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Determining the R values for 12 inch deep Z-purlins and girts with through-fastened panels under suction loading

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DETERMINING THE R VALUES FOR 12 INCH DEEP Z-PURLINS AND GIRTS WITH THROUGH-FASTENED PANELS UNDER SUCTION LOADING

by

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ABSTRACT

Designing purlins for roof systems attached to through-fastened panels has been a subject well researched in the past. The current design specification uses a simplified approach to the designing of these members where the fully braced moment capacity of such members is multiplied by a reduction factor, commonly referred to as the R-value. This value represents the point in between the fully braced and fully unbraced member behavior. However, the current AISI Specification, S100, only contains R-values for purlins and girts up to 11.5 inches in depth. Since manufacturers are now rolling sections up to 12 inches deep, two confirmatory tests were performed with the goal of expanding the limits of the current design provisions. The intent of this research was to demonstrate that the R-value for the 11.5 inch deep Zee members is representative for members with depths of 12 inches as well. One continuous span and one simple span test were performed.

Based on the findings of this test program, 12" deep Z-purlins do meet or exceed the required strength computed using the current AISI S100 R-values. Thus, it is recommended that the limitations of Section D6.1.1 be expanded to include these deeper 12" Z-purlins. With the increase in depth, the depth-to-flange width ratio should also be expanded. It is recommended that the upper limit of the depth-to flange width ratio be expanded to include members with depth/flange width ratios up to a value of 5.5. It is also recommended that Section D6.1.1 be changed to ensure ductile steel rather than limiting the yield stress of the material. It is suggested that the limiting F_u/F_y ratio of the member be 1.20.

ACKNOWLEDGMENTS

I would like to take this opportunity to thank Dr. LaBoube, for all of his guidance and support he has given me throughout my college career. It has been an awesome experience to work with someone with so much knowledge and passion towards the subject of cold-formed steel.

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I would also like to mention my family and friends, including my parents, David and Cecilia, as well as, my brothers and sister, Ben, Elaine and Neil. They are sincerely cherished.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1. INTRODUCTION

1.1. GENERAL

The metal building industry uses C- and Z-purlins and girts as part of a system connecting panels to the metal building's main structure. These members are flexural members with either screw attached or standing seam panels. A photo, courtesy of NCI Building Systems, of such a roof system is shown in Figure 1.1. When a purlin or girt is under wind suction, the compression flange of this member is not fully braced by the panels. However, the purlin or girt is also not completely unbraced. The member has a capacity somewhere in between these two extremes. Early research has shown how complex a set of formulas can be when trying to mathematically model the rotational restraint and distortion of the purlin-to-panel connections. A simpler approach is now used by the cold-formed design community.

Figure 1.1. Metal Building Roof System

The design provisions used today are based on the fully braced purlin capacity under uplift conditions. This capacity is then multiplied by a reduction factor based on the size and shape of the member under loading. This reduction factor is call the R-value and is based on experimental test results. The R-value represents that the system's bending strength lies in between fully braced and fully unbraced. The current AISI specification (S100) provision only list R-values for purlins or girts with screw attached panels.

1.2. APPLICATION

When the previous sets of continuous span and simple span R-value tests were performed, metal building companies were not using Z-purlins as large as in today's industry. Several metal building manufactures are now using 12" deep Z-purlins and girts. Since the American Iron and Steel Institute's North American Specification for Cold-Formed Steel Structural Members (2007) does not currently include R-values for 12" deep Z-sections, tests were completed to confirm that these deeper members have R-values which fit within the range of previous tests for Z-section that had shallower depths. Continuous and simple span tests were conducted as part of this study. These tests were to be considered confirmatory tests if the results did, in fact, fit within the data which supports current R-values. If the deeper members did not meet or exceed the strength required for the current provisions for smaller sections, more tests would need to be performed and new R-values for the deeper sections would be required to better represent 12" deep members.

2. LITERATURE REVIEW

2.1 GENERAL

Prior to 1986, flexural members having one flange through-fastened to deck or sheathing were designed as laterally unsupported members. This was highly conservative and several approaches were researched to determine what factors affected the bracing support gained by the through-fastened panel and to what degree were, in fact, the purlins supported by the panel or deck. In other words, what capacity between fully braced and laterally unbraced were the members reaching? Early design approaches were shown to be quite complex and tests results deviated significantly from calculations. The current provision, the North American Specification for the Design of Cold-Formed Steel Structural Members (AISI S100) uses a simpler approach to determining the design strength of Z-purlins and girts under uplift loading conditions.

Calculations in Appendix A show an example of how much design strength can be gained when taking into account that the roof sheathing is providing some support for the purlins. Section C.3.1.2 of the AISI S100 was used to calculate the fully unbraced moment capacity, while Section D.6.1.1 was used to calculate the moment capacity with the support gained from the through fasten roof sheathing taken into account.

2.2. PREVIOUS RESEARCH

The current design provisions contained in the AISI S100 Section D6.1.1 are based primarily on the research programs summarized by Fisher (1996) and LaBoube et al. (1988). The S100 gives R-values for through-fastened roof panels only. For systems with standing seam panels, designers are referred to either Appendix A or B for provisions. For the United States and Mexico, the Reduction factor must be determined in accordance with the AISI S908 test method (AISI S908, 2004) also referred to as the Base Test Method.

2.2.1 Haussler and Pabers, 1973. The paper "Connection Strength in Thin Metal Roof Structures," among other things, presents an analytical method for evaluating the moment capacity of purlins and girts with through-fastened panels under suction loading. Haussler and Pabers assumed that roof panels are rigid enough to provide an elastic brace to the bottom flange of a purlin (or girt) under suction loading. The panels will allow the top (tension) flange to remain straight, and also provides some restraint against rotation of the bottom (compression) flange. The system's moment capacity can be represented by the Engesser's formula:

$$P_a = 2\sqrt{b_5 E_t I_y} \qquad \text{(Eq. 2.1)}$$

Where,

P_a = the critical axial load in the compression flange

b_5 = the spring constant of the elastic system providing lateral support for the compression flange

E_t = the tangent modulus of elasticity

I_y = the moment of inertia of the compression flange about its Y-axis

Where the elastic spring constant for the lateral support of the system, b_5, can be found by the following equation:

$$b_5 = \frac{1}{S * a_1^{\,2} * \left[\dfrac{L_2}{4E_s I_S} + \dfrac{4(1v^2) * \left(\dfrac{t^3 bn}{t_f^{\,3}} + a_1 \right)}{E t^3 L_e} + K_f \right]} \qquad \text{(Eq. 2.2)}$$

Where,

a_1 = the height of the section from the tension edge to the shear center of the compression flange

E = the modulus of elasticity of the web material

E_S = the modulus of elasticity of the panel material

I_S = the moment of inertia of the panel about its horizontal axis

K_F = the experimentally determined joint flexibility in radians/inch lb

L_e = the effective length of the web at each panel

L_2 = the beam spacing

N = zero for unlipped flanges, 1 for lipped channels or zee sections, and 0.25 for lipped I sections

S = panel spacing

t = web thickness

μ = Poisson's ratio for the web material

Haussler and Pabers discuss a test setup and analysis procedure that can be used to find the K_F value.

2.2.2 Pekoz and Soroushian, 1981 and 1982. In their report "Behavior of C- and Z-Purlins under Wind Uplift," Pekoz and Soroushian present research performed with the goal of developing "simple equations" for purlins under suction loading conditions. The report states that previous equations assumed that the compression flange did not move laterally under initial loading and that the "initial sweep and twist was not accounted for" (Pekoz 1982). It was assumed that the purlin could be considered a beam-column on an elastic foundation where the purlin-to-panel connections were represented as linear extensional springs for rotational stiffness with a stiffness of "k." This "k" represents the combined "effect of the restraint provided by the roof panels and the web of the purlin to the compression portion of the purlin" (Pekoz 1982).

2.2.3 LaBoube, 1986. The paper "Roof Panel to Purlin Connection: Rotational Restraint Factor" investigated the factors affecting the rotational stiffness provided to the purlin by the panel and screw connection. The report suggests an empirical equation:

$$F = 20.41(t - 0.061) + 2.3 \qquad \text{(Eq. 2.3)}$$

Where,

F = the rotational restraint factor in units of lb/in/in

t = the purlin thickness in inches

The factors examined in this test program were:

1) Purlin depth and thickness
2) Roof sheet depth and thickness
3) Insulation thickness
4) Fastener type
5) Fastener location

The purlin depth and thickness were thought to be important influences on the rotational restrain factor because the web and bottom flange were thought to resemble a cantilever beam. Based on the test results, the purlin thickness proved to be much more significant than the member depth.

Tested roof sheet depth and thickness were limited to the range used in the metal building industry at the time. It was determined that the thickness of the panels had a minor impact on the rotational stiffness and the depth of the paneling had no significance. The member thickness had a very significant impact on rotational stiffness. Setups with insulation were also tested. It was found that insulation thickness did not significantly affect the "F" factor. It was suggested that this is due to the fact that the insulation was compressed to similar thicknesses at the connection no matter how thick the blankets of insulation.

Fastener types tested were self-drilling or self-tapping screws. Self-tapping screws provided slightly more rotational restraint than self-drilling screws. It was also found that the rotational restraint significantly increased when the screw was located near the edge stiffener, and decreased when the fastener was located towards the web of the purlin. When designing, it is typically assumed that the fastener is located at the center of the flange. This is why it was also suggested that the self-tapping screws had a significant advantage because the screw location could be more controlled with a prepunched hole. Graphs from LaBoube's report "Roof Panel to Purlin Connection: Rotational Restraint Factor" can be found in Appendix B: Graphical Results from LaBoube's "Roof Panel to Purlin Connection: Rotational Restrain Factor" Report (LaBoube 1986). These results can show insight as to why there are certain limitations to the use of the current D6.1.1 section of AISI S100.

2.2.4 Haussler, 1988. The paper "Theory of Cold-Formed Steel Purlin/Girt Flexure" written in 1988 by Haussler addressed the importance of the effects of member distortion and panel bending. Haussler suggests using theoretical equations in combination with rotational restraint and stiffness tests to find the best design solution for beams with partial support from deck or sheathing.

2.2.5 LaBoube, et al., 1988. This paper, "Behavior of Continuous Span Purlin Systems" discussed a simplified approach to calculating the nominal strength of a Z- or C-shaped purlin attached to a through-fastened roof system under uplift loading. This is one of the early studies that forms the basis for the design method used today. The following equation was introduced:

$$M_n = R\, S_e\, F_y \tag{Eq. 2.4}$$

Where,

R = Strength reduction factor

S_e = Elastic section modulus of the effective section at F_y

F_y = yield stress

This research determined that R values of 0.7 for continuous span Z-sections and 0.6 for continuous span C-sections were valid representations for design. Since this equation was based on experimental data (R-values determined experimentally), there are certain conditions that should be met to use the given R-values which include the following:

1. Purlin depths less than 10 in (25.5 cm)
2. The free flange is a stiffened compression element
3. 60 < web depth/thickness < 170
4. 2 < web depth/flange width < 5
5. 16 < flange flat width/thickness < 43

The first limitation was later extrapolated to include depths up to 11.5 inches, and the fourth limitation was expanded to include 2.8≤depth/flange width ≤ 4.5

2.2.6 Fisher, 1996. Fisher composed a report "Uplift Capacity of Simple Span Cee and Zee Members with Through-Fastened Roof Panels" in 1996 which had significant impact on the AISI design provision at the time. The purpose of Fisher's tests (Fisher 1996) was to show that the existing R-values used for simple span C- and Z-sections were "overly conservative." Prior to this test program, the value used for Z-sections was 0.5. Table 2.1 shows the R-values determined for simple span members from Fisher's study.

Table 2.1. Resulting R-Values from Fisher's Study

Depth Range, in. (mm)	**Profile**	**R**
$d \leq 6.5$ (165)	C or Z	0.7
6.5 (165) < $d \leq 8.5$ (216)	C or Z	0.65
8.5 (216) < $d \leq 9.5$ (241)	C or Z	0.5
9.5 (241) < $d \leq 11.5$ (292)	C	0.4
8.5 (216) < $d \leq 11.5$ (292)	Z	0.4

Fisher also discusses the effects of insulation on the R values. The r value (insulation correction factor) was introduced as:

$$r = 1.00 - 0.01t \quad \text{(in inches)} \qquad \text{(Eq.2.4)}$$

$$r = 1.00 - 0.0004t \quad \text{(in mm)} \qquad \text{(Eq. 2.5)}$$

Where,

t = the depth of uncompressed insulation

This reduction factor (r) found from Equations 2.4 and 2.5 represents the loss of strength due to the inclusion of insulation. The R-value found in Table 2.2 should be multiplied by the r value from Equations 2.4 and 2.5. Reducing the values found in Table 2.2. (These R-values do not include the affect of insulation) will give you an R-Value that takes into consideration the fact that the strength of the system will be lower with insulation included.

2.3 AISI DESIGN PROVISIONS

Prior to 1986, there was no design specification for beams having one flange through-fastened to deck or sheathing. Much research went into trying to find an equation to model the system; however, the R-value method was the only design equation ever adopted by the AISI Specification.

2.3.1 AISI Specification 1980 Edition. AISI had not yet adopted any design provisions for addressing that through-fastened panels provided some bracing for members under uplift conditions when the tension flange was connected to a through-fastened panel. The members were designed as laterally unbraced beams.

2.3.2 Cold-Formed Steel Specification 1986 Edition. This specification used the same R-value design equation with similar conditions as the S100; however, the R-values differed slightly from the ones being used currently in the S100. The R-values used in this edition are shown in Table 2.2.

Table 2.2. AISI 1986 Edition R-Values

	R-Value
Simple Span Cee	0.40
Simple Span Zee	0.50
Continuous Span Cee	0.60
Continuous Span Zee	0.70

2.3.3. Cold-Formed Steel Design Specification 1996 Edition. The only change made from the 1986 edition was the lap length for channel sections was limited to 1.5d rather than 3.0d. The lap for zee sections remained at 1.5d, where d is the cross-section depth.

2.3.4. Cold-Formed Steel Design Specification 2001 Edition. After the Fisher tests were conducted, the simple span R-values changed. Fisher's report showed that the previous values adopted by AISI for Simple span systems were very conservative for members of smaller depth. Table 2.3 shows the R-values for simple span systems which appeared in the 2001 design specification.

Table 2.3. AISI 2001 Edition Simple Span R-Values

Depth Range, in. (mm)	**Profile**	**R**
d ≤ 6.5 (165)	C or Z	0.70
6.5 (165) < d ≤ 8.5 (216)	C or Z	0.65
8.5 (216) < d ≤ 11.5 (292)	Z	0.50
8.5 (216) < d ≤ 11.5 (292)	C	0.40

Another significant change made to the design parameters was the approach to designing simple span systems with insulation. If insulation was used, the R-value from Table 2.3 was multiplied by a reduction factor to represent the loss of capacity of the purlin due to the addition of insulation. The equations that appeared in this specification are the same as Eq. 2.4 and Eq. 2.5 which were in Fisher's report "Uplift Capacity of Simple Span Cee and Zee Members with Through-Fastened Roof Panels."

The 2001 design specification also differed from the previous design specifications by the addition of certain yield strengths of the materials used in the systems. The roof or wall panels were limited to no greater than 50 ksi while the purlins or girts should not exceed 60 ksi. The paneling minimum rib depth limitation was also changed from 1inch to 1-1/4 inch.

2.3.5 AISI Specification 2007 Edition Section D6.1.1. The 2007 Edition of the AISI S100 is the current provision used for the design of cold-formed steel members. The current specification contains R-values for both simple span C- and Z-sections up to 11.5 inches deep. These values are shown in Table 2.4 shown below.

Table 2.4. AISI 2007 Edition Simple Span R-Values

Depth Range, in. (mm)	Profile	R
d ≤ 6.5 (165)	C or Z	0.70
6.5 (165) < d ≤ 8.5 (216)	C or Z	0.65
8.5 (216) < d ≤ 11.5 (292)	Z	0.50
8.5 (216) < d ≤ 11.5 (292)	C	0.40

As stated in the AISI S100, the R-values for continuous span test are 0.60 for C-sections and 0.70 for Z-sections. These R-values are valid if the following conditions are met:

1. Member depth is less than or equal to 11.5 in. (292 mm),
2. Member flanges with edge stiffeners,
3. 60 ≤ depth/ thickness ≥ 170
4. 2.8 ≤ depth/ flange width ≤ 4.5
5. 16 ≤ flat width/ thickness of flange ≤ 43
6. For continuous span systems, the lap length at each interior support in each direction (distance from center of support to end of lap) is not less than 1.5d,
7. Member span length is not greater than 33 feet (10 m),
8. Both flanges are prevented from moving laterally at the supports,
9. Roof or wall panels are steel sheets with 50 ksi (340 MPa or 3520 kg/cm^2) minimum yield stress, and a minimum of 0.018 in. (0.46 mm) base metal thickness, having a minimum rib depth of 1-1/8 in. (29 mm), spaced a maximum of 12 in (305 mm) on centers and attached in a manner to effectively inhibit relative movement between the panel and purlin flange,
10. Insulation is glass fiber blanket 0 to 6 in. (152 mm) thick compressed between the member and panel in a manner consistent with the fastener being used,
11. Fastener type is, at minimum, No. 12 self-drilling or self-tapping sheet metal screws or 3/16 in. (4.76 mm) rivets, having washers ½ in. (12.7 mm) diameter,
12. Fasteners are not standoff type screws,
13. Fasteners are spaced not greater than 12 in. (305 mm) on centers and placed near the center of the beam flange, and adjacent to the panel high rib, and
14. The design yield stress of the member does not exceed 60 ksi (410 MPa or 4220 kg/ cm^2)

If the roof system varies from the above conditions, then a full-scale test should be run in accordance with Section F1 of the AISI S100. The current R-values in the S100 are based on the research programs summarized by Fisher (1996) and LaBoube et al. (1988).

2.4 EUROPEAN DESIGN PROVISIONS

The 2007 European Design Code for Steel Structures (Eurocode 3) prescribes a design approach for purlin design that is very different from the one prescribed in the AISI S100. Reviewing this design approach can provide insight to the parameters that affect the strength and stability of these members. Chapter 1-3 of Eurocode 3 provides design guidance specifically for cold-formed steel members. Section 10 titled Special Considerations for Purlins, Linear Trays, and Sheetings, specifically 10.1 titled Beams Restrained by Sheetings provides design provisions for situations similar to the ones designed by AISI S100 Section D6.11.

2.4.1 Design Provisions. Chapter 1-3 Section 10.1- provides guidance for designing purlins or other similar types of members which are attached to sheeting. This section applies to both positive and negative bending moment conditions with or without an additional applied axial load. There are two main design considerations for these members, strength (referred to as resistance of cross-sections) and stability (referred to as buckling resistance of free flange), the latter only applying to situations when the free flange is in compression. The Eurocode provides Figures 2.1, 2.2, 2.3, 2.4, and 2.5 to explain the design procedure used. It is important to note the coordinate system defined by Figure 2.4 as it does vary compared to the coordinate system used in AISI design provisions.

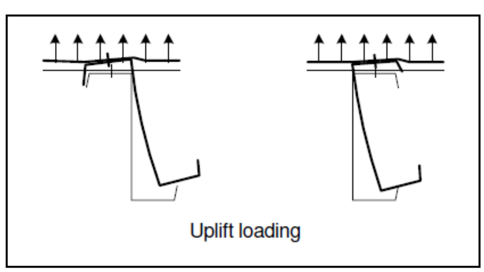


Figure 2.1. Distortion of Purlins Subject to Uplift Loading

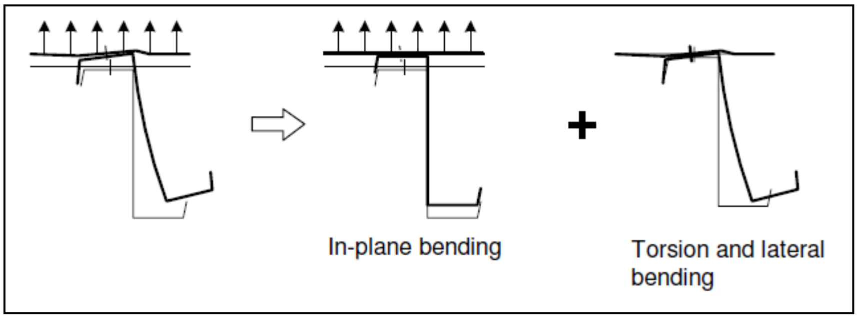


Figure 2.2. Total Deformation Split into Two Parts

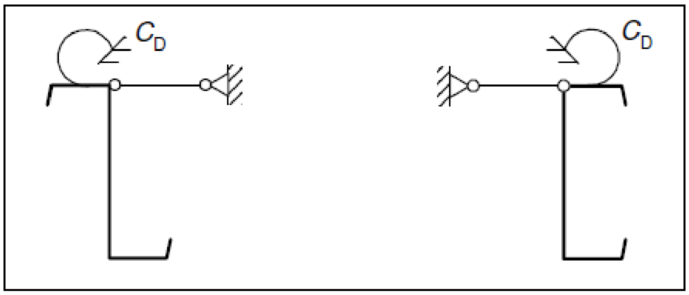


Figure 2.3. Model Purlin as Laterally Braced with Rotational Spring Restraint C_D from Sheeting

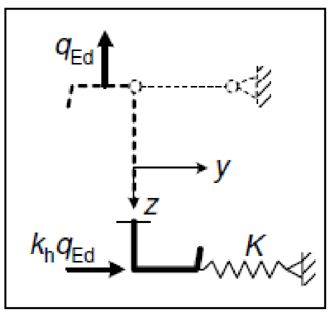


Figure 2.4. Rotational Spring Simplified by a Lateral Spring Stiffness *K*

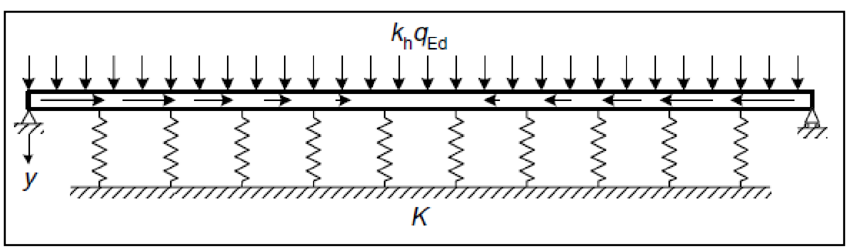


Figure 2.5. Free Flange Modeled as a Spring on an Elastic Foundation

The equation used to analyze the buckling resistance of the free flange in compression of the cross-section superimposes stresses due to in-plane bending, applied axial forces, and the additional stresses acting on the compression flange due to torsion and lateral bending. This equation is as follows:

$$\frac{1}{X_{LT}}\left(\frac{M_{y,Ed}}{W_{eff,y}}+\frac{N_{Ed}}{A_{eff}}\right)+\frac{M_{fz,Ed}}{W_{fz}}\leq f_{yb}/\gamma_{M1} \qquad \text{(Eq. 2.6)}$$

Where,

X_{LT} is the reduction factor for lateral torsional buckling chosen from a country's National Annex

$M_{y,Ed}$ is the moment in the member due to the applied uniform load

N_{Ed} is the applied axial load

A_{eff} is the effective area of the cross-section for only uniform compression

$f_{y,b}$ is the yield strength of the material

$M_{fz,Ed}$ is the bending moment of the free flange due to the lateral load q_{hEd}

$W_{eff,y}$ is the effective section modulus of the cross-section for only bending about the y-y axis

W_{fz} is the gross elastic section modulus of the free flange plus the contributing part of the web for bending about the z-z axis

γ_{M1} is a reduction factor which represents the model uncertainties and dimensional variations

The contributing part of the web for bending about the z-z axis can be taken as one fifth of the web height for Cee and Zee sections unless another method is used to determine the contributing height of the member's web.

To find the moment resulting from torsion and lateral bending, the uplift load (q_h) is multiplied by a k_h factor. This k_h factor is based on the shape of the cross section. To find this k_h factor, first a k_{h0} factor, which represents loading through the shear center, must be found by the following equation:

$$k_{h0} = \frac{I_{yz}}{I_y}\frac{g_s}{h} \qquad \text{(Eq. 2.7)}$$

Where,

G_s is the y distance from the loaded flange to the center of gravity

Then to find k_h for uplift loading,

$$k_h = k_{h0} - a/h \qquad \text{(Eq. 2.8)}$$

Where,

a is the distance in the y direction from the face of the web to the line of fasteners on the loaded flange.

After this is found the equivalent lateral load on the free flange can be found by:

$$q_{h,Ed} = k_h \; q_{Ed} \qquad \text{(Eq. 2.9)}$$

This lateral load can be used to find an internal lateral bending moment in the free flange, $M_{fz,Ed}$. To finding this moment the equations from the table in Figure 2.6 can be used to calculate the initial lateral bending moment (not recognizing the spring support) then multiply this initial lateral bending moment by a correction factor, κ_R which accounts for the spring support.

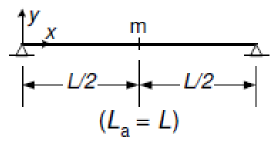
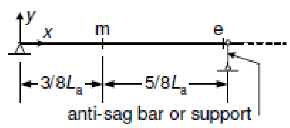
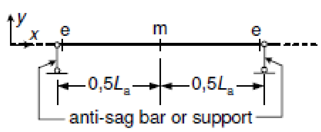
<table>
<tr><th>System</th><th>Location</th><th>$M_{0,fz,Ed}$</th><th>κ_R</th></tr>
<tr><td>
</td><td>m</td><td>$\frac{1}{8} q_{h,Ed} L_a^2$</td><td>$\kappa_R = \frac{1-0,0225R}{1+1,013R}$</td></tr>
<tr><td rowspan="2">
</td><td>m</td><td>$\frac{9}{128} q_{h,Ed} L_a^2$</td><td>$\kappa_R = \frac{1-0,0141R}{1+0,416R}$</td></tr>
<tr><td>e</td><td>$-\frac{1}{8} q_{h,Ed} L_a^2$</td><td>$\kappa_R = \frac{1+0,0314R}{1+0,396R}$</td></tr>
<tr><td rowspan="2">
</td><td>m</td><td>$\frac{1}{24} q_{h,Ed} L_a^2$</td><td>$\kappa_R = \frac{1-0,0125R}{1+0,198R}$</td></tr>
<tr><td>e</td><td>$-\frac{1}{12} q_{h,Ed} L_a^2$</td><td>$\kappa_R = \frac{1+0,0178R}{1+0,191R}$</td></tr>
</table>

Figure 2.6. Internal Moment and Correction Factor

The L_a in the internal moment equations from Figure 2.6 are to be taken as the distance between the antisag bars (known as intermediate braces in the United States) or as the span length of the purlin centerline to centerline of the supports if no antisag bars are present. The "R" in the correction factor equations can be found by the following equation:

$$R = \frac{K\ L_a^4}{\pi^4 EI_{fz}} \qquad \text{(Eq. 2.10)}$$

Where,

I_{fz} is the second moment of area of the free flange plus the contributing part of the web for bending in the z-z axis

K is the lateral spring stiffness per unit length

This K factor is based on joint stiffness of the sheet-to-purlin connection, lateral stiffness due to the distortion of the cross-section, and the flexural stiffness of the sheeting shown by the following equation:

$$\frac{1}{K}=\frac{1}{K_A}+\frac{1}{K_B}+\frac{1}{K_C} \qquad \text{(Eq. 2.11)}$$

Where.

K_A is the lateral stiffness due to the rotational stiffness of the joint between the sheeting and the purlin

K_B is the lateral stiffness due to the distortion of the cross-section

K_C is the lateral stiffness due to the flexural stiffness of the sheeting

However, the flexural stiffness of the paneling is typically small relative to the other two contributing sources of lateral stiffness and thus is often neglected. K can be found by either testing or calculation. An equation which can be used for calculating K is as follows:

$$\frac{1}{K}=\frac{4(1-\nu^2)h^2(h_d+b_{\mathrm{mod}})}{Et^3}+\frac{h^2}{C_D} \qquad \text{(Eq. 2.12)}$$

Where (for uplift loading),

$b_{mod} = 2a + b$

t is the thickness of the purlin

a is the distance in the z-direction from the sheet-to-purlin fastener to the purlin web

b is the width of the purlin flange connected to the sheeting

C_D is the total rotational spring stiffness

h is the overall height of the purlin

h_d is the developed height of the purlin web (=h for zee-sections)

The total rotational spring stiffness is the reciprocal of the sum of the reciprocal of the rotational stiffness due to the connection between the sheeting and the purlin ($C_{D,A}$) and the reciprocal of the rotational stiffness due to the flexural stiffness of the sheeting ($C_{D,C}$). However, $C_{D,C}$ is often neglected especially if the effects of cross-sectional distortion must be considered. $C_{D,A}$ can be found by the following equation:

$$C_{D.A} = C_{100}\ k_{ba}\ k_t\ k_{bR}\ k_A\ k_{bT} \qquad \text{(Eq. 2.13)}$$

Where (for uplift loading),

$k_{ba} = (b_a/100)^2$ if $b_a < 125$mm

$k_{ba} = 1.25\ (b_a/100)$ if $125\text{mm} <= b_a < 200\text{mm}$

$k_t = (t_{nom}/0.75)^{1.1}$ if $t_{nom} \geq 0.75$mm; positive position

$k_t = (t_{nom}/0.75)^{1.5}$ if $t_{nom} \geq 0.75$mm; negative position

$k_t = (t_{nom}/0.75)1.1$ if $t_{nom} < 0.75$mm

$k_{bR} = 1.0$ if $b_R \leq 185$mm

$k_{bR} = 185/b_R$ if $b_R > 185$mm

$k_A = 1.0$

$k_{bT} = (b_{T,max}/b_T)^{0.5}$

And where,

b_a is the width of the purlin flange (in mm)

b_R is the corrugation width

b_T is the width of the sheeting flange (called panel rib in the United States) through which it is fastened to the purlin

$b_{T,max}$ is given in the Table 2.5

C_{100} is a rotational coefficient, representing the value of $C_{D,A}$ if ba is 100mm also given in the following table provided that there is no insulation used

Table 2.5. C_{100} and $B_{T,max}$ Values

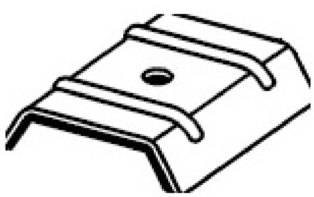
Positioning of sheeting		Sheet fastened through		Pitch of fasteners		Washer diameter [mm]	C_{100}	$b_{T,max}$
Positive 1)	Negative 1)	Trough	Crest	$e = b_R$	$e = 2b_R$		[kNm/m]	[mm]
For gravity loading:								
×		×		×		22	5,2	40
×		×			×	22	3,1	40
	×		×	×		K_a	10,0	40
	×		×		×	K_a	5,2	40
	×	×		×		22	3,1	120
	×	×			×	22	2,0	120
For uplift loading:								
×		×		×		16	2,6	40
×		×			×	16	1,7	40

Key:

b_R is the corrugation width;

b_T is the width of the sheeting flange through which it is fastened to the purlin.

K_a indicates a steel saddle washer as shown below with $t \geq 0,75$ mm

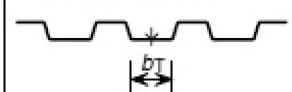


The values in this table are valid for:

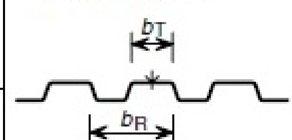
- sheet fastener screws of diameter: $\phi = 6,3$ mm;
- steel washers of thickness: $t_w \geq 1,0$ mm.

Sheet fastened:

- through the trough:



- through the crest:



Alternatively $C_{D,A}$ can be taken as 130p where p is the number of sheet to purlin fasteners per meter. This is only valid if the panel rib through which the purlin is connected is not greater than 120mm, the panel thickness is at least 0.66 mm, and the distance between the fastener and center of rotation of the purlin is at least 120 mm.

2.4.2 Comparison to Other Literature. When comparing the design provisions from the Eurocode to previous research that has led to the AISI S100 provisions there are several things that are noticed. One similarity is that the Eurocode does recognize that the paneling does provide support for the compression flange of a purlin when under uplift loading. The Eurocode also provides an equation to determine if the system can be considered fully laterally braced, this is something that AISI S100 does not provide.

Some similarities between the Eurocode provisions and the research done in the United States which ultimately lead to the current AISI provisions can be seen. The Eurocode does recognize that this type of system can be represented by an elastic spring to the bottom flange, and that joint flexibility is an important parameter to determine the strength of this elastic spring. This is similar to the findings of Haussler and Pabers (1973). Pekoz and Soroushian (1982) determined that the system could be represented by a beam column on an elastic foundation. This idea is also presented in the Eurocode as shown in Figure 2.4.

In LaBoube's report "Roof Panel to Purlin Connection: Rotational Restraint Factor" (LaBoube 1986) several parameters were found to influence the rotational stiffness of the panel-to-purlin connection. It is interesting to see that the equations for finding the lateral spring support from the panel-to-purlin connection in the Eurocode include many of these parameters. Some of these parameters include: purlin thickness, fastener location relative to the purlin web, and height of the purlin. The thickness of the paneling is also accounted for in the Eurocode when evaluating $C_{D,A}$, which is used to find the stiffness of the rotational spring due to the connection between the panel and purlin.

Haussler (1988) recognized that the effects of member distortion and panel bending would affect the bracing support of the free flange under compression. This is recognized in the Eurocode as K_B and K_C as shown in Equation 2.11. Previous research recognized that simple span and continuous span members behaved differently. The Eurocode also recognizes this and accounts for the difference for both uplift and gravity loading cases.

3. EXPERIMENTAL INVESTIGATION

3.1. INTRODUCTION

The experimental investigation was conducted in collaboration with NCI Building Systems located in Houston, Texas. Both a simple span and continuous span test setup were required. The testing was performed at the MBCI testing facility also located in Houston, Texas because the university did not have the resources to perform the continuous span test.

Two twelve inch deep Z-purlin uplift tests were performed at NCI Building System's test laboratory to confirm that purlins with deeper cross-sections would meet or exceed the strength requirements achieved by the previous R-value tests and the AISI S100 provisions. The previous tests did not include purlins this deep and the AISI Specification does not include R-values for members of this web depth.

3.2. SCOPE OF INVESTIGATION

The investigation included two tests; one continuous span test and one simple span test. These tests were considered to be confirmatory tests with the goal of meeting or exceeding the required strength computed using the current AISI S100 R-values.

3.3. TEST PARAMETERS

The test setups were as similar as possible to the previous R-value tests; the major change being the larger web depth of the purlins. Since AISI S100 has limitations to the design equation, comparisons were done between the test setup and the equation limitations. Refer back to Section 2.3.3 for the list of limiting conditions as they appear in the AISI S100.

Condition (1) states that the member depth is not to exceed 11.5 inches. This is the varied parameter in this test program. Twelve inch deep Z-purlins were used in this set of tests. Edge stiffeners are required by condition (2). The purlins that NCI rolled for these tests had edge stiffeners of an average length of 0.9375 inches and an approximate angle of 50 degrees from the plane of the flange.

Condition (3) places depth-to-thickness ratio limitations of less than 170 and greater than 60. With purlins of 12” depth and 0.073” thickness, the h/t ratio for these test purlins was 164.4 which falls between the upper and lower limits for this condition.

Currently Condition (4) sets a depth-to-flange width ratio requirement of between 2.8 and 4.5. Due to the larger web depth of this test purlin, this limitation was not met. This ratio was slightly higher at 5.33 with a given flange width of 2.25 inches.

Condition (5) sets limitations for the flat width/thickness ratio. This ratio needs to be between 16 and 43. This value was calculated to be 31; therefore, this condition was met.

As stated in Condition (6), the lap length for the continuous span test should not be less than 1.5d. The lap length was 19.5 inches which is greater than 1.5d (18” for 12” deep sections).

Condition (7) sets a limitation on the span length at 33 feet. The largest span length was 30’1/8”, thus this condition was also met.

Purlins were restricted from lateral movement at the supports as Condition (8) states. The purlins were connected with flange bolts. The typical connection used in these tests is shown in Figure 3.1.

Figure 3.1. Flange-Bolted Connection

MBCI’s PBR panel was used for both tests. A section detail of the panel is shown in Figure 3.2, and a full detail of the panel can be found in Appendix C.

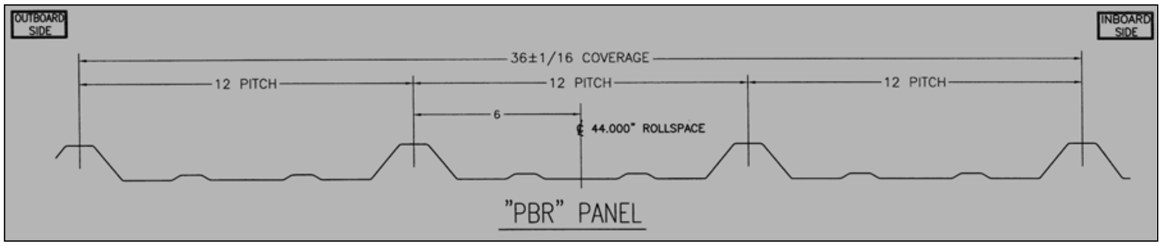


Figure 3.2. MBCI’s PBR Roof Panel Profile

This panel meets Condition (9), which states that the panel should be steel sheets with a minimum of 50 ksi yield stress. The thickness of the base sheet metal was 26 gage (0.018 inches thick) which meets the minimum 0.018 in. thickness requirement. The high ribs are 1.25 inches high and are spaced at 12 inches center to center, which also meets the AISI S100 stated criteria.

Condition (10) gives stipulations on insulation; however, no insulation was used in this set of tests.

The type of screws used for the panel-to-purlin connection were UltiMates #12-14 x 1 1/4" Long-Life Self-Drilling Screws made by Atlas Fasteners. These screws meet Condition (11) of Section D6.1.1.

These screws were not standoff type screws; therefore, Condition (12) was not applicable to this setup.

The location of these screws does affect the R-value of a given system; therefore, Condition (13) regulates the spacing and location of the screws connecting the paneling to the purlins. It states that fasteners were not to be spaced farther than one foot on center, located near the center of the beam flange, and adjacent to a high rib on the panel. The test specimens were assembled with this condition in mind.

Condition (14) states that the design yield stress of the purlin material was not to exceed 60 ksi. The specified yield stress of the material used for the purlins in these tests was 57 ksi. The material, however, tested to be on average 74.7 ksi which is much higher than the specified 60 ksi limitation. With an average ultimate strength of 92.2 ksi, the material still had a F_u/F_y ratio of 1.23. The yield strength limitation was not met; yet, the ductility was such that the material met AISI S100 minimum ductility requirements.

3.4. TEST SETUP

For both the simple span and the continuous span tests, a test chamber was set up similar to AISI S908: Base Test Method for Purlins Supporting Standing Seam Roof System (AISI 2004). This test standard exists to test standing seam panels under gravity or uplift conditions. AISI S908 prescribes a setup for a simple span simulated wind uplift or gravity loaded roof system. This test standard was also used as a guide to set up the continuous span chamber.

3.4.1. Test Fixture. A test chamber was used to support the roof system and to hold pressure to simulate wind uplift. Pictures as well as sketches of the chambers used for the simple span and continuous span tests are shown in Figures 3.3, 3.4, 3.5, and 3.6.

Figure 3.3. Simple Span Chamber

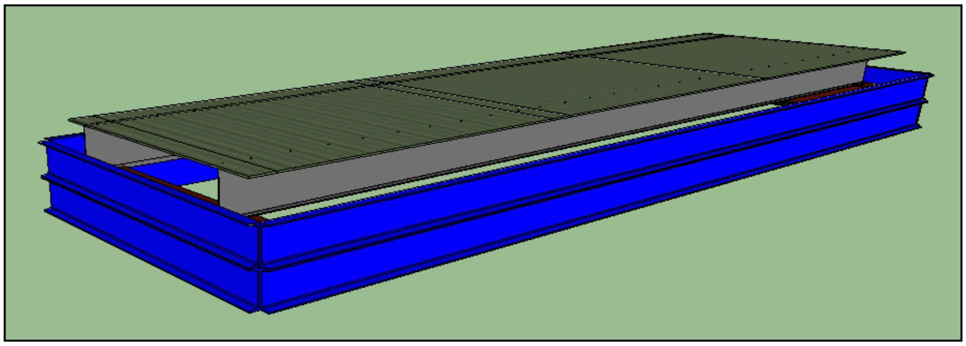


Figure 3.4. Simple Span Chamber Sketch

Figure 3.5. Continuous Span Chamber

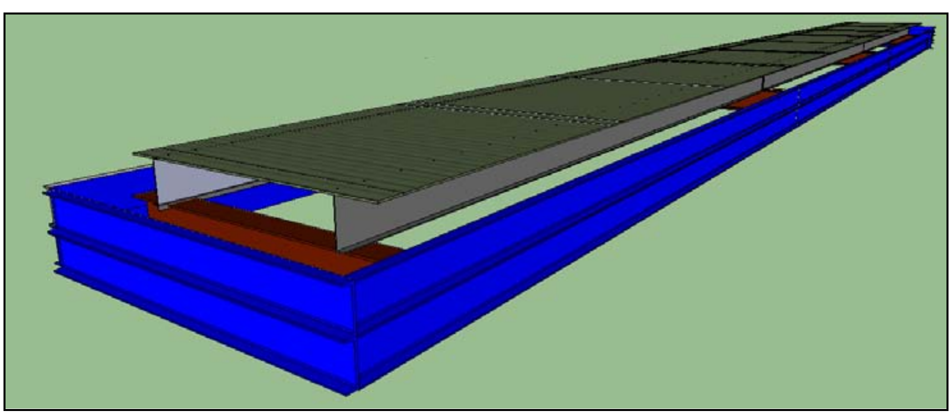


Figure 3.6. Continuous Span Sketch

Structural channels were used to form the walls of the chamber (shown in blue in the sketches) and W 12x40 sections were used for the support beams (shown in grey in the sketches). The beams that formed the walls of the chamber were secured to the concrete floor, and the support beams where connected to the chamber with 5/8 inch A325 structural bolts. Seams and holes were filled with caulking to make the chamber more air tight. The chamber-to-floor connection and the purlin support beam-to-chamber connection are shown in Figures 3.7 and 3.8.

Figure 3.7. Chamber-to-Floor Connection

Figure 3.8. Purlin Support Beam to Chamber Connection

The purlins were connected by a flange bolted connection to clips that were connected to the support beams shown in Figure 3.9. The purlins were assembled facing each other to achieve anti-rolling. For the continuous span test, the laps, which were measured from centerline of the purlin support beam to the end of the lapped purlin sections, were 19.5 inches long. A typical lap connection for the continuous span test is shown in Figure 3.10.

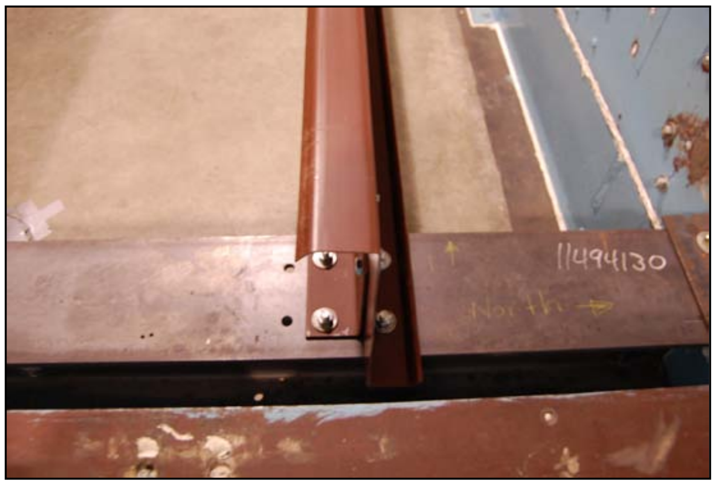


Figure 3.9. Purlin to Support Beam Connection

Figure 3.10. Continuous Span Typical Lap Connection

Once the purlins were assembled to the tests chamber, six mil polyethylene plastic was used to create a more airtight seal to help hold pressure for the test. This plastic was folded in such a way so that it would not add any lateral support in addition to the support given by the panels. This ensured that the test specimen did not gain strength from the plastic which would not be present in systems on actual buildings. Thicker 60 mil EPDM black plastic lined the edges of the panels to ensure that the metal would not tear the thinner plastic. One inch by one inch angles were then screwed to the edges of the panels. See Figures 3.11 and 3.12 for details.

Figure 3.11. Six Mil. Polyethylene Plastic

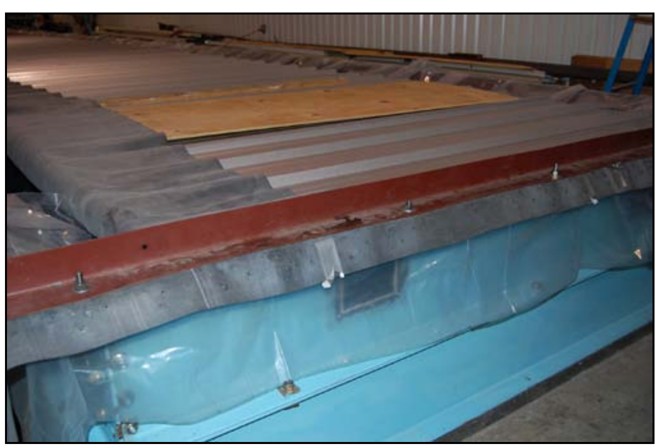


Figure 3.12. Sixty Mil EDPM Lining and Angle Connections

As illustrated by Figure 3.13, measurements were then taken so that tributary areas could be determined. The tributary widths for the individual purlins were determined as the average length between the two purlins divided in half (X) plus the average length of the screw to the edge of the panel on a given side (Y) plus half the average length of edge of panel to edge of chamber (Z).

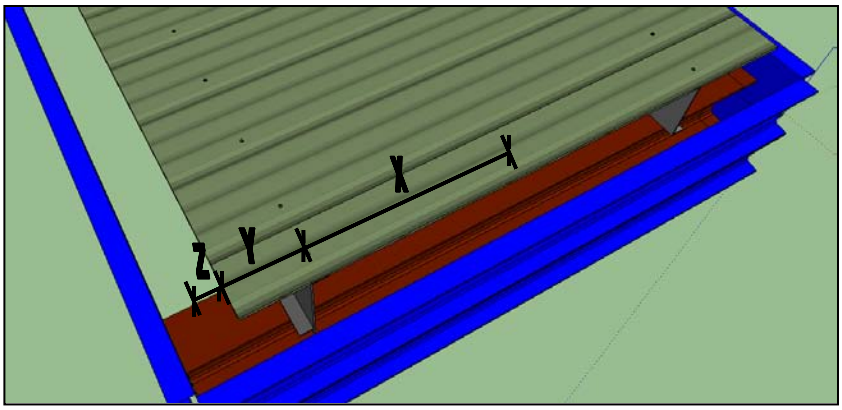


Figure 3.13. Determination of Tributary Width

Deflection gauges were set up at midspan of the simple span chamber and at midspan of the west span of the continuous span chamber as seen in Figure 3.14. An auto-level was used to read these gauges.

Figure 3.14. Deflection Gauges

Compressors were used to apply pressure to the inside of the chamber, which simulated an uplift wind load. The compressor fans were Cincinnati model # HP-12E26, and the motor is a Baldor Motor with 40 Horse Power. The motor can spin up to 3525 RPM and has a frequency of 60 HZ. The compressor to chamber hookup can be seen in Figure 3.15.

Figure 3.15. Compressor Hookup

For the continuous span test, digital manometers were used to check the pressure readings at various locations in the big chamber. During the testing, when the chamber was under pressure, the readings were consistent with each other; thus validating that the entire system was under uniform pressure.

3.4.2. Test Specimens. The purlins used in these tests were manufactured by NCI Building Systems. The steel's specified minimum yield strength of 57 ksi and an ultimate strength specified at 70ksi.

The same type of paneling and the same size purlins were used in both the simple span and the continuous span tests. The purlins were formed with a depth of 12 inches. They had equal flanges of approximately 2.25 inches and flange stiffeners approximately 0.9375 inches in length. A typical purlin shape is shown in Figure 3.16. and the actual measured values are shown in Table 3.1.

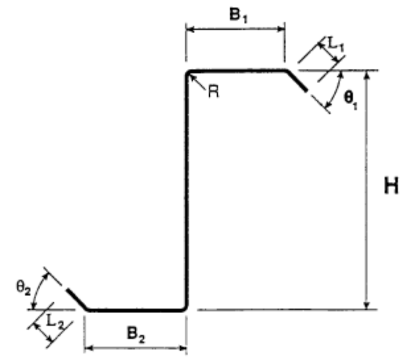


Figure 3.16. Cross-Sectional Shape

Table 3.1. Test Purlin Dimensions

Purlin	H (in)	Thickness (in)	B_1 (in)	B_2 (in)	L_1 (in)	L_2 (in)	θ_1 (deg)	θ_2 (deg)	Radius (in)
1	12.0	0.075	2.25	2.3125	1	0.875	50	50	0.1875
2	12.0	0.073	2.25	2.25	0.9375	0.9375	47	47	0.1875
3	12.0	0.073	2.25	2.25	0.9375	0.9375	47	49	0.1875
4	12.0	0.073	2.25	2.3125	0.9375	0.9375	48	49	0.1875
5	12.0	0.074	2.25	2.25	0.9375	0.9375	46	47	0.1875
6	12.0	0.0735	2.25	2.1875	0.875	0.9375	46	47	0.1875
Average	**12.0**	**0.0736**	**2.25**	**2.2604**	**0.9375**	**0.9271**	**47.3**	**48.2**	**0.1875**

4. TEST PROCEDURE

4.1. INTRODUCTION

Like the test setup, the procedure for these tests followed closely to the procedure prescribed in AISI S908: Base Test Method for Purlins Supporting Standing Seam Roof System (AISI 2004).

4.2. CHAMBER LOADING PROCEDURE

The test specimen was loaded with approximately 5 psf of pressure and held for 60 seconds. AISI S908 states that this initial pressure load shall be applied and released to set a zero pressure reading. Pressures were checked in various locations to ensure that there was uniform loading. The chamber pressure was then zeroed and the test loading began. The pressures were measured both by a water manometer and an electronic differential pressure manometer. Applied loading was measured in inches of water and then converted to pressure, as summarized by Table 4.1. Each pressure interval indicated by Table 4.1 was held for 60 seconds before increasing the pressure to the next level.

Table 4.1. Test Pressure Intervals

Pressure Interval	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
Inches of water	0.00	1.00	2.00	3.00	4.00	4.50	5.00	5.25	5.50	5.75	6.00	6.25	6.50	6.75	7.00	7.25	7.50	7.75	8.00	8.25	8.50
psf	0.0	5.2	10.4	15.6	20.8	23.4	26.0	27.3	28.6	29.9	31.2	32.5	33.8	35.1	36.4	37.7	39.0	40.3	41.6	42.9	44.2

Load was applied to the test specimen until failure of a purlin occurred. Failure was a buckling of the cross-section at the location of maximum moment. Once the purlins buckled, no additional load was applied and the test was stopped.

Horizontal and vertical deflections were measured at every pressure interval. The deflection measurements were taken at midspan of the simple span test. The continuous span deflection measurements were taken at midspan of the end span.

5. TEST RESULTS

5.1 INTRODUCTION

Upon completion of the load test, the purlins were visually examined. In both tests, the failure mode was determined to be lateral-torsional buckling. A typical failed purlin is shown in Figure 5.1.

Figure 5.1. Typical Purlin Failure

After examining the failure, the test specimens were disassembled and coupons were cut from the failed purlins for material properties.

5.2. MATERIAL TESTING

Three coupon tests were performed according to ASTM A370 to find the tested yield and ultimate strengths of the failed purlins. A typical stress-strain curve is show in Figure 5.2.

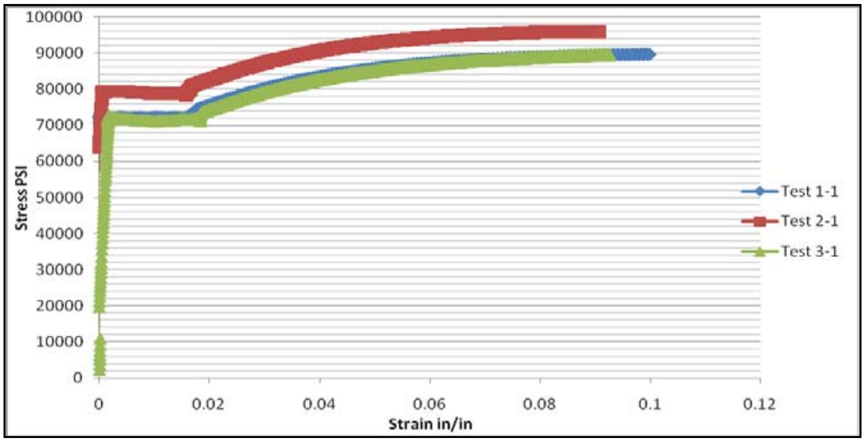


Figure 5.2. Stress-Strain Curve

The average yield strength for the three coupons was 74.7 ksi, and the average ultimate tensile strength was found to be 92.2 ksi. This gives the F_u/F_y ratio of 1.23. The percent elongation was also determined to be an average elongation of 10.3%.

5.3. SIMPLE SPAN TEST RESULTS

A graph of the simple span deflection readings is shown in Figure 5.3. The horizontal deflections were minimal. The vertical deflections of the north and south purlins were comparable and varied fairly linearly with increased pressure.

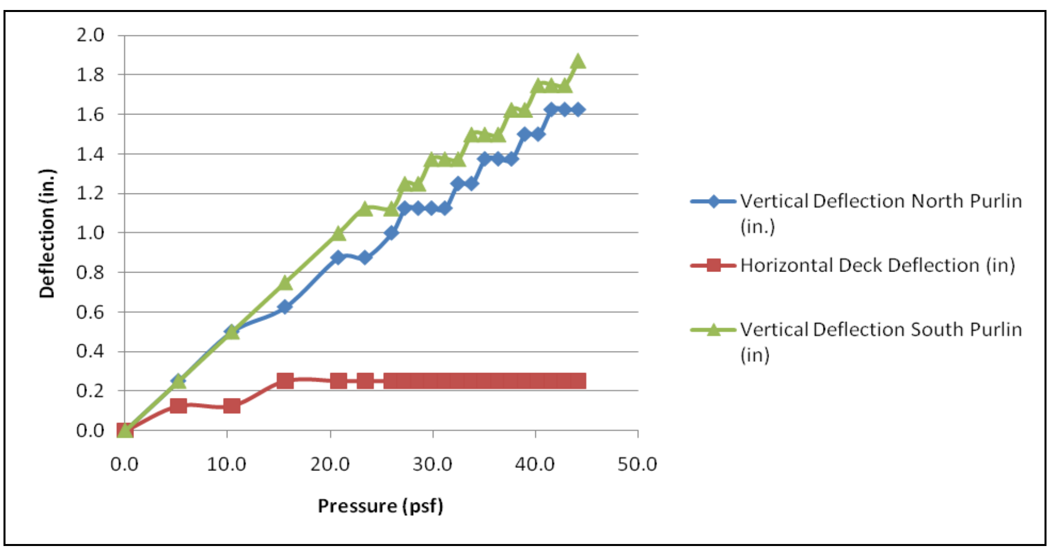


Figure 5.3. Simple Span Test Deflections

A failure pressure of 44.2 psf was recorded for the simple span test. Subtracting the panel weight of 0.91 psf yielded a uniform uplift load of 43.29 psf for the whole area of the chamber. With a tributary width for the failure purlin of 3.29 feet, and a member weight (subtracted out) of 4.51 plf, the uniform load on the purlin was computed as 137.9 plf. Since the purlin was simple span, the maximum applied moment was found as follows:

$$M_u = \frac{wL^2}{8} \quad \text{(Eq. 5.1)}$$

Where,

w= is the uniform load applied to the purlin, kip/ft

L= length of the purlin from centerline to centerline of support beams

The maximum applied moment for the simple span test was computed as 10.35 kip feet.

Based on the coupon test results per ASTM A370, the average yield stress of the tested purlins was found to be 74.7 ksi. The section modulus of the purlin was then computed to be 2.78 in^3. The fully braced moment capacity of the purlins was found as follows:

$$M_N = S_e F_y \quad \text{(Eq. 5.2)}$$

Where,

S_e = effective section modulus at Fy (74.7 ksi) = 2.78 in.^{3}

F_y = yield stress of the material = 74.7 ksi

This fully braced moment capacity of the simple span test purlin was 17.31 kip-feet. The R-value was determined by dividing the actual tested moment capacity by the calculated fully braced moment capacity. The R-value for the 12 in. simple span Zee purlin uplift test was computed to be 0.60. A summary table of these simple span calculations can be found in Appendix D: Summary Table of Test Results.

5.4. CONTINUOUS SPAN TEST RESULTS

A graph of the continuous span deflection readings is shown in Figure 5.4. Similar to the simple span test deflections, the horizontal deflections were minimal. The vertical deflections of the north and south purlins were comparable and varied fairly linearly with increased pressure. For the horizontal deflection readings positive deflections are deflections to the right while negative deflections are deflections to the left.

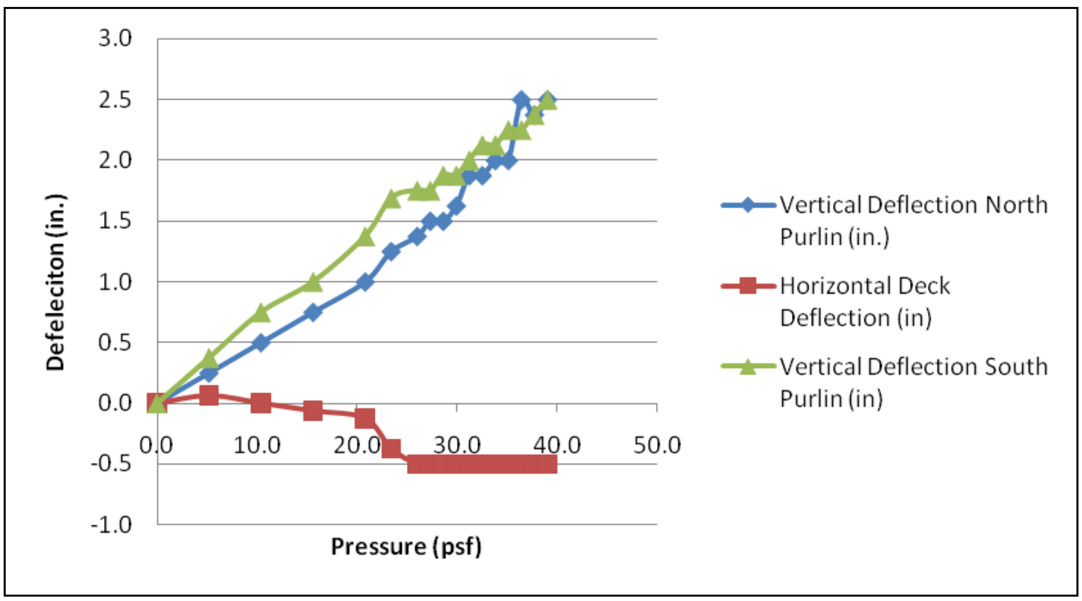


Figure 5.4. Continuous Span Test Deflections

The continuous span test reached a pressure of 40.04 psf when both end span purlins failed almost simultaneously. Subtracting the panel weight yielded 39.13 psf applied load. The tributary area for the failed purlin was 5.01 feet. Multiplying the 39.13

psf by the tributary width and subtracting the purlin weight resulted in a uniform load on the purlin of 191.5 plf. The span of the failed purlin was 30.01 feet. The maximum moment of the failed purlin was found by the following:

$$M_u = 0.08wL^2 \qquad \text{(Eq. 5.3)}$$

The maximum moment was found to be 13.8 kip-ft.

The fully braced moment capacity of the purlin was the same as in the simple span purlin test, 17.31 kip-feet. Dividing the applied moment by the moment of a fully braced purlin gives an R-value of 0.80. A summary of the continuous span test results are found in Appendix D: Summary Table of Test Results.

6. ANALYSIS OF TEST RESULTS

6.1. INTRODUCTION

Since the goal of the tests was to expand the depth limitation of the previous R-values for members with depths of 11.5 inches, a statistical analysis was performed to determine how well the new 12 inch deep test data fit into the previous test data.

6.2. SIMPLE SPAN TEST RESULT ANALYSIS

Appendix E: Statistical Analysis shows comparisons between the average R-value and the standard deviation for the previous test data only and the average R-value and standard deviation including the 12 inch test data. The average R-value for the previous simple span tests with purlins of depths ranging from 8.5 inches to 11.5 inches was 0.6333. When adding in this test result for this simple span test with purlins 12 inches deep, the average improved to 0.632. This shows that the test results for the 12 inch Z-purlins fit into the range of the previous test data.

6.3. CONTINUOUS SPAN TEST RESULT ANALYSIS

Similarly to the simple span 12 inch Z-purlin test, the statistical analysis (shown in Appendix E) proves that the continuous span 12 inch deep test result fits within the range of previous test data for a similar setup. The previous average without this test data included was 0.6964 while the new average with this test data included is 0.7031. Summary tables of these comparisons can also be found in Appendix E: Statistical Analysis.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1. CONCLUSIONS

Based on the findings of this test program, 12" deep Z-purlins do meet or exceed the required strength computed using the current AISI S100 R-values. The R-value for the 12" deep Z-purlin simple span test, 0.60, surpasses the S100 specified 0.50 R-value for purlins up to 11.5" deep. Similarly the continuous span test's R-value of 0.80 surpasses the specified 0.70 for the 11.5" Z-purlins.

7.2. RECOMMENDATIONS

Based on this study it is recommended that, the limitations of Section D6.1.1 be expanded to include 12" Zee purlins. The tables in Appendix E support this recommendation. It is recommended that condition (1) state:

(1) Member depth ≤ 11.5 in. for C-sections and ≤ 12 in for Z-sections

When analyzing the F_u/F_y ratios of the steel used in previous R-value tests, it was found that the lowest F_u/F_y ratio was 1.21. It is suggested that condition (14) of Section D6.1.1 be changed to state that the steel's F_u/F_y ratio should not be less than 1.20. The data for the ultimate and yield stresses for the Fisher tests were used to determine the lowest previous F_u/F_y ratio used. A table of F_u/F_y ratios for purlins from the Fisher's report and from the tests performed at NCI in 2009 are presented in Appendix F: F_u/F_y Ratio Table. Therefore, it is also recommended that Condition (14) of Section D6.1.1 be changed to ensure a ductile steel be used rather than limiting the yield stress of the material. The original intent of this limitation was to ensure a ductile failure. The yield stress does not need to be limited for the use of this equation. It is suggested that condition (14) of Section D6.1.1 state the following:

(14) The F_u/F_y ratio of the member ≥ 1.20.

With the inclusion of deeper members, it is also recommended that the limitation of the depth-to-flange width ratio be extended from the current value of 4.5 to a value of 5.5.

8. RECOMMENDATIONS FOR FUTURE RESEARCH

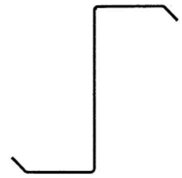
Because R-values are empirical, the use of this approach to design girts and purlins under suction loading comes with several limitations (or conditions as summarized in Section 2.3.3). As the metal building industry undergoes changes in member dimensions, sheet metal properties, fabrication procedures or other similar changes, other desires to extrapolate the current conditions, such as the depth limitation being expanded to include 12” deep members, may arise. Future research will then be needed to ensure that these systems do, in fact, meet or exceed current strength requirements.

More research may also be to clarify Condition (8) which states that both flanges are to be prevented from moving laterally at the supports. The tests that the current R-values are based on all had similar purlin to rafter connections; purlins were flange-bolted to the supporting members. Web-bolts or other connection methods may not provide the same lateral or rotational restraint as the flange-bolted setup provides. More research may be needed to determine what exactly will prevent a purlin from moving laterally at the supports.

Since more and more metal building companies are also using standing seam panels rather than through-fastened panels, R-values for these standing seam panels could be researched in the future as well.

APPENDIX A

EFFECTS OF RECOGNIZING PARTIAL BRACING PROVIDED BY THROUGH-FASTENED SHEATHING



Z 12 x 2.25 x 15/16

$F_y = 55\ ksi$

$F_u = 70\ ksi$

$K_y = K_x = K_t = 1.0$

$L_y = L_x = L_t = 30.01\ ft = \ 360.12\ in$

Section C3.1.2 (No lateral support from roof sheathing)

$$\sigma_{ey} = \frac{\pi^2 E}{\left(\dfrac{K_y L_y}{r_y}\right)^2} = 2.355\ ksi$$

Where,

$E = 29500\ ksi$

$r_y = 1.0243\ in$

$$\sigma_t = \frac{1}{A\, r_o^2}\left[GJ + \frac{\pi^2 EC_w}{(K_t L_t)^2}\right] = 4.143\ \text{ksi}$$

Where,

$A = 1.2807\ in^2$

$r_o = 4.5259\ in$

$G = 11300\ ksi$

$J = 0.002170\ in^4$

$C_w = 37.475\ in^6$

For a point symmetric section

$$F_e = \frac{C_b r_o A}{2S_f}\sqrt{\sigma_{ey}\sigma_t} = 2.1823\ ksi$$

Where,

$S_f = 4.1483\ in^3$

$C_b = 1.0$

$0.56F_y = 30.8$ ksi

$F_e \le 0.56F_y$

$\therefore F_c = F_e = 2.1823\ ksi$

$S_c = 3.4017\ in^3$

$M_n = S_c\ F_e = 7.4234\ kip\text{-}in$

Section D6.1.1 (Recognizing lateral support from roof sheathing)

$$M_n = R\, S_e\, F_y$$

Where (for continuous spans Z-sections),

$$R = 0.70$$
$$S_e = 3.4017\ in^3$$

$$M_n = 130.97\ kip - in$$

Conclusion

For this particular example, recognizing the bracing support from the sheathing will greatly increase the design capacity of the system. The calculated design strength without the recognition of the bracing support provided by the paneling is so small for this setup, a span length of this length likely would not have even been designed if the S100 did not have provisions that recognized the lateral support from the through fastened sheathing.

APPENDIX B

GRAPHICAL RESULTS FROM LABOUBE'S "ROOF PANEL TO PURLIN CONNECTION: ROTATIONAL RESTRAINT FACTOR" REPORT (LaBoube 1986)

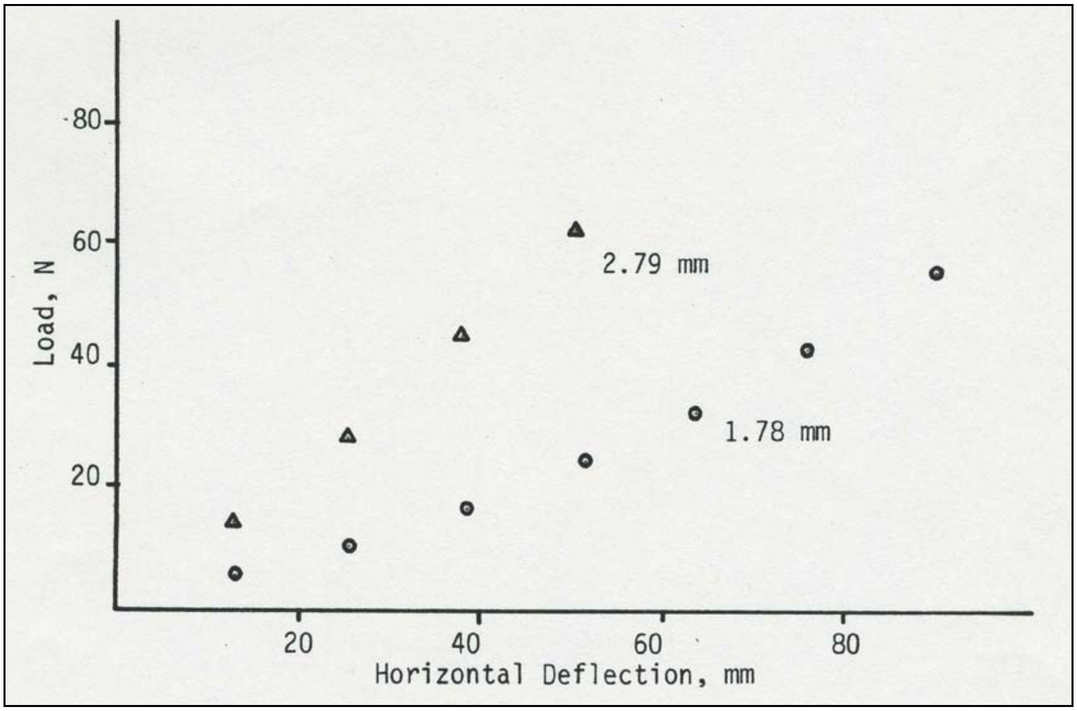


Figure B.1. Affect of Purlin Thickness

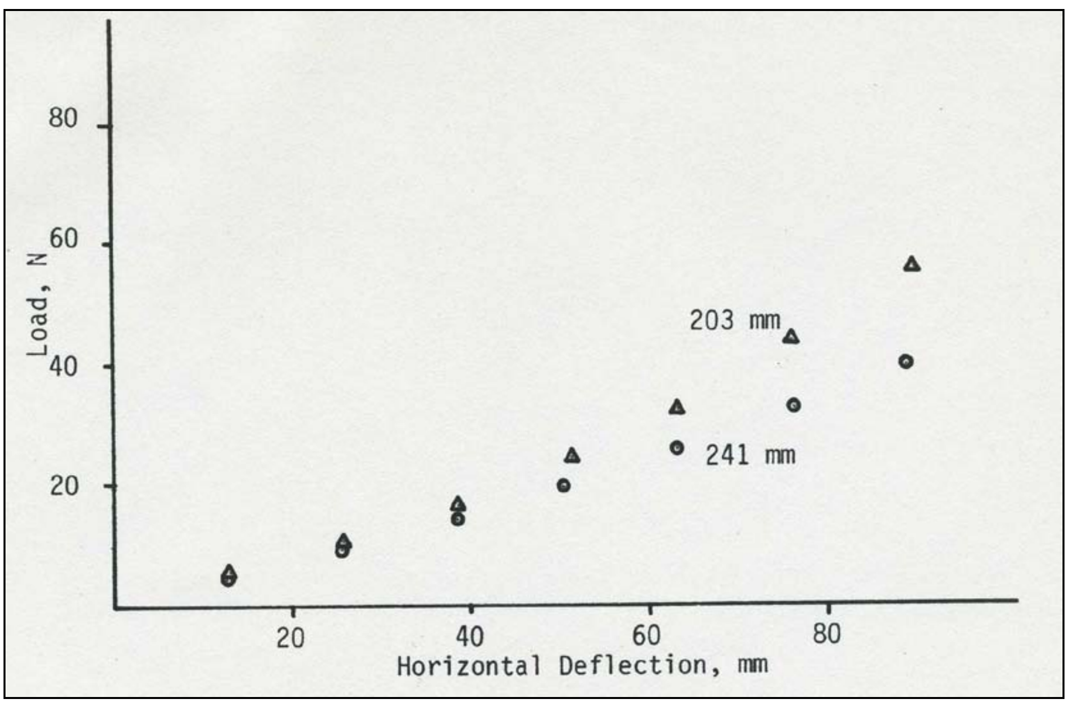


Figure B.2. Affect of Purlin Depth

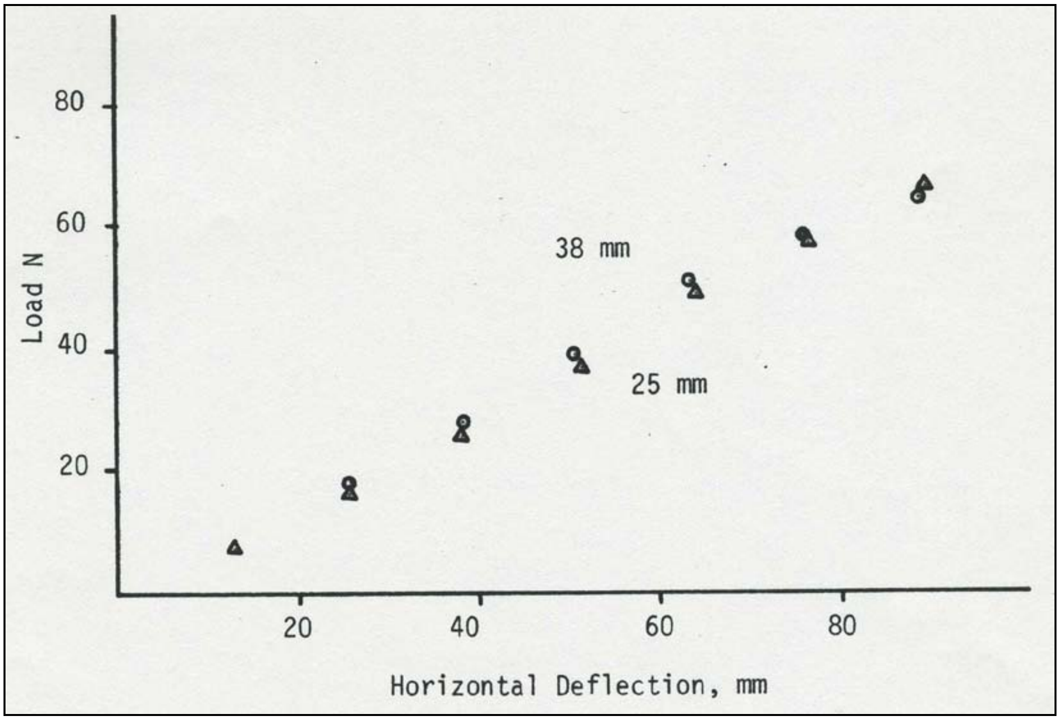


Figure B.3. Affect of Panel Depth

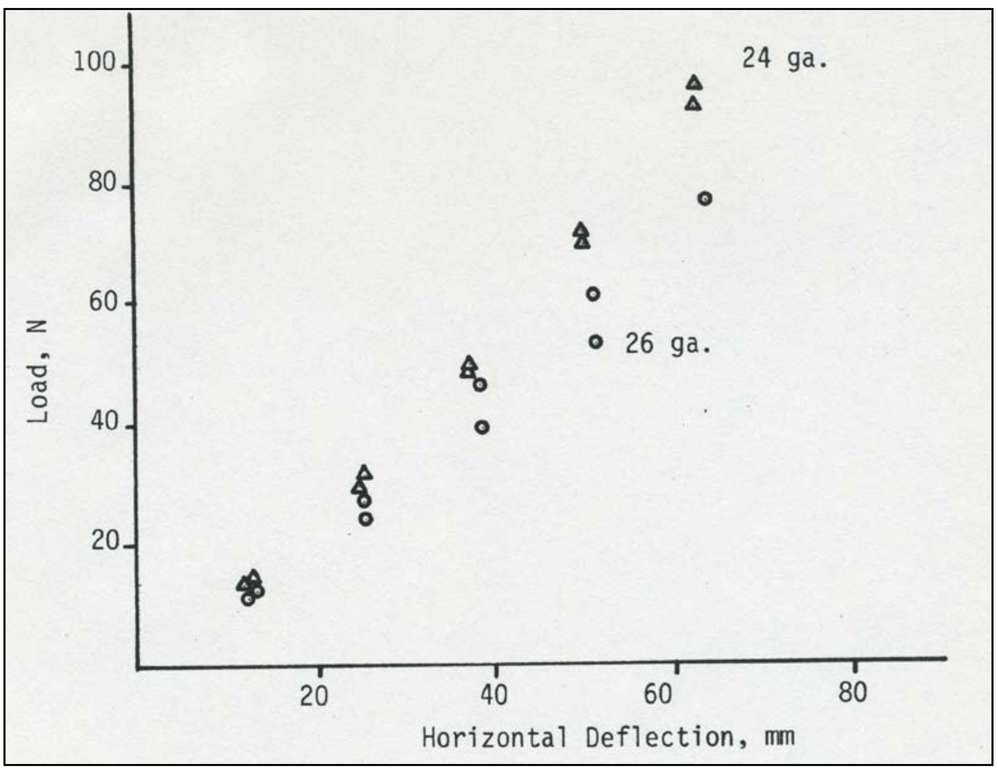


Figure B.4. Affect of Panel Thickness

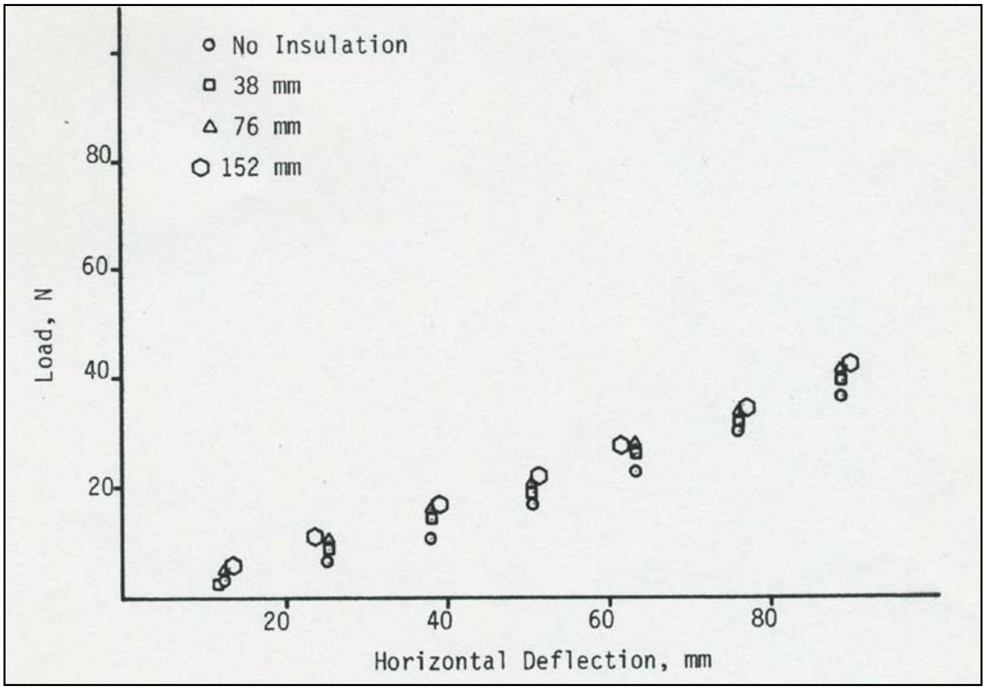


Figure B.5. Affect of Insulation Thickness

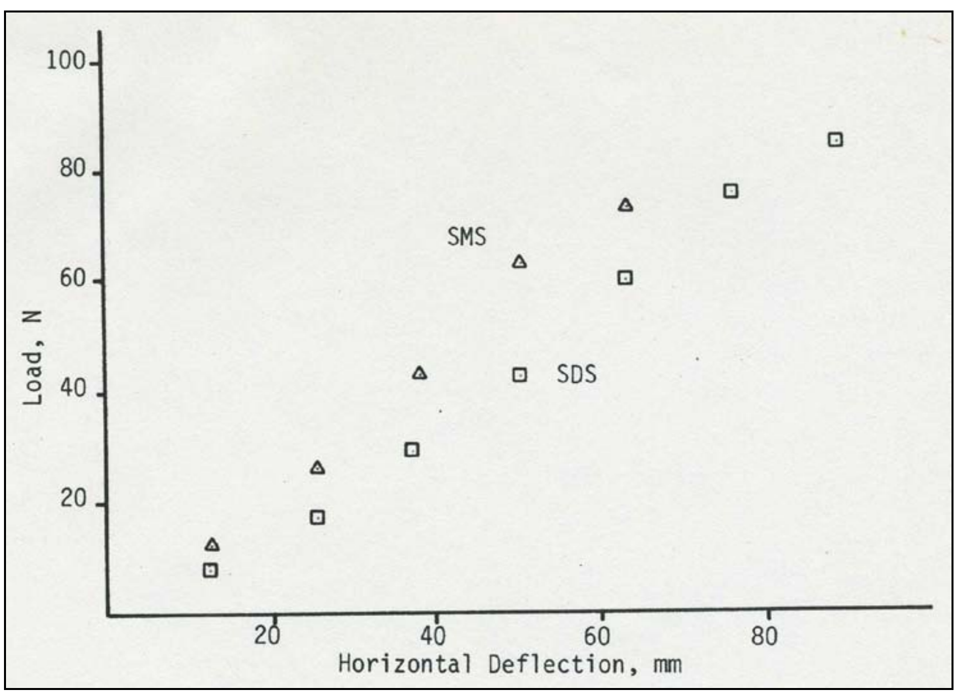


Figure B.6. Affect of Fastener Type

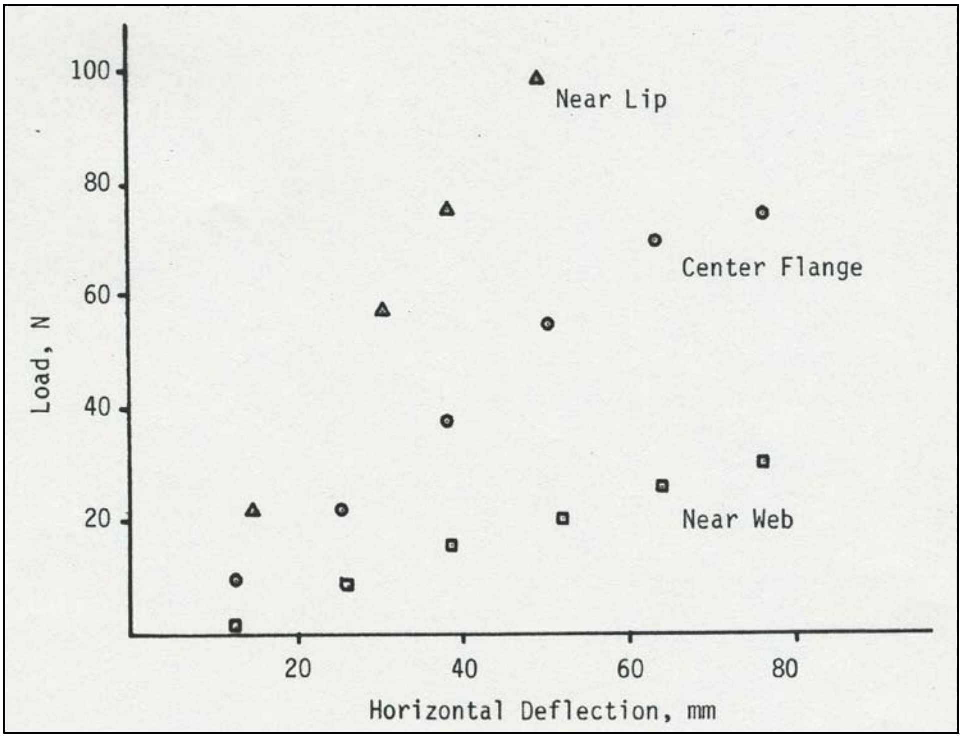
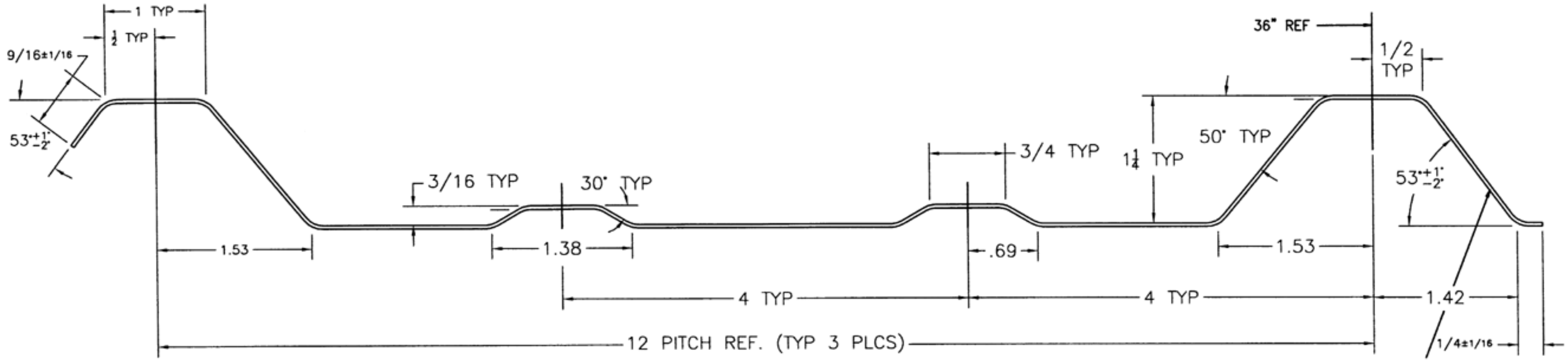


Figure B.7. Affect of Fastener Location

APPENDIX C

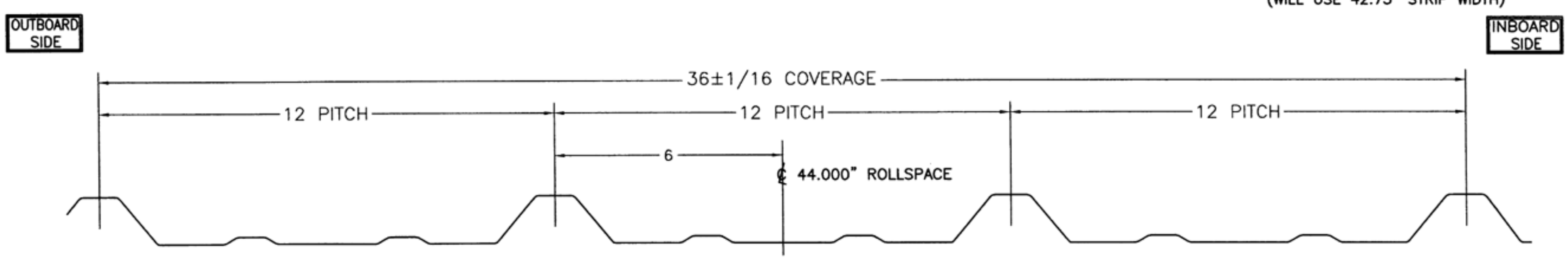
FULL DETAIL OF PBR PANEL



PURLIN–BEARING EDGE ONE SIDE ONLY. TOOLING ALLOWS FORMING OF THIS EDGE ON EITHER SIDE. CALCULATED STRIP WIDTH FOR THE PANEL WITH PURLIN–BEARING EDGE IS 42.87". (WILL USE 42.75" STRIP WIDTH)

PITCH & EDGE DETAILS

(FULL SCALE)



"PBR" PANEL

(3/8 SCALE)

NOTES:

1. ALL INSIDE BEND RADII ARE .172".
2. MATERIAL: 29–22 GA. GALV. & PAINTED STL. (.017 MIN. .035 MAX.). GRADES "D" AND "E" (50 KSI AND 80 KSI MINIMUM YIELDS).
3. STRIP WIDTH USED = 42.75
4. MAJOR AND MINOR RIB HEIGHT TOLERANCE: ±1/32. COVERAGE TOLERANCE END TO END:±1/16.
5. WHEN FORMING THIS PBR PANEL, .062 SHIM MUST BE LOCATED BETWEEN SPACER 14T1 & ROLL 14T2 ON INBOARD SIDE AND BETWEEN ROLL 14T15 & SPACER 14T16 ON OUTBOARD SIDE.

APPENDIX D

SUMMARY TABLE OF TEST RESULTS

Table D.1. Summary Table of Test Results

Continuous Span

Se	2.78	in^3
Fy	74.7	ksi
Trib_width	5.01	ft
Span Length	30.01	ft
Gamma_liquid	62.4	pcf
Mano. Reading	7.7	in
Panel Wt.	0.91	psf
Member Wt	4.51	plf

Max Applied Moment

Uniform Load	39.13	psf
Uniform Load	191.53	plf
Max_Mom.	13.80	kip-ft

Fully Braced Nom. Mom. Capacity

Mn	17.31	kip-ft

R Value

R Value	0.797	

Simple Span

Se	2.78	in^3
Fy	74.7	ksi
Trib_width	3.29	ft
Span Length	24.5	ft
Gamma_liquid	62.4	pcf
Mano. Reading	8.5	in
Panel Wt.	0.91	psf
Member Wt	4.51	plf

Max Applied Moment

Uniform Load	43.29	psf
Uniform Load	137.91	plf
Max_Mom.	10.35	kip-ft

Fully Braced Nom. Mom. Capacity

Mn	17.31	kip-ft

R Value

R Value	0.598	

APPENDIX E

STATISTICAL ANALYSIS

Table E.1. Statistical Analysis

Continuous Span

		R-Value
LaBoube (1988)	1	0.72
	2	0.83
	3	0.72
	4	0.8
	5	0.67
	6	0.64
	7	0.65
	8	0.68
	9	0.7
	10	0.62
	11	0.69
	12	0.72
	13	0.72
	14	0.59
NCI 2009	1	0.797

Previous Data		New Data Included	
Average	0.696	Average	0.703
Stand. Dev	0.065	Stand. Dev	0.067

Simple Span

	Depth		R-Value
Fisher (1996)	8.5"	M1	0.64
		M1	0.65
		M1	0.62
		M1	0.62
		M1	0.57
		M1	0.70
		M1	0.74
		M1	0.66
		M1	0.54
		M2	0.72
		M2	0.67
		M2	0.68
		M2	0.63
		M2	0.65
		M2	0.63
		M2	0.70
		M2	0.83
		M2	0.84
		M2	0.68
	9.5"	9	0.47
		10	0.54
		27	0.53
		M1	0.51
		M1	0.54
	11.5"	11	0.47
NCI 2009	12"	1	0.60

Previous Data		New Data Included	
Average	0.633	Average	0.632
Stand. Dev	0.097	Stand. Dev	0.110

APPENDIX F

F_u/F_y RATIO TABLE

Table F.1. F_u/F_y Ratios

	F_u (ksi)	F_y (ksi)	F_u/F_y
Fisher (1996)	77.7	58.7	1.32
	78.6	59.0	1.33
	78.2	59.1	1.32
	77.8	58.8	1.32
	81.7	59.6	1.37
	82.9	61.1	1.36
	81.7	63.4	1.29
	78.2	58.0	1.35
	84.0	62.5	1.34
	85.0	60.9	1.40
	76.0	55.9	1.36
	83.4	61.9	1.35
	82.8	62.1	1.33
	80.6	60.9	1.32
	79.2	59.2	1.34
	75.6	62.4	1.21
	81.8	61.0	1.34
	81.4	62.1	1.31
	81.9	56.7	1.44
	80.0	58.0	1.38
	80.4	48.5	1.66
	79.8	58.7	1.36
	80.5	59.6	1.35
	81.3	58.6	1.39
	80.6	60.7	1.33
	76.8	59.5	1.29
	76.0	49.5	1.54
	75.7	51.8	1.46
	75.6	55.9	1.35
NCI (2009)	92.2	74.7	1.23
		Min:	1.21

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VITA

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