe that groups C and D were different in plate thickness. In each tested group, three identical specimens were tested to examine the effect of lap length, which is equal to the bearing length, on web crippling strength. It is worth mentioning that the tested specimens herein had unbraced length of 2h that was proved in the sensitivity study to be sufficient to obtain reliable web crippling strength of single-web channels lapped at the interior support. Also, in each group three lap lengths of 63.5, 92.0, and 152.4 mm (2.5", 3.625" and 6") were considered. It should be noted that the bearing plate length were maintained equal to the lap length as it changes.

Table 4.2 presents the experimental results of the tested specimens in each group of channel section. One may observe that each test was repeated twice times to provide more confidence of the experimental results. In Group A, it can be observed that web crippling strength increased from 20.35 to 27.75 kN with an increase in channel lap length as well as bearing plate length from 63.50 mm to 152.40 mm. Similar behavior was observed in other groups B, C and D as shown in Table 4.2. For the same bearing length of 63.5 mm, and channel depth of 254 mm, it can be observed that web crippling strength increased from 13.44 kN to 20.35 kN with increase of channel thickness from 1.43 mm (group B) to 1.719 mm (group A).

Table 4.2 Database for experimental results

Group	D (mm)	t (mm)	N (mm)	Experimental web crippling strength (kN)		
				Test # 1	Test # 2	Average value
A	254	1.719	63.05	20.67	20.02	20.35
	254	1.719	92.10	20.59	23.00	21.80
	254	1.719	152.4	27.73	27.77	27.75
B	254	1.430	63.05	12.62	14.25	13.44
	254	1.430	92.10	16.24	15.20	15.72
	254	1.430	152.4	22.50	23.98	23.24
C	203	1.403	63.05	16.18	15.12	15.65
	203	1.403	92.10	18.35	18.50	18.43
	203	1.403	152.4	23.65	21.89	22.77
D	203	1.196	63.05	11.66	12.73	12.20
	203	1.196	92.10	13.35	15.90	14.63
	203	1.196	152.4	15.98	17.52	16.75

Figures 4.12 to 4.39 presents a summary of the results for each tested group shown in Table 4.2. The results include (i) a sample specimen from each group before and after testing showing the web crippling failure mode; (ii) the applied load-vertical displacement relationship of the channel at load location as obtained from tests No. 1 and 2 for each bearing length; and (iii) the applied load-lateral displacement relationship of the channel at mid-height of the web at load location as obtained from tests No. 1 and 2 for each bearing length. All reported load-displacement relationship reported herein showed the web crippling behaviour of the tested panel rather than local yielding of the web at load location. The results reported in Table 4.2 will be further used in Chapter 5 to deduce design coefficients for the available web crippling strength equation in the North American Specification for CFS members.

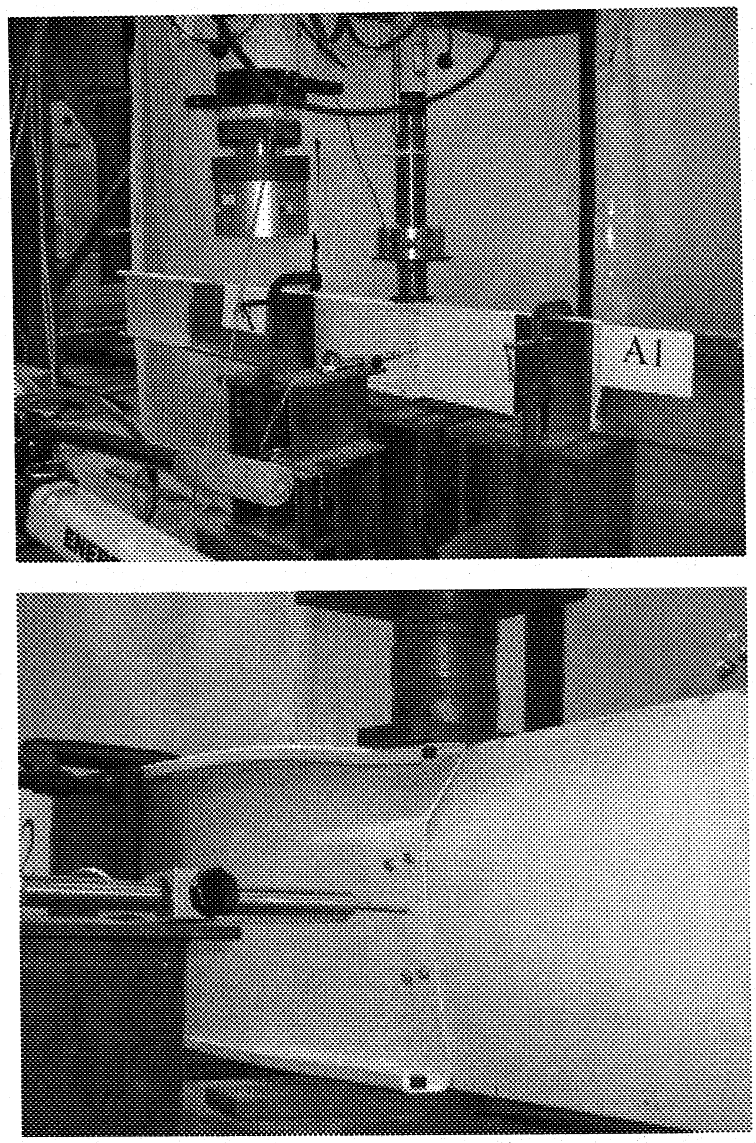


Fig. 4.12 Views of a specimen in group A before and after test

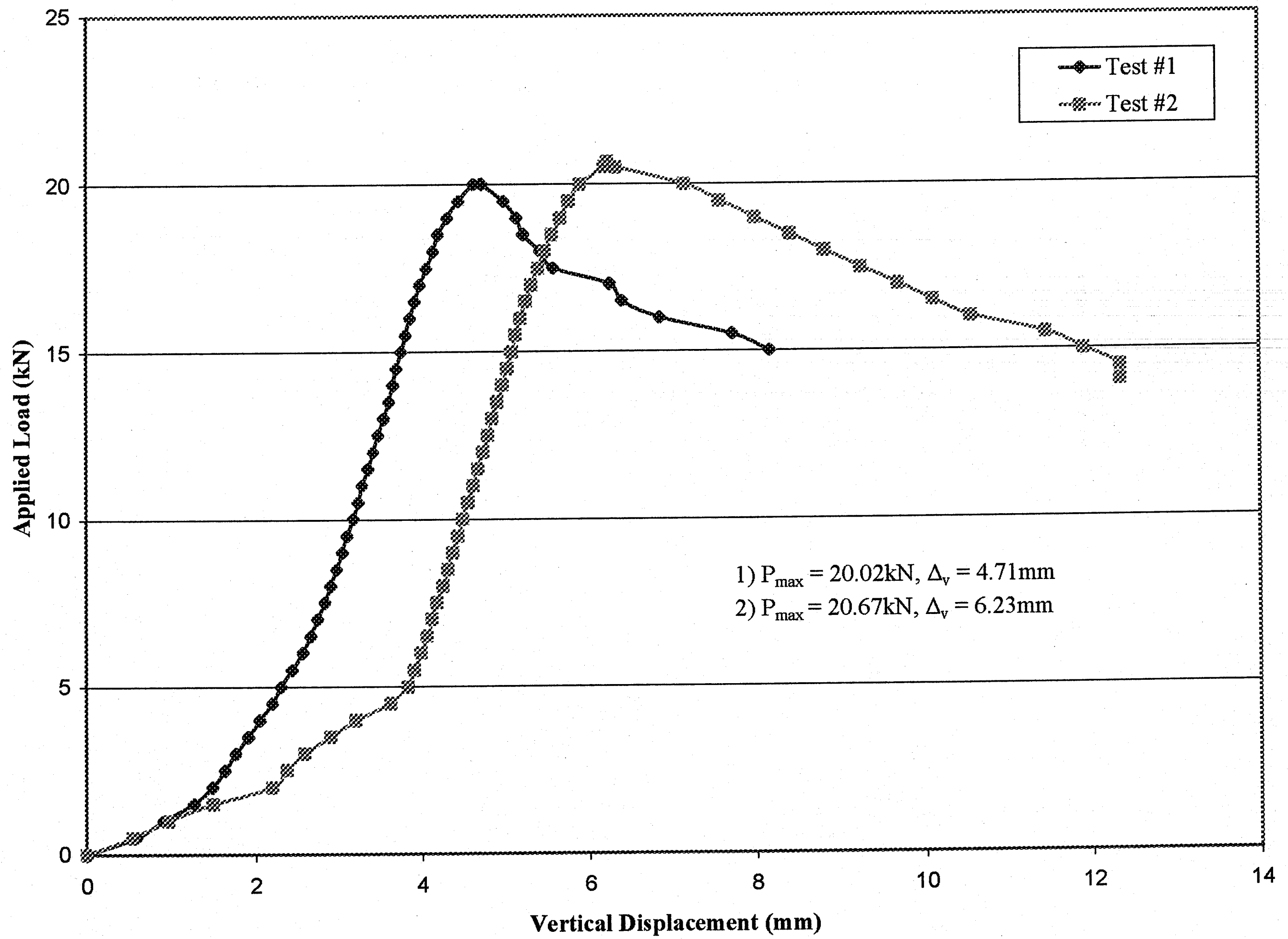


Fig. 4.13 Load-vertical displacement relationship for a specimen in group A with bearing length of 63.5 mm

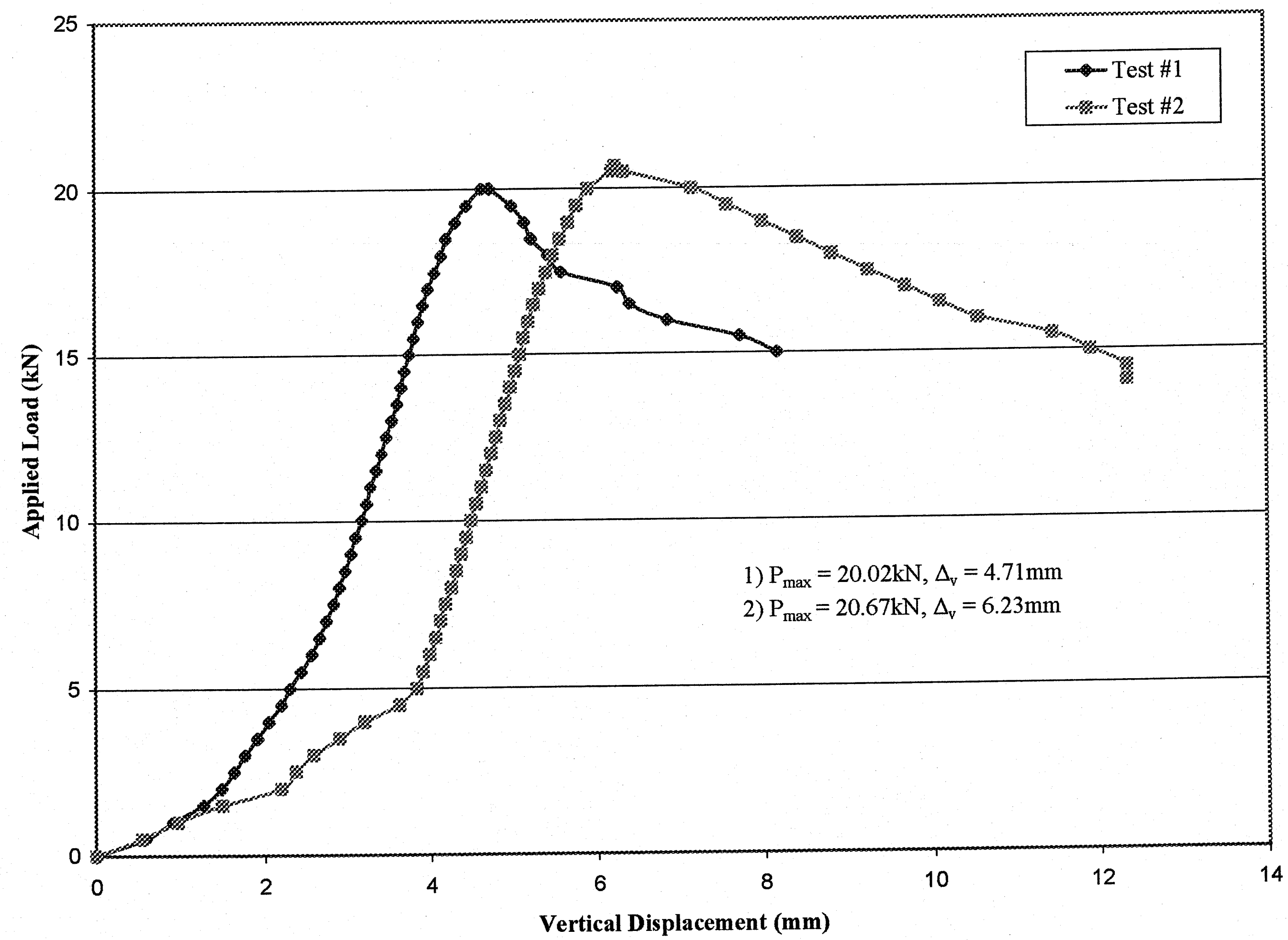


Fig. 4.13 Load-vertical displacement relationship for a specimen in group A with bearing length of 63.5 mm

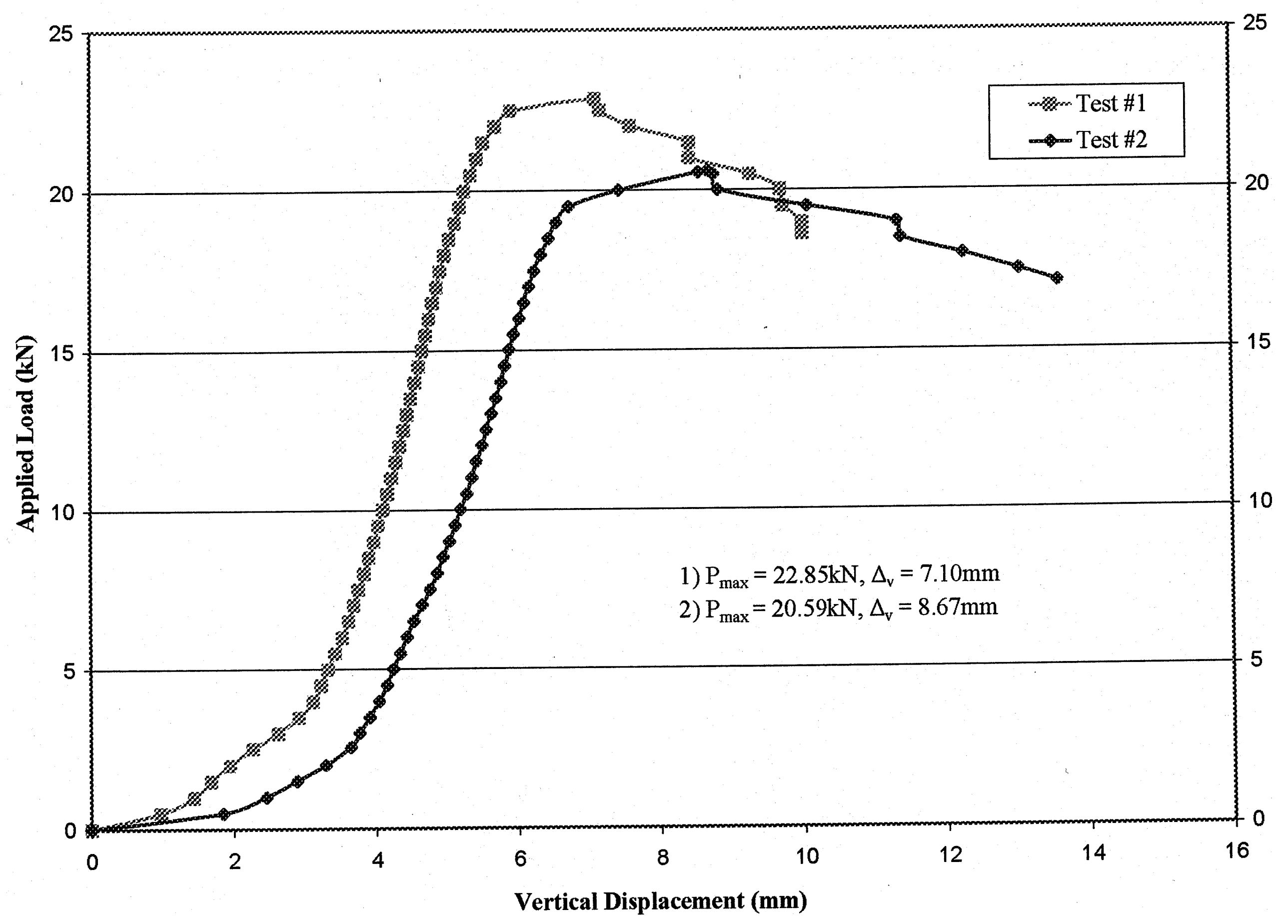


Fig. 4.14 Load-vertical displacement relationship for a specimen in group A with bearing length of 92.1 mm

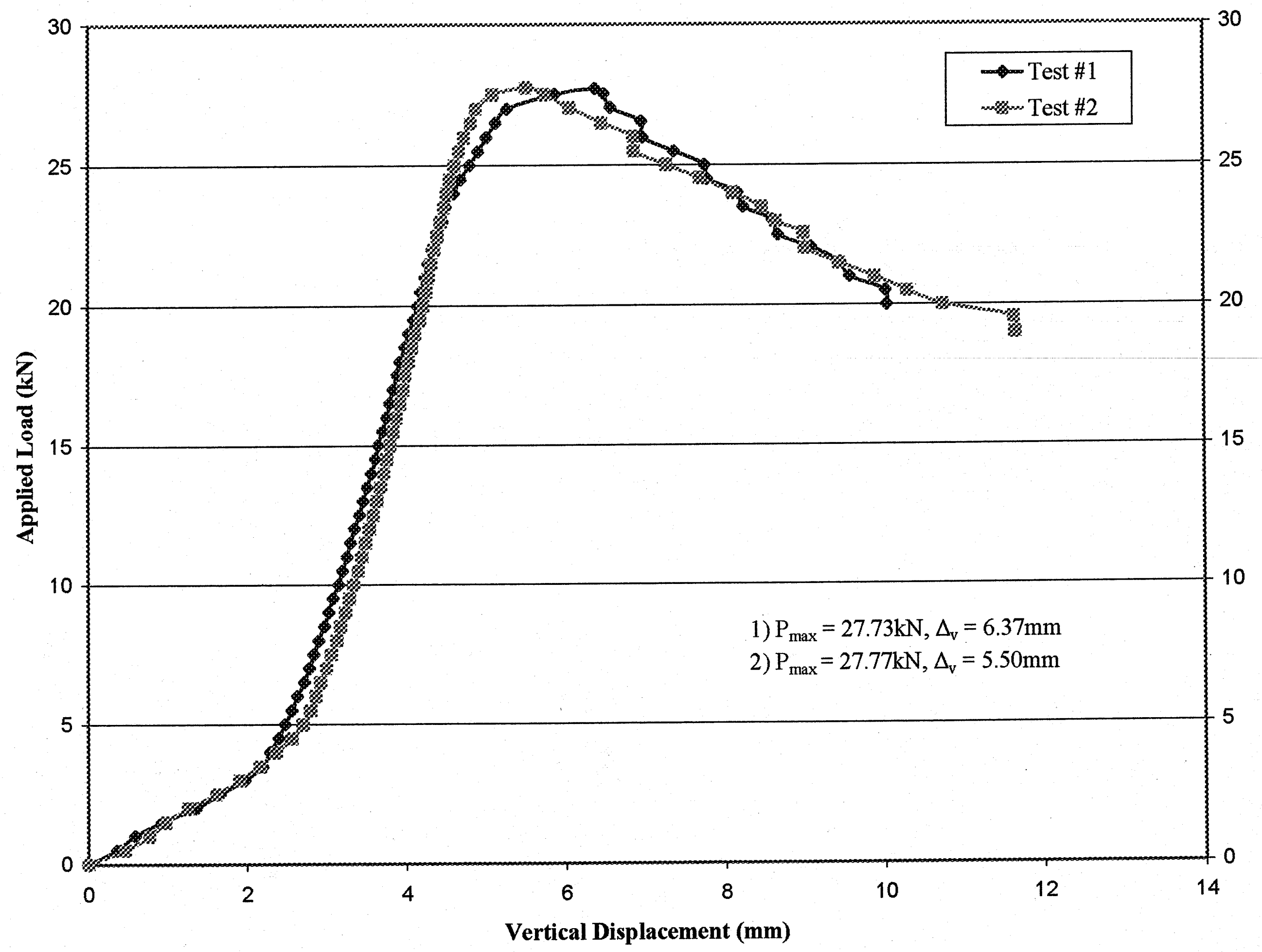


Fig. 4.15 Load-vertical displacement relationship for a specimen in group A with bearing length of 152.40 mm

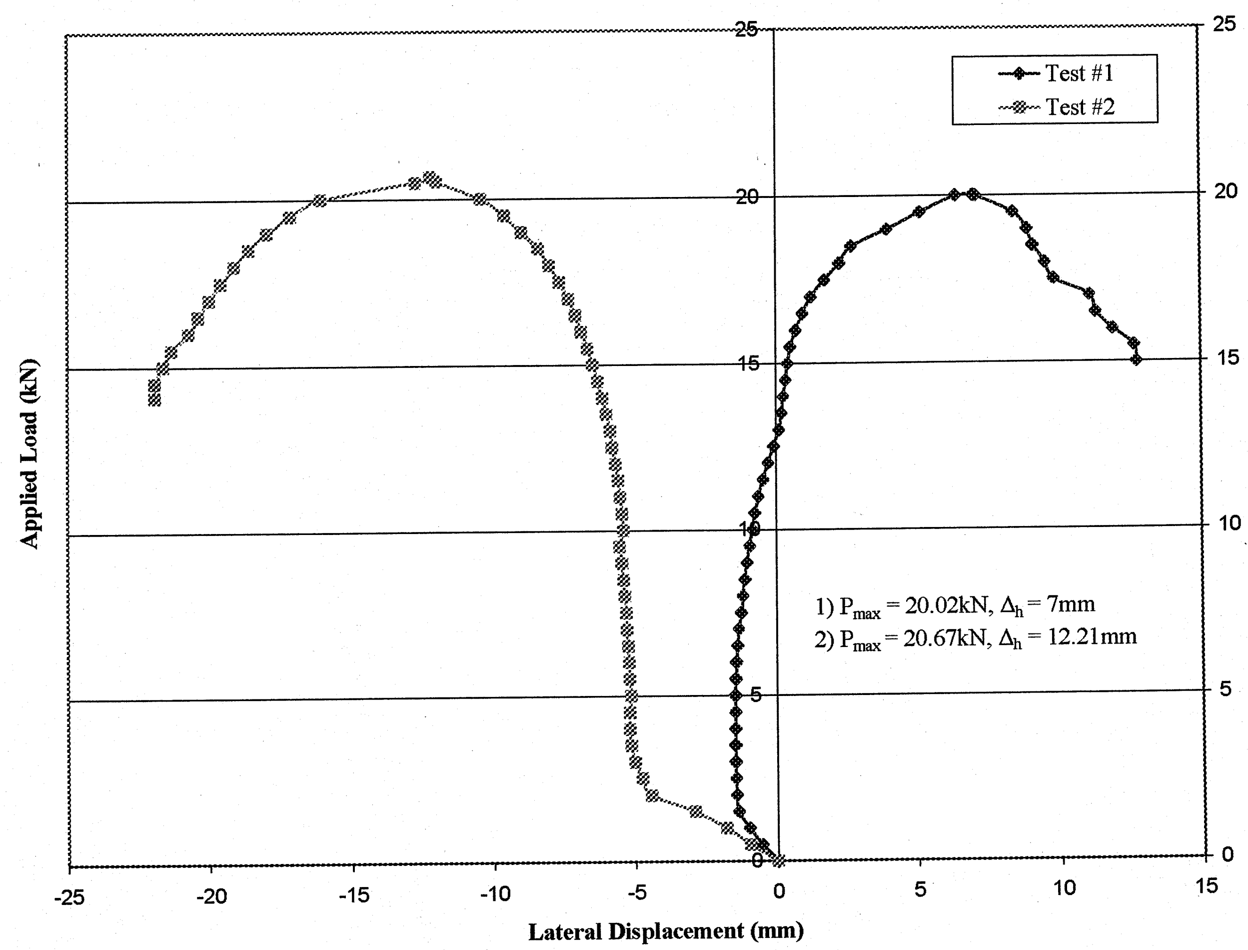


Fig. 4.16 Load-lateral displacement relationship for a specimen in group A with bearing length of 63.5 mm

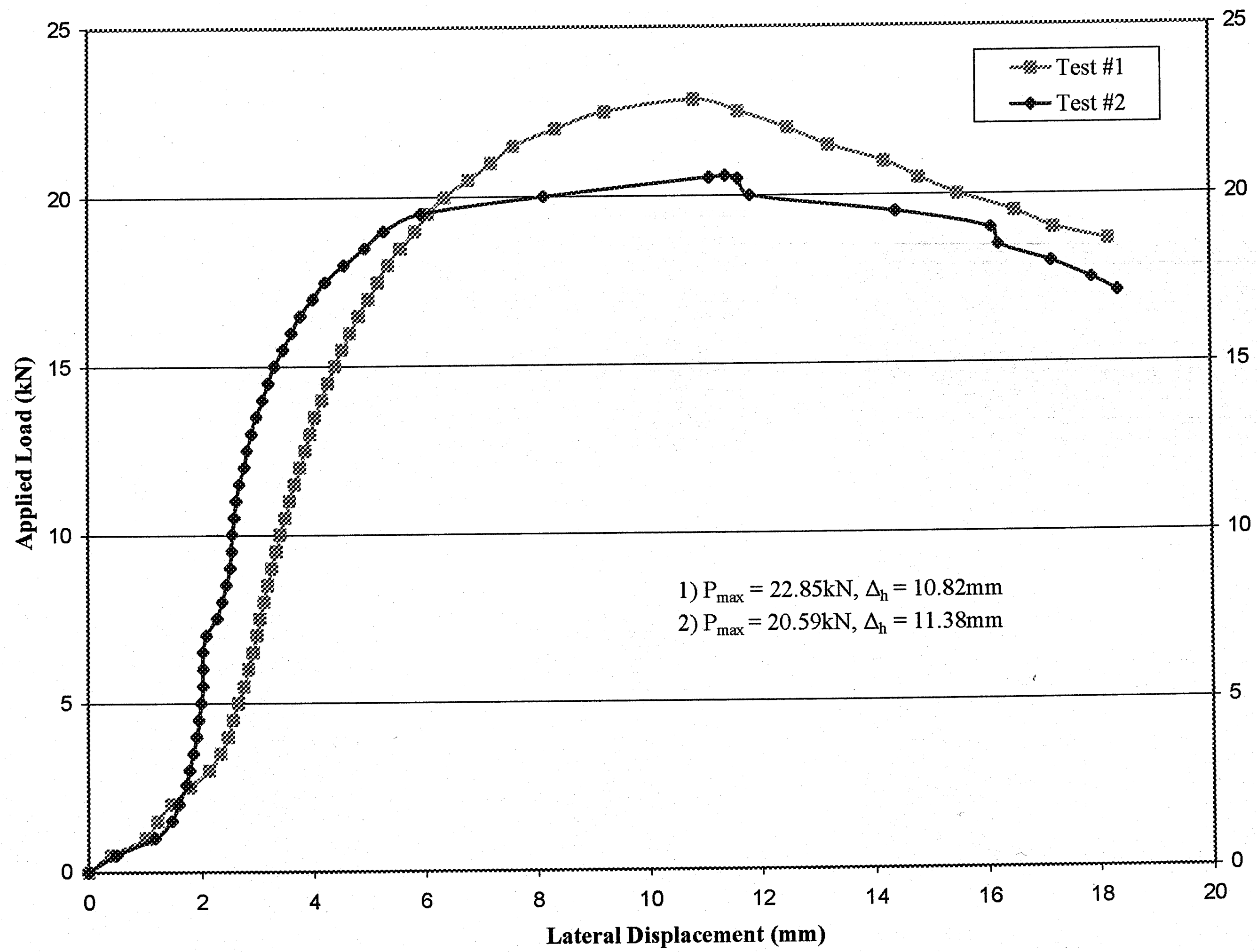


Fig. 4.17 Load-lateral displacement relationship for a specimen in group A with bearing length of 92.1 mm

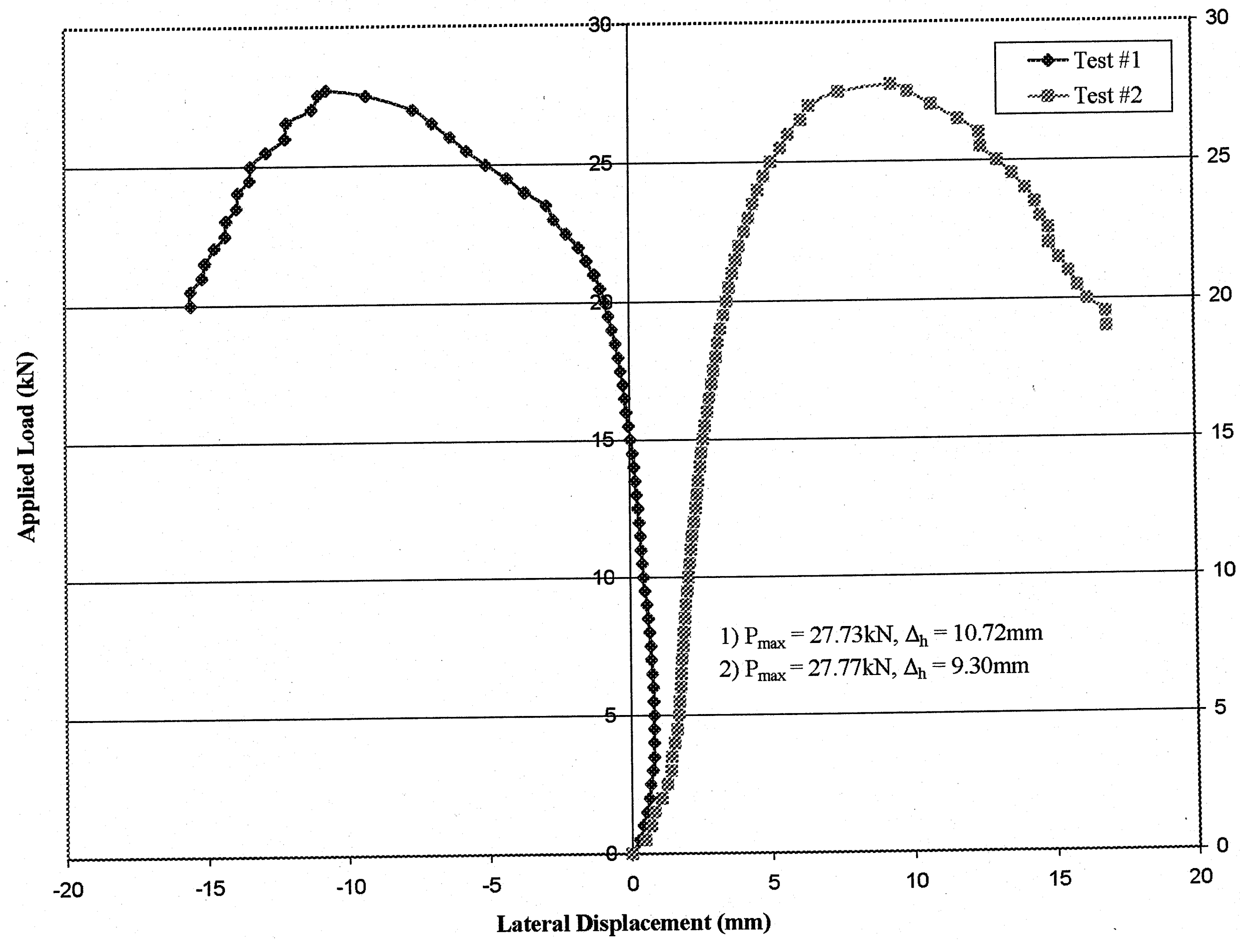
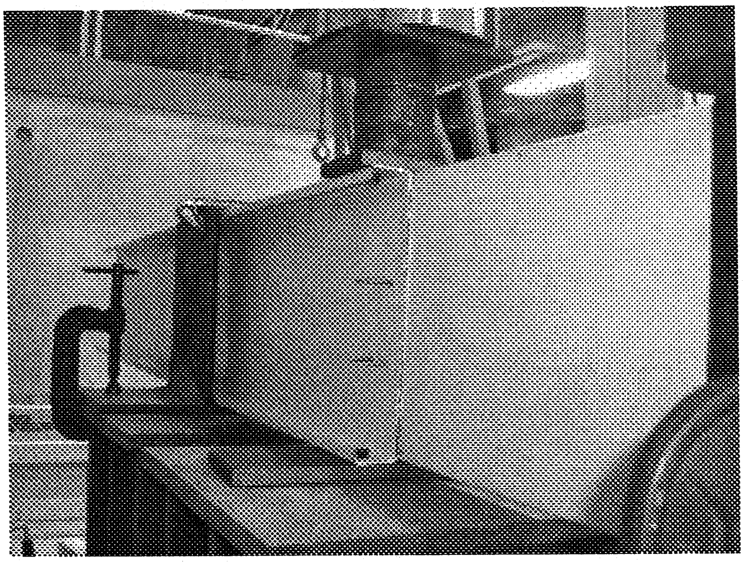


Fig. 4.18 Load-lateral displacement relationship for a specimen in group A with bearing length of 152.4 mm



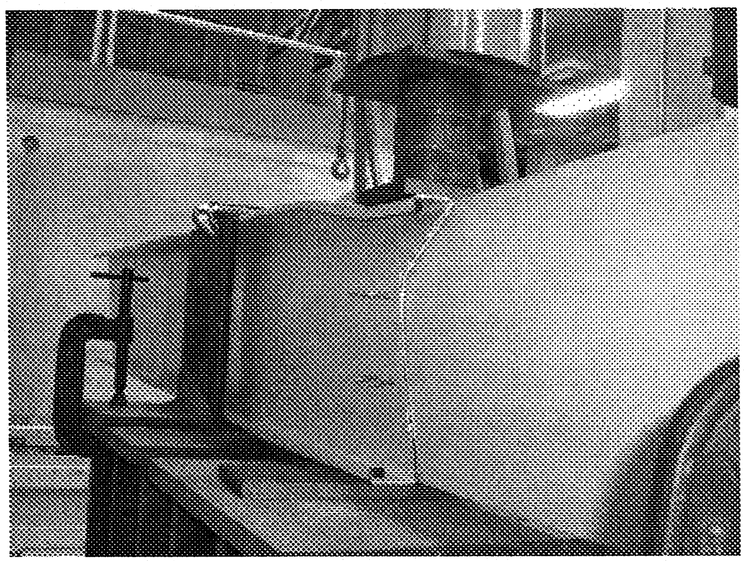


Fig. 4.19 Views of a specimen in group B before and after test

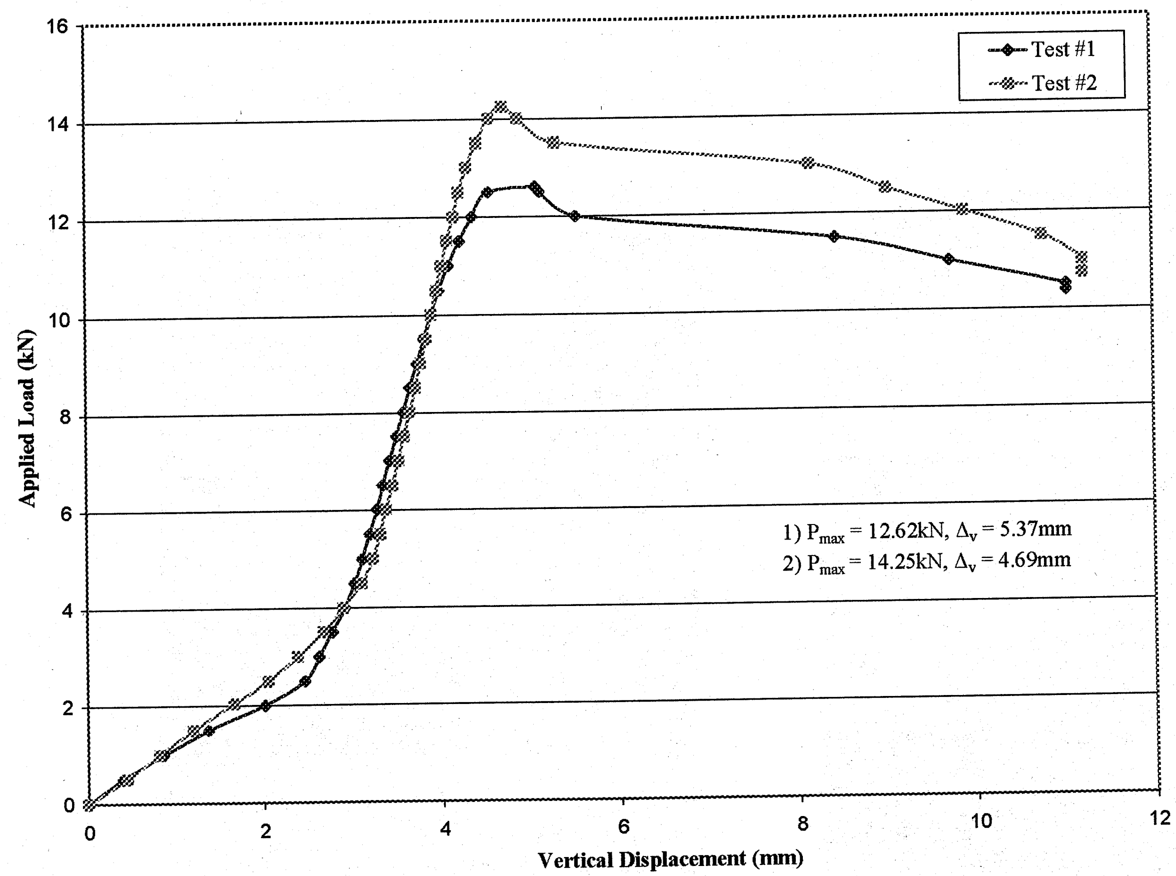


Fig. 4.20 Load-vertical displacement relationship for a specimen in group B with bearing length of 63.50 mm

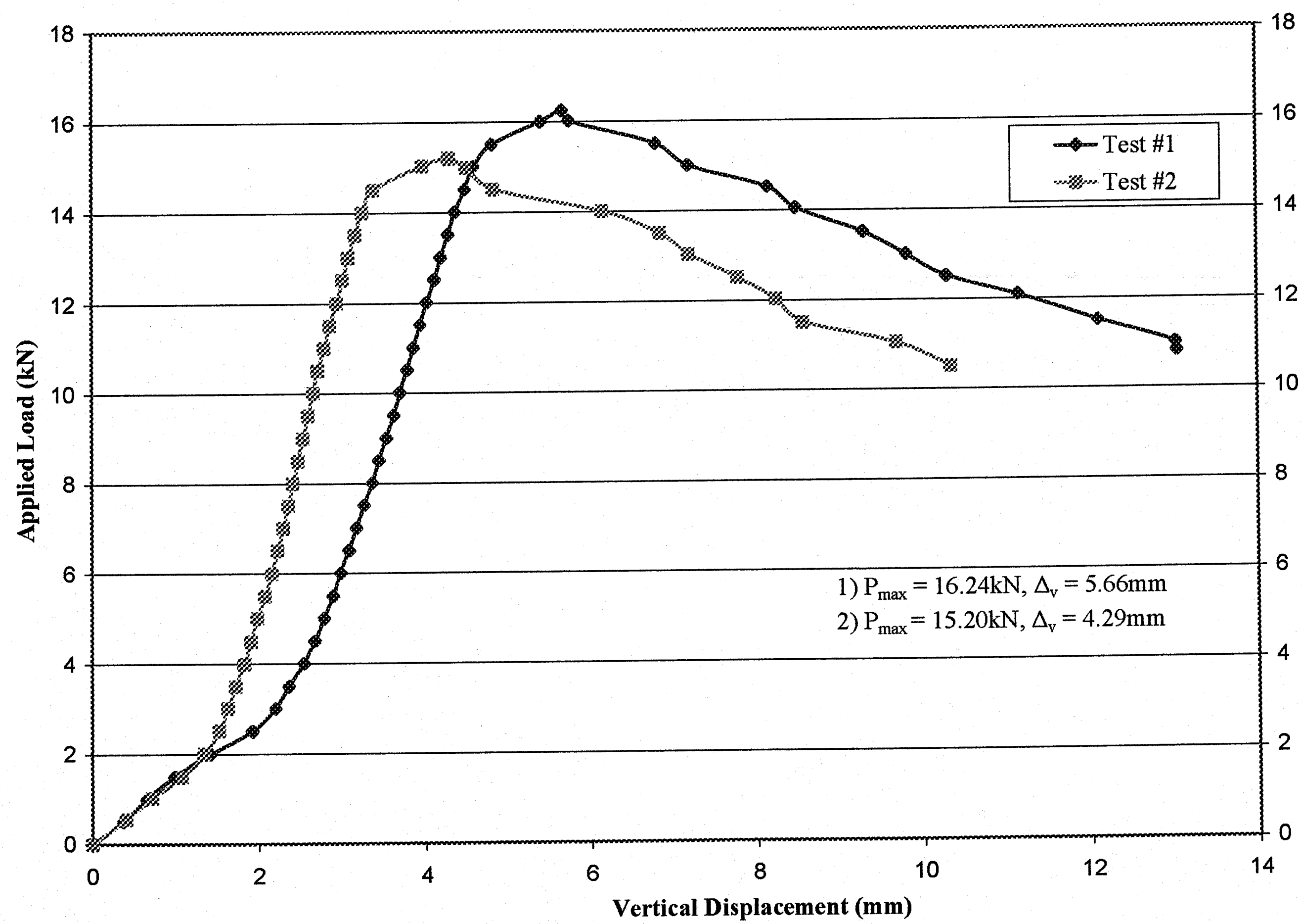


Fig. 4.21 Load-vertical displacement relationship for a specimen in group B with bearing length of 92.1 mm

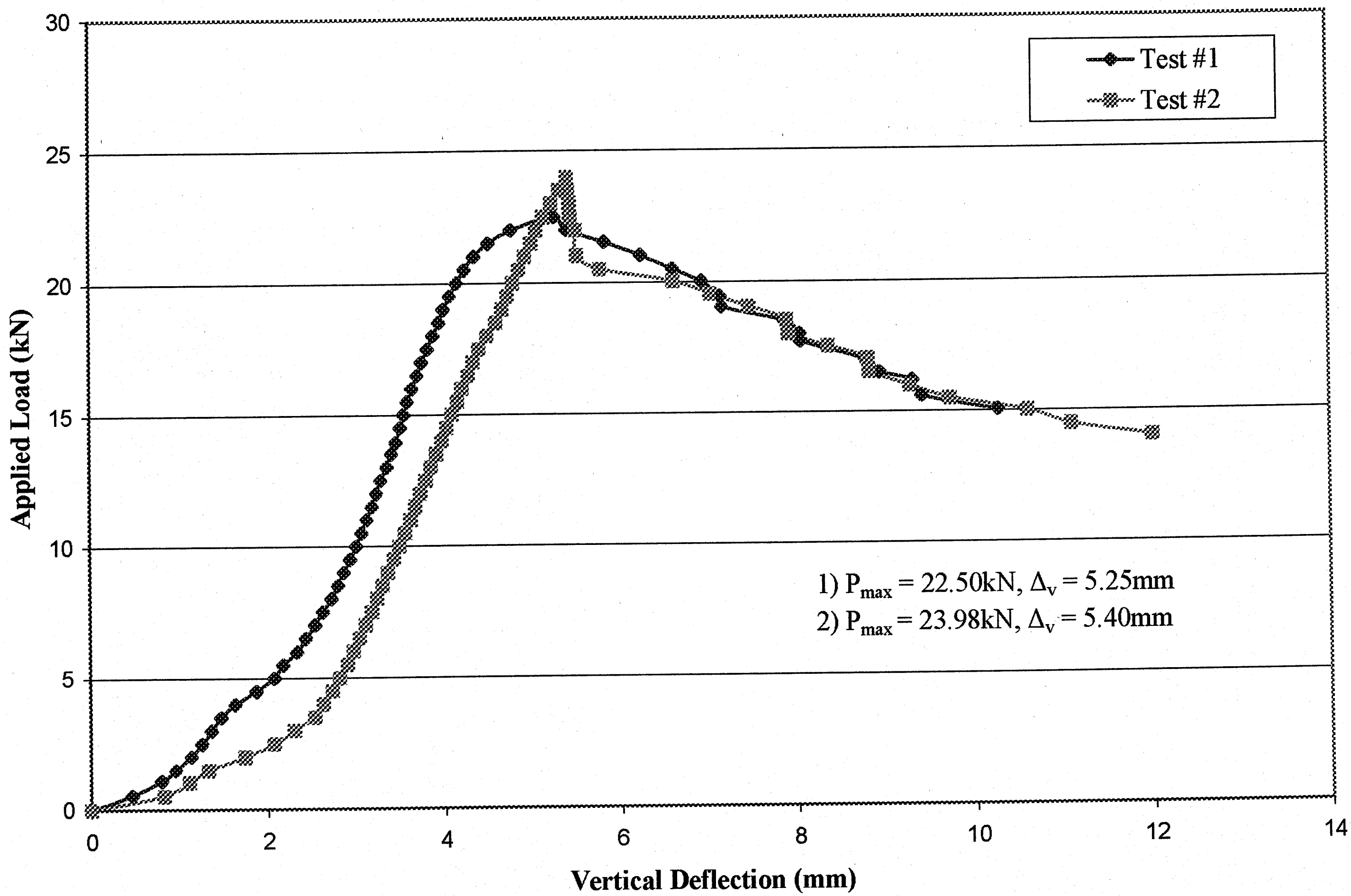


Fig. 4.22 Load-vertical displacement relationship for a specimen in group B with bearing length of 152.40 mm

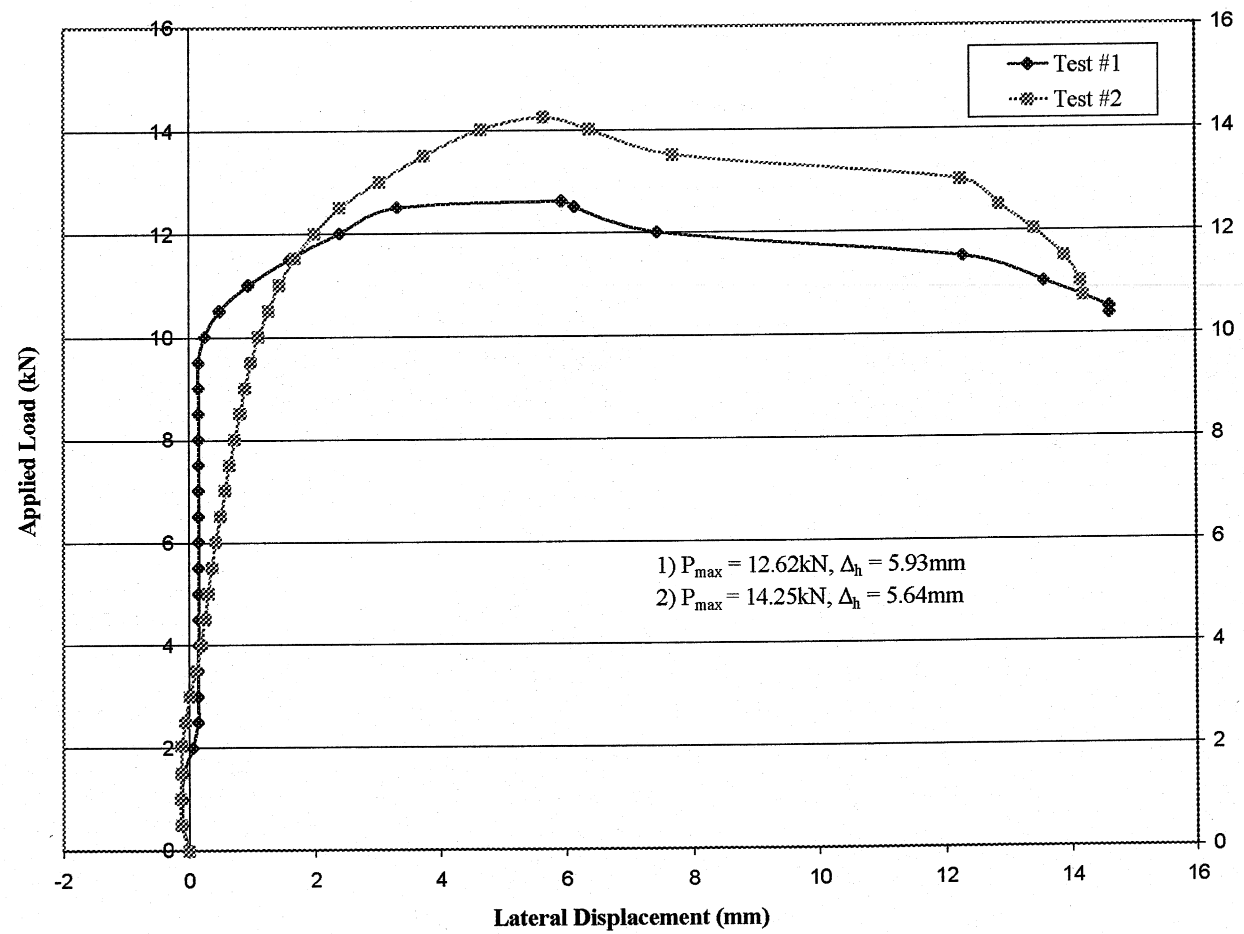


Fig. 4.23 Load-lateral displacement relationship for a specimen in group B with bearing length of 63.5 mm

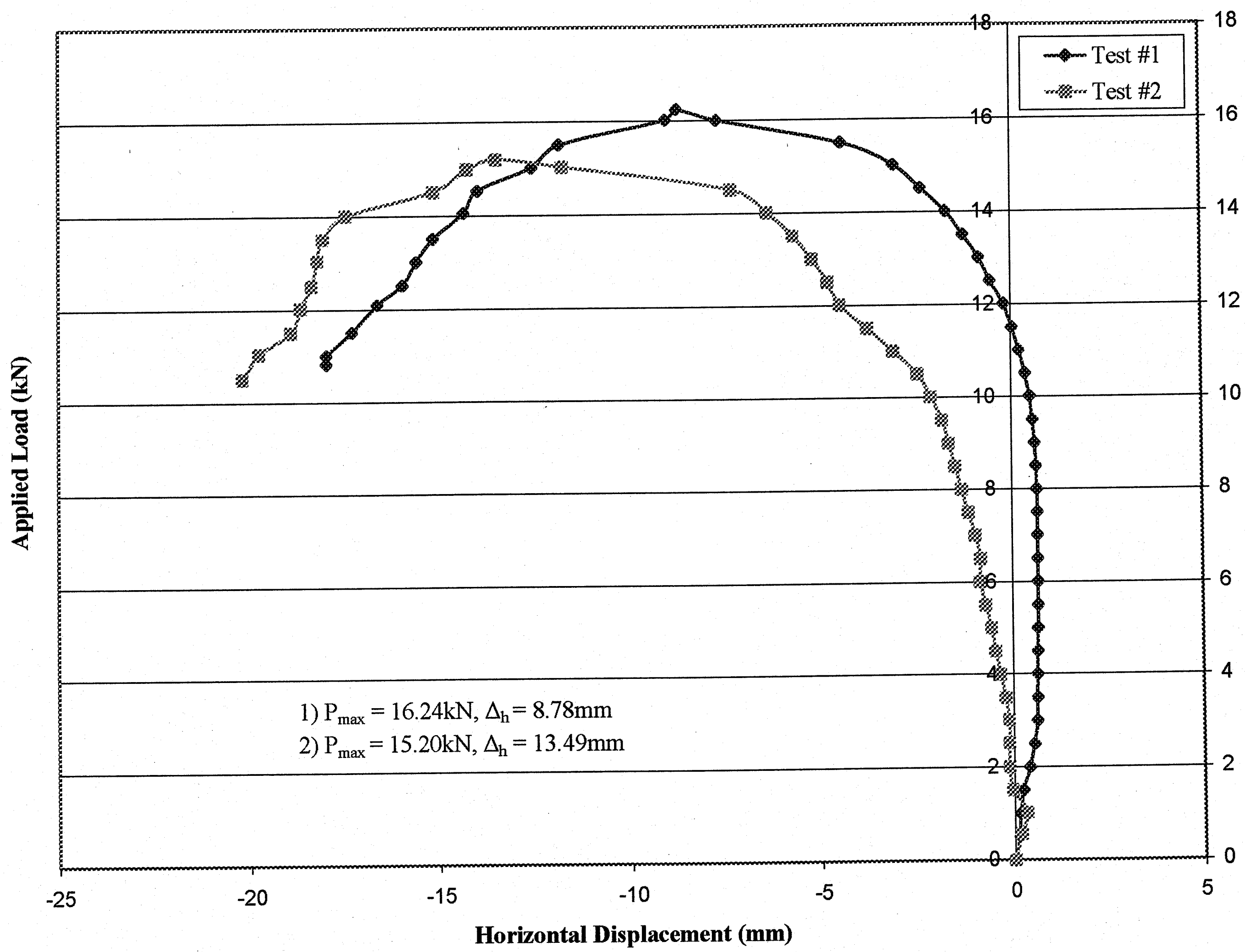


Fig. 4.24 Load-lateral displacement relationship for a specimen in group B with bearing length of 92.1 mm

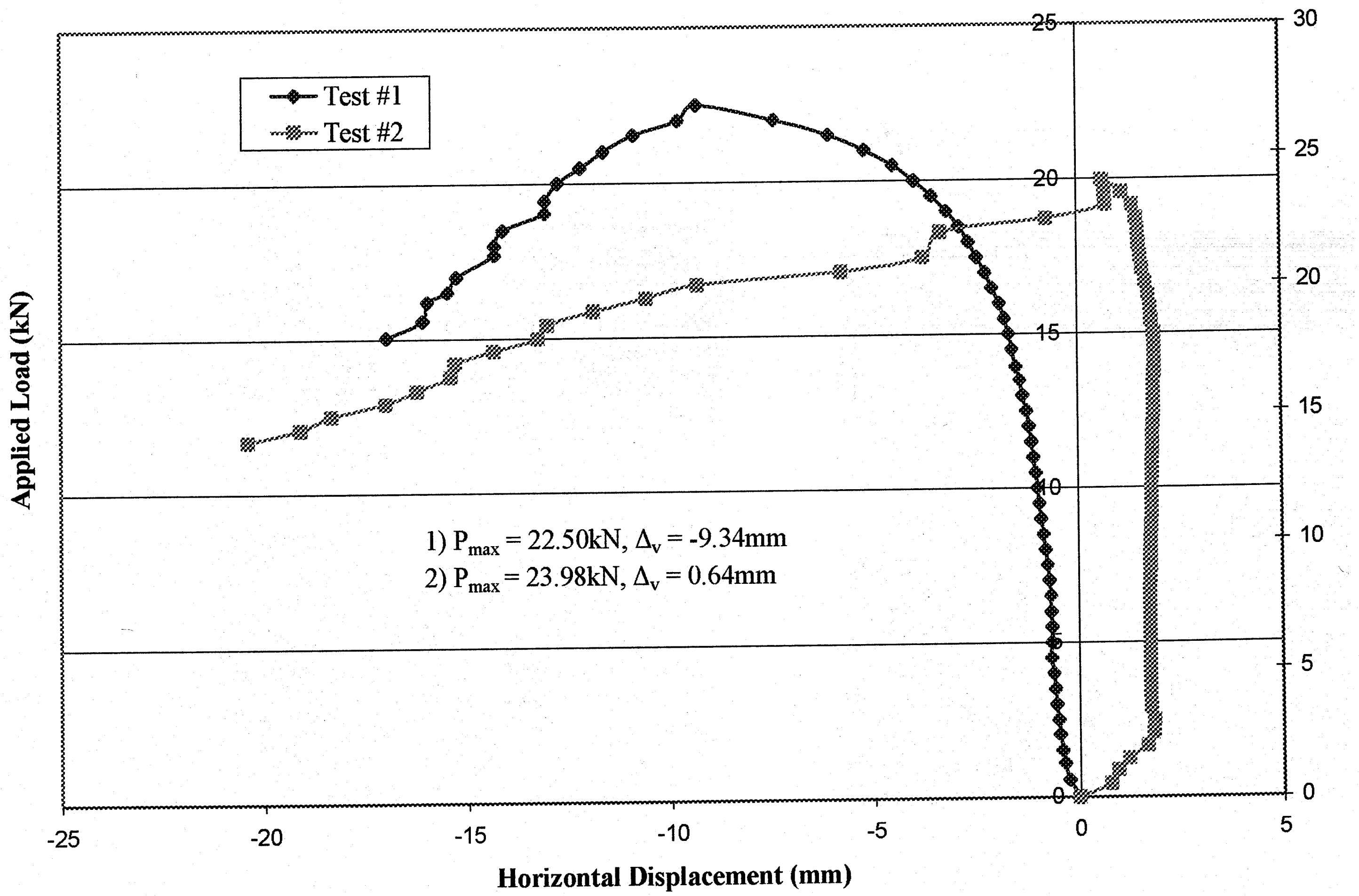
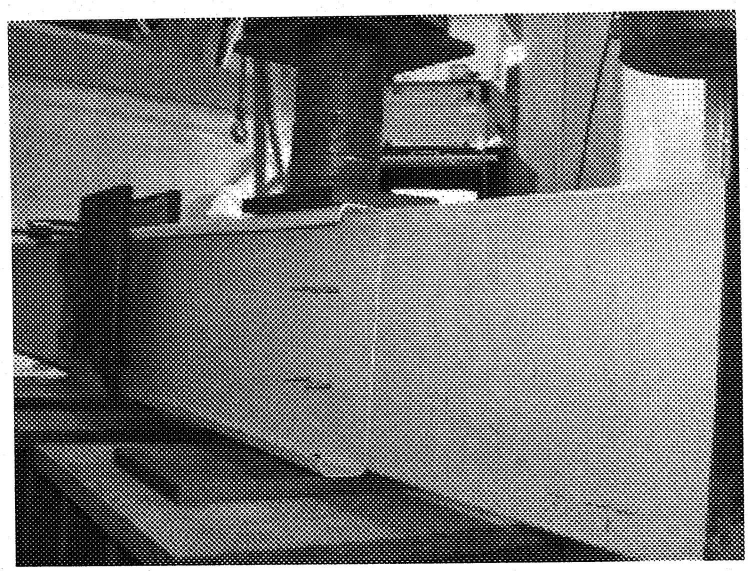


Fig. 4.25 Load-lateral displacement relationship for a specimen in group B with bearing length of 152.4 mm



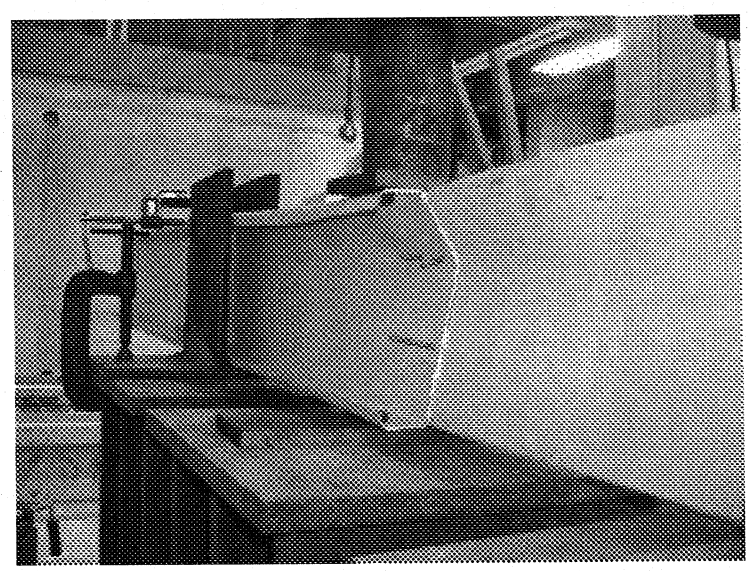


Fig. 4.26 Views of a specimen in group C before and after test

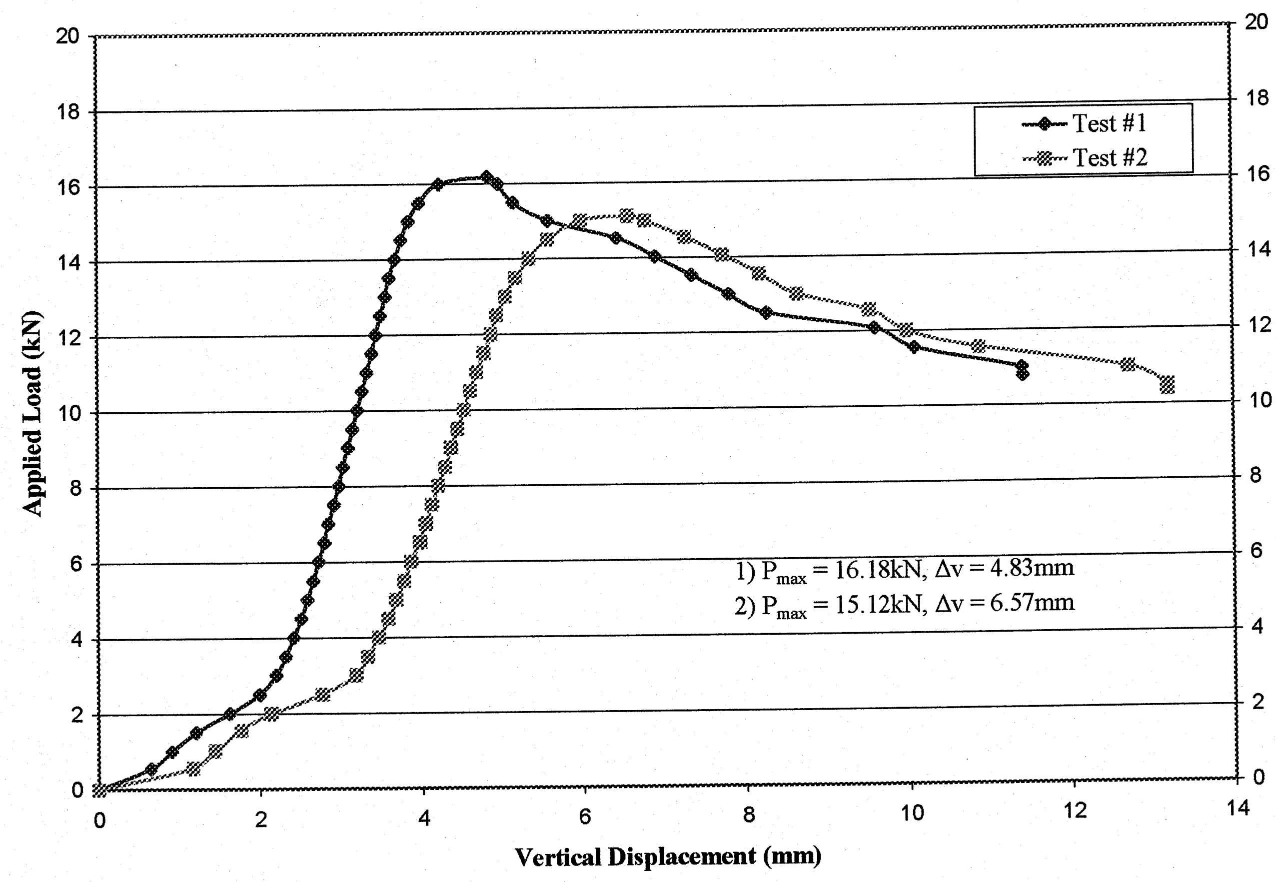


Fig. 4.27 Load-vertical displacement relationship for a specimen in group C with bearing length of 63.5 mm

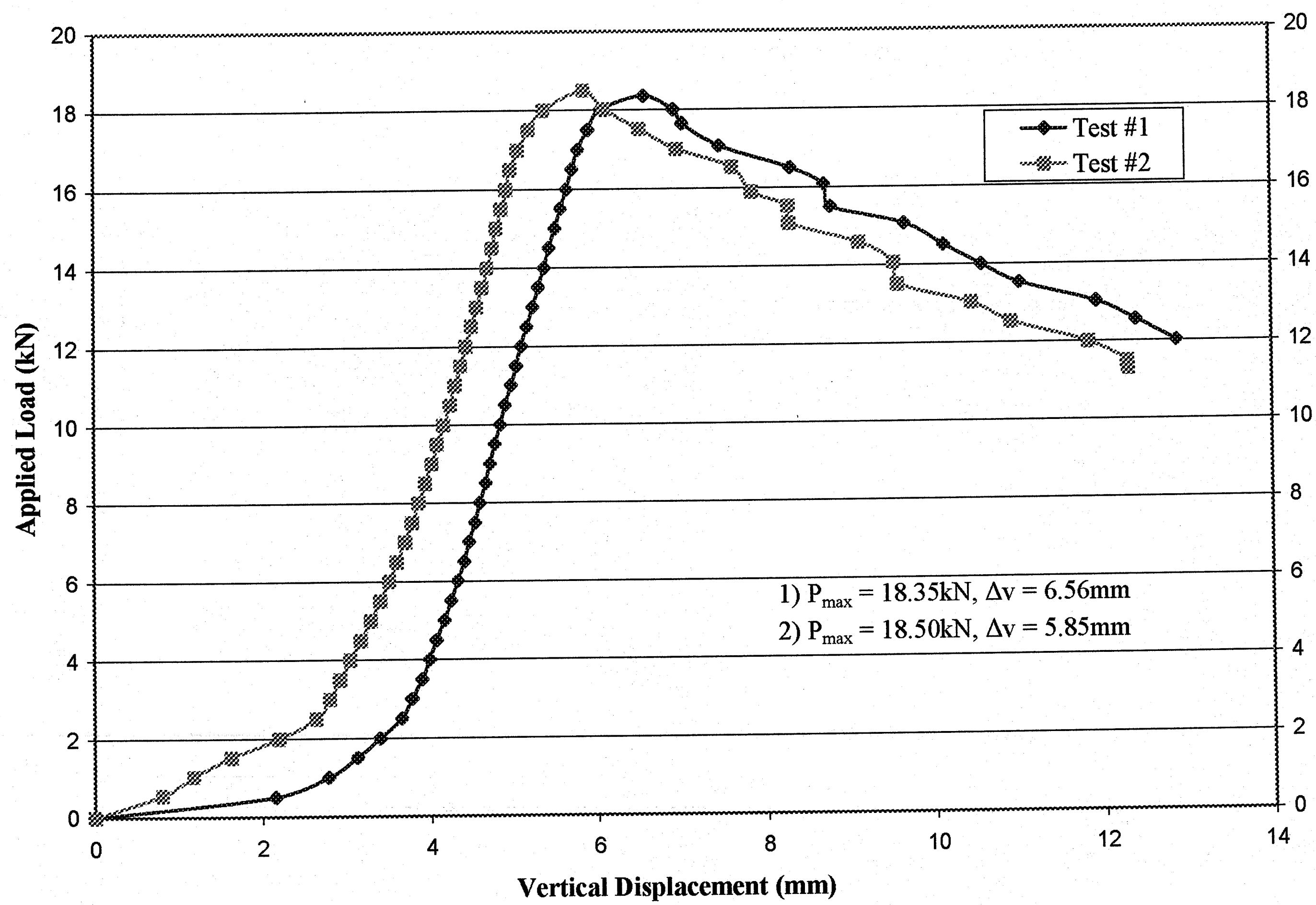


Fig. 4.28 Load-vertical displacement relationship for a specimen in group C with bearing length of 92.1 mm

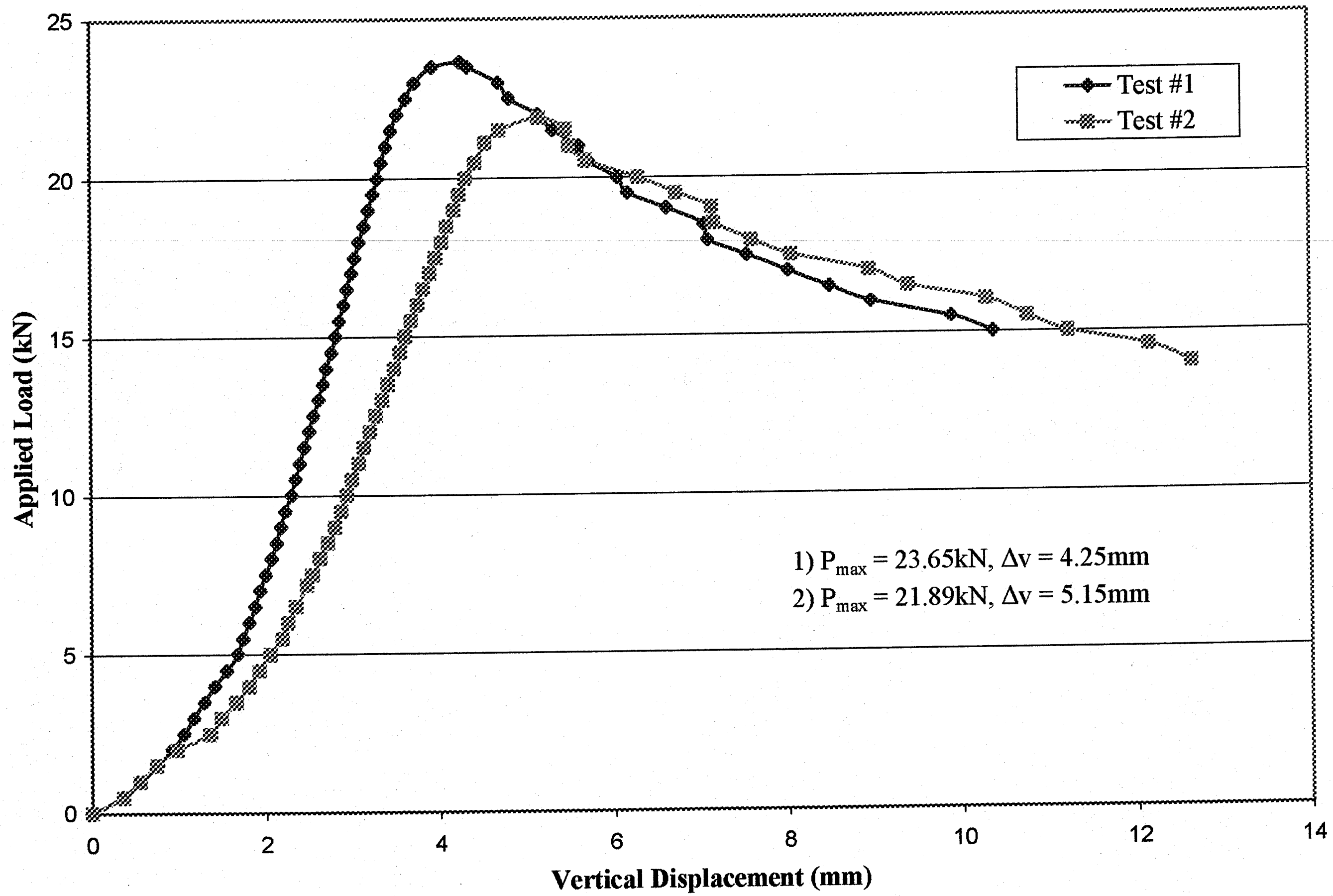


Fig. 4.29 Load-vertical displacement relationship for a specimen in group C with bearing length of 63.50 mm

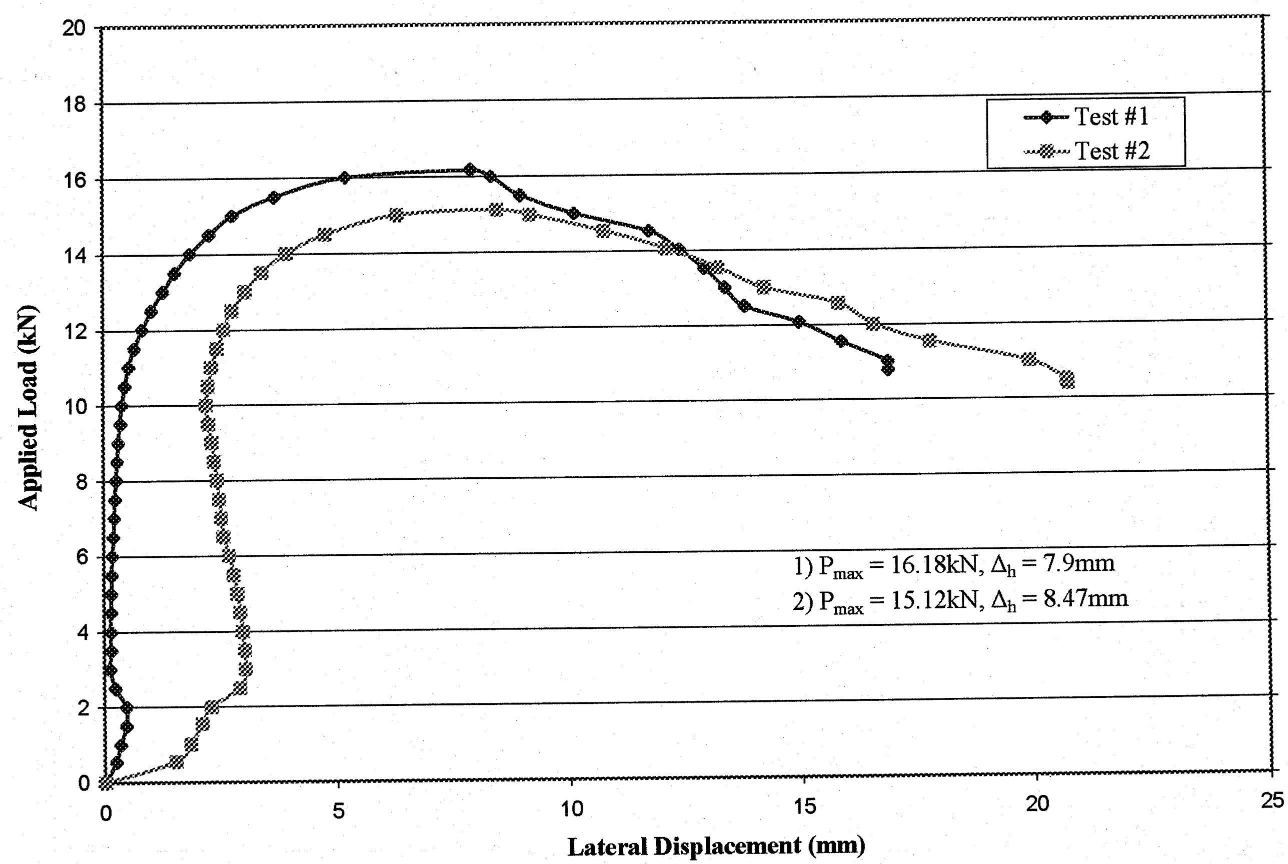


Fig. 4.30 Load-lateral displacement relationship for a specimen in group C with bearing length of 63.50 mm

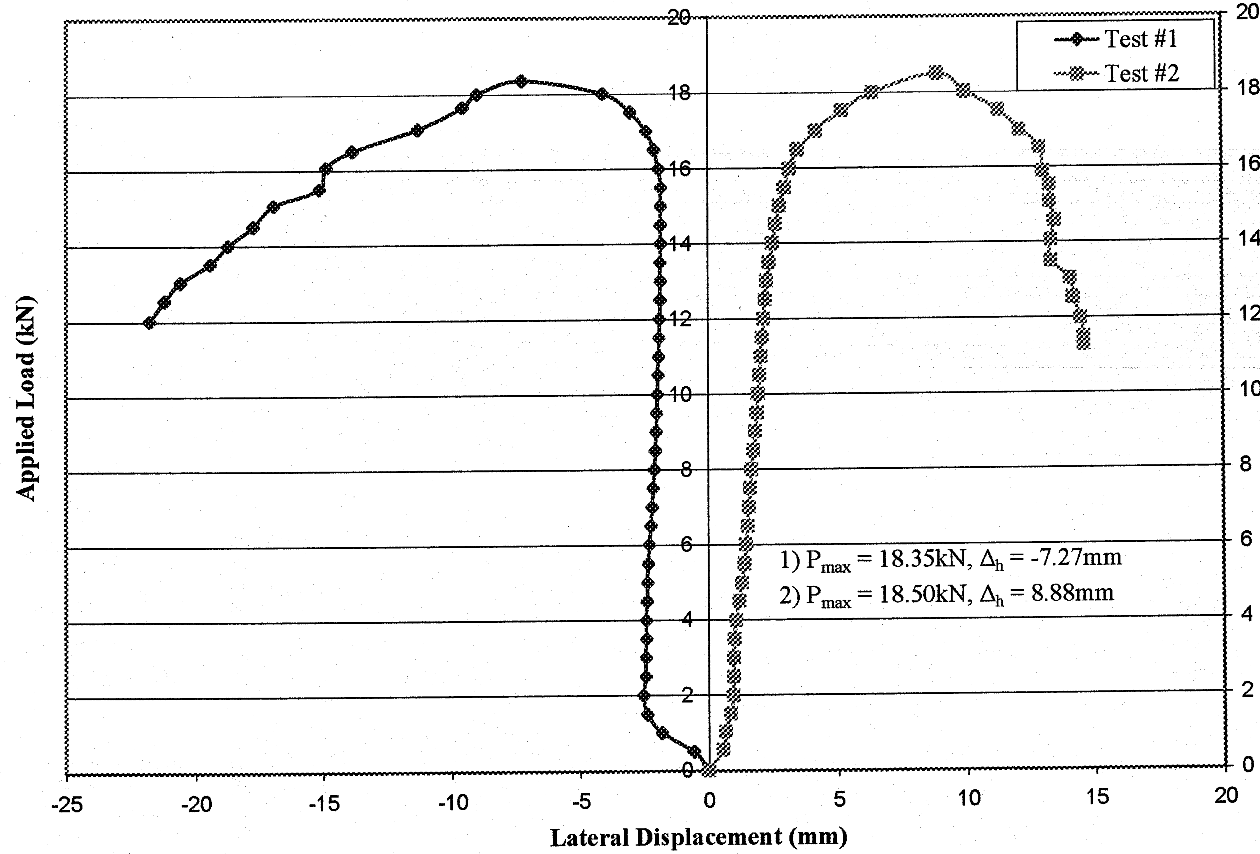


Fig. 4.31 Load-lateral displacement relationship for a specimen in group C with bearing length of 92.1 mm

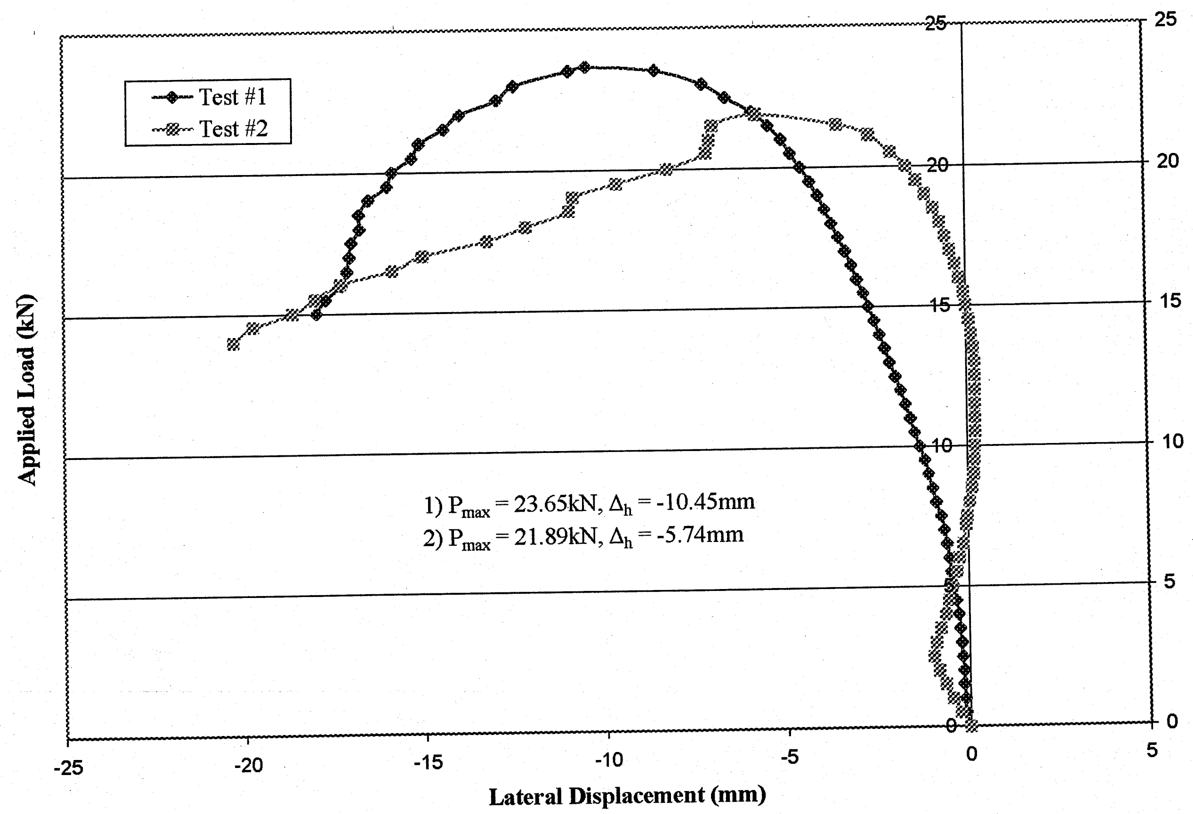
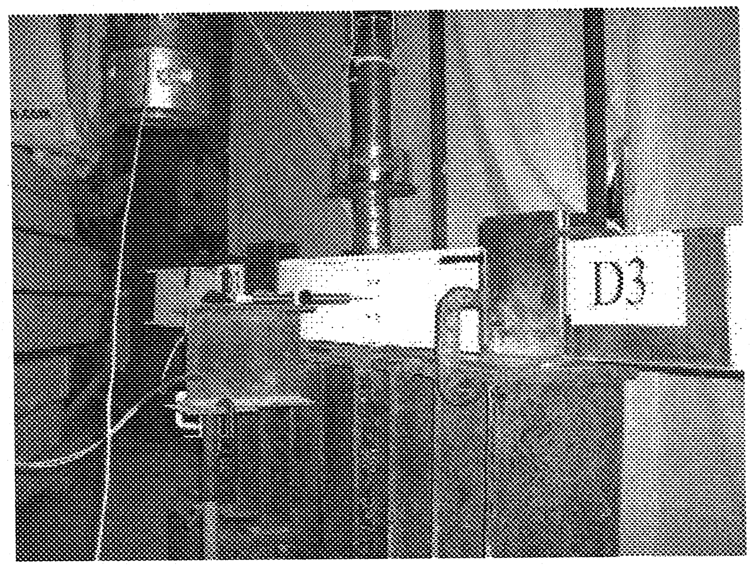


Fig. 4.32 Load-lateral displacement relationship for a specimen in group C with bearing length of 152.4 mm



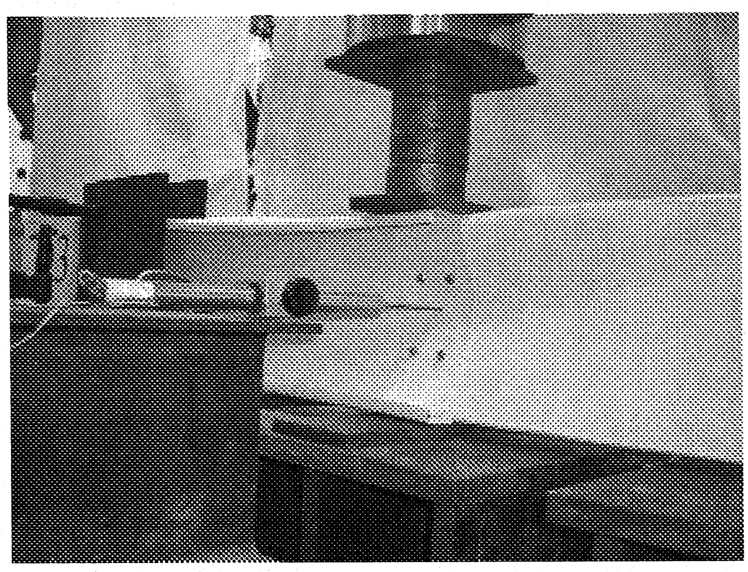


Fig. 4.33 Views of a specimen in group D before and after test

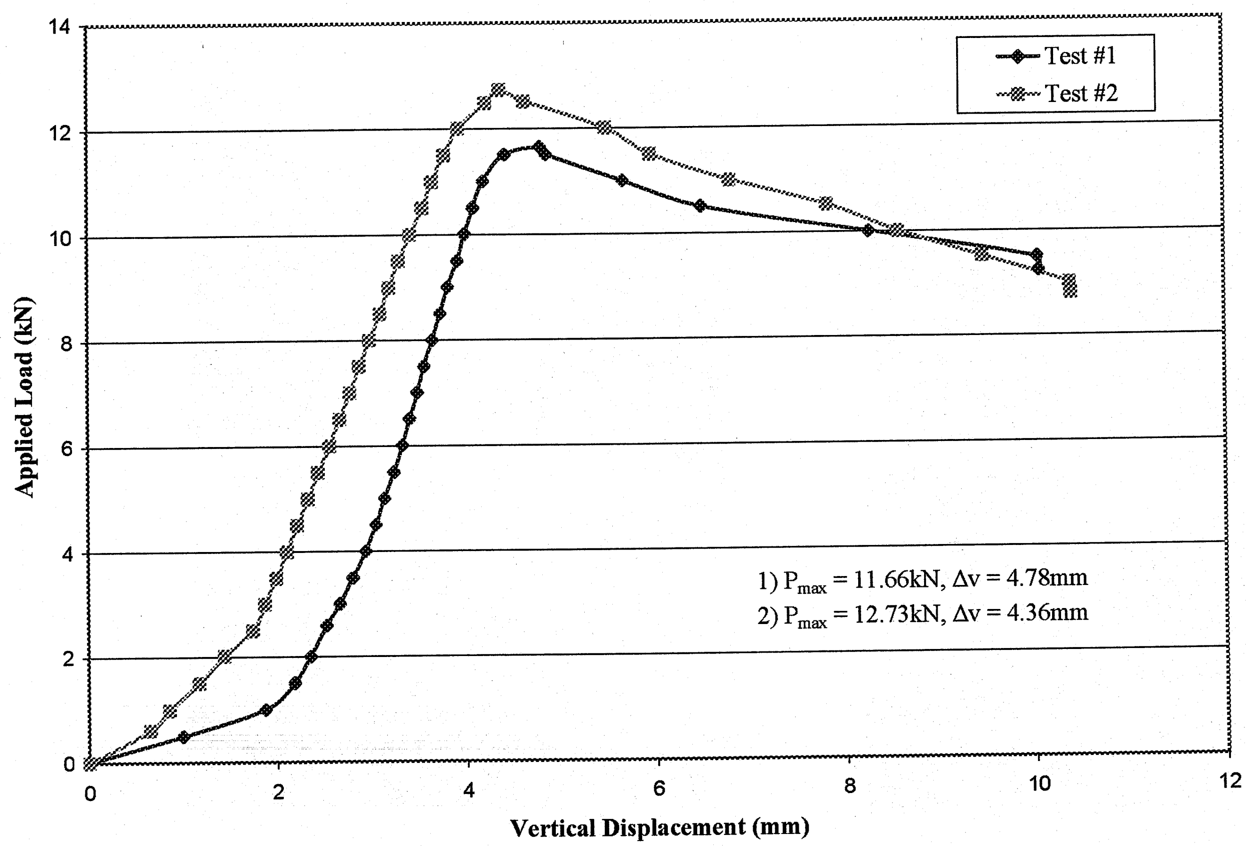


Fig. 4.34 Load-vertical displacement relationship for a specimen in group D with bearing length of 63.50 mm

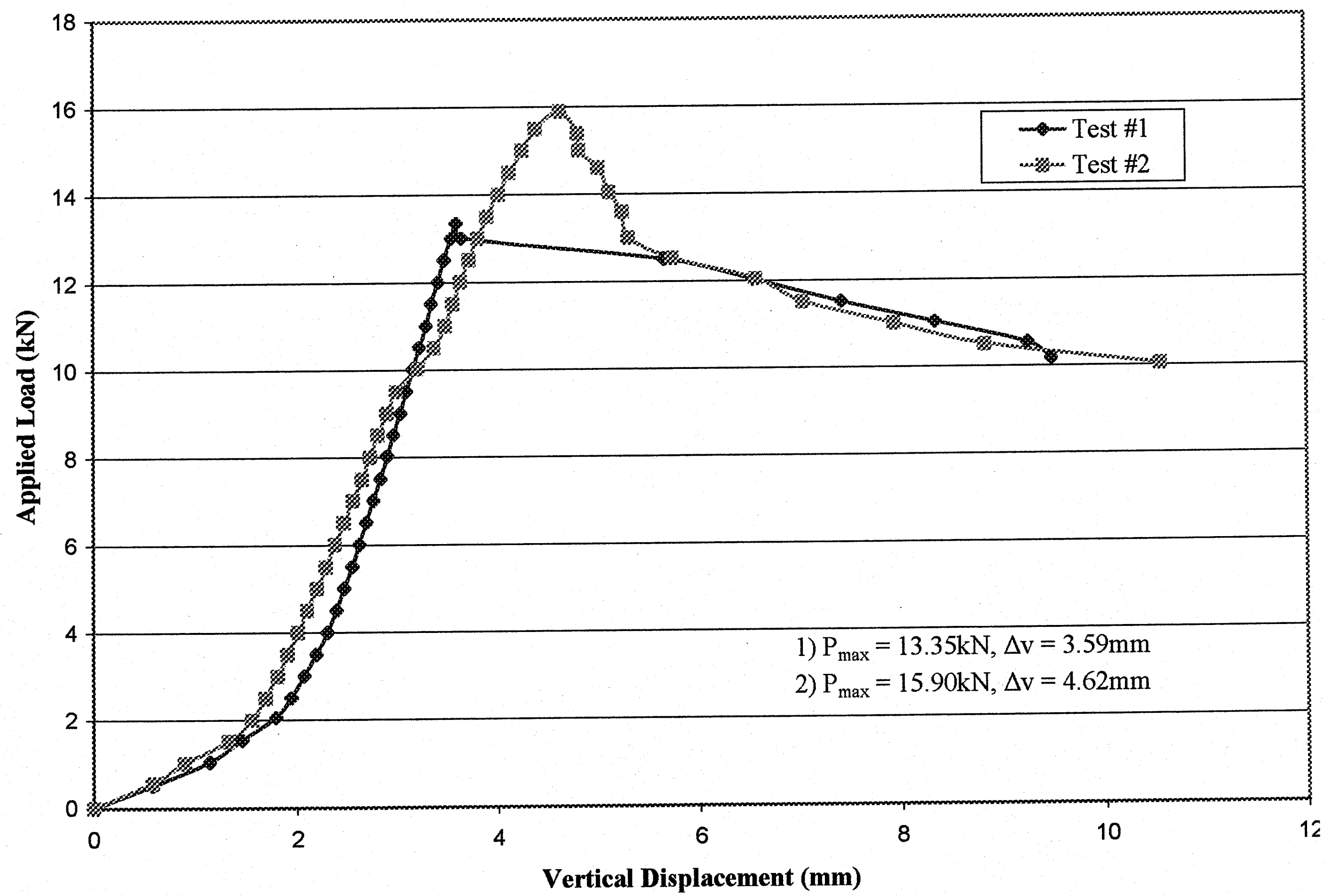


Fig. 4.35 Load-vertical displacement relationship for a specimen in group **D** with bearing length of 92.1 mm

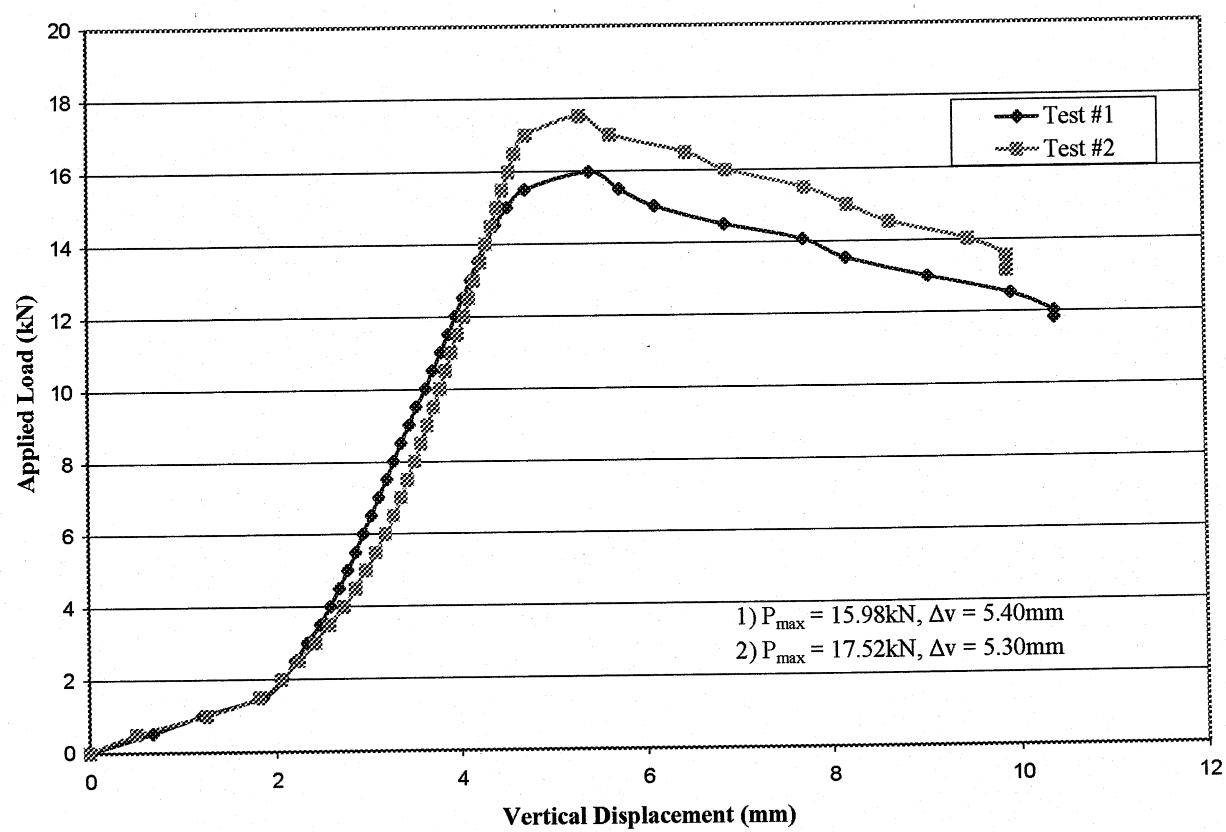


Fig. 4.36 Load-vertical displacement relationship for a specimen in group D with bearing length of 152.4 mm

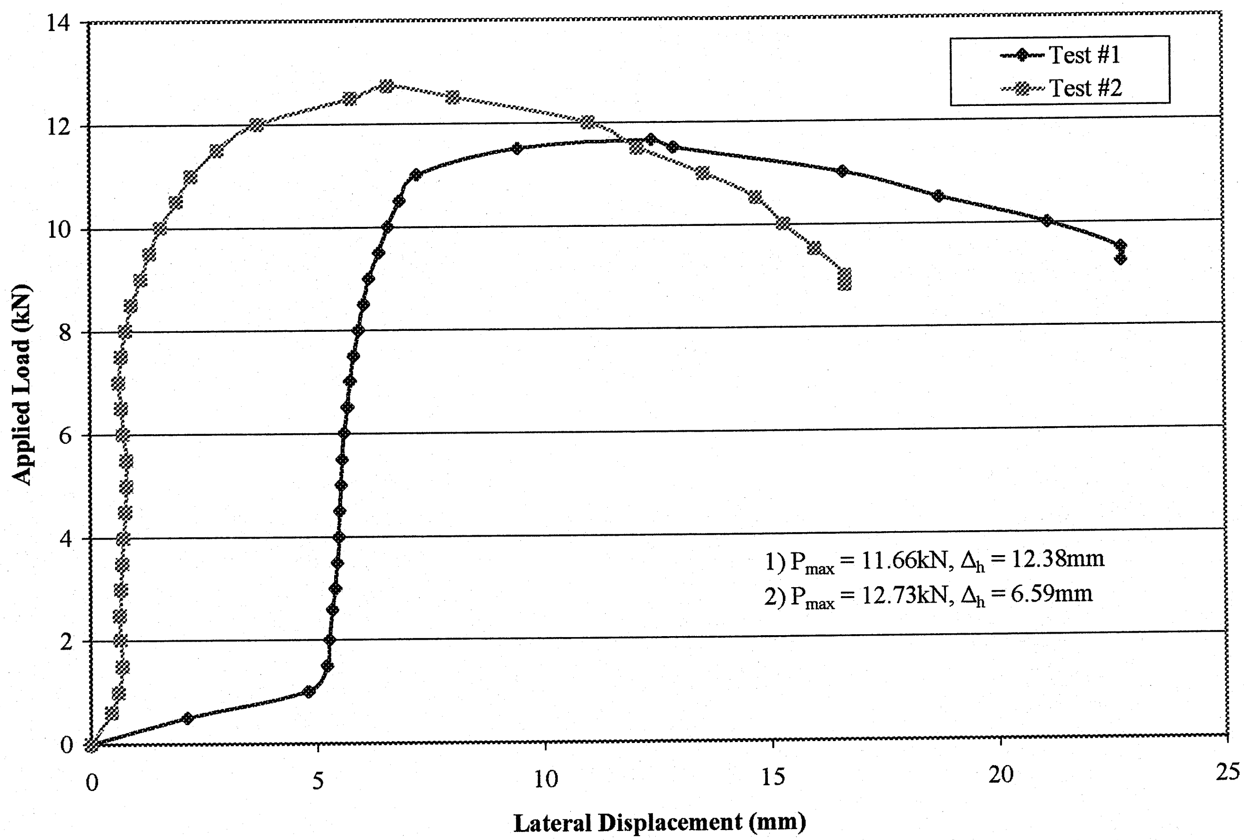


Fig. 4.37 Load-lateral displacement relationship for a specimen in group D with bearing length of 63.5 mm

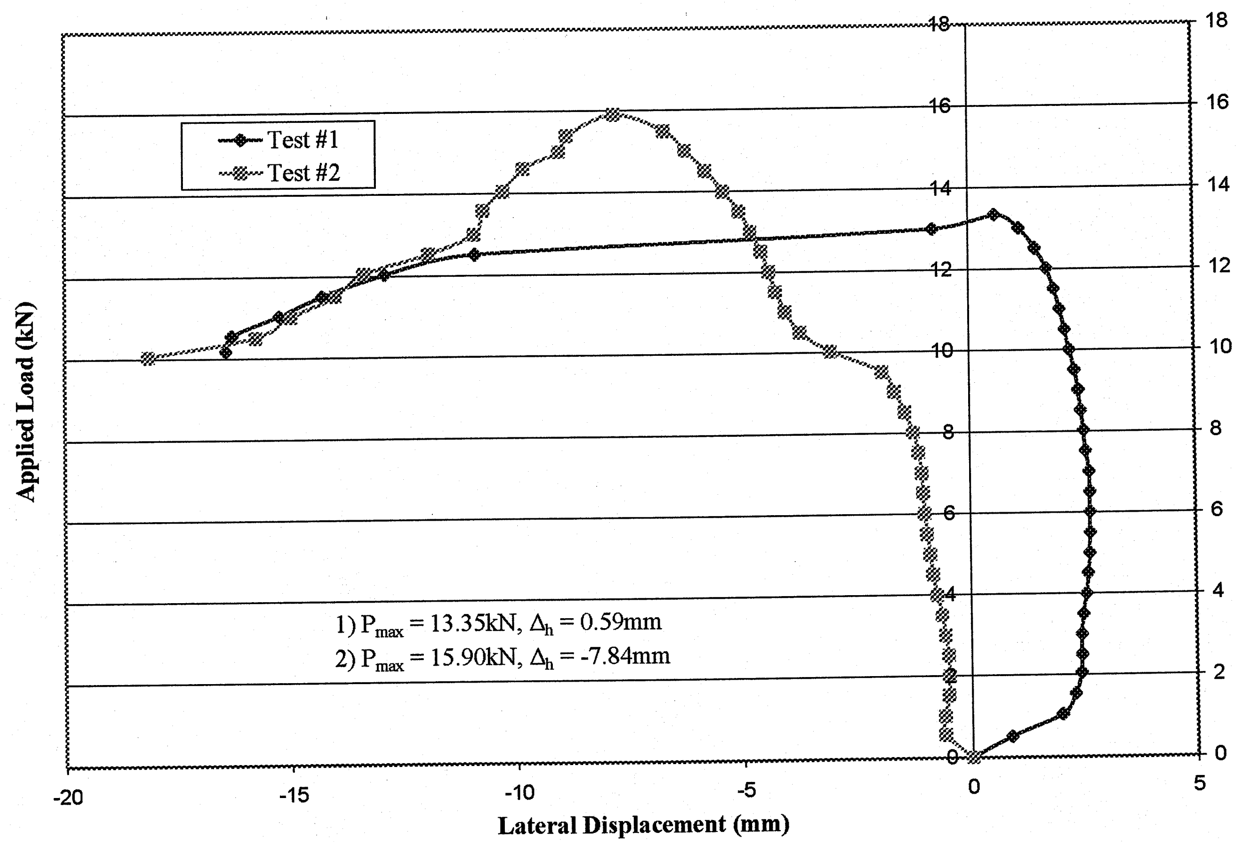


Fig. 4.38 Load-lateral displacement relationship for a specimen in group D with bearing length of 92.5 mm

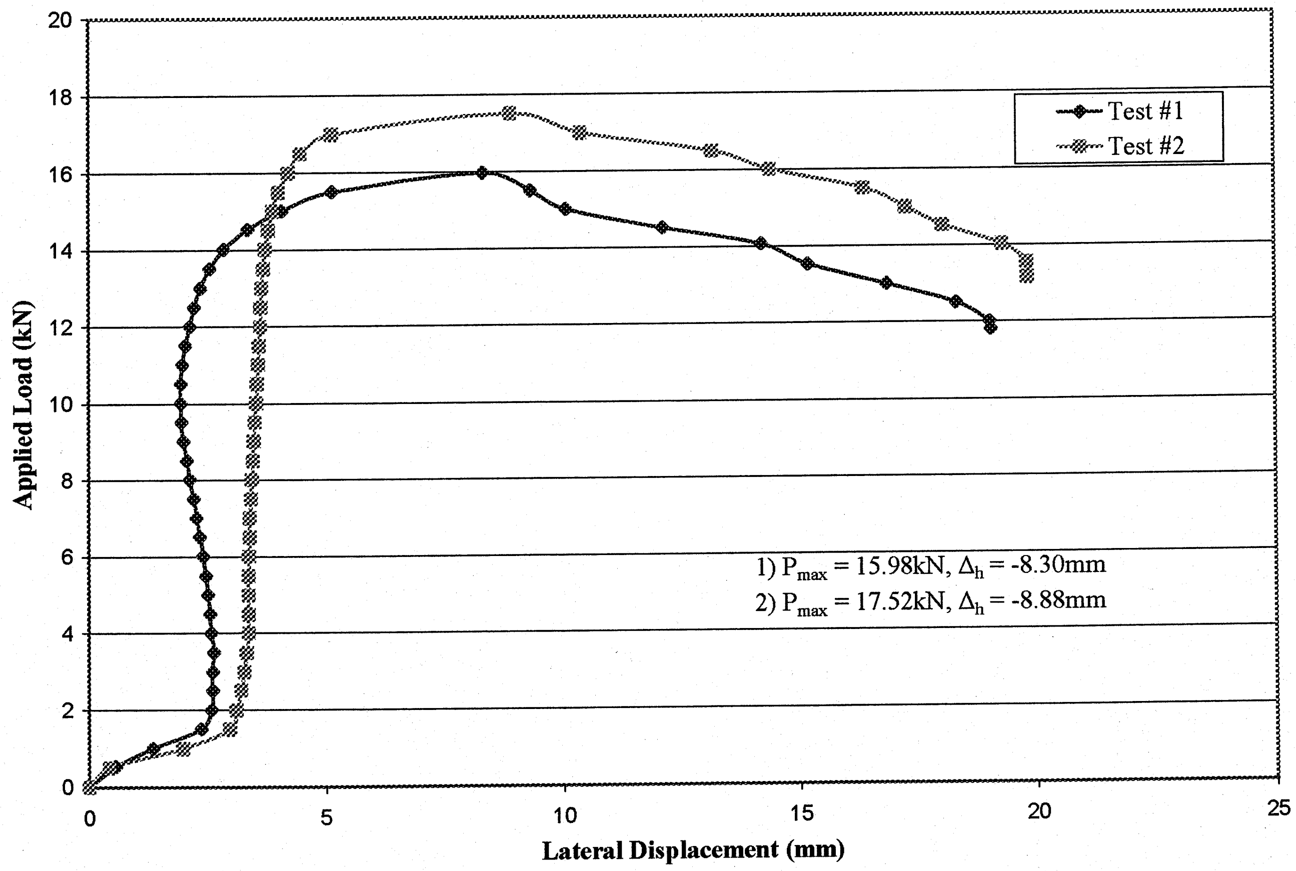


Fig. 4.39 Load-lateral displacement relationship for a specimen in group D with bearing length of 152.4 mm

CHAPTER 5

DEVELOPMENT OF NEW COEFFICIENTS

5.1 Development of New Coefficients

There are five major parameters that can affect the web crippling resistance in cold formed steel members. These five parameters are (i) web thickness, t; (ii) yield strength, F_y; (iii) web slenderness ratio, h/t; (iv) inside bend ratio, R/t; and (v) the bearing plate length to thickness ratio, N/t. The Canadian standard (CAN/CSA-S136-01) specifies the following equation to calculate web crippling strength of CFS members subjected to one-flange or two-flange loading conditions.

$$P_n = Ct^2F_y \sin\theta\left(1-C_R\sqrt{\frac{R}{t}}\right)\left(1+C_N\sqrt{\frac{N}{t}}\right)\left(1-C_h\sqrt{\frac{h}{t}}\right) \qquad (5.1)$$

Where P_n is the nominal calculated web crippling resistance and C, C_R, C_N, and C_h are correction coefficients. A nonlinear regression analysis was performed using the unified web crippling strength expression stated above to develop new coefficient for single channel lapped at the interior support and subjected to two-flange loading. To establish these new coefficients, the computer program "MinRes", developed by A. Victor Lewis, [40] was used. Table 5.1 presents the developed coefficients for the studied channel configuration and loading condition. These design coefficients produced an average experimental-to-design web crippling strength ration of 1.045, with a standard deviation of 0.114 and coefficient of variation of 0.109.

Table 5.1 New Coefficients for single channel lapped at interior support and subjected to two-flange loading

Support and Flange Conditions		Load Cases		C	C_R	C_N	C_h
Fastened To Support	Stiffened Flanges	Two Flange Loading	Interior Lap	2.5	0.02	1.01	0.001

5.2 Evaluation of the Developed Coefficients

To present the correlation between the experimental web crippling strength, Pt, and the theoretical web crippling strength, Pn, obtained from Equation (5.11) with the developed coefficients listed in Table 5.1, Fig. 5. presents the relation between the experimental and calculated values, Pt/Pn, for each specimens. It should be mentioned that Pn used to produce this graph is the average value of the two tested reported in Table 4.2. Per listed design parameters in Table 5.2, one may observe that S-136 expression with the new coefficients generally overestimate the web crippling strength by a maximum of 21% and underestimate it by a maximum of 11%.

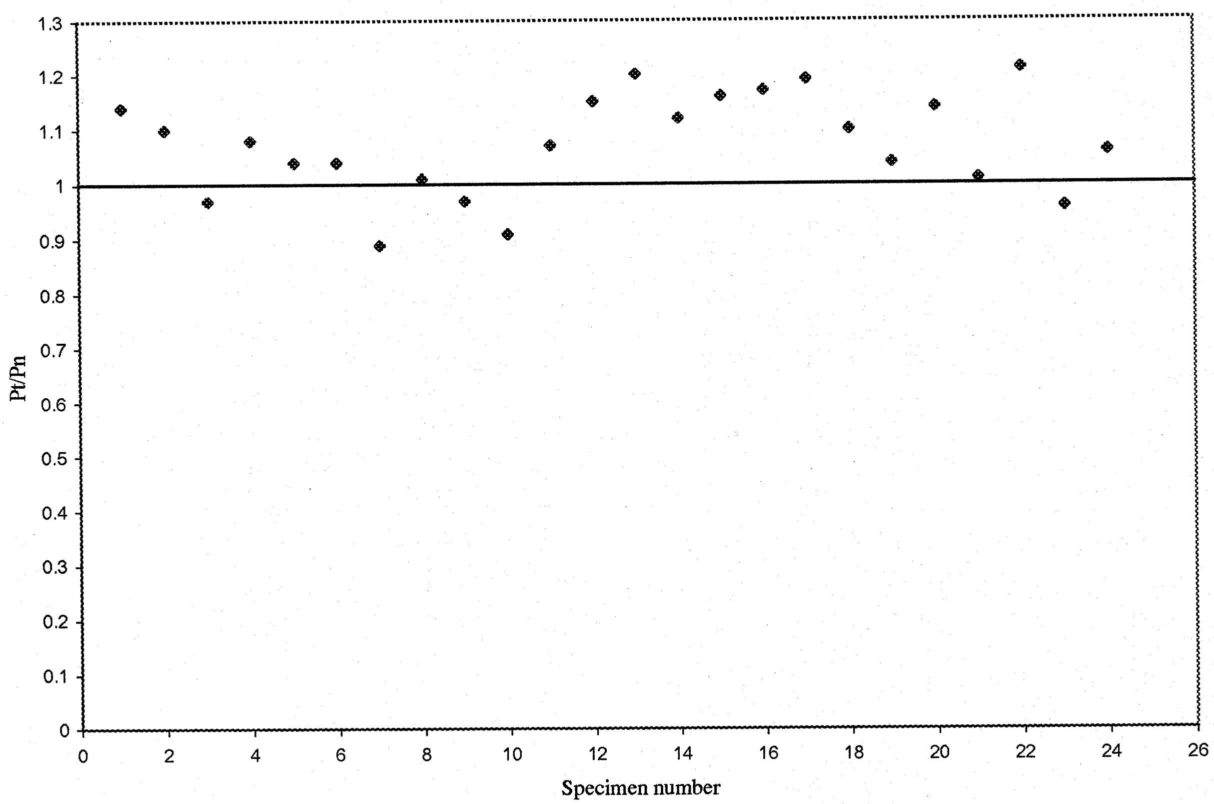


Fig. 5.1 P_t/P_n Value Vs Test Number

Table 5.2 Ratio of Experimental to Design Values, Pt/Pn

Specimen	t (mm)	h (mm)	N (mm)	R (mm)	Fy MPa	C	C_r	C_n	C_h	P_n (kN)	P_t (kN)	P_t/P_n
A	1.719	242	63.50	3.38	357.1	2.5	0.02	1.01	0.001	18.09	20.67	1.14
	1.719	242	63.50	3.38	357.1	2.5	0.02	1.01	0.001	18.09	20.02	1.10
	1.719	242	92.10	3.38	357.1	2.5	0.02	1.01	0.001	21.26	20.59	0.97
	1.719	242	92.10	3.38	357.1	2.5	0.02	1.01	0.001	21.26	23.0	1.08
	1.719	242	152.4	3.38	357.1	2.5	0.02	1.01	0.001	26.63	27.73	1.04
	1.719	242	152.4	3.38	357.1	2.5	0.02	1.01	0.001	26.63	27.77	1.04
B	1.430	246	63.50	2.77	377.1	2.5	0.02	1.01	0.001	14.30	12.62	0.89
	1.430	246	63.50	2.77	377.1	2.5	0.02	1.01	0.001	14.30	14.25	1.01
	1.430	246	92.10	2.77	377.1	2.5	0.02	1.01	0.001	16.84	16.24	0.97
	1.430	246	92.10	2.77	377.1	2.5	0.02	1.01	0.001	16.84	15.20	0.91
	1.430	246	152.4	2.77	377.1	2.5	0.02	1.01	0.001	21.13	22.50	1.07
	1.430	246	152.4	2.77	377.1	2.5	0.02	1.01	0.001	21.13	23.98	1.15

Table 5.2 Ratio of Experimental to Design Values, P_t/P_n (continued)

Specimen	t (mm)	h (mm)	N (mm)	R (mm)	Fy MPa	C	C_r	C_n	C_h	P_n (kN)	P_t (kN)	Pt/P_n
C	1.403	194	63.50	2.77	366.5	2.5	0.02	1.01	0.001	13.92	16.18	1.20
	1.403	194	63.50	2.77	366.5	2.5	0.02	1.01	0.001	13.92	15.12	1.12
	1.403	194	92.10	2.77	366.5	2.5	0.02	1.01	0.001	16.39	18.35	1.16
	1.403	194	92.10	2.77	366.5	2.5	0.02	1.01	0.001	16.39	18.50	1.17
	1.403	194	152.4	2.77	366.5	2.5	0.02	1.01	0.001	20.57	23.65	1.19
	1.403	194	152.4	2.77	366.5	2.5	0.02	1.01	0.001	20.57	21.89	1.10
D	1.196	197	63.50	1.78	389.5	2.5	0.02	1.01	0.001	11.21	11.66	1.04
	1.196	197	63.50	1.78	389.5	2.5	0.02	1.01	0.001	11.21	12.73	1.14
	1.196	197	92.10	1.78	389.5	2.5	0.02	1.01	0.001	13.23	13.35	1.01
	1.196	197	92.10	1.78	389.5	2.5	0.02	1.01	0.001	13.23	15.90	1.21
	1.196	197	152.4	1.78	389.5	2.5	0.02	1.01	0.001	16.64	15.98	0.96
	1.196	197	152.4	1.78	389.5	2.5	0.02	1.01	0.001	16.64	17.52	1.06

5.3 Calibration of the New Safety Coefficients

The objective of structural design is to produce safe, serviceable, economic, durable and aesthetic structures. Structures must be able to withstand the loads acting on them during a reasonable lifetime. North American Specification for CFS member design specifies two different design methods, namely: Limit State Design (LSD) and Allowable Stress Design (ASD). Based on a probabilistic concept, the structural safety can be measured in terms of a reliability index, β. The theory of probability can be applied to both design methods to achieve the same degree of structural safety. The following subsections elaborate on both methods of design as stated in S-136-01.

5.3.1 Limit State Design (LSD)

The use of LSD is mandatory in Canada but it is specified also in United States. It is a method in which the performance of a structure is checked against various limiting conditions at appropriate load levels. These limits are the ultimate strength limit state, and serviceability limit state. Ultimate limit state design (LRFD as known in the United States) is concerned with those states concerning safety, such as exceeding the load carrying capacity. Load factors and resisting factors are used in the proportioning of the members to minimize the risk arising from overloads and under strength elements. This relation is expressed in S136-01 [41] as follows.

$$\Sigma \gamma i Q i \leq \phi R_n \quad \text{(for LRFD)} \tag{5.1}$$

Or

$$\phi R_n \geq \Sigma \gamma i Q i \quad \text{(for LSD)} \tag{5.2}$$

Where

$\Sigma\gamma iQi$ =Required strength [effect of factored loads];

R_n =Average value of all test results;

ϕ =Resistance factor;

γi =load factors;

Q_i =load effects; and

ϕR_n=design strength.

The resistance factor, ϕ, is specified to cover the variability of material properties, dimensions, and uncertainty in the predication of the resistance and is usually less than 1. While the load factors compensate for uncertainties and variability's of the loads and usually greater than 1. The advantage of strength limit state design is that the uncertainties and variability of varies types of loads are different (e.g. dead load is less variable than wind load), and so these differences can be accounted for by the use of multiple factors, which leads to a more consistent reliability in design.

5.3.2 Allowable Stress Design (ASD)

In this method, the expected resistance of the structural member is divided by a factor of safety to obtain an allowable or working stress, and the part is then chosen so that the stress induced by the expected service load, or service load combination is equal to or less than the allowable value. The allowable design strength specified in S136-01 is calculated as:

$$R = Rn / \Omega \quad (5.3)$$

Where

Rn =Average value of all test results;

Ω =Factor of safety to be computed as: $\Omega = 1.6 / \phi$

The fundamental nature of the factor of safety is to compensate for uncertainties inherent in the design, fabrication, or erection of building components, as well as uncertainties in the estimation of the applied loads [1]. The ASD method considers only one factor of safety for a certain mode of failure, regardless of the combination of applied loads.

5.3.3 Calibration of New Values for Resistance Factors

Procedure for calculating both the resisting factor, ϕ, for load resistance factored design (LRFD), and the factor of safety, Ω, for allowable stress design (ASD), is well described in North American Specifications S136-01. The resistance factor ϕ can be calculated as follows.

$$\phi = C_{\phi}\left(M_m F_m P_m\right)e^{-\beta_0\sqrt{V_M^2+V_F^2+C_P V_P^2+V_Q^2}} \tag{5.4}$$

Where:

C_{ϕ} =Calibration coefficient and is equal to 1.52 for the United States and Mexico and 1.42 for Canada;

M_m =Mean value of material factor, M, listed in Table 5.3 for type of component involved;

F_m =Mean value of fabrication factor, F, listed in Table 5.3 for type of component involved;

P_m =Mean value of professional factor, P, for tested component =1.0;

β_o =Target reliability index, equal to 2.5 for structural members and 3.5 for connections for the United States and Mexico, and 3.0 for structural members and 4.0 for connections for Canada;

V_M = Coefficient of variation of material factor listed in Table 5.3 for type of component involved;

V_F = Coefficient of variation of fabrication factor listed in Table 5.3 for type of component involved;

C_P =Correction factor and is equal to $(1+1/n)m/(m-2)$ for $n \geq 4$, and 5.7 for $n = 3$;

m =Degrees of freedom and is equal to (n-1);

n =Number of tests;

V_P =Coefficient of variation of test results, but not less than 6.5%;

V_Q =Coefficient of variation of load effect =0.21;

e =Natural logarithmic base (2.718);

To find the factor of safety (Ω) for ASD method, the following equation can be used:

$\Omega = 1.6 / \phi$

Where: Ω =factor of safety and ϕ = calculated from equation (5.4).

TABLE 5.3 (CAN/CSA-S136-01)

Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Transverse Stiffeners	1.10	0.10	1.00	0.05
Shear Stiffeners	1.00	0.06	1.00	0.05
Tension Members	1.10	0.10	1.00	0.05
Flexural Members				
Bending Strength	1.10	0.10	1.00	0.05
Lateral Torsional Buckling Strength	1.00	0.06	1.00	0.05
One Flange Through-Fastened to Deck or Sheathing	1.10	0.10	1.00	0.05
Shear Strength	1.10	0.10	1.00	0.05
Combined Bending and Shear	1.10	0.10	1.00	0.05
Web Crippling Strength	1.10	0.10	1.00	0.05
Combined Bending and Web Crippling	1.10	0.10	1.00	0.05
Concentrically Loaded Compression Members	1.10	0.10	1.00	0.05
Combined Axial Load and Bending	1.05	0.10	1.00	0.05
Cylindrical Tubular Members				
Bending Strength	1.10	0.10	1.00	0.05
Axial Compression	1.10	0.10	1.00	0.05
Wall Studs and wall stud Assemblies				
Wall Studs in compression	1.10	0.10	1.00	0.05
Wall Studs in Bending	1.10	0.10	1.00	0.05
Wall Studs with Combined Axial load and Bending	1.05	0.10	1.00	0.05
Structural Members Not Listed Above	1.00	0.10	1.00	0.05

TABLE 5.3 (Continued) (CAN/CSA-S136-01)

Statistical Data for the Determination of Resistance Factor

Type of Component	M_m	V_M	F_m	V_F
Welded Connections				
Arc Spot Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Arc Seam Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Tearing	1.10	0.10	1.00	0.10
Fillet Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.08	1.00	0.15
Flare Groove Welds				
Shear Strength of Welds	1.10	0.10	1.00	0.10
Plate Failure	1.10	0.10	1.00	0.10
Resistance Welds	1.10	0.10	1.00	0.10
Bolted Connections				
Minimum Spacing and Edge Distance	1.10	0.08	1.00	0.05
Tension Strength on net section	1.10	0.08	1.00	0.05
Bearing Strength	1.10	0.08	1.00	0.05
Screw Connections				
Minimum Spacing and Edge Distance	1.10	0.10	1.00	0.10
Tension Strength on net section	1.10	0.10	1.00	0.10
Bearing strength	1.10	0.10	1.00	0.10
Connection Not Listed Above	1.10	0.10	1.00	0.15

5.3.4 Determination of Resistance Factors, ϕ

(a) LRFD method (used in United States and Mexico)

The resistance factor for LRFD design method in United States and Mexico can be calculated using equation (5.4) with the following values. This would produce a resistance factor of **0.85**.

$C\phi = 1.52$

$M_m = 1.10$ (Table 5.3)

$F_m = 1.00$ (Table 5.3)

$P_m = 1.0$

$\beta o = 2.5$

$V_M = 0.10$ (Table 5.3)

$V_F = 0.05$ (Table 5.3)

$V_P = 10.944\%$

$V_Q = 0.21$

$C_P = 1.324$

To find the factor of safety (Ω) for ASD method following equation can be used:

$\Omega = 1.6 / \phi$

$\Omega = 1.6 / 0.85 = \mathbf{1.90}$

(b) LSD method (used in Canada only)

The resistance factor for LSD design method in Canada can be calculated using equation (5.4) with the following values. This would produce a resistance factor of **0.70**.

$C\phi = 1.42$

$M_m = 1.10$ (Table 5.3)

$F_m = 1.00$ (Table 5.3)

$P_m = 1.0$

$\beta o = 3.0$

$V_M = 0.10$ (Table 5.3)

$V_F = 0.05$ (Table 5.3)

V_P =10.944%

V_Q =0.21

C_P =1.324

To find the factor of safety (Ω) for ASD method following equation can be used:

$\Omega = 1.6 / \phi$

$\Omega = 1.6 / 0.70 = 2.30$

Table 5.4 Proposed design coefficients and resistance factors for web crippling of C-sections

Support and flange conditions		Load cases		C	C_R	C_N	C_h	S136-01		AISI	
								Ω	ϕ	Ω	ϕ
Fastened to support	Stiffened flanges	Two-flange loading	Interior lap	2.5	0.02	1.01	0.001	2.30	0.70	1.90	0.85

Notes: The above coefficient apply when $139 \le h/t \le 172$, $37 \le N/t \le 127$, $1.48 \le R/t \le 1.97$, $\theta = 90°$.

5.3.5 Limitation of the proposed design equations

Based on experimental findings and scope of research presented in this thesis, the following parameter limitations are presented for design coefficients and resistance factors proposed in Table 5.4.

1- The two bearing plates and the channel lap over the interior support are of equal length;

2- Two rows of self-drilling screws should be located at third point of the web depth; and

3- The geometric properties of the channel section should be within the limits specified in the footnote of Table 5.4.

4- Arrangement of fasteners at 38mm lateral spacing.

CHAPTER 6
CONCLUSIONS

6.1 General

The objective of this research was to study the web crippling capacity of lapped cold-formed steel C-channels subjected to interior two flanges loading, with ultimate goal of developing new web crippling design coefficients to extend the applicability of the available Standard web crippling strength equation to the studied CFS channel configuration and loading condition. A literature review of web crippling research and design specifications was presented in chapter 2. As described in Chapter 2, the computation of the web crippling strength by theoretically analysis is complex, as it involves a large number of factors, such as initial imperfection of web element, instability of the web element, local yielding in the region of load application and some other factors. Therefore, an experimental study on 24 specimens was conducted in this thesis to obtain the web crippling capacity for C-sectional CFS members that are lapped at the interior support location and subjected to interior two flange loading. Few key parameters affecting web crippling strength of lapped channels were considered, namely: channel thickness, channel depth, inside bend radius and lap length.

6.2 Conclusions

Based on the experimental findings, the following conclusions were drawn:

1- The sensitivity study on web crippling of lapped cold-formed steel channels subjected to interior two-flange loading condition revealed that the unbraced length of twice the joist depth on each side of the interior support is sufficient to obtain a reliable web crippling strength of single-web channels lapped at the interior support, and having rotationally-restraint flanges. This is in contrast to the recommendations specified in the 2002 AISI document on "Standard Test Method for Determining the Web Crippling Strength of Cold-Formed Steel Members."

2- The sensitivity study also revealed that the presence of metal deck has insignificant effect on web crippling strength as long as the unbraced length of the joist is maintained as 2h. Moreover, channel lap length should be considered when developing empirical expression for web crippling resistance of such channel members at interior supports.

3- Experimental findings proved that web crippling strength of the tested channel specimens increases with an increase of bearing length, provided that bearing length equals the channel lap length at interior support location.

4- Experimental findings proved that web crippling strength increases with an increase in channel plate thickness, as expected.

5- In calibration with the safety requirements in accordance with both the Canadian and American specifications for CFS member designs, new design coefficients and resistance factors for web crippling strength of lapped cold-formed steel channels at interior support subjected to two-flange loading were deduced based on experimental findings.

6.3 Recommendations for Future Research

Possible areas of research interests for future studies include:

1- Tests on single channel lapped at intermediate support for one-flange loading.

2- Experimental study on the effect of the change in lap length opposed to the length of bearing plate for lapped channel subjected to two-flange loading.

3- Experimental study on the web crippling strength of unstiffened single channel lapped at interior location and subjected to two-flange loading and with wide range of N/t, h/t and R/t similar to other CFS sections specified in S136-01 for web crippling strength.

4- Study the effect of clip angle bearing stiffener (required to connect transverse joist to the channels under consideration) on web crippling strength of "C" channels lapped at interior support.

5- Tests on single "Z" shapes lapped at interior support, subjected to either one-flange or two-flange loading.

6- Study the effect of self-drilling screw configuration at lap splices on web crippling strength.

7- Conduct finite-element analysis and yield-line analysis to correlate the experimental results with experiment findings for possible extension of the scope of the applicability of the developed design expressions.

REFERENCES

1. American Iron and Steel Institute (AISI). (2001). "North American Specification for the Design of Cold-formed Steel Structures Members". Washington D.C.
2. Wei-Wen Yu. (2000). "Cold Formed Steel Design". New York: John Wiley & Sons.
3. Schuster, R.M., and Beshara, B. (1999). "Web Crippling of Cold Formed Steel Members." Final Report, Department of Civil Engineering, University of Waterloo.
4. Young, B., and Hancock, G. (2001). "Design of cold-formed channels subjected to web crippling" Journal of structural engineering, Vol. 127, No. 10, 1137-1144.
5. Winter, G., and Pian, R.H. (1946). "Crushing of Thin Steel Webs." Engineering Experiment Station, Bulletin No. 35, Cornell University.
6. Hetrakul, N., and Yu, W. (1978). "Structural Behaviour of Beam Webs Subjected to Web Crippling and a Combination of Web Crippling and Bending", Final Report, Civil Engineering Study 78-4, University of Missouri-Rolla, Rolla, Missouri, U.S.A.
7. Bhakta, B., LaBoube, R., and Yu, W. (1992). "The Effect of Flange Restraint on Web Crippling Strength", Final Report, Civil Engineering Study 92-1, University of Missouri-Rolla, Rolla, Missouri, U.S.A.
8. Prabakaran, K. (1993). "Web Crippling of Cold Formed Steel Sections." Final Report, Department of Civil Engineering, University of Waterloo.
9. Gerges, R. R. (1997). "Web Crippling of Single Web Cold Formed Steel Members Subjected to End One-Flange Loading", M.A.Sc. Thesis, University of Waterloo, Waterloo, Ontario, Canada.
10. Holesapple, M. and LaBoube, R. (2003). "Web crippling of cold formed steel beams at end supports". Engineering Structures, 25: 1211-1216.
11. Young, B., and Hancock, G. (2004). "Web Crippling of Cold-formed Unlipped Channels with Flanges Restrained." Thin-Walled Structures, 42: 911-930.
12. Zhou, F., and Young, B. (2007). "Cold-Formed High-Strength Stainless Steel Tubular Sections Subjected to Web Crippling." ASCE Journal of Structural Engineering, Vol. 133, No. 3, 368-377.

13. Young, B., and Hancock, G. (2002). "Tests of Channels Subjected to Combined Bending and Web Crippling." ASCE Journal of Structural Engineering, Vol. 128, No. 3, 300-308.
14. AS/NZS. (1996). "Cold-Formed Steel Structures." Australian/New Zealand Standard, AS/NZS 4600, Sydney, Australia.
15. Zhou, F., and Young, B. (2007). "Experimental Investigation of Cold-Formed High-Strength Stainless Steel Tubular Members Subjected to Combined Bending and Web Crippling." ASCE Journal of Structural Engineering, Vol. 133, No. 7, 1027-1034.
16. Stephens, S., and LaBoube. (2003). "Web Crippling and Combined Bending and Web Crippling of Cold-Formed Steel Beam Headers." Thin-Walled Structures, 41: 1073-1087.
17. Serrette, R. (2002). "Web Crippling Behavior of Floor Joist/Rim Members in Built Assemblies." ASCE Journal of Structural Engineering, Vol. 128, No. 9, 1177-1185.
18. LaBoube, R., Yu, W., Deshmukh, S., and Uphoff, C. (1999). "Crippling Capacity of Web Elements with Openings." ASCE Journal of Structural Engineering, Vol. 125, No. 2, 137-141.
19. LaBoube, R., Nunnery, J., and Hodges, R. (1994). "Web Crippling Behavior of Nested Z-Purlins." Engineering Structures, Vol. 16, No. 5: 332-336.
20. Korvink, S., Berg, G., and Merwe, P. (1995). "Web Crippling of Stainless Steel Cold-Formed Beams." Journal of Constructional Steel Research, 34: 225-248.
21. Timoshenko, S.P., and Gere, G.M. (1961). "Theory of Elastic Stability, Second Edition," New York: McGraw-Hill.
22. Yamaki, N. (1953). "Buckling of a Rectangular Plate Under Locally Distributed Force Applied on the Two Opposite Edges." First and Second Report, The Institute of High Speed Mechanics, Tohoku University, Japan, Vol. 3.
23. Walker, A.C. (1975). "Design and Analysis of Cold Formed Sections," New York: John Wiley & Sons.
24. Ren, W., Fang, S., and Young, B. (2006). "Finite-Element simulation and design of cold-formed steel channels subjected to web crippling". Journal of structural engineering, Vol. 132, No. 12, 1967-1975.
25. Ren, W., Fang, S., and Young, B. (2006). "Analysis and Design of Cold-Formed Steel Channels Subjected to Combined Bending and Web Crippling." Thin-Walled Structures, 44: 314-320.

26. Zhou, F., and Young, B. (2008). "Aluminium Tubular Sections Subjected to Web Crippling - Part I: Tests and Finite-Element Analysis." Thin-Walled Structures, 46: 339-351.

27. Young, B., and Zhou, F. (2008). "Aluminium Tubular Sections Subjected to Web Crippling – Part II: Proposed Design Equations." Thin-Walled Structures, 46: 352-361.

28. Zhou, F., and Young, B. (2006). "Yield Line Mechanism Analysis on Web Crippling of Cold-Formed Stainless Steel Tubular Sections under Two-Flange Loading." Engineering Structures, 28: 880-892.

29. Fox, S. (2004). "Design Expressions Based on a Finite-Element Model of a Stiffened Cold-Formed Steel C-Section." ASCE Journal of Structural Engineering, Vol. 130, No. 5, 708-714.

30. Cevik, A. (2007). "Unified Formulation for Web Crippling Strength of Cold-Formed Steel Sheeting using Stepwise Regression." Journal of Constructional Steel Research, 63: 1305-1316.

31. Cevik, A. (2007). "A New Formulation for Web Crippling Strength of Cold Formed Steel Sheeting Using Generic Programming." Journal of Constructional Steel Research, 63: 867-883.

32. Guzelbey, I., Cevik, A., and Erklig, A. (2006). "Prediction of Web Crippling Strength of Cold-Formed Steel Sheeting Using Neural Networks." Journal of Constructional Steel Research, 62: 962-973.

33. Akhand, A., Badaruzzaman, W., and Wright, H. (2004). "Combined Flexure and Web Crippling Strength of a Low Ductility High Strength Steel Decking: Experimental and A Finite Elemnt Model." Thin-Walled Structures, 42: 1067-1082.

34. Avci, O., and Easterling, S. (2004). "Web Crippling Strength of Steel Deck Subjected to Eng One Flange Loading." ASCE Journal of Structural Engineering, Vol. 130, No. 5: 697-707.

35. Hofmeyer, H., Kerstens, J., Snijder, H., and Bakker, M. (2002). "Combined Web Crippling and Bending Moment Failure of First Generation Trapezoidal Steel Sheeting." Journal of Constructional Steel Research, 58: 1509-1529.

36. Hofmeyer, H., Kerstens, J., Snijder, H., and Bakker, M. (2001). "New Generation Model for Failure of Steel Sheeting Subjected to Concentrated Load (Web Crippling) and Bending." Thin-Walled Structures, 39: 773-796.

37. Bakker, M. (1994). "Theoretical and Experimental Research on Web Crippling of Cold-Formed Flexural Steel Members." Thin-Walled Structures, 18: 261-290.

38. American Society of Testing and Materials A370-92, ASTM. (1992). "Standard Method and Definitions for Mechanical Testing of Steel Products."

39. LaBoube, R., and Schuster, R. (2002). "Standard Test Method for Determining the Web Crippling Strength of Cold-Formed Steel Members." AN AISI Sponsored Project, American Iron and Steel Institute, Washington D.C.

40. Victor Lewis (2007). "FORTRAN", MinRes; Canada Sheet Building Institute.

41. Canadian Standards Association, CSA S136-94. (1994). "Cold Formed Steel Structural Members," Canada.