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Compression Behavior of Thin Gusset Plates

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Abstract

The use of cold-formed steel members for truss applications is gaining widespread acceptance in the United States. However, there is little technical information regarding the behavior and design of thin gusset plates in compression. Thus, a study has been initiated at the University of Missouri-Rolla aimed at investigating the behavior of thin gusset plates in compression. Key parameters that were considered in the experimental study were the thickness of the gusset plate sheet steel which ranges from 0.058 inches (1.47 mm) to 0.103 inches (2.62 mm); width and length of the gusset plates, fastener location, and fastener pattern.

Introduction

The use of cold-formed steel members for truss applications is gaining widespread acceptance in the United States. Cold-formed steel truss design in the United States is based on the American Iron and Steel Institute (AISI) publication North American Specification for the Design of Cold-Formed Steel Structural Members (2001) and the Standard for Cold-Formed Steel Framing – Truss Design (2001). However, neither the specification nor the standard includes guidelines for the design of gusset plates. These plates are critical for connecting structural elements in the same plane. Truss systems commonly utilize gusset plate connections, as illustrated by Figure 1.

This study focused on the general behavior of thin flat gusset type plates in compression. The test data was compared against current design methodologies

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and modifications were suggested. Both edge stiffened and unstiffened gusset plates were studied.

Literature Review

One of the earliest studies in the United States on gusset plates was performed by Whitmore (1952) at the University of Tennessee. His study focused on the stress distribution within aluminum plate connections. Whitmore concluded that the stresses start at the outside edge of the top row of fasteners and propagate out at an angle close to 30°. The stresses continued to spread out at this angle until they reached the last row of fasteners. The stresses then continued nearly parallel to the original line of fasteners. This pattern has become known as the Whitmore Section and is illustrated in Figure 2.

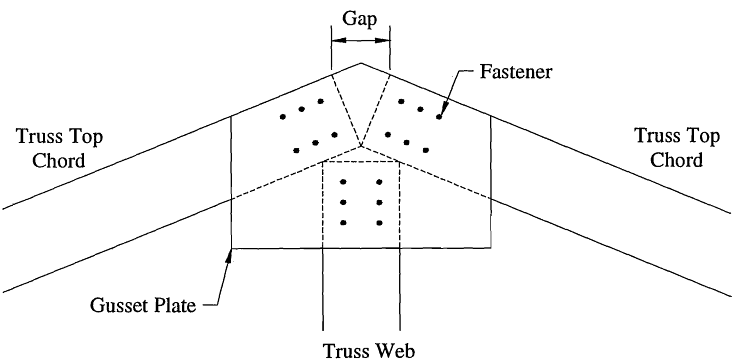

Figure 1 Roof Truss with Gusset Plate Connection

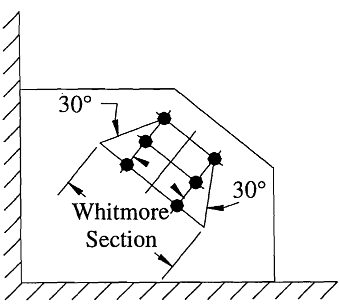

Figure 2 Whitmore Section

Thornton (1984) presented a design methodology for brace connections in heavy construction. Thornton proposed a method for calculating the capacity of a gusset plate in compression. The author suggested that the plate be analyzed as an effective column. Thornton's analytical model is based on assumptions that have only been verified for thick gusset plates. The effect of a thin gusset plate in the connections had not been previously examined.

An unpublished technical paper by Babich (2002) described a design methodology for thin, flat gusset plates used for cold-formed steel truss design. Babich suggested two methods for the design of compression plate connections. The first method was based on a modified Thornton approach. Babich also suggested a method that utilized a plate buckling approach. The critical buckling stress was calculated and then averaged to take into account the non-linear and post-buckling stress distribution within the plate. A plate buckling coefficient, k, equal to 1.25 was used.

The plate buckling design methodology proposed by Babich is based on plate theory and research performed by others on thick gusset plates. The axial compressive strength of the compression plate is calculated as follows:

$$P_{compute} = A_e f_{av} \quad (1)$$

Where:

$$f_{av} = \sqrt{f_{cr} F_y}\left(1 - 0.22\sqrt{\frac{f_{cr}}{F_y}}\right) \quad (2)$$

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{W_{eff}}\right)^2 \quad (3)$$

Where:

A_e	=	Effective area of member = $W_{eff}\, t$
F_y	=	Specified minimum yield point
E	=	Modulus of elasticity
k	=	Plate buckling coefficient = 1.25
μ	=	Poisons Ratio = 0.3
t	=	Plate thickness, in.

W_{eff} is to be taken as the lesser of the following:

The gusset plate width
1.25 times the least width of the connected members
The Whitmore Section as shown in Figure 2.

A compilation of test data on gusset plates loaded in compression was assembled by Dowswell and Barber (2004). The data is based on both experimental tests and finite element models. The test data was separated into five different categories. These included compact corner-braces, non-compact corner-braces, extended corner-braces, single-brace gusset plates, and Chevron-brace gusset plates. A statistical analysis was performed on the calculated strength using Thornton's method and the tested load capacity. Suggestions were then offered as to what value should be used for the effective length, and the effective length factor, for each category of brace. Of the five test data categories, the single-brace gusset plate tests most closely resembled that of a typical roof truss gusset plate connection. The smallest thickness that was tested in the study was a 0.256 in. thick plate.

UMR EXPERIMENTAL INVESTIGATION

An experimental study was initiated at the University of Missouri-Rolla to explore the behavior of the thin steel gusset plates in compression. A testing apparatus was developed that would accommodate the variables of the testing program and ensure consistent dimensions between tests. Three variables were considered in the testing program: plate thickness, plate width, and fastener pattern. The fastener pattern controlled two of the parameters, the effective length and the effective plate width.

Eleven different fastener patterns were chosen for testing. These patterns were designed to show the effect of the parameters that were to be investigated. A sketch of the general test setup can be seen in Figure 3. Details regarding the fastener patterns are given by Lutz (2004). A summary of the dimensions of each individual test can be found in Table 1.

Material Properties. The mechanical properties of the gusset plate material were determined by performing standard tensile coupon tests (ASTM A370, 2002). A summary of the average results can be found in Table 2.

Test Specimen Fabrication. Each test specimen consisted of two rectangular hollow structural steel (HSS) sections oriented perpendicular to each other with two thin gusset plates on each side, as shown in Figure 4. Bolts were used instead of screws to provide a test fixture that would be reusable during the testing program. The bolts were ¼ in. (6.4 mm) diameter Grade 8. Holes were drilled into both HSS sections for use both during testing and to provide a template for drilling the holes in the plates. These holes were precisely placed using the digital readout on the milling machine. A spacer was placed between the two tubes to maintain a consistent gap, Lg, (Fig. 3) as specified by the layouts of the different tests.

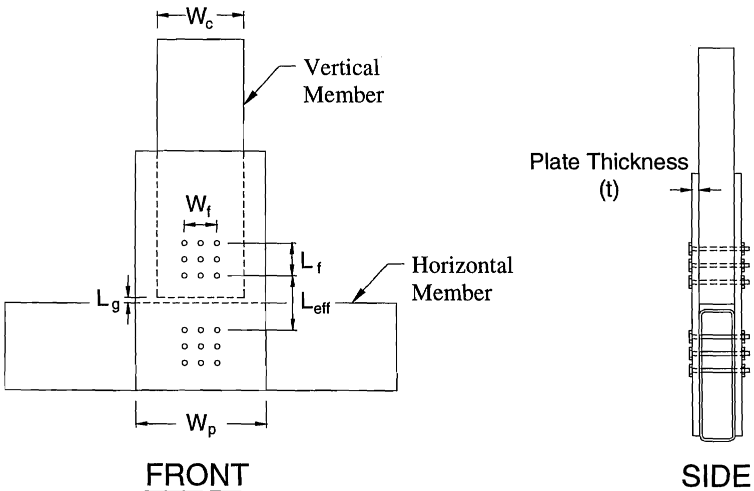

Figure 3 Typical Connection Detail

The small bolts provided a fastener pattern that was comparable to a screw fastener pattern and ensured that the plates failed before the fasteners. The nuts were finger tightened to represent the low clamping action provided by screws.

Edge Stiffened Plate Fabrication. The effect of an edge stiffener on the gusset plate was also studied. A typical stiffened plate with a 5/8 inch edge stiffener is shown in Figure 5.

Test Procedure. The tests were performed using a universal testing machine. The load was continuously applied until the test specimen either stopped taking load or the bottom of the vertical HSS section came into contact with the horizontal HSS section.

TEST RESULTS

Unstiffened Gusset Plates. The flat unstiffened gusset plates comprised the majority of the test program. Each test specimen configuration was duplicated with similar failure modes and maximum loads achieved.

Table 1 Test Parameters

Specimen No.	Plate Thickness t (in)	Effective Width W_{eff} (in)	Effective Length L_{eff} (in)	Length of Gap L_g (in)	Fastener Length L_f (in)	Fastener Width W_f (in)	HSS Width W_t (in)	Plate Width W_p (in)
1A	0.0585	3.23	1	0.125	1.5	1.5	4	6
1B	0.0749	3.23	1	0.125	1.5	1.5	4	6
1C	0.1027	3.23	1	0.125	1.5	1.5	4	6
2A	0.0585	4.73	1	0.125	1.5	3	4	6
2B	0.0749	4.73	1	0.125	1.5	3	4	6
2C	0.1027	4.73	1	0.125	1.5	3	4	6
3A	0.0585	5.00	1	0.125	3	3	4	6
3B	0.0749	5.00	1	0.125	3	3	4	6
3C	0.1027	5.00	1	0.125	3	3	4	6
4B	0.0749	4.00	1	0.125	3	3	4	4
5B	0.0749	3.23	1	0.125	1.5	1.5	4	12
6A	0.0585	3.23	2.5	0.125	1.5	1.5	4	6
6B	0.0749	3.23	2.5	0.125	1.5	1.5	4	6
6C	0.1027	3.23	2.5	0.125	1.5	1.5	4	6
7B	0.0749	3.23	1.125	0.25	1.5	1.5	4	6
8B	0.0749	4.73	2.5	0.125	1.5	3	4	6
9A	0.0585	3.23	1.75	0.125	1.5	1.5	4	6
9B	0.0749	3.23	1.75	0.125	1.5	1.5	4	6
9C	0.1027	3.23	1.75	0.125	1.5	1.5	4	6
10A	0.0585	4.96	1	0.125	3	1.5	4	6
10B	0.0749	4.96	1	0.125	3	1.5	4	6
10C	0.1027	4.96	1	0.125	3	1.5	4	6
11A	0.0585	3.23	4	0.125	1.5	1.5	4	6
11B	0.0749	3.23	4	0.125	1.5	1.5	4	6
11C	0.1027	3.23	4	0.125	1.5	1.5	4	6

Note: See Figure 3 for parameter definitions; 1 inch = 25.4 mm.

The typical failure mode for specimen type 14ga and 16ga having either 6 in. or 4 in. widths was a lateral buckling of the plates as is shown in Figure 6. For the 12 in. wide 14ga and 16ga specimen types, the failure mode was a localized buckling of the plate in the vicinity of the Whitmore Section. The 12ga specimen types deformed axially until the vertical and horizontal HSS sections came in bearing contact with each other. The contact between the HSS sections was considered a failure. Minor buckling was noted in the plates.

Table 2. Material Properties

Specimen Type	Uncoated Thickness	Yield Point	Tensile Strength	Percent Elongation
	t	F_y	F_u	
	(in)	(psi)	(psi)	(%)
12ga	0.1027	44,385	56,052	41
14ga	0.0749	51,563	60,051	36
16ga	0.0585	33,750	48,161	27

Note: 1 ksi = 6.89 MPa, 1 inch = 25.4 mm.

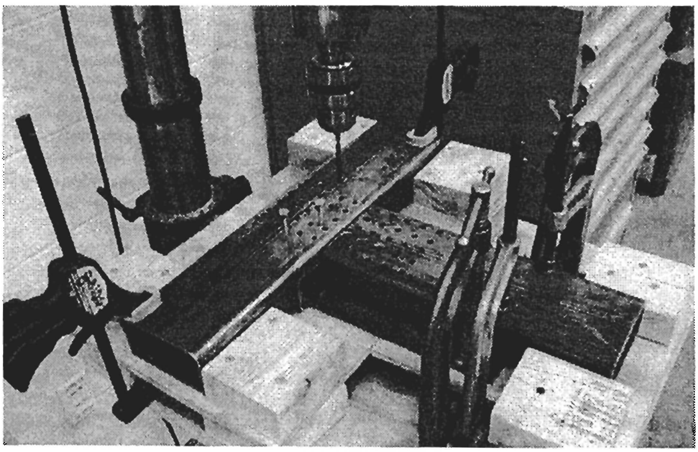

Figure 4 HSS Test Fixture Sections

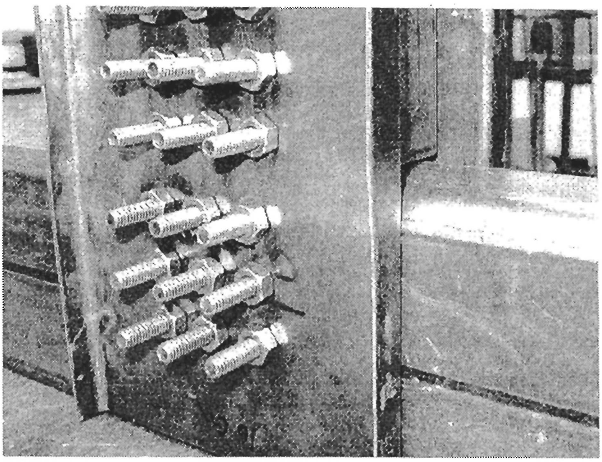

Figure 5 Typical Stiffened Plate

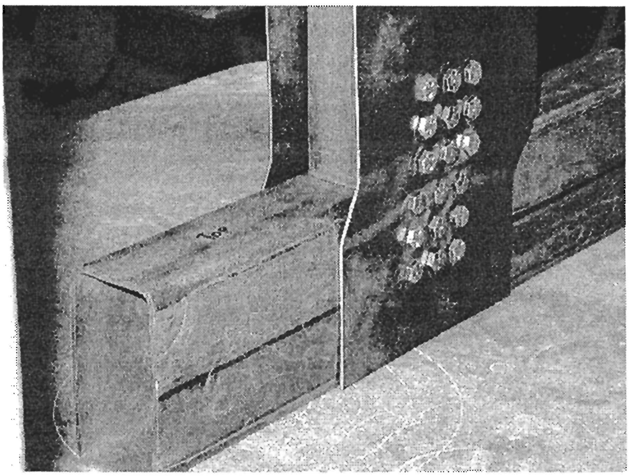

Figure 6 Typical Lateral Buckling of Unstiffened Plate

Table 3 summarizes the results of the tests conducted on the unstiffened plates. Test 2C-1 failed with a local buckling in the webs of the horizontal HSS and therefore test 2C-2 was not performed.

Edge Stiffened Gusset Plates. The edge stiffened gusset plates were tested to gain insight into the increased strength that the stiffeners might provide. The typical failure mode for these plates was a plate buckling in the Whitmore Section with deformation in the stiffeners and a slight out-of-plane sway. Table 4 summarizes the results of the edge stiffened tests. For the two specimen types investigated, a similar strength increase of approximately 25% was achieved.

UMR Data Analysis

The data obtained from the test program was used to develop both a plate model and a column model for determining compressive strength of a gusset plate (Lutz, 2004). This paper presents only the plate model.

The test results were used to back calculate the plate buckling coefficient, k, required to achieve the tested load capacity. This process yielded an average k value of approximately 4.0. According to Yu (2000), k equal to 4.0 would correspond to a situation where all four sides of the plate are simply supported. The connected members and fastener acted together to provide a simply supported condition for the edges of the plates, justifying the use of k equal to 4.0.

The three effective width criteria, W_{eff}, as proposed by Babich, were considered but the test results and analysis justified using only the smaller of the Whitmore section and the plate width. There was no justification found in the test data for the use of the 1.25 times the width of the connected member.

One variable that was not taken into account in the Babich plate model was the effective length of the plate, L_{eff} (Fig. 3). As depicted by Figure 7 a bilinear relationship existed between the $P_{test}/P_{compute}$ ratio and the width to length, W_{eff}/L_{eff}, ratio.

Table 3 Unstiffened Gusset Plate Test Results

Specimen No.	$P_{test\ \#1}$ (lbs/plate)	$P_{test\ \#2}$ (lbs/plate)	Specimen No.	$P_{test\ \#1}$ (lbs/plate)	$P_{test\ \#2}$ (lbs/plate)
1A	5,188	6,313	6C	11,775	11,400
1B	12,275	10,175	7B	9,188	9,588
1C	17,000	17,500	8B	13,863	11,888
2A	7,175	7,400	9A	5,850	4,738
2B	14,538	14,363	9B	9,050	10,500
2C	19,150	-	9C	14,750	14,000
3A	5,900	5,500	10A	5,263	6,588
3B	14,400	16,125	10B	12,363	12,613
3C	22,725	17,950	10C	17,000	17,050
4B	11,550	10,625	11A	3,625	3,113
5B	13,800	11,613	11B	6,875	5,188
6A	4,263	5,813	11C	10,550	9,775
6B	8,888	9,125			

Table 4 Edge Stiffener Test Results

Specimen No.	$P_{test\ \#1}$ (lbs/plate)	$P_{test\ \#2}$ (lbs/plate)	Test Average (lbs/plate)	% Strength Increase
1A	5,188	6,313	5,750	27%
S1A	7,595	7,000	7,298	
6A	4,263	5,813	5,038	24%
S6A	5,950	6,530	6,240	

Note: S1A and S6A were edge stiffened Plates

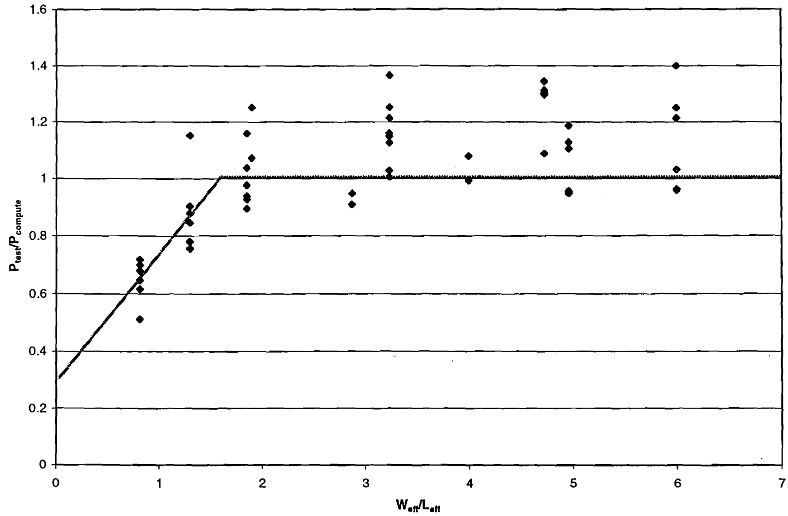

Figure 7 P_{test}/Pcompute versus W_{eff}/L_{eff}

A modified analytical model was thus developed as follows:

For $W_{eff} / L_{eff} \leq 1.5$

$$P_n = P_{compute}\left(0.47\frac{W_{eff}}{L_{eff}} + 0.3\right) \tag{4}$$

For $W_{eff} / L_{eff} > 1.5$

$$P_n = P_{compute} \tag{5}$$

Where:

$$P_{compute} = A_e f_{av} \tag{6}$$

$$f_{av} = \sqrt{f_{cr}F_y}\left(1 - 0.22\sqrt{\frac{f_{cr}}{F_y}}\right) \tag{7}$$

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{W_{eff}}\right)^2 \tag{8}$$

Where:

A_e	=	Effective area of gusset plate = $W_{eff}\,t$
F_t	=	Specified minimum yield point, ksi
E	=	Modulus of elasticity = 29,500 ksi
k	=	Plate buckling coefficient = 4.0
μ	=	Poisons ratio = 0.3
t	=	Uncoated plate thickness, in.

W_{eff} shall be taken as the lesser of the following:
The gusset plate width
The Whitmore width as shown in Figure 2

L_{eff} shall be taken as the average length between the last row of fasteners of adjacent members, shown as Avg. (l1,l2,l3) in Figure 8.

The P_{test}/P_n ratio was calculated using the analytical model represented by Equations 4 through 8, and is given in Table 5. The mean and standard deviations were calculated to be 1.07 and 0.15, respectively, and is illustrated by Figure 9.

DESIGN RECOMMENDATION

The factors of safety for Allowable Stress Design and Resistance factors for Load and Resistance Factor Design and Limit State Design were calculated in accordance with the AISI Specification (AISI, 2001)

The nominal axial strength, P_n, of a thin, flat gusset plate in compression is calculated as follows:

$$P_n = A_e f_{av} \tag{9}$$

USA and Mexico		Canada
Ω_c(ASD)	Φ_c(LRFD)	Φ_c(LSD)
2.50	0.60	0.50

For $W_{eff} / L_{eff} \leq 1.5$

$$f_{av} = f'_{av}\left(0.47\frac{W_{eff}}{L_{eff}} + 0.3\right) \quad (10)$$

For $W_{eff} / L_{eff} > 1.5$

$$f_{av} = f'_{av} \quad (11)$$

Where:

$$f'_{av} = \sqrt{f_{cr}F_y}\left(1 - 0.22\sqrt{\frac{f_{cr}}{F_y}}\right) \quad (12)$$

$$f_{cr} = \frac{k\pi^2 E}{12(1-\mu^2)}\left(\frac{t}{W_{eff}}\right)^2 \quad (13)$$

Where:

A_e	=	Effective area of gusset plate = $W_{eff}\,t$
F_y	=	Specified minimum yield point, ksi
E	=	Modulus of elasticity = 29,500 ksi
k	=	Plate buckling coefficient = 4.0
μ	=	Poisons Ratio = 0.3
t	=	Uncoated plate thickness, in.

W_{eff} shall be taken as the lesser of the following:
The gusset plate width
The Whitmore plate width as shown in Figure 2

L_{eff} shall be taken as the average length between the last row of fasteners of adjacent truss members, $L_{eff} = (l_1 + l_2 + l_3)/3$, (Figure 11).

The equations are applicable within the limits of the investigation.

- Uncoated design base metal thickness: 0.059 in. – 0.103 in.
- Design yield point: 33 ksi – 52 ksi
- W_{eff} / L_{eff} ratio: 0.8 – 6.0
- Plates with edge stiffeners could be conservatively designed using these equations.

Conclusions

A total of 49 specimens were tested to study the behavior of thin gusset plates in compression. Based on the test results a design methodology was established for thin gusset plate connections in compression.

A limited number of tests were performed to determine the strength gain in gusset plates with edge stiffeners. The test results showed an approximate strength increase of 25% for the plates.

Table 5 P_{test}/P_n Ratio for the Analytical Model

Test No.	P_n lbs/plate	$P_{test\#1}$ lbs/plate	$P_{test\#2}$ lbs/plate	$P_{test\#1}/P_n$	$P_{test\#2}/P_n$
1A	5,044	5,188	6,313	1.03	1.25
1B	10,110	12,275	10,175	1.21	1.01
1C	15,088	17,000	17,500	1.13	1.16
2A	5,505	7,175	7,400	1.30	1.34
2B	11,077	14,538	14,363	1.31	1.30
2C	17,581	19,150	-	1.09	-
3A	5,715	5,900	5,500	1.03	0.96
3B	11,517	14,400	16,125	1.25	1.40
3C	18,716	22,725	17,950	1.21	0.96
4B	10,696	11,550	10,625	1.08	0.99
5B	10,110	13,800	11,613	1.36	1.15
6A	4,557	4,263	5,813	0.94	1.28
6B	9,133	8,888	9,125	0.97	1.00
6C	13,629	11,775	11,400	0.86	0.84
7B	10,110	9,188	9,588	0.91	0.95
8B	11,077	13,863	11,888	1.25	1.07
9A	5,044	5,850	4,738	1.16	0.94
9B	10,110	9,050	10,500	0.90	1.04
9C	15,088	14,750	14,000	0.98	0.93
10A	5,551	5,263	6,588	0.95	1.19
10B	11,174	12,363	12,613	1.11	1.13
10C	17,832	17,000	17,050	0.95	0.96
11A	3,415	3,625	3,113	1.06	0.91
11B	6,845	6,875	5,188	1.00	0.76
11C	10,216	10,550	9,775	1.03	0.96
			Mean	1.07	
			Std. Dev.	0.15	

Note: 1 lb. = 1.18 N

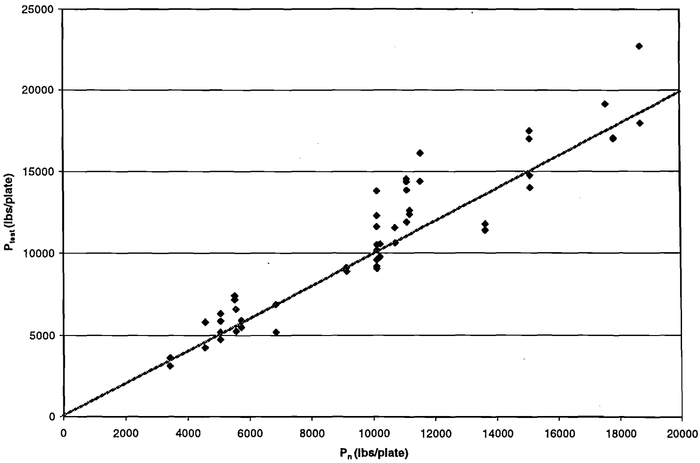

Figure 9 Evaluation of Analytical Model

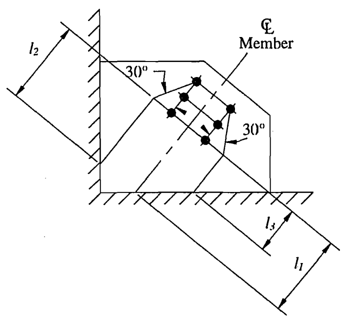

Figure 10 Effective Length

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