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A GUIDE FOR DESIGNING WITH STANDING SEAM ROOF PANELS

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AUGUST 1997

Committee on Specifications for the Design of Cold-Formed Steel Structural members

American Iron and Steel Institute
1101 17th Street, NW
Washington, DC 20036
A GUIDE FOR DESIGNING
WITH
STANDING SEAM ROOF PANELS

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Committee on Specifications
for the Design of Cold-Formed
Steel Structural Members
American Iron and Steel Institute
1101 17th Street, N.W.
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The following design guide has been developed under the direction of the American Iron and Steel Institute Committee on Specifications for the Design of Cold-Formed Steel Structural Members. The development of the guide was co-sponsored by the AISI and the Metal Building Manufacturers Association. The AISI Committee and MBMA wish to acknowledge and express gratitude to Dr. James M. Fisher of Computerized Structural Design, Inc. and Dr. Roger A. LaBoube of the University of Missouri-Rolla who were principal authors of the guide.

With anticipated improvements in understanding of the behavior of cold-formed steel and the continuing development of new technology, this material might become dated. It is possible that AISI will attempt to produce updates of this guide, but it is not guaranteed.

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Copyright 1997, American Iron and Steel Institute
The Design Guide for Designing with Standing Seam Roof Panels provides information to
the designer of standing seam panels, cold formed purlins and steel joists supporting standing
seam roof panels. The Guide is based on the American Iron and Steel Institute’s Specification
for the Design of Cold-Formed Steel Structural Members, 1996 Edition. Where the Specification
is silent on design issues the procedures are based on published references and on the opin­
ions of the authors.

The Guide was co-sponsored by the Metal Building Manufacturer’s Association (MBMA).

AISI and MBMA acknowledge the efforts of Dr. James M. Fisher of Computerized Structural
Design, Inc., and Dr. Roger A. LaBoube of the University of Missouri-Rolla in the development of
this Guide.

Users of the Design Guide for Designing with Standing Seam Roof Panels are invited to
offer comments and suggestions. User response will be critical in improving design procedures
and for enhancing the use of standing seam roof systems.
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1. INTRODUCTION AND BACKGROUND

A typical roof system is composed of three primary components: purlins, roof panels and purlin braces. The term system is used to describe the roof assembly because of the interaction and synergism of the components. The purlins are considered the primary load carrying components of the roof system but are commonly called secondary members with regard to the entire building structural system. They support the dead load, gravity load, and wind load, and transfer these loads to the primary structural framing. The roof panel is a multi-functional component. It transfers the applied loads to the purlin, as well as serves as a bracing member for the purlin. Additional discrete purlin braces are often located along the length of the purlin. The intent of this design guide is to define the roof system components, their behavior, and their design requirements.

The behavior and subsequently the design of the roof system is primarily dependent on the type of roof panel. There are two general categories of roof panels, the conventional through-fastened panel, and the standing seam panel. The through-fastened panel, although it has its place in the marketplace, is rapidly being replaced by the standing seam panel. The standing seam panel with sliding clips, since its introduction in the 1960's, has shown to be a cost effective roof membrane solution because of its superior weathertightness, its ability to provide consistent thermal performance, and low maintenance requirements due to its ability to adjust to thermal expansion and contraction. For structural design, the through-fastened panel or the through-fastened panel in conjunction with discrete point braces is used to provide lateral support to the top flange of the purlin. For standing seam roofs the bracing effectiveness of the standing seam panels or the bracing effectiveness of the standing seam panels in conjunction with discrete point braces must be determined by tests.

Purlins are typically either cold-formed steel shapes or steel joists. Cold-formed members must be designed in accordance with the design rules promulgated by the American Iron and Steel Institute (1996). Steel joist design is within the purview of the Steel Joist Institute (1995). However, when supporting a standing seam roof panel, the Specifications do not fully address the design requirements for purlins. The AISI Specifications do not address the bracing effectiveness of standing seam roofs on purlins subjected to uplift or axial loads. Therefore, the intent of this design guide is to supplement the recognized design specifications for purlins supporting standing seam panels.

1.1 Through-Fastened Roof System

Section 4.0 reviews the issues pertinent to the design of purlins with a standing seam roof panel. In general these design issues apply equally to purlin systems having through-fastened roof panels. Issues unique to the through-fastened roof panel are examined in the following discussion.

1.1.1 Continuous Bracing

Bracing provided by the through-fastened panel attached to the top flange will enable the use of AISI Specification Section C3.1.1 (a) for positive moment from gravity loads. For wind uplift loading, panel bracing permits the use of AISI Specification Section C3.1.3.
1.1.2 Discrete Bracing

For wind uplift loading, when AISI Specification Section C3.1.3 does not offer the requisite design strength, discrete braces are to be considered for the bottom flange. The purlin design for uplift is then based on AISI Specification Section C3.1.2 and the brace is designed per AISI Specification Section D3.2.2. However, when using discrete braces for a Z-purlin, a minimum of third point braces should be considered. Full-scale tests have shown that a single mid-point brace may not offer the moment strength of a Z-purlin as would be indicated by AISI Specification Section C3.1.2.

1.2 Other Design Considerations

Section 4.6 of this design guide provides a comprehensive discussion of the array of design issues that must be considered for a purlin system. Although the specific emphasis in Section 4.6 is directed toward the standing seam panel, the provisions apply equally to the through-fastened panel. The exception is the evaluation of the nominal axial strength, \( P_0 \), which is defined by AISI Specification Section C4.4 and the nominal moment strength, \( M_0 \), which is defined by AISI Specification Section C3.1.

2. EXPLANATION OF SYSTEMS AND THEIR COMPONENTS

A roof system is a synergistic system composed of the following components:

2.1 Roof Panels

Steel roof panels serve as an environmental barrier as well as contributor to the structural integrity of the purlins which support them. A standing seam panel and a through-fastened panel are shown in Fig. 2.1. The standing seam panel has no fasteners that penetrate the steel membrane, except where the ends of the panels are joined.

![Typical Roof Panels](image)

Standing seam panels are either of the pan type or rib type shown in Fig. 2.2.
2.2 Fasteners/Clips

Fasteners and clips are used to connect the standing seam panels to the purlins. Clips are specially designed connection elements that are embedded in the seam of standing seam roof panels. The clips may be either sliding or fixed (Fig. 2.3). A sliding clip allows thermal movement of the roof membrane. Self-drilling screws are normally used to attach the clip to the purlin. This screw type combines the functions of drilling and tapping. For through-fastened roof systems, the panels are attached directly to the purlin with the aid of self-drilling screws.

2.3 Purlins

Purlins are secondary roof structural members which transfer gravity and wind loads from the roof panel to the rafter. Cold-formed steel purlins are generally either C or Z-sections. For longer spans, when a C- or Z-section is not economical, steel joists are used. Steel joists may be fabricated using hot-rolled or cold-formed steel chord and web members.

2.4 Purlin Braces

Purlin braces are elements within a roof system that provide lateral restraint to the purlin. The braces may be discrete braces or continuous braces. Discrete braces are generally angles or
straps which may function in either compression or tension (Fig. 2.4). Continuous braces may be provided by the diaphragm roof panels. Forces that accumulate in the bracing system must be anchored.

![Fig. 2.4 Example of Discrete Braces](image)

C and Z sections generate braces forces larger than the stability forces required to restrain vertically symmetric steel joists or wide flange beams. C-sections loaded through their gravity axes tend to twist because the centroid is not coincident with their shear center. To keep the C-section from twisting, so that the simple bending equation can be used to design the C-section, brace forces larger than those normally used for stability requirements are necessary. The Z-section when loaded through its vertical axis translates laterally. This occurs in a Z-section because its principal axis is not coincident with its vertical axes. To prevent translation and again to permit the use of simple bending theory to design a Z-section significant bracing may be required.

For general information concerning C and Z-section behavior the reader is referred to Salmon, Charles G. and Johnson, John E. (1996).

2.5 Purlin Clips

Cold-formed purlins are attached to rafters either by bolting the bottom flange directly to the rafter top flange or by using a clip which is bolted to the purlin web and bolted or welded to the rafter top flange. The clip can serve as an “anti-roll” mechanism for the purlin as well as eliminating purlin web crippling. Steel joists are attached to rafter either by bolting or welding the joist seat to the rafter.

2.6 Rafters

Rafters are beams or trusses supporting the roof purlins. The rafter serves as a load path from the purlin to the column.

3. PANEL DESIGN

3.1 Gravity Load Design

Although standing seam roof panels are composed of thin structural elements that may distort and/or buckle in cross section when subject to gravity loading, the panels can be designed using the AISI Specification. The Specification, in Section B1.1, recognizes the tendency of the panel to deform by noting that stiffened elements having w/t ratios larger than 500 may be used with
safety to support loads. The AISI Commentary goes on to state that the intent of the Specification is to caution the designer, not to preclude the use of compression elements having w/t ratios greater than 500. For gravity load situations panel deformation is normally not serious enough to preclude the use of the AISI equations.

The design of a standing seam panel to resist gravity load follows the same methodology and design considerations as any flexural member. That is, the panel must be investigated for the following design considerations: (1) bending, (2) shear, (3) web crippling, (4) combinations of bending and shear, and (5) deflection. The combination of bending and web crippling is explicitly excluded from design consideration by the exception clause of Specification Section C3.5. Design Example 11.1 demonstrates the application of the Specification to the design of a standing seam panel.

3.2 Panel Uplift Design

Panels and fasteners must be designed for the appropriate wind uplift pressures of the applicable building code. They must be checked for the different pressures as defined by the code for the field of the roof, rakes, eaves, corners, overhangs and ridge areas. Most building codes specify pressures that vary inversely with influence area. Thus, fasteners are usually required to resist greater pressures than the panel in each of the areas defined. It is usually incorrect to evaluate fasteners with the same pressure as the panel.

Through-fastened panel resistance may be checked using the AISI defined section properties. This approach is deemed sufficient for through-fastened roofs, but may be inaccurate for standing seam roofs. In the case of standing seam roofs, disengagement of the panel from the clip tends to be the limiting failure mechanism. The resistance of screws used in roof systems may be calculated using pull-out or pull-over allowables defined in the AISI Specification. The resistance of standing seam clips must be determined by test methods.

Several rating methods are currently available for roof panels. These include Underwriter’s Laboratories UL 580, ASTM E-1592, and Factory Mutual Research Corporation (FMRC) Approval Standard 4471. The UL 580 and FMRC 4471 procedures provide a relative measure of the roof system’s resistance to uplift. ASTM E-1592 provides a quantitative measure of the static uplift load that a roof panel system can sustain. All of these test methods use a uniform pressure distribution. None of the tests take into account the dynamic, non-uniform nature of actual wind loading. All currently available methods of determining wind uplift ratings must be used with caution recognizing that they do not give an actual measure of a roof panel system’s ability to resist specified design wind loads.

4. REVIEW OF COLD-FORMED STEEL PURLIN DESIGN

4.1 Structural Analysis

Elastic analysis assumptions and techniques are applied when determining the internal design forces and moments in a purlin. Simple span purlins are always analyzed as prismatic members.

Based on the judgment of the engineer, continuous span purlins may be analyzed as either prismatic or non-prismatic members. For example, when the purlin system is analyzed using the prismatic assumption larger positive moments will result as compared to those obtained from a non-prismatic analysis. From the design standpoint this may result in larger end span purlins as compared to a design based on the results of a non-prismatic analysis. Designs based on a non-
prismatic analysis will often be controlled by combined bending and shear calculations at the end of the purlin laps because the non-prismatic analysis generates larger negative moments as compared to a prismatic analysis.

For a C-purlin, the adjacent span purlins are connected back-to-back over the main frame, whereas the Z-section is lapped or nested over the main frame (Fig. 4.1). For both cross sections, within the lapped region, i.e. over the main frame, the purlins are connected in such a manner that full moment continuity between the individual purlins can be achieved.

![Fig. 4.1 Purlin Continuity](image)

In subsequent discussions, positive and negative moments are defined by Figure 4.2.

![Fig. 4.2 Moment Definitions](image)

### 4.2 Continuous Bracing

Bracing is critical to the successful performance of a purlin roof system. Typically, when a purlin has a roof panel attached to its top flange, the tendency is to assume that the flange has full lateral support for bending behavior. This is an acceptable assumption when the panel is a through-fastened roof panel. However, when the panel is a standing seam panel, the presence of full lateral support can only be verified through testing. The following discussion will summarize the design methodology for a purlin system with particular emphasis given to the design nuances of a standing seam panel.
The presence of a standing seam panel may not result in full bracing for the purlin. There is no formal theoretical design criteria in the AISI Specification for computing the synergism of the standing seam panel and the purlin. The Specification (Section C3.1.4) thus requires that the gravity moment strength be determined by the AISI Base Test Method (See Section 5). The 1996 AISI Specification does not contain provisions relative to wind uplift design of purlins using lateral and torsional support provided by standing seam roofs. Section 7 of this design guide addresses the moment capacity for wind uplift.

### 4.3 Discrete Point Bracing

Discrete point braces are often employed to enhance the structural performance of a compression flange that is not attached to sheathing. Discrete braces are also used to enhance performance for wind uplift loading.

For gravity loading, if the standing seam panel is to be considered as a contributor to bracing of the purlins in conjunction with discrete point bracing, the positive moment strength must be defined by test as discussed in Section 5 of this design guide. When panel bracing is discounted, the purlin moment capacity is defined by AISI Specification Section C3.1.2. For wind uplift loading, the purlin design is based on AISI Specification Section C3.1.2 and the brace is designed per AISI Specification Section D3.2.2 for lack of a more applicable design approach. However, when using discrete braces for a Z-purlin, a minimum of third point braces should be considered for gravity and wind load cases. Full-scale tests have shown that a single mid-point brace may not offer the moment strength of a Z-purlin as would be indicated by AISI Specification Section C3.1.2.

### 4.4 Unbraced Purlins

Unbraced purlins are not routinely encountered in purlin systems. If such a condition should occur, the member's flexural strength is to be determined by AISI Specification Section C3.1.2.

In the region of an intermediate support, that is the negative moment region of a continuous span purlin, the flexural capacity in the negative moment region beyond the end of the lap is computed by AISI Specification Section C3.1.2. In the strength evaluation it is normally assumed that the inflection point is a free end. Thus the unbraced length is taken as the distance from the end of the lap to the inflection point. Correspondingly, C_b is conservatively taken as 1.0.

### 4.5 Anchorage of Purlin Bracing

The load carrying capacity of purlin systems attached to roof sheeting is dependent on the ability of the roof sheeting to torsionally and laterally restrain the purlins. The torsional restraint is provided by the bending strength and stiffness of the sheeting, and the clip/fastener assembly which connects the roof system to the purlins. Lateral restraint is provided by the diaphragm capacity of the sheets and any discrete point bracing designed into the system.

The torsional restraint is self contained in the sheeting; however, brace forces and diaphragm forces accumulate and must be transferred to other structural elements, i.e. rigid frames, vertical bracing, etc.

Purlins having their compression flange attached to deck or sheathing are designed as laterally supported members. Forces which are developed in the bracing system and the deck or sheathing must be calculated and anchored in accordance with AISI Specification Section D3.2.1.
The AISI equations depend on the location and type of lateral bracing system. The cases included in the Specification are:

1. Torsional Bracing at the Purlin Ends.
2. Third Point Bracing, and
3. Mid-span Bracing.

The brace forces in the AISI Specification are contingent upon having a roof diaphragm system which meets the span divided by 360 requirement of AISI Specification Section D3.2.1.

Designers using discrete point bracing systems generally anchor the bracing by balancing the bracing forces across the building ridge. See Fig. 4.3. This requires that the structure have equal slopes, equal loading and equal lengths on each side of the ridge, in order for the bracing forces to be balanced. If these equalities do not exist then bracing members must be added to resist the unbalanced forces, or a proper number of purlins have their flanges facing one another to eliminate the unbalanced forces.

For single sloped buildings or for buildings where the discrete braces cannot be anchored across the ridge, the bracing forces must be transferred to an anchor location. Rigid frame lines or braced frame lines are generally selected as anchorage points. To transfer the forces out of the discrete braces to these anchorage locations horizontal trusses are generally installed in the plane of the roof.

Other designers use a system that incorporates only the standing seam roof diaphragm. The diaphragm is anchored at discrete points along the rafter or at the building eave. For example, see Fig. 4.4. The primary advantage of this system over discrete point bracing systems is that fewer parts need to be handled during erection. In addition, this system does not require modification for single slope buildings and does not require alteration for the inequalities mentioned above.
No matter what anchorage system is used the designer must prove by calculation or tests that the diaphragm can deliver the accumulated purlin anchorage forces into the anchorage points.

**4.6 Other Design Considerations**

**4.6.1 Shear**

The shear capacity of a purlin web is defined by AISI Specification Section C3.2. In the region of a support where the purlin is lapped, each web shall be considered as a separate element carrying its share of the shear force.

**4.6.2 Bending and Shear**

The interaction of bending and shear must be considered by using AISI Specification Section C3.3. The bending capacity is based on initiation of yielding per AISI Specification Section C3.1.1(a). Shear capacity is defined above. For continuous purlin systems the most critical location is generally at the end of the purlin laps.

**4.6.3 Web Crippling**

Web crippling is a design consideration for gravity loading conditions. At the free end of a purlin, for example at the end wall of a building, the purlin is subject to the AISI end-one-flange loading condition. The web crippling capacity for this condition is defined by AISI Specification Section C3.4. The use of purlin web reinforcement or support clips can eliminate web crippling at the supports.

**4.6.4 Web Crippling and Bending**

At interior supports and overhangs of continuous span purlins, combined web crippling and bending must be evaluated unless purlin support clips are provided. AISI Specification Section C3.5 contains design rules for single unreinforced webs, that is C-purlins back-to-back, and for nested Z-purlins. When evaluating the bending capacity of each section, AISI Specification Section C3.1.1(a) applies to bending, whereas AISI Specification Section C3.4 pertains to web crippling. The section strengths are additive when evaluating the interaction equation.

**4.6.5 Bending and Axial Load**

The structural integrity must be investigated for combined bending and axial load (strut purlins) for purlins that are designed to resist the combined effects of gravity and seismic loads or the effects of wind uplift on the roof surface and wind on the endwall of the building. The interaction equations of AISI Specification Section C5 are applicable for purlins supporting through-fastened roof systems. Section 8 of this design guide discusses the design strength of strut purlins in conjunction with standing seam panels.

**4.6.6 Connections**

AISI Specification Sections E2, E3, and E4 summarize the design rules for welded, bolted and screw connections.
5. THE BASE TEST

For gravity loading many designers assume that the standing seam roof panels brace purlins, some assume a reduction of full constrained bending capacity, and some assume no lateral support.

These assumptions range from possibly very liberal to possibly very conservative. There is no formalized theoretical criteria to measure the assumptions. Based on testing, the amount of resisting moment which the supporting purlins can achieve can vary from that of a fully braced condition to an unbraced condition for different roof systems. Because of this wide variation in behavior it was determined that it is not practical to develop a generic analytical method to predict the interaction of standing seam roof systems and supporting structure, thus the base test method was developed.

The major advantage of the base test is that a simple span test may be used to predict performance of continuous span systems, thereby reducing experimental costs. A copy of the AISI Base Test Method is contained in Appendix I. The concept for the base test was developed by T.M. Murray and several graduate students at Virginia Tech University. If the reader is not familiar with the base test, Appendix I should be read before continuing with the reading of the remainder of this guide.

The purpose of the base test is to determine the ability of a particular standing seam roof system to provide lateral and rotational support to gravity loaded purlins. The 1996 AISI Specification does not address using the base test method for uplift; however, it is the opinion of the writers that a similar test procedure should be used to determine the uplift resistance of purlins supporting standing seam roofs unless independent discrete point bracing is used.

The test method provides a means of establishing a nominal resisting moment for purlins in a simple span or continuous, multiple purlin line, standing seam roof system from the results of tests on a single span, two purlin line, sample of the system.

This test method only applies to an assembly consisting of the standing seam panel, purlin, and attachment devices used in the system being tested. It is not a test for the strength of the individual components of the assembly.

The base test procedure allows for the following two different test arrangements relative to purlin orientation.

1. Purlin flanges opposed.
2. Purlin flanges facing the same direction.

Both test methods are discussed below.

5.1 Purlin Flanges Opposed

Testing with purlin flanges opposed does not place requirements on the diaphragm, i.e. the lateral forces generated by the purlins are counterbalanced. This procedure does not address the issue of how the diaphragm anchorage forces are resisted and hence does not consider all of the lateral stability issues. The method does demonstrate the effectiveness of the panel/clip torsional resistance for the purlin. It also demonstrates if the lateral forces can be transmitted into the roof panels.
5.2 Purlin Flanges Facing the Same Direction

Conducting the base test with the purlin flanges facing the same direction demonstrates the torsional effectiveness of the panel/clip to brace the purlin, and also demonstrates if the lateral forces can be transmitted from the purlin to the roof panels.

When two Z-purlins are tested facing the same direction, the test procedure necessitates that the failure load, \( w_{\text{fl}} \), be adjusted by the factor \( 2P_L \left( \frac{d}{B} \right) \). This adjustment factor is applicable only to the Z-section because it is not loaded along its principal axes. The justification for the adjustment factor, \( \Delta w_{\text{fl}} \), is explained as follows.

Each purlin develops the \( P_L \) force at the top of the purlin. See Fig. 5.1. If the sheeting is not anchored in the test then the resistance to the lateral force is supplied by the reactions at the purlin support points. Summing moments about one of the supports yields:

\[
\Delta w_{\text{fl}} = 2P_L \left( \frac{d}{B} \right)
\]  
(Eq. 5.2-1)

The AISI Specification equation D3.2.1-1 represents the lateral force developed for single-span systems, i.e. the base test case. The Specification requires that a minimum of four purlins be used in equation D3.2.1-1. Substituting \( n_p = 4 \) and conservatively not taking the 1.1 multiplier for systems less than four purlin lines, the AISI equation D3.2.1-1 for zero roof slope reduces to Equation 5.2-2.

\[
P_L = 0.041 \left( \frac{b^{1.5}}{d^{0.90} t^{0.60}} \right) (p_{\text{fl}} + p_{\text{g}}) s
\]  
(Eq. 5.2-2)

where

- \( b \) = flange width of the purlin
- \( d \) = depth of the purlin
- \( t \) = purlin thickness
- \( p_{\text{fl}} \) = failure load (force/area) of the single span system tested
The increase in load, \( \Delta w_{tu} \), must be added to the achieved test load to arrive at the ultimate load on the purlin.

The AISI Base Test procedure allows the simulation of the eave condition in a building by using a 3 in. by 3 in. by 1/4 in. maximum thickness (See Appendix I) continuous angle at the end of the test panels. It is permitted to anchor the continuous angle to prevent lateral displacement at its ends. The use of an anchored simulated eave condition can result in unconservative results, Fisher and Nunnery (1996). This unconservative situation occurs in standing seam roof systems which possess little to no diaphragm strength and stiffness. In these cases, the total anchorage strength is derived from the eave member and not the diaphragm. Based on the AISI anchorage requirements (Section D3.2.1.) the demands on the simulated eave condition would be greater if more than two purlins are to be braced, i.e. in the actual structure. If the tests are conducted not anchoring to the simulated eave, the results will be conservative; in fact, this may result in overly conservative test results. This is due to the high strength and stiffness demands on the relatively shallow diaphragm which may allow the purlins to fail prematurely, Fisher and Nunnery (1996). A discussion is presented in the next section on the procedures to be used to properly assess the diaphragm and base angle eave condition.

### 5.3 Determining Diaphragm Requirements

Based on the foregoing discussion it is apparent that the strength and stiffness of the diaphragm system should be evaluated when base tests are conducted with purlin flanges opposed (unless the purlins have their flanges opposed in field usage), or when the simulated eave condition is used in the test. Based on diaphragm test results, it may be demonstrated that the diaphragm satisfies the strength and stiffness requirements as established in Section D3.2.1 of the AISI Specification. If so then the design procedure for standing seam roof systems is to conduct the base test and to provide the required anchorage system per AISI Section D3.2.1. If the diaphragm system does not satisfy either the strength or stiffness criteria, then a discrete point bracing system can be designed in accordance with Section D3.2.2 of the AISI Specification.

The majority of diaphragm stiffness loss comes from side lap slip. Many designers provide an eave member to which the panels are secured. The fasteners that are used to attach the panels to the eave member provide significant restraint, thus they can significantly increase the diaphragm strength and stiffness.

The effect of the eave attachment can be determined from diaphragm tests. The difficulty of including the eave attachments in a diaphragm test is that the benefit of the fasteners in the eave can lead to unconservative assumptions relative to the diaphragm strength and stiffness if the results are not evaluated properly. For example, if the cantilever test method is used to determine the strength and stiffness, and the values obtained from the test are then used to predict the strength and stiffness of a larger diaphragm, the effects of the eave member on the strength and stiffness will be overstated. Stating this in another way, assume that a particular roof system has no ability to resist side lap slip, then the total stiffness is derived from the fasteners in the eave member. The strength does not necessarily increase when the size of the diaphragm is increased, the resistance can decrease if the diaphragm depth increases and the width remains constant.

If the attachment to the eave member is intended to be used to help provide additional strength and stiffness, to the diaphragm system, then the benefits from the eave in a test attachment must be isolated from the basic diaphragm strength and stiffness. The benefit from the eave member can then be added to the basic behavior of the diaphragm without the eave member.
5.3.1. Establishing Diaphragm Strength and Stiffness

The strength and stiffness of any size diaphragm can be obtained by first obtaining the strength and stiffness of the diaphragm (i.e. multiplying test values by the physical dimensions of the actual diaphragm) and then adding this strength and stiffness to the eave attachment strength and stiffness, Fisher and Nunnery (1996).

Thus, the diaphragm shear strength can be represented as:

\[ S_{nt} = S_n + S_e \]  
(Eq. 5.3-1)

where,

- \( S_{nt} = \) the total nominal diaphragm shear strength including eave attachment effects
- \( S_n = \) the nominal diaphragm shear strength without eave attachment
- \( S_e = \) the nominal shear strength of the eave attachment

The stiffness of the system can be represented similarly as:

\[ G'_{nt} = K_n + K_e \]  
(Eq. 5.3-2)

where,

- \( G'_{nt} = \) the total diaphragm stiffness including eave attachment effects
- \( K_n = \) the diaphragm stiffness without eave attachment
- \( K_e = \) the stiffness of the eave attachment

To obtain \( S_n, S_e, K_n \) and \( K_e \) diaphragm tests must be conducted as illustrated in Figures 5.2 and 5.3.

Shown in Figure 5.2 is the cantilever test assembly for determining the diaphragm properties. The test should be conducted following the cantilever diaphragm test procedure as outlined in the AISI Cold-Formed Manual, 1996. This test arrangement should be used to determine the nominal diaphragm shear strength and diaphragm stiffness without eave attachments. Thus, the system attachments at the eave are not included in this test. The test load is delivered into the sheeting through an edge member parallel to the load. This member is connected to the paneling using only the system's purlin-panel clips. A similar edge member is connected between the diaphragm reaction points at the opposite edge. Members simulating typical interior purlins are positioned at appropriate spacing within the assembly again connected to the panels with purlin-panel clips. Along the remaining two sides edge members are provided. These edge members can be attached to the test panels with self drilling fasteners.
Shown in Figure 5.3 is the cantilever test assembly for determining the diaphragm properties of the sheeting and the contribution from the attached eave member. The construction of the test assembly is identical to that shown in Figure 5.2, except for the inclusion of the eave members and their attachments. The eave member has been included along both edges in the assembly. This is done to eliminate the panel warping effects along the ends of free panel edges. It is the writers' opinion that the warping if not prevented would unrealistically reduce the stiffness of the effects of the eave member in the test. However, when the additional eave member is included in the test arrangement, then the strength and stiffness effects of the eave member may have been doubled.
It is the writers' opinion that the strength and stiffness contribution of the eave member can be determined by subtracting the test load obtained from the tests using the arrangement in Figure 5.2 from the load obtained from tests using the arrangement shown in Figure 5.3 and dividing by two to account for the use of the two eave members.

In some systems the ridge condition may have a strength and stiffening effect on the diaphragm. For these systems the diaphragm tests can be conducted using the ridge and eave details for the system. The test results for this case would not have to be divided by two.

**Strength Contribution from the Eave Attachments:**

Based on the diaphragm tests shown in Figs. 5.2 and 5.3, the shear strength per foot, $S_e$, furnished by the attachment of the standing seam panels to the eave member can be determined from the equation:

$$S_e = \frac{(P_2 - P_1)}{2b} \quad \text{(Eq. 5.3-3)}$$

The strength furnished by the eave attachment is analogous to a series of cantilever shear walls. See Figure 5.4.

The eave connection furnishes a "fixed" moment resistance. The resistance is derived from the moment capacity of the panel - fastener - eave member connection. The shear strength furnished by an individual panel equals the base moment resistance divided by the distance from the base to the line of action of the shear force. For the diaphragm test, the eave attachment contribution to the resisting shear per foot is $Q_e/a$, where $Q_e$ is a constant for the panel representing the eave attachment moment resistance. From the diaphragm tests:

$$Q_e = (S_e)(a) \quad \text{(Eq. 5.3-4)}$$

![Diagram of diaphragm shear and eave connection strength](image-url)

**Fig. 5.4 Eave Connection Strength (One Eave Member)**

For any diaphragm, the total diaphragm strength per foot can be obtained from the equation:
\[
S_m = S_n + Q_e / d
\]
(Eq. 5.3-5)

where, \(d\) is the panel length in the field installed diaphragm, i.e. the diaphragm depth.

**Stiffness Contribution from the Eave Connection:**

From the diaphragm tests when two eave members are used, the stiffness, \(K_e\), per unit length furnished by the eave attachment equals:

\[
K_e = (G_2 - G_1) / 2
\]
(Eq. 5.3-6)

where \(G_2\) and \(G_1\) are the average diaphragm shear stiffnesses calculated from diaphragm tests with and without eave connections, respectively. Since the deformation from the eave restraint occurs almost totally at the eave attachment, (See Fig. 5.5), with the remainder of the panel being rigid, \(K_e\) can be expressed as a constant \(C_e\) divided by the height of the panel used in the diaphragm tests. From the diaphragm tests:

\[
C_e = (K_e)(a)
\]
(Eq. 5.3-7)

For a given diaphragm of depth \(d\), the diaphragm stiffness can be determined from:

\[
G' = K_n + C_e / d
\]
(Eq. 5.3-8)

where \(d\) is the panel length of the field installed diaphragm.

![Fig. 5.5 Diaphragm Stiffness per Unit Length](image)
The use of $S_e$ and $G'$ are further described in Example 11.5.

5.3.2. Evaluation of the Diaphragm Strength and Stiffness

As discussed, depending upon the manner in which the base test is conducted, the diaphragm must be evaluated to determine if it meets the required strength and stiffness per Section D3.2.1 of the AISI Specification. The requirements of the diaphragm depend upon the bracing system used for the purlins. The strength and stiffness requirements can be calculated based on the lateral forces generated by the purlin system. The Specification requires that the diaphragm deflection at the purlin mid-span not exceed the span length divided by 360. The deflection requirement is intended to apply at the load producing the nominal strength of the purlin. Diaphragm deflections are typically evaluated from shear deflection equations using the tested diaphragm shear stiffness $G'$. Traditionally $G'$ has been established using the secant modulus at 0.4 times the maximum diaphragm test load.

Because load deflection curves for diaphragms are normally non-linear, using the secant modulus established at $0.4P_u$ per the AISI diaphragm test procedure could lead to unconservative results. The writers' suggest that for both ASD and LRFD designs $G'$ be determined at $0.65P_u$ for both ASD and LRFD designs.

The diaphragm strength and stiffness requirements are established based on the AISI anchorage equations as follows:

**C-Sections**

From the AISI specifications, for roof systems using C-sections, with the flanges facing in the same direction, the load on the diaphragm equals $0.05W$ including the down slope load component. $W$ is defined as the total load supported by the purlins. The uniform in-plane load per foot on the diaphragm equals $0.05W$ divided by the purlin span including the total down slope load component. The shear deflection for a diaphragm between supports is calculated by the equation:

$$\Delta_s = \frac{w_u L^2}{8G'd}$$  (Eq. 5.3-9)

where,

- $w_u$ = the uniform in-plane load, kips/ft.
- $L$ = the diaphragm span between diaphragm reactions, ft.
- $G'$ = the diaphragm shear stiffness evaluated at $0.65P_u$, kips/ft.
- $d$ = the diaphragm depth, ft.

Substituting the appropriate variables $w_u$, $L$, $b$ and calculating $G'$ based on the diaphragm geometry, the diaphragm system shear deflection is obtained. The bending deflection of the dia-
phragm system is normally neglected because it is small compared to the shear deflection. To guarantee purlin stability the total deflection $\Delta$ must be less than $L/360$.

The diaphragm design shear strength equals:

$$\phi_s S_n = \frac{w_n L}{2d} \text{ (LRFD)}$$

(Eq. 5.3-10)

The design shear strength must then be compared to the required strength.

**Z-Sections**

For Z-sections, the deflection and strength requirements must be established for each type of restraint system being used. It should be noted that the gravity load down slope force component is included in the AISI equations and must not be added to the diaphragm load. For example, for a single-span system with midspan restraint, $P_L$ can be established from the AISI Specification equation D3.2.1.-3. Since only a midspan brace exists the diaphragm must span from midspan brace to midspan brace. Thus, the load on the diaphragm equals $P_L / L$.

The diaphragm loading for all other restraint conditions can be determined in a similar manner.

6. **STANDING SEAM ROOF SYSTEMS SUBJECTED TO GRAVITY LOADING**

To design purlin systems which are subjected to gravity loading, the AISI Base Test Method must be used, unless the purlins are designed as laterally unsupported using only discrete point bracing.

**6.1 Purlin Design Procedure**

The following steps must be taken to design purlins subjected to gravity loading when using the base test method:

1. Conduct the base tests in accordance with the AISI Base Test Method.

2. Select the proper size purlins to provide the required moment capacity, $M_n = R M_e / \Omega$, in which the "R" values are obtained from the base tests and $M_e / \Omega$ is determined by the AISI Specification ($M_e = S_S F_y$).

3. Provide a proper diaphragm system design.

4. Provide the proper anchorage for the diaphragm system and for the discrete point bracing system (if discrete bracing is a part of the purlin bracing system). Unlike through-fastened systems where the discrete point bracing can be placed intermittently, discrete point bracing must be continuous in nature when used with standing seam roofs.

The requirements for steps 3 and 4 depend upon exactly how the base tests are conducted as discussed in Section 6.2.

**6.2 System Design Requirements**

Based on how the base tests were conducted, the design requirements for the diaphragm system and for discrete point bracing vary. Summarized below are typical system design requirements.
Test Condition 1:

Purlin flanges opposed.
Simulated eave member used.
Discrete point bracing used.

System Design Requirements:

For the structure in question, determine if the diaphragm system meets the AISI stiffness requirement of span / 360, and if the diaphragm strength requirements are met.

If both the diaphragm stiffness and strength requirements are met, design the discrete point braces for the requirements of AISI Section D3.2.1.

Design anchorage for the discrete point braces for the calculated forces.

Design anchorage for the diaphragm for the imposed loads.

If either the strength or stiffness diaphragm requirements are not met, design the discrete point braces for the AISI requirements in Section D3.2.2.

Design anchorage for the discrete point braces for the calculated forces.

Design anchorage for the diaphragm as anchored in the base test.

Test Condition 2

Purlin flanges opposed.
Simulated eave member used.
Discrete point bracing not used.

System Design Requirements:

For the structure in question, determine if the diaphragm system meets the AISI stiffness requirement of span / 360, and if the diaphragm strength requirements are met.

If both the diaphragm stiffness and strength requirements are met, as above, anchor the diaphragm for the calculated imposed loads.

If either of the diaphragm requirements are not met, add discrete point bracing and design the discrete point braces for the AISI requirements in Section D3.2.2, and design anchorage for the discrete point braces for the calculated forces.

Test Condition 3:

Purlin flanges opposed.
Simulated eave member not used.
Discrete point bracing used.

System Design Requirements:

For the structure in question, determine if the diaphragm system meets the AISI stiffness requirement of span / 360, and if the diaphragm strength requirements are met.
If both the diaphragm stiffness and strength requirements are met, design the discrete point braces for the AISI requirements in Section D3.2.1.

Design anchorage for the discrete point braces for the calculated forces.

If either of the diaphragm requirements are not met, design the discrete point braces for the AISI requirements in Section D3.2.2.

Anchor the discrete point braces for the calculated forces.

**Test Condition 4:**

- Purlin flanges opposed.
- Simulated eave member not used.
- Discrete point bracing not used.

**System Design Requirements:**

For the structure in question, determine if the diaphragm system meets the AISI stiffness requirement of span / 360, and if the diaphragm strength requirements are met.

If both the diaphragm stiffness and strength requirements are met, anchor the diaphragm for the calculated imposed loads.

If the requirements are not met, add discrete point bracing and design the discrete point braces for the AISI requirements in Section D3.2.2.

Design anchorage for the discrete point braces for the calculated forces.

**Test Condition 5:**

- Purlin flanges facing the same direction.
- Simulated eave member used.
- Discrete point bracing used.

**System Design Requirements:**

For the structure in question, determine if the diaphragm system meets the AISI stiffness requirement of span / 360, and if the diaphragm strength requirements are met.

If both the diaphragm stiffness and strength requirements are met, design the discrete point braces for the AISI requirements in Section D3.2.1.

Design anchorage for the discrete point braces for the calculated forces.

Design anchorage for the diaphragm for the imposed loads.

If either of the requirements are not met, design the discrete point braces for the AISI requirements in Section D3.2.2.

Design anchorage for the discrete point braces for the calculated forces.

**Test Condition 6:**

- Purlin flanges facing the same direction.
Simulated eave member used.
Discrete point bracing not used.

**System Design Requirements:**

For the structure in question, determine if the diaphragm system meets the AISI stiffness requirement of span / 360, and if the diaphragm strength requirements are met.

If both the diaphragm stiffness and strength requirements are met, anchor the diaphragm for the calculated imposed loads.

If the requirements are not met, add discrete point bracing and design the discrete point braces for the AISI requirements in Section D3.2.2.

Design anchorage for the discrete point braces for the calculated forces.

Design anchorage for the diaphragm as in the tested condition.

**Test Condition 7:**

- Purlin flanges facing the same direction.
- Simulated eave member not used.
- Discrete point bracing used.

**System Design Requirements:**

For the structure in question, determine if the diaphragm system meets the AISI stiffness requirement of span / 360, and if the diaphragm strength requirements are met.

If both the diaphragm stiffness and strength requirements are met, design the discrete point braces for the AISI requirements in Section D3.2.1.

Design anchorage for the discrete point braces for the calculated forces.

Design anchorage for the diaphragm for the imposed loads.

If either of the diaphragm requirements are not met, design the discrete point braces for the AISI requirements in Section D3.2.2.

Design anchorage for the discrete point braces for the calculated forces.

**Test Condition 8:**

- Purlins facing the same direction.
- Simulated eave member not used.
- Discrete point bracing not used.

**System Design Requirements:**

No system requirements.

7. **STANDING SEAM ROOF SYSTEMS SUBJECTED TO UPLIFT LOADING**

The AISI Specification currently contains no criteria for the design of purlin systems attached to standing seam roofs subjected to uplift loadings. It is the opinion of the writers that the base test
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method (with reversed loading) should be used to evaluate purlins supporting standing seam roofs subjected to uplift loading, unless discrete point bracing is used. If discrete point bracing is used the braces must be evaluated in accordance with Section D3.2.2 of the Specification. Until further research is conducted the adjustment factor Eq. 5.2-1 must not be used in evaluating the base test data. In addition, the AISI diaphragm strength and stiffness requirements remain applicable. These requirements can be evaluated using the safety factors and load factors associated with wind loading. The proper design procedure again depends on the standing seam roof characteristics and the manner in which the base tests are conducted.

7.1 Purlin Design Procedure

The following steps must be taken to properly design purlins subjected to uplift loadings:

1. Conduct uplift (reverse loading) using the AISI Base Test method.

2. Select the proper size purlins to provide the required moment capacity, \( M_a = RM_p/\Omega \), in which the “R” values are obtained from the base tests and \( M_p/\Omega \) is determined by the AISI Specification \( (M_a = S_yF_v) \).

3. Provide the anchorage for the diaphragm or the discrete point bracing.

7.2 System Design Requirements

If the base tests are conducted without discrete point bracing then the diaphragm requirements and anchorage forces should be calculated using the requirements stipulated in the AISI Specification Section D3.2.1. The system design requirements are identical to those discussed for C-sections and for Z-sections for purlins subjected to gravity loads in Section 6.2 herein (Test conditions 2, 4, 6, and 8).

If the purlin uplift design is dependent on discrete point braces in conjunction with the standing seam roof (using the “reverse” base test) then the AISI equations (Section D3.2.1) are not directly applicable. Conservatively, the designer can use the AISI equations contained in AISI Section D3.2.2. The total brace force equals the sum of the brace force for each purlin in the roof system. Alternately additional testing can be used to determine the brace forces. The following procedure is suggested:

1. Conduct the base tests using the intended bracing system, i.e. use the number of discrete braces to be used in the system.

2. Conduct the base tests without discrete braces.

Based on these two sets of tests, the discrete point brace forces can be obtained for a given purlin line as following:

1. Determine the moment capacity of the given purlin based on the base test R value obtained from the base tests conducted with the braces.

2. Determine the moment capacity of the given purlin based on the base test R value obtained from the base tests conducted without the braces.

3. Subtract the value obtained from (2) from the value obtained from (1).
4. Determine the load along the length of the purlin which would produce the moment obtained in (3), \( W = \frac{8M_J}{L^2} \).

5. Use the load \( W \) obtained from step (4) as the load \( W \) to be used in the appropriate anchorage equation from the equations contained in AISI Section D3.2.1. This calculation must be based on one purlin. The total anchorage force equals the force obtained from the calculations for one purlin multiplied by the total number of purlins being braced in the roof system.

8. **STANDING SEAM ROOF SYSTEMS SUBJECTED TO IN-PLANE FORCES**

Purlins as a part of the horizontal roof truss bracing system are often subjected to a combination of axial forces and bending moments. As discussed in Section 4.6.5, the AISI Specification is to be used to calculate the nominal strength of these beam-columns using the equations in AISI Section C5. The Specification is silent on the calculation of the nominal axial force component \( P_a \) when the purlins support standing seam roofs.

The nominal axial force for a purlin supporting a standing seam roof can be determined based on the discrete point bracing provided to the purlin by using the equations in AISI Chapter C4. If no discrete point bracing exists then tests must be conducted. Contained in Appendix II is a suggested test procedure.

The nominal moment capacity for the purlin is determined from the gravity or uplift base tests depending upon the loading condition being investigated for the strut purlin.

The applied forces and moments are obtained from the analysis of the structure. The strut purlin may have to be investigated for several moment conditions along its length. In determining the end moments applied to the strut purlin the eccentricity between the horizontal brace rods and the purlin centroid must be considered.

9. **STANDING SEAM ROOFS ON STEEL JOISTS**

The Standard Specification for Open Web Steel Joists, K-Series, 1995, requires that roof deck fasteners be spaced at no more than 36 inches, and be capable of carrying a lateral load of 300 pounds. The Steel Joist Institute currently does not recognize standing seam roofs as being capable of providing the necessary lateral support for steel joists. Some joist manufacturers have conducted proprietary tests to determine the strength of their joists with various standing seam roof systems. Currently there are no standardized test procedures to determine the lateral bracing effectiveness of the standing seam panel.

Without supportive test data, joist manufacturers typically add extra bridging lines and design the joist chords based on the spacing of the bridging.

The Steel Joist Institute does not address anchorage requirements for bridging lines, other than stating, “The ends of all bridging lines terminating at walls or beams shall be anchored thereto”. It is also stated in the Standard Specifications For Open Web Steel Joists, K-Series that “Bridging shall support the top chords against lateral movement during construction period and shall hold the steel joists in the approximate position as shown on the plans”. For K-series joists each horizontal bridging attachment to the joists must be capable of resisting a horizontal force of not less than 700 pounds. It is not stated in the Specification whether the 700 pounds is a factored load or a service load. The writers presume that it is a service load.

Design Recommendations
The following guidelines are presented for the design of joists laterally supported by standing seam roof systems. These recommendations are based on research by Fisher, et al. 1995.

These considerations have been presented in ultimate strength terms that would be appropriate for limit state design. Adjustments must be made for allowable stress design.

The designer of such systems should be aware that the use of standing seam roofs to laterally support steel joists is not recognized by the Steel Joist Institute.

1. The use of standing seam roofs to laterally support steel joists should be limited to K-series joists.

2. Each type of roof system must be tested using a procedure similar to the AISI Base Test Method. The writers believe that it is conservative to test the roof system on K-Series joists that have the largest chord axial load in combination with the largest bridging spacing to be used in the system.

3. A reasonable estimate of the bracing force that must be resisted by the roof system can be determined using the work of Lutz and Fisher, 1985.

Lutz and Fisher developed the following expression for the ideal stiffness of continuous bracing, $K_e$, as a function of the critical compression load of the member $P_{cr}$.

$$K_e = \frac{2.5P_{cr}}{L_e^2} \quad \text{(Eq. 9.1-1)}$$

where

$$L_e = \pi \sqrt{\frac{E_t I}{P_{cr}}} \quad \text{(Eq. 9.1-2)}$$

$I$ is the moment of inertia about the axis of buckling and the tangent modulus, $E_t$, is given by

$$E_t = \frac{4EP_{cr}}{P_{y}}(1 - \frac{P_{cr}}{P_{y}}) \quad \text{(Eq. 9.1-3)}$$

For an imperfect member that has an initial displacement $d_0$ in the plane of buckling, the required stiffness, $K_r$, is defined by

$$K_r = K_e(1 + d_0 / d) \quad \text{(Eq. 9.1-4)}$$

where $d$ is the additional displacement when buckling occurs, which is often taken as $d_0$ for design purposes. The required bracing force, $P_b$, in force/length is the stiffness times $d$ or

$$P_b = K_d = K_e(d_0 + d) \quad \text{(Eq. 9.1-5)}$$

The value of $d_0$ for the region between bridging lines should be conservatively estimated from experience or construction tolerances. The writers suggest a value of span over 500 (distance between braced points).

4. The critical load in upper chord of the joist must be determined. For joist proportions and material strength similar to those in the test program, it should not exceed 80% of the yield load.
5. An eave member is to be provided. The stability loads are collected by the eave member based on the following load transfer path:

a. The lateral bracing forces are transferred to each roof panel by friction, panel envelopment and clip panel interaction. This force transfer occurs continuously along the length of each joist.

b. Once transferred into the individual panels and bridging lines, forces are accumulated along the length of the panels and bridging, and are transferred into the fasteners at the eave member. Some of the bracing force may be distributed to the frame lines via the roof diaphragm and does not reach the eave member. The amount of force transferred to the frame lines through the diaphragm is dependent on the ratio of the diaphragm stiffness to the total stiffness of the roof system.

c. The accumulated bracing forces are transferred laterally along the eave beam through the fasteners in the eave beam until reaching the end supports of the eave beam at the frame lines.

d. The forces that are transferred to each joist top chord at the frame lines through diaphragm action must be transferred to the frame lines through the clips and the joist attachment to the frame.

e. The forces that remain in the eave member must be transferred to the frame lines by the connection of the eave member to the frame line.

For the above described force path to be permissible, fasteners in the eave member must have sufficient resistance to transfer the forces, and the eave member must have sufficient shear and bending resistance. The maximum shear in any fastener and the maximum bending in the eave member can be found from statics.

6. The force that the sheeting must carry in tension or compression must reflect the number of joists that are being braced. Conservatively, the total number of joists could be used, or the suggestion in the draft of the International Standards Organization Code that \(0.2n + 0.8\sqrt{n}\) joists be considered, where \(n\) is the number of joists being braced, can be adopted. For sloping roofs, the in-plane component of the load on the sheeting must be included in the analysis.

7. To determine the in-plane shear and associated moment in the collecting member, a distribution of the sheeting force must be assumed. It is conservative to use a uniform lateral force distribution. A parabolic or sinusoidal distribution would also be reasonable.

10. ROOF TOP UNITS AND HANGING LOADS

The lateral support provided to purlins from a standing seam roof is available from roof clips, friction and panel envelopment. The amount of support provided by each of these effects is unknown. Since the base test is conducted in an air chamber, whereby the loads are applied through the standing seam roof to the purlins, the stability effect of concentrated loads applied to the purlin flanges is unknown. Thus, purlins supporting concentrated loads should be braced independently of the standing seam roof system. From a practical point of view, the stability forces from small collateral loads such as sprinkler lines, ceiling, etc. may be neglected.
11. DESIGN EXAMPLES

11.1 Standing Seam Panel Design

Dead Load = 2.0 PLF, Snow Load = 70 PLF, Wind Uplift = 140 PLF

Fig. 11.1 Geometry and Loading

Fig. 11.2 Shears and Moments

Fig. 11.3 Panel Cross Section

Given: 1. Four span standing seam roof panel.
2. $F_y = 50$ ksi
3. Use ASD approach.

Required:

1. Calculate Section Properties.
2. Check the design for gravity loads.
3. Check the design for uplift loads.

Solution:
1. **Calculation of Section Properties**

Based on the design procedures of the AISI Specification, the following section properties can be obtained:

\[
\begin{align*}
I_x &= 0.191 \text{ in.}^4 \\
S_f &= 0.113 \text{ in.}^3 \text{ (top), } 0.612 \text{ in.}^3 \text{ (bottom)} \\
S_e &= 0.101 \text{ in.}^3 \text{ (top), } 0.618 \text{ in.}^3 \text{ (bottom)} - \text{ Positive Bending} \\
&= 0.086 \text{ in.}^3 \text{ (top), } 0.085 \text{ in.}^3 \text{ (bottom)} - \text{ Negative Bending}
\end{align*}
\]

Note: When computing effective section properties, the AISI Specification permits w/t ratios larger than 500. This is the situation for the computations of \( S_e \) (bottom).

2. **Check gravity loads**

a. **Strength for Bending Only (Section C3.1.1)**

**Required Strength:**

\[
M = M_D + M_s
\]

Maximum positive moment: \( M = 0.004 + 0.135 = 0.139 \text{ kip-ft.} \)

Maximum negative moment: \( M = 0.005 + 0.187 = 0.192 \text{ kip-ft.} \)

Positive moment is defined as a moment producing compression stresses on the top of the panel.

**Allowable Design Strength:**

Positive Moment:

\[
M_a = S_e F_y = (0.101)(50) \left(\frac{1}{12}\right) = 0.421 \text{ kip-ft.}
\]

\[
M_a = M_a / \Omega = 0.421 / 1.67 = 0.252 \text{ kip-ft.} > 0.139 \text{ kip-ft.} \text{ o.k.}
\]

Negative Moment:

\[
M_n = S_e F_y = (0.085)(50) \left(\frac{1}{12}\right) = 0.354 \text{ kip-ft.}
\]

\[
M_n = M_n / \Omega = 0.354 / 1.67 = 0.212 \text{ kip-ft.} > 0.192 \text{ kip-ft.} \text{ o.k.}
\]

b. **Strength for Shear Only (Section C3.2)**

**Required Strength:**

\[
V = V_D + V_s = 0.006 + 0.212 = 0.218 \text{ kips}
\]

**Allowable Design Strength**
For $t = 0.024$ in., $h = 1.856$ in., $h/t = 77.3 < 1.415 \sqrt{E_{k_v}/F_y} = 79.4$

$$V_n = 0.64t^2 \sqrt{k_v F_y E}$$

$$V_n = 0.64 \times 0.024^2 \sqrt{5.34 \times 50 \times 29500} = 1.035 \text{ kips}$$

$$V_a = V_n / \Omega = 1.035 / 1.67 = 0.620 \text{ kips per web}$$

$$V_a = 2 \times 0.620 = 1.240 \text{ kips} > 0.218 \text{ kips} \text{ o.k.}$$

c. **Strength for Combined Bending and Shear (Section C3.3)**

**Required strength:**

For the first interior support

$$M = M_D + M_S = 0.005 + 0.187 = 0.192 \text{ kip-ft.}$$

$$V = V_D + V_S = 0.006 + 0.212 = 0.218 \text{ kips}$$

$$(M/M_{w0})^2 + (V/V_a)^2 \leq 1.0$$

$$(0.192/0.212)^2 + (0.218/1.240)^2 = 0.85 < 1.0 \text{ o.k.}$$

d. **Web Crippling Strength (Section C3.4)**

**Required strength:**

$$P = P_D + P_S$$

Supports

End support $= 0.004 + 0.138 = 0.142 \text{ kips}$

First interior support $= 0.011 + 0.400 = 0.410 \text{ kips}$

**Allowable design strength:**

The following assumes a bearing length of 2-1/2 inches.

At end supports use Eq. C3.4-1 of the AISI Specification

$$P_n = t^2 k C_3 C_4 C_9 C_{10} [331 - 0.61 h/t] [1 + 0.01 N/t]$$

$$k = \frac{894 F_y}{E}$$

$$= \frac{(894)(50)}{(29500)} = 1.515$$

$$C_3 = 1.33 - 0.33k$$

$$= 1.33 - (0.33)(1.515) = 0.8300$$

$$C_4 = 1.15 - 0.15 R/t$$

$$= 1.15 - (0.15)(0.048)/0.024 = 0.8500$$

-28-
\[ C_9 = 1.0 \]
\[ C_0 = 1.0 \]
\[ P_n = (0.024)^2(1.515)(0.8300)(0.8500)(1)(1) \left[ 331 - 0.61 \frac{1.856}{0.024} \left( 1 + 0.01 \frac{2.5}{0.024} \right) \right] \]
\[ P_n = 0.357 \text{ kips per web} \]
\[ P_s = P_n/\Omega = 0.357 \times 2 \text{ webs}/1.80 = 0.396 \text{ kip} > 0.142 \text{ kip} \text{ o.k.} \]

At interior supports use Eq. C3.4-4 of the AISI Specification

\[ P_n = t^2 k C_1 C_2 C_3 C_4 [538 - 0.74 h/t][0.75 + 0.011 N/t] \]

where

\[ C_1 = 1.22 - 0.22k \]
\[ = 1.22 - 0.22(1.515) = 0.8867 \]
\[ C_2 = 1.06 - 0.06 R/t \]
\[ = 1.06 - (0.06)(0.048)/0.024 = 0.9400 \]
\[ C_3 = 1.0 \]
\[ C_4 = 1.0 \]
\[ P_n = (0.024)^2(1.515)(0.8867)(0.9400)(1)(1) \left[ 538 - 0.74 \frac{1.856}{0.024} \left( 0.75 + 0.011 \frac{2.5}{0.024} \right) \right] \]
\[ P_n = 0.663 \text{ kip per web} \]
\[ P_s = P_n/\Omega = 0.663 \times 2 \text{ webs}/1.80 = 0.737 \text{ kips} > 0.410 \text{ kip} \text{ o.k.} \]

e. Combined Bending and Web Crippling (Section 3.5)

\[ 1.2(P/P_s) + (M/M_{xxo}) \leq 1.5 \]

The AISI Specification excludes the application of the above equation to deck or panel sections. This implies that combined bending and web crippling is not a strength design consideration.

3. Check uplift loads

a. Strength for Bending Only (Section C3.1.1)

Required strength:

\[ M = (M_D - M_u)0.75, \text{ where 0.75 is per Section A5.1.3.} \]

Maximum positive moment: \[ M = \left| (0.005 - 0.375) \right| (0.75) = 0.278 \text{ kip-ft.} \]

Maximum negative moment: \[ M = \left| (0.004 - 0.270) \right| (0.75) = 0.198 \text{ kip-ft.} \]
Positive moment is defined as a moment producing compression stresses on the top of the panel.

Allowable design strength:

Positive Moment, \( M_a = 0.252 \text{ kip-ft.} < 0.278 \text{ kip-ft.} \) n.g.

Negative Moment, \( M_a = 0.212 \text{ kip-ft.} > 0.198 \text{ kip-ft.} \) o.k.

b. Strength for Shear Only (Section C3.2)

Required Strength

\[
V = (V_D - V_w)0.75 = \left| (0.006 - 0.425) \right| (0.75) = 0.314 \text{ kip}
\]

\[
V_s = 1.240 \text{ kip} > 0.314 \text{ kip} \text{ o.k.}
\]

c. Strength for Combined Bending and Shear (Section C3.3)

Required Strength:

For the first interior support

\[
M = (M_D - M_w)0.75 = \left| (0.005 - 0.375) \right| (0.75) = 0.278 \text{ kip-ft.}
\]

\[
V = (V_D - V_w)0.75 = \left| (0.006 - 0.425) \right| (0.75) = 0.314 \text{ kip}
\]

\[
\left(\frac{M}{M_{aro}}\right)^2 + \left(\frac{V}{V_s}\right)^2 \leq 1.0
\]

\[
(0.278/0.252)^2 + (0.314/1.240)^2 = 1.28 > 1.0 \text{ n.g.}
\]

Summary:

The standing seam panel is adequate for the applied dead load and snow load. However, for uplift loads, the panel is inadequate for bending at the first interior support. Because the panel is inadequate for bending only, it also fails to pass the combined bending and shear check at the first interior support.

11.2 Base Test Evaluation

Using the AISI Base Test Method for Purlins Supporting a Standing Seam Roof System, tests were conducted for the purlin cross section shown in Figure 11.4 below. Apply the evaluation procedure as prescribed in the AISI Base Test Method and determine the appropriate reduction factors.
In accordance with Section 7.2 of the AISI Base Test Method, three tests were conducted for the thickest profile and three tests were conducted for the thinnest profile. The span length for all tests was 25 ft. The following summarizes the data:

Thickest profile, \( t = 0.135 \) in. (measured)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Failure Load, ( p_{ts} ) (lb/ft(^2))</th>
<th>Specimen wt., ( p_d ) (lb/ft(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>55.6</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>56.8</td>
<td>2.0</td>
</tr>
<tr>
<td>3</td>
<td>56.4</td>
<td>2.0</td>
</tr>
</tbody>
</table>

Thinnest profile, \( t = 0.06 \) in. (measured)

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Failure Load, ( p_{ts} ) (lb/ft(^2))</th>
<th>Specimen wt., ( p_d ) (lb/ft(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18.4</td>
<td>1.0</td>
</tr>
<tr>
<td>2</td>
<td>18.0</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>17.6</td>
<td>1.0</td>
</tr>
</tbody>
</table>

The maximum anticipated purlin spacing, \( B \), and the tributary width of the purlins tested, \( s \), are both 5.0 ft.

Determine the lateral anchorage force.

In accordance with Section 8.1 of the AISI Base Test Method, the anchorage force is

\[
P_L = 0.041 \left[ \frac{b^{1.5}}{d^{0.90} t^{0.60}} \right] (p_{ts} + p_d)s
\]

For 0.135 in. profile,

\[
P_L = 0.041 \left[ \frac{(3)^{1.5}}{(9)^{0.9} (0.135)^{0.60}} \right] (p_{ts} + p_d)s
\]

\[
P_L = 0.098 (p_{ts} + p_d)s
\]

For 0.060 in. profile,

\[
P_L = 0.041 \left[ \frac{(3)^{1.5}}{(9)^{0.9} (0.060)^{0.60}} \right] (p_{ts} + p_d)s
\]

\[
P_L = 0.159 (p_{ts} + p_d)s
\]

Compute the failure load, \( w_{ts} \)

Per Section 8.1 of the AISI Base Test Method, the tested failure load is determined by

\[
w_{ts} = (p_{ts} + p_d)s + 2P_L (d/B)
\]
Compute the modification factor, R_t,

The purlin strength modification factor, R_t, is determined by Sections 8.2 through 8.5 of the AISI Base Test Method.

\[ M_{ts} = \frac{w_t L^2}{8}, \text{single span failure moment} \]

\[ L = 25 \text{ ft.}, \text{the test purlin span length} \]

\[ M_{nt} = S_{\alpha} F_{\alpha}, \text{computed moment capacity of the fully braced section} \]

When evaluating \( M_{nt} \), the measured yield stress, \( F_{\alpha} \), and the measured geometry for each Test No. must be used to compute \( M_{nt} \). Because the intent of this example is to demonstrate the application of the AISI Base Test Method, \( F_{\alpha} \) and \( S_{\alpha} \) were taken as constants for each of the three tests.

The computed moment capacity is determined using Section C3.1.1(a) of the AISI Specification. \( S_{\alpha} \) is the computed effective section modulus using the measured section properties and the measured yield strength, \( F_{\alpha} \). The measured yield strength is determined using the procedures of ASTM A370.
Compute the reduction factor, R

The purlin strength reduction factor, R, is computed by using Section 8.6 of the AISI Base Test Method.

\[
R = \frac{R_{\text{max}} - R_{\text{min}}}{M_{\text{max}} - M_{\text{min}}} (M_R - M_{\text{min}}) + R_{\text{min}} \leq 1.0
\]

where the symbols are defined in Section 8.6 of the AISI Base Test Method.

\[
R_{\text{min}} = 0.827 - 0.013 = 0.814
\]
\[
R_{\text{max}} = 0.907 - 0.013 = 0.894
\]
\[
M_{\text{min}} = 109.36 \text{ kip-in}
\]
\[
M_{\text{max}} = 306.87 \text{ kip-in}
\]

Using the above numerical values, the reduction factor for any purlin thickness can be determined. For example, in the table below R values for 0.135, 0.090, and 0.060 thicknesses are shown.

<table>
<thead>
<tr>
<th>Purlin Thickness (in.)</th>
<th>M_p (kip-in)</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.135</td>
<td>300.14</td>
<td>0.891</td>
</tr>
<tr>
<td>0.090</td>
<td>179.86</td>
<td>0.843</td>
</tr>
<tr>
<td>0.060</td>
<td>106.57</td>
<td>0.813</td>
</tr>
</tbody>
</table>

The nominal flexural strength for each purlin is the product of RMR.

It should be noted that the R values for this example decrease with decreasing thicknesses. The reader should not assume that this is typical for all standing seam systems.
11.3 Continuous Purlin Design for Gravity and Wind Uplift Loads

Given:

1. Four span Z-purlin system using laps at interior support points to create continuity (see Figures 1 and 2).

2. Roof covering is attached with standing seam panel clips along entire length of purlins.

3. $F_y = 55$ ksi

4. Roof Slope $= 0.5/12$

5. No discrete bracing lines.

6. Purlin flanges are bolted to support member and anti-roll clips are provided at each support at every fourth purlin line.

7. Tested R values using the AISI Base Test Method:

   For gravity loads:
   
   $R = 0.85$ for the 0.090 in. thick purlin.
   
   $R = 0.90$ for the 0.060 in. thick purlin.

---
For uplift load:
R = 0.70 for the 0.090 in thick purlin.
R = 0.70 for the 0.060 in. thick purlin.

**Required:**

1. Check the design for gravity loads using the ASD approach.
2. Check the design for uplift loads using the ASD approach.
3. Compute the anchorage forces at the supports.

**Solutions:**

Note: The equations and Sections referenced in the example refer to the AISI Specification Sections and the AISI equation numbers.

**Assumptions for Analysis and Application of the AISI Specification Provisions**

The AISI Specification does not define the methods of analysis to be used; these judgments are the responsibility of the designer. The following assumptions are considered good practice but are not intended to prohibit other approaches:

a. The purlins are connected within the lapped portions in a manner that achieves full continuity between the individual purlin members.

b. The continuous beam analysis to establish the shear and moment diagrams assumes continuous non-prismatic members in which $l_x$ within the lapped portions is the sum of the individual members.

c. The strength within the lapped portions is assumed to be the sum of the strengths of the individual sections.

d. For gravity loads, the compression (bottom) flange at and near the interior supports is assumed to be fully braced between the support and the end of the lap. The negative moment region between the end of the lap and the inflection point is treated as a cantilever with an unbraced free end.

e. Since the loading, geometry and materials are symmetrical, check the first two spans only.

**Calculation of Section Properties**

Based on the design procedures illustrated in Examples No. I-3 and No. I-10 and Table I-3 and II-3 of the AISI Cold-Formed Steel Design Manual, the following sections properties can be obtained for the two Z-sections:

For: $t = 0.060$ in.

- $I_x = 8.146$ in.$^4$
- $S_y = 2.037$ in.$^3$
- $S_e = 1.71$ in.$^3$
- $I_y = 1.225$ in.$^4$

For: $t = 0.090$ in.

- $I_x = 12.014$ in.$^4$
- $S_y = 3.004$ in.$^3$
- $S_e = 2.826$ in.$^3$
- $I_y = 1.787$ in.$^3$

1. **Check Gravity Loads**
a. Strength for Bending Only (Section C3.1)

Required Strength

By inspection, ASD load combination 2 from Section A5.1.2 controls:

\[ M = M_D + M_L \]

**End Span, from left to right:**

- Maximum positive moment: \[ M = 0.68 + 4.53 = 5.21 \text{ kip-ft.} \]
- Negative moment at end of right lap: \[ M = 0.69 + 4.57 = 5.26 \text{ kip-ft.} \]
- Negative moment at support: \[ M = 1.12 + 7.46 = 8.58 \text{ kip-ft.} \]

**Interior span, from left to right:**

- Negative moment at end of left lap: \[ M = 0.49 + 3.28 = 3.77 \text{ kip-ft.} \]
- Maximum positive moment: \[ M = 0.30 + 1.98 = 2.28 \text{ kip-ft.} \]
- Negative moment at end of right lap: \[ M = 0.49 + 3.29 = 3.78 \text{ kip-ft.} \]
- Negative moment at center support: \[ M = 0.66 + 4.37 = 5.03 \text{ kip-ft.} \]

**Allowable Design Strength**

**End span:**

At the location of maximum positive moment, the section is assumed to be braced by the standing seam panel. The ability of the panel to brace the purlin has been quantified by the AISI Base Test Method (R = 0.85).

Use allowable moment per Section C3.1.4.

For the end span purlin, \( t = 0.090 \)

\[ M_a = R_S F_y = (0.85)(2.826)(55) = 132.1 \text{ kip-in.} = 11.01 \text{ kip-ft.} \] (Eq. C3.1.4-1)

\[ \frac{M_a}{\Omega_b} = \frac{11.01}{1.67} = 6.59 \text{ kip-ft.} > 5.21 \text{ kip-ft. o.k.} \] (Eq. A5.1.1-1)

In the region of negative moment between the end of the lap and the inflection point, treat the section as an unbraced cantilever with a free end:

Determine the allowable moment using the distance from the inflection point to the lap as the unbraced length per Section C3.1.2.(b) with \( C_b = 1.0 \).

\[ L = 5.96 - 2.00 = 3.96 \text{ ft.} = 47.5 \text{ in.} \]

\[ I_{yc} = \frac{I_{y}}{2} = \frac{1.787}{2} = 0.894 \text{ in.}^4 \]
C_b = 1.0.

\[ M_e = \frac{\pi^2 E C_b d I_{y_{f}}}{2L^2} \]  
\[ = \frac{\pi^2 \times 29500 \times 1.0 \times 8.0 \times 0.894}{2 \times 47.5^2} = 461.4 \text{ kip-in.} \]

\[ M_y = S_r F_y = (3.004)(55) = 165.2 \text{ kip-in.} \]

Since \( M_e = 461.4 \text{ kip-in.} > 2.78 \) \( M_y = 459.3 \text{ kip-in.}, \) \( M_e = M_y = 165.2 \text{ kip-in.} \)

\[ M_a = S_c \frac{M_e}{S_c} = 2.826 \frac{165.2}{3.004} = 155.4 \text{ kip-in. or 12.95 kip-ft.} \]  
\[ \text{(Eq. C3.1.2-1)} \]

\[ \frac{M_a}{\Omega_b} = \frac{12.95}{1.67} = 7.75 \text{ kip-ft.} > 5.26 \text{ kip-ft. o.k.} \]  
\[ \text{(Eq. A5.1.1-1)} \]

At the negative moment at the support, the section is assumed to be fully braced:

Use allowable moments per Section C3.1.4, summing the strength of the two overlapped purlins:

For the exterior purlin, \( t = 0.090 \) in.

\[ M_a = S_c F_y = (2.826)(55) = 155.4 \text{ kip-in. or 12.95 kip-ft.} \]

For the interior purlin, \( t = 0.060 \) in.

\[ M_a = S_c F_y = (1.71)(55) = 94.1 \text{ kip-in. or 7.84 kip-ft.} \]  
\[ \text{(Eq. C3.1.1-1)} \]

Combined strength of purlins

\[ \frac{M_a}{\Omega_b} = \frac{12.95 + 7.84}{1.67} = 12.45 \text{ kip-ft.} > 8.58 \text{ kip-ft. o.k.} \]  
\[ \text{(Eq. A5.1.1-1)} \]

Interior Span:

In the region of negative moment between the end of the left lap and the inflection point, treat the section as an unbraced cantilever with a free end:

Determine the allowable moment using the distance from the inflection point to the lap as the unbraced length per Section C3.1.2(b) with \( C_b = 1.0 \)

\[ L = 7.45 - 3.50 = 3.95 \text{ ft. or 47.4 in.} \]

\[ I_{y_{f}} = \frac{I_f}{2} = \frac{1.225}{2} = 0.613 \text{ in.}^4 \]
At the location of maximum positive moment, the section is assumed to be braced by the standing seam panel. The ability of the panel to brace the purlin has been quantified by the AISI Base Test Method ($R = 0.90$).

\[ M_n = RS_eF_y = (0.90)(1.71)(55) = 84.65 \text{ kip-in. or 7.05 kip-ft.} \]

\[ \frac{M_n}{\Omega_b} = \frac{7.05}{1.67} = 4.22 \text{ kip-ft.} > 3.77 \text{ kip-ft. o.k.} \quad (\text{Eq. A5.1.1-1}) \]

In the region of negative moment between the right lap and the inflection point, treat the section as an unbraced cantilever with a free end:

Determine the allowable moment using the distance from the inflection point to the lap as the unbraced length per Section C3.1.2(b) with $C_b = 1.0$.

\[ L = 4.98 - 1.00 = 3.98 \text{ ft. or 47.8 in.} \]

By inspection, this condition is less severe than the left lap, since the unbraced length is about equal and the required strength is less, therefore the section is o.k.

At the negative moment at the center support, the section is assumed to be fully braced:

Use allowable moments based on initiation of yielding per Section C3.1.1(a), summing the strength of the two overlapped purlins:

Combined strength of purlins

\[ \frac{M_h}{\Omega_b} = \frac{7.84 + 7.84}{1.67} = 9.39 \text{ kip-ft.} > 5.03 \text{ kip-ft. o.k.} \quad (\text{Eq. A5.1.1-1}) \]

b. Strength for Shear Only (Section C3.2)

Required Strength:

By inspection, load combination 2 from Section A5.1.2 controls:

\[ V = V_D + V_{lr} \]
AISI — A Guide for Designing with Standing Seam Roof Panels

End Span, from left to right:

At left support: \( V = 0.14 + 0.95 = 1.09 \) kip
At end of right lap: \( V = 0.20 + 1.35 = 1.55 \) kip
At first interior support: \( V = 0.23 + 1.55 = 1.78 \) kip

Interior Span, from left to right:

At first interior support: \( V = 0.21 + 1.37 = 1.58 \) kip
At end of left lap: \( V = 0.15 + 1.02 = 1.17 \) kip
At end of right lap: \( V = 0.15 + 1.03 = 1.18 \) kip
At center support \( V = 0.17 + 1.13 = 1.30 \) kip

Allowable Design Strength

End Span:

At the left support and right lap, \( t = 0.060 \). By inspection the end of the right lap controls.

For \( t = 0.090 \) in. and \( h = 7.445 \) in.

\[
\frac{h}{t} = 82.7 > 1.415\sqrt{\frac{E_k v}{F_y}} = 75.7
\]

\[
V_n = \frac{\pi^2 E_k t^3}{[12(1 - \mu^2)h]} = 13.94 \text{ kip} 
\]

\[
V_n = \frac{\pi^2 \times 29500 \times 5.34 \times 0.090^3}{12(1 - 0.3^2)7.445} = 13.94 \text{ kip} 
\]

\[
\frac{V_n}{\Omega_v} = \frac{13.94}{1.67} = 8.35 \text{ kip} > 1.55 \text{ kip} \quad \text{o.k.} 
\]

(Eq. C3.2-3)

At the first interior support, sum the strength of the two overlapped purlins:

For \( t = 0.060 \) in. and \( h = 7.505 \) in., \( h/t = 125.1 \)

\[
\frac{h}{t} > 1.415 \sqrt{\frac{E_k v}{F_y}} = 1.415 \sqrt{29500 \times 5.34 / 55} = 75.7
\]

\[
V_n = \frac{\pi^2 \times 29500 \times 5.34 \times 0.060^3}{12(1 - 0.3^2)7.505} = 4.10 \text{ kip} 
\]

(Eq. C3.2-3)

For the combined section:
\[
\frac{V_n}{\Omega_v} = \frac{4.10 + 13.94}{1.67} = 10.80 \text{ kip} > 1.78 \text{ kip} \quad \text{o.k.} \quad (\text{Eq. A5.1.1-1})
\]

Interior span:

By inspection of the left and right laps, the left lap controls

\[
\frac{V_n}{\Omega_v} = \frac{4.10}{1.67} = 2.45 \text{ kip} > 1.17 \text{ kip} \quad \text{o.k.} \quad (\text{Eq. A5.1.1-1})
\]

At the center support, sum the strength of the two overlapped purlins:

For the combined section:

\[
\frac{V_n}{\Omega_v} = \frac{4.10 + 4.10}{1.67} = 4.91 \text{ kip} > 1.30 \text{ kip} \quad \text{o.k.} \quad (\text{Eq. A5.1.1-1})
\]

c. Strength for Combined Bending and Shear (Section C3.3)

End Span:

\[
\left( \frac{M}{M_{exo}} \right)^2 + \left( \frac{V}{V_n} \right)^2 \leq 1.0
\quad (\text{Eq. C3.3.1-1})
\]

where

\[M_{exo} = \frac{M_n}{\Omega_b}\]

where \(M_n\) is calculated on the initiation of yielding per Section C3.1.1

\[V_n = \frac{V_n}{\Omega_v}\]

At start of lap, \(t = 0.090\) in.

\[
\left( \frac{5.26}{7.75} \right)^2 + \left( \frac{155}{8.35} \right)^2 = 0.50 < 1.0 \quad \text{o.k.} \quad (\text{Eq. C3.3.1-1})
\]

At interior support,

\[
\left( \frac{8.58}{7.75 + 4.69} \right)^2 + \left( \frac{1.78}{8.35 + 2.45} \right)^2 = 0.50 < 1.0 \quad \text{o.k.} \quad (\text{Eq. C3.3.1-1})
\]

Interior Span:

At end of laps, \(t = 0.060\) in. Right lap controls by inspection.
(\frac{3.78}{4.69})^2 + \left(\frac{1.18}{2.45}\right)^2 = 0.88 < 1.0 \text{ o.k.} \quad (\text{Eq. C3.3.1-1})

At center support,

\left(\frac{5.03}{4.69 + 4.69}\right)^2 + \left(\frac{1.30}{2.45 + 2.45}\right)^2 = 0.36 < 1.0 \text{ o.k.} \quad (\text{Eq. C3.3.1-1})

d. Web Crippling Strength (Section C3.4)

Required Strength

By inspection, load combination 2 from Section A5.1.2 controls:

\( P = P_D + P_L \)

Supports, from left to right:

- At left support: \( P = 0.14 + 0.95 = 1.09 \text{ kip} \)
- At first interior support: \( P = 0.44 + 2.92 = 3.36 \text{ kip} \)
- At center support: \( P = 0.34 + 2.25 = 2.59 \text{ kip} \)

Allowable Design Strength:

The following assumes a bearing length of 5 inches.

At outside supports use Eq. C3.4-1 of the AISI Specification.

\[
P_a = t^2kC_3C_4C_9C_0 \left[ 331 - 0.61 \frac{h}{t} \left( 1 + 0.01 \frac{N}{t} \right) \right] \quad (\text{Eq. C3.4-1})
\]

where

\[
k = \frac{894F_{/E}}{E} = \frac{(894)(55)}{29500} = 1.67 \quad (\text{Eq. C3.4-21})
\]

\[
C_3 = 1.33 - 0.33k = 1.33 - (0.33)(1.667) = 0.78 \quad (\text{Eq. C3.4-12})
\]

\[
C_4 = 1.15 - 0.15\frac{R}{t} = 1.15 - (0.15)(0.1875)/0.090 = 0.838 \quad (\text{Eq. C3.4-13})
\]

\[
C_9 = 1.0
\]

\[
C_0 = 1.0
\]

\[
P_a = 0.090^2(1.67)(0.78)(0.838)(1.0)(1.0) \left[ 331 - 0.61 \frac{7.445}{0.090} \left( 1 + 0.01 \frac{5.0}{0.090} \right) \right]
\]
= 3.86 kip  

(Eq. C3.4-1)

Per Section C3.4 of the AISI Specification $P_n$ may be multiplied by 1.3.

$$P_n = 1.3(3.86) = 5.02 \text{ kip}$$

$$\frac{P_n}{\Omega_w} = \frac{5.02}{1.80} = 2.79 \text{ kip} > 1.09 \text{ kip} \quad \text{o.k.}$$

(Eq. A5.1.1-1)

At interior supports use Eq. C3.4-4 of the AISI Specification.

$$P_n = t^2kC_1C_2C_9C_0\left[538 - \frac{0.74h}{t}\left[0.75 + \frac{0.011N}{t}\right]\right]$$

(Eq. C3.4-4)

where

$$C_1 = 1.22 - 0.22k = 1.22 - (0.22)(1.67) = 0.853$$

(Eq. C3.4-10)

For $t = 0.090$ in.,

$$C_2 = 1.06 - 0.06R/t \leq 1.0 = 1.06 - (0.06)(0.1875)/0.090 = 0.935$$

(Eq. C3.4-11)

$$C_9 = 1.0$$

$$P_n = 0.090^2(1.67)(0.853)(0.935)(1.0)\left[538 - \frac{0.74}{0.090} - \frac{7.445}{0.090}\left[0.75 + \frac{0.011}{0.090} - \frac{5.0}{0.090}\right]\right]$$

$$= 7.0 \text{ kip}$$

(Eq. C3.4-4)

For $t = 0.060$ in.,

$$C_2 = 1.06 - 0.06R/t \leq 1.0 = 1.06 - (0.06)(0.1875)/0.060 = 0.873$$

(Eq. C3.4-11)

$$P_n = 0.060^2(1.67)(0.853)(0.873)(1.0)\left[538 - \frac{0.74}{0.060} - \frac{7.505}{0.060}\left[0.75 + \frac{0.011}{0.060} - \frac{5.0}{0.060}\right]\right]$$

$$= 3.32 \text{ kip}$$

(Eq. C3.4-4)

At first interior support,

$$\frac{P_n}{\Omega_w} = \frac{7.0+3.32}{1.80} = 5.73 \text{ kip} > 3.36 \text{ kip} \quad \text{o.k.}$$

(Eq. A5.1.1-1)

At center support,

$$\frac{P_n}{\Omega_w} = \frac{3.32+3.32}{1.80} = 3.69 \text{ kip} > 2.59 \text{ kip} \quad \text{o.k.}$$

(Eq. A5.1.1-1)

e. Combined Bending and Web Crippling (Section C3.5)
\[ \frac{M}{M_{no}} + \frac{P}{P_n} \leq \frac{1.67}{\Omega} \]  
(Eq. C3.5.1-3)

where

\[ M_{no} = \text{the sum of } M_n \text{ of each purlin at the support calculated based on the initiation of yielding per Section C3.1.1} \]

\[ P_n = \text{the sum of } P_n \text{ of each purlin at the support} \]

\[ \Omega = 1.67 \]

At the first interior support,

\[ \frac{8.58}{12.95 + 7.84} + \frac{3.36}{7.0 + 3.32} = 0.74 < 1.0 \text{ o.k.} \]  
(Eq. C3.5.1-3)

This check should also be performed at the other interior supports.

2. **Check Uplift Loads**

   a. Strength for Bending Only (Section C3.1.4)

   Required Strength:

   By inspection, ASD load combination 3 from Section A5.1.2 controls. Per Section A5.1.3, the resulting forces may be multiplied by 0.75:

   \[ M = 0.75(M_D + M_u) \]

   End Span:

   Moment near center of span: \[ M = 0.75(0.68 - 5.21) = -3.40 \text{ kip-ft.} \]

   Interior Span:

   Moment near center of span: \[ M = 0.75(0.30 - 2.27) = -1.48 \text{ kip-ft.} \]

   Allowable Design Strength:

   \[ M_n = R S \epsilon F_y \]  
(Eq. C3.1.4-1)

   \[ R = 0.70 \text{ for both purlin thicknesses} \]

   End Span:

   For \( t = 0.090 \text{ in.} \)

   \[ M_n = (0.70)(2.826)(55) = 108.8 \text{ kip-in. or 9.07 kip-ft.} \]  
(Eq. C3.1.4-1)

   \[ \frac{M_n}{\Omega_b} = \frac{9.07}{1.67} = 5.43 \text{ kip-ft. > 3.40 kip-ft. o.k.} \]  
(Eq. A5.1.1-1)
Interior Span:

For \( t = 0.060 \) in.

\[
M_b = (0.70)(1.71)(55) = 65.8 \text{ kip-in.} \text{ or } 5.49 \text{ kip-ft.} \quad \text{(Eq. C3.1.4-1)}
\]

\[
rac{M_b}{\Omega_b} = \frac{5.49}{1.67} = 3.28 \text{ kip-ft.} > 1.48 \text{ kip-ft.} \text{ o.k.} \quad \text{(Eq. A5.1.1-1)}
\]

b. Other Comments

Since the magnitude of the shears, moments and reactions are approximately 65 percent of those of the gravity case, it can be concluded that the design satisfies the Specification criteria for uplift.

3. Compute Anchorage Forces

Compute the anchorage forces at the supports (Section D3.2.1) with anchorage at every fourth purlin, Case (b4) - Multiple-Span System with Restraints at the Supports.

End Span:

\[
W = n(L)(DL + LL) = (4)(25)(15 + 100) = 11,500 \text{ lbs.}
\]

\[
\theta = \arctan(0.5/12) = 2.3859 \text{ degrees}
\]

\[
t = 0.090 \text{ in.}
\]

\[
P_L = C_u \left[ \frac{0.053b^{1.88}L^{0.13}}{n_p d^{1.07} t^{0.94}} - \sin \theta \right] W 
\]

\[
P_L = C_u \left[ \frac{(0.053)(2.5^{1.88})(25 \times 12)^{0.13}}{4^{0.95}8.0^{1.07}0.090^{0.94}} - \sin(2.3859) \right] 11,500
\]

\[
= C_u 1516
\]

Interior Span:

\[
P_L = C_u \left[ \frac{(0.053)(2.5^{1.88})(25 \times 12)^{0.13}}{4^{0.95}8.0^{1.07}0.060^{0.94}} - \sin(2.3859) \right] 11,500
\]

\[
= C_u 2441
\]

At outside supports, \( C_u = 0.63 \)

\[
P_L = (0.63)(1516) = 955 \text{ lbs.}
\]

At interior supports, average the contributions from adjacent purlins
At first interior support, \( C_L = 0.87 \)

\[
P_L = 0.87(1516 + 2441)/2 = 1721 \text{ lbs.}
\]

At center support, \( C_L = 0.81 \)

\[
P_L = 0.81(2441 + 2441)/2 = 1977 \text{ lbs.}
\]

### 11.4 Diaphragm Calculations to Determine Purlin Stability

Determine the diaphragm requirements to stabilize the 8ZS2.5x090 purlins in Example 11.3.

**Solution:**

From Example 11.3, \( P_L = 955 \) lbs. at the exterior support and \( P_L = 1721 \) lbs. at the first interior support. These values were calculated in Example 11.3 based on anchorage at every fourth purlin. Therefore the \( P_L \) values apply to a 20 foot width of diaphragm.

The worst loading case on the diaphragm comes from the exterior reaction. Based on \( P_L = 955 \) lbs. the load on the 20 foot diaphragm width equals \( 955/12.5 = 76.4 \) lb/ft. The shear in the diaphragm equals \( 955/20 = 47.8 \) lbs/ft.

Using Eq. 5.3-9 the required shear stiffness can be found as follows:

\[
\frac{L}{360} = \frac{w_u L^2}{8 G'd}
\]

\[
G' = \frac{45 w_u L}{d}
\]

\[
= \frac{(45)(1.67)(76.4)(25)(12)}{(20)(12)}
\]

\[
= 7,177 \text{ lbs./ft.}
\]

The diaphragm must have an allowable shear greater than 47.8 lbs/ft. and a shear modulus greater than 7,177 lbs./ft.

It should be noted that if the eave member is used by the designer in evaluating the diaphragm strength and stiffness an adjustment may be required in the supplied diaphragm strength and stiffness. For example, if the roof system consists of sixteen purlin lines the resisting strength and stiffness of the diaphragm per 4 purlin sets would be calculated as follows:

Find \( S_m \) from Eq. 5.3-1:

\[
S_m = (4)(20)S_u + S_e
\]

\[
= 80S_u + S_e
\]

\( S_m \) per purlin set = \( S_m / 4 \)

The allowable strength per purlin set equals
\[ S_s = \frac{S_{sd}}{4\Omega_d} \]

where \( \Omega_d = 2.45 \) from the AISI Specification Table D5.

The stiffness would be evaluated in an identical manner.

11.5 Diaphragm Design Example

Determine the shear deflection of the diaphragm shown in Fig. 11.6. The diaphragm has a factored edge loading of 250 lbs./ft. Also determine if the shear capacity of the diaphragm exceeds the shear capacity provided by the standing seam system.

**Fig. 11.6 Example 11.5**

*Given*

Diaphragm tests have been conducted for the standing seam system. Tests 1 and 2 were conducted without eave attachment, and Tests 3 and 4 contained eave members with connections. The tests were conducted using the configuration shown in Figure 11.7.
The test results are shown below:

<table>
<thead>
<tr>
<th>Test No. - Eave Condition</th>
<th>( P_{\text{max}} )</th>
<th>( G )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>lbs.</td>
<td>lbs./in.</td>
</tr>
<tr>
<td>1 - No eave connections</td>
<td>1000</td>
<td>1200</td>
</tr>
<tr>
<td>2 - No eave connections</td>
<td>800</td>
<td>750</td>
</tr>
<tr>
<td>3 - Eave connections</td>
<td>1800</td>
<td>3500</td>
</tr>
<tr>
<td>4 - Eave connections</td>
<td>2000</td>
<td>4000</td>
</tr>
</tbody>
</table>

Based on the test results:

\[
S_a = \frac{(P_1 + P_2)}{(2)(18)}
\]

where

\( P_1 = 1000 \) lbs.
\( P_2 = 800 \) lbs.

\[
S_a = \frac{(1000 + 800)}{(2)(18)} = 50 \text{ plf}
\]

\[
Q_a = (P_3 - P_4)a/2b
\]

where

\( P_3 = (1800 + 2000)/2 = 1900 \) lbs.
\( P_4 = (1000 + 800)/2 = 900 \) lbs.

\[
Q_a = (1900 - 900)(15)/[(2)(18)] = 417 \text{ lbs.}
\]
\[ K_n = \frac{G_2 + G_1}{2} \]

where

\[ G_2 = 1200 \text{ lbs./in.} \]
\[ G_1 = 750 \text{ lbs./in.} \]
\[ K_n = \frac{1200 + 750}{2} = 975 \text{ lbs./in.} \]
\[ K_e = \frac{(G_3 - G_4)}{2} \]

where

\[ G_3 = \frac{(3500 + 4000)}{2} \]
\[ G_4 = \frac{(1200 + 750)}{2} \]
\[ K_e = \frac{(3750-975)}{2} = 1388 \text{ lbs./in.} \]
\[ C_e = (K_e)(a) \]
\[ C_e = (1388)(15)(12) = 249,840 \text{ lbs.} \]

**Solution:**

**Deflection**

The deflection of a uniformly loaded diaphragm equals:

\[ \Delta_s = \frac{wL^2}{8G'b} \]

\[ G' = K_n + C_e/d \]
\[ G' = 975 + 249,840/[150(12)] = 1391 \text{ lbs./in.} \]

\[ \Delta_s = \frac{(250)(25)^2}{8(1391)(50)} = 0.29 \text{ in.} < \frac{L}{360} = \frac{(25)(12)}{360} = 0.833 \text{ in.} \]

**Shear Strength**

Design Shear Strength = \( \frac{wL}{2d} \)

\[ = \frac{(250)(25)}{(2)(50)} = 62.5 \text{ plf} \]
\[ S_{ut} = S_n + Q_e/50 = 50 + 417/50 = 58 \text{ plf} < 62.5 \text{ n.g.} \]
11.6 Strut Purlin Design

Given:
1. Wind uplift loading per Example 11.3.
2. Four span continuous Z-purlin system (Ex. 11.3).
3. Braced bay is the second interior bay.
4. Axial load = 5 kip (Reflects 0.75 factor from Section A.5.1.3).

Required:
Check the 8ZS 2.5 x 0060 as a strut-purlin


The AISI Specification does not define the methods of analysis to be used; these judgments are the responsibility of the designer. The following assumptions are considered good practice, but are not intended to prohibit other approaches.

a. The purlins are connected within the lapped portions in a manner that achieves full continuity between the individual purlin members.
b. The continuous beam analysis to establish the shear and moment diagrams assumes continuous non-prismatic members in which \( I_x \) within the lapped portions is the sum of the individual members.
c. The strength within the lapped portions is assumed to be the sum of the strengths of the individual sections.
d. Braced bay analyzed as a horizontal truss; the critically loaded strut-purlin will be the nearest strut-purlin to the sidewall of the building.

Calculation of Section Properties.

Based on the design procedures illustrated in Examples No. I-3 and No. I-10 and Table I-3 and II-3 of the AISI Cold-Formed Steel Design Manual, the following sections properties can be obtained for the two Z-sections:

For: \( t = 0.060 \text{ in.} \)
\[ s = 8.146 \text{ in.}^4 \]
\[ S_r = 2.037 \text{ in.}^3 \]
\[ I_x = 1.71 \text{ in.}^3 \]
\[ I_y = 1.225 \text{ in.}^4 \]
\[ r_x = 3.105 \text{ in.} \]

Solution:
1. Determine the strength for bending only.
2. Determine the strength for axial load only.
3. Determine the strength for combined compressive axial load and bending.

1. **Strength for Bending Only (Section C3.1.4.)**
Required Strength:

By inspection, load combination 3 from Section A5.1.2 controls. Per Section A5.1.3, the resulting forces may be multiplied by 0.75:

\[ M = 0.75(M_D + M_w) \]

Interior Span:

Moment near center of span: \[ M = 0.75(0.30 - 2.27) = -1.48 \text{ kip-ft.} \]

Allowable Design Strength

\[ M_a = RS_y \]

Tested R value using the Uplift Base Test Method:

\[ R = 0.70 \]

Interior Span:

For \( t = 0.060 \text{ in.} \)

\[ M_a = (0.70)(1.71)(55) = 65.8 \text{ kip-in. or 5.49 kip-ft.} \]

\[ \frac{M_a}{\Omega_b} = \frac{5.49}{1.67} = 3.29 \text{ kip-ft.} > 1.48 \text{ kip-ft. o.k.} \quad (\text{Eq. A5.1.1-1}) \]

Other Comments

All other regions of the system have their compression flange braced by the roof panel. Since the magnitude of the shears, moments and reactions are approximately 65 percent of those of the gravity case, it can be concluded that the design satisfies the Specification criteria for uplift.

2. Strength for Axial Load Only

a. Weak Axis Buckling

The nominal axial strength must be determined by test. Section 7 of this design guide discusses a recommended test procedure.

Based on the test procedure, assume \( P_u = 12 \text{ kips} \) was determined.

b. Strong Axis Buckling (Section C4)

\[ KL = \text{distance from end-of-lap to end-of-lap} \]

\[ = 25 - 3.5 - 1 = 20.5 \text{ ft.} \]

\[ F_e = \frac{\pi^2 E}{(KL/r)^2} \quad (\text{Eq. C4.1-1}) \]
\[
\lambda_c = \frac{F_y}{F_e} = \sqrt{\frac{55}{46.38}} = 1.089 < 1.5
\]

\[
F_n = (0.658\lambda_c^\frac{3}{2}) F_y = (0.658(1.089)^{\frac{3}{2}}) 55 = 33.48 \text{ ksi}
\]

\[
A_e = 0.531 \text{ in.}^2 \text{ at stress } F_n
\]

\[
P_n = A_e F_n = 0.531 \times 33.48 = 17.78 \text{ kips}
\]

Axial capacity is governed by weak axis buckling

\[
P_n = 12 \text{ kips (tested value)}
\]

3. **Strength for Combined Compressive Axial Load and Bending**

The combined strength must be evaluated for Eq. C5.2.1-1 and C5.2.1-2 near midspan and for Eq. C5.2.1-2 at the end of the lap.

a. Check Near Mid-Span

\[
C_{mx} = 0.85, \quad L_x = 25 - 3.5 - 1 = 20.5 \text{ ft.}
\]

\[
P_{EX} = \frac{\pi^2 E I_x}{(K_x L_x)^2} \quad (\text{Eq. C5.2.1-6})
\]

\[
= \frac{\pi^2 \times 29500 \times 8.146}{(1 \times 20.5 \times 12)^2} = 39.19 \text{ kips}
\]

\[
\alpha_x = 1 - \frac{\Omega_c P}{P_{EX}} \quad (\text{Eq. C5.2.1-4})
\]

\[
= 1 - \frac{1.80 \times 5}{39.19} = 0.770
\]
The applied moment is the sum of the uplift moment plus the moment resulting from the eccentric axial load. The eccentric moment may be taken as \( P \times \frac{d}{2} \). See Fig. 11.6. If a flange brace or braces are located at the strut purlin location the rafter will not rotate, and the eccentric moment is virtually eliminated.

![Diagram of Purlin with Flange Brace](image)

**Fig. 11.6 Purlin with Flange Brace**

\[
M_x = 1.48 \text{ kip-ft., flange braces preclude rafter rotation}
\]

\[
\frac{\Omega_c P}{P_n} + \frac{\Omega_k C_{mx} M_x}{M_{mx} \alpha_x} \leq 1.0
\]

\[
\frac{1.80 \times 5}{12} + \frac{1.67 \times 0.85 \times 1.48}{5.49 \times 0.770} = 0.75 + 0.50 = 1.25 > 1.0 \text{ n.g.}
\]

A different section must be used.

b. Check at End of Lap

\[
P_{no} = A_x F_y
\]

\[
A_x = 0.435 \text{ in}^2
\]

\[
P_{no} = 0.435 \times 55 = 23.93 \text{ kips}
\]

\[
M_{mx} = 7.84 \text{ kip-ft. from Example 11.3}
\]

\[
M = 0.75 (3.77 - 0.49)
\]

\[
= 2.46 \text{ kip-ft.}
\]

\[
\frac{\Omega_c P}{P_{no}} + \frac{\Omega_k M_x}{M_{mx}} \leq 1.0
\]

\[
\frac{1.80 \times 5}{23.93} + \frac{1.67 \times 2.46}{7.84} = 0.38 + 0.52 = 0.90 < 1.0 \text{ o.k.}
\]
11.7 Joist Stability Calculation

Determine if a 24K4 spanning 40 ft. can safely support a dead load of 8 psf and a snow load of 24 psf. The joists are spaced 5’-0” apart. The standing seam roof is anchored at the building eave and must support the lateral stability load from 20 joists. The roof slope is 1/2” per ft. The joist manufacturer has supplied the following information regarding the joist top chord angles:

A = 0.718
I_y = 0.639
F_y = 50 ksi
4 - rows of top chord bridging

**Solution (LRFD):**

Determine the chord yield load:

\[ P_y = F_yA \]
\[ = (50)(0.718) \]
\[ = 35.9 \text{ kips} \]

Determine the critical axial load:

Factored load:

\[ w_n = [1.2(8) + 1.6(24)]5 = 240 \text{ lbs/ft. or 0.24 kip/ft.} \]

Critical load:

\[ M = l/8wL^2 = (1/8)(.240)(40)^2 = 48 \text{ kip-ft.} \]

For the 24 in. deep joist, axial compression in the top chord is

\[ P_{a} \equiv M/d = (48)(12)/24 = 24 \text{ kips} \]

Note that \( P_{a}/P_y = 24 / 35.9 = 0.67 \), which is less that the maximum ratio of 0.8 recommended in Section 9.4.

\[ E_t = (4E_P_{a} / P_y)(1 - P_{a} / P_y) \]  
\[ E_t = [(4)(29000)(24)/35.9][1 - 24/35.9] \]
\[ = 25706 \text{ ksi} \]

\[ L_e = \pi\sqrt{E_tI/P_{a}} \]  
\[ L_e = \pi\sqrt{(25706)(0.639)/24} \]
\[ = 82.2 \text{ in.} \]
\[ K_c \approx \frac{2.5P_e}{L_e^2} \]
\[ = \frac{(2.5)(24)}{(82.2)^2} \]
\[ = 0.0089 \]

Take \( d_0 \) equal to the bridging spacing divided by 500.

\[ d_0 = \frac{(10)(12)}{500} = 0.24 \text{ in.} \]

Conservatively use \( d = 2d_0 = 0.48 \text{ in.} \)

\[ P_b = K_c(d_0 + d) = (0.0089)(0.72) = 0.0064 \text{ kips/in.} \]
\[ = 76.8 \text{ lbs/ft.} \]

Total bracing force:

\[ P_t = P_b(.2n + .8\sqrt{n}) \]
\[ = (76.8)[(.2)(20) + (.8)\sqrt{20}] \]
\[ = 582 \text{ lbs/ft.} \]

In addition to the bracing force the standing seam roof must resist the downward force component of the snow load.

\[ P_s = (1.6)(24)(100)(\sin 2.39^\circ) \]
\[ = 160 \text{ lbs/ft.} \]

An eave member, or eave horizontal truss system must be provided to resist a load of \((582 + 160) = 742 \text{ lbs/ft.} \) Alternately the capability of the standing seam diaphragm could be examined for its ability to resist the 742 lb/ft. load.
REFERENCES


American Institute of Steel Construction, Inc. (1993), "Load and Resistance Factor Design Specifications for Structural Steel Buildings", Chicago, IL

American Iron and Steel Institute (1996), "Specification for the Design of Cold-Formed Steel Structural Members", Washington, DC

American Iron and Steel Institute (1996), Cold-Formed Steel Design Manual, Washington, DC


APPENDIX I
AISI — A Guide for Designing with Standing Seam Roof Panels

AISI
BASE TEST METHOD
FOR
PURLINS SUPPORTING
A
STANDING SEAM ROOF SYSTEM

1. Scope

1.1 The purpose of this test is to obtain the reduction factor to be used in determining the nominal flexural strength of a purlin supporting a standing seam roof system. The reduction factor reflects the ability of a particular standing seam roof system to provide lateral and rotational support to the purlins to which it is attached. This applies to discrete lateral and torsional bracing when the sheeted flange of the purlin is the compression flange, as in gravity loading cases.

1.2 This test method applies to an assembly consisting of the standing seam panel, purlin, and attachment devices used in the system being tested. The test specimen boundary conditions described in Section 6.6 apply only to standing seam roof systems for which the roof deck is positively anchored to the supporting structural system at one or more purlin or eave member lines.

1.3 Due to the many different types and construction of standing seam roof systems and their attachments, it is not practical to develop a generic method to predict the interaction of a particular standing seam roof system and supporting structure. Therefore, the amount of resisting moment which the supporting purlins can achieve can vary from the fully braced condition to the unbraced condition for a given system.

1.4 This test method provides the designer with a means of establishing a nominal flexural strength reduction factor for purlins in a simple span or continuous span, multiple purlin line, supporting a standing seam roof system, from the results of tests on a single-span, two-purlin line, sample of the system. Test procedures are given for gravity loads only. The validity of this test method has been established by a research program at Virginia Polytechnic Institute and State University and documented in References 1 through 3.

2. Applicable Documents

2.1 ASTM Standards:
   A370 - Standard Test Methods and Definitions for Mechanical Testing of Steel Products

2.2 **AISI Specification for the Design of Cold-Formed Steel Structural Members, 1996 Edition.**

3. Terminology

3.1 ASTM Definition Standards:
   E6 - Definitions of Terms Relating to Methods of Mechanical Testing.
   E380 - Practice for Use of the International System of Units (SI).

3.2 Description of terms specific to this standard:
   *fixed clip* - a hold down clip which does not allow the roof panel to move independently of the roof substructure
   *insulation* - glass fiber blanket or rigid board
lateral - a direction normal to the span of the purlins in the plane of the roof sheets
thermal block - strips of rigid insulation located directly over the purlin between clips
pan type standing seam roof - a "U" shaped panel which has vertical sides
positive moment - a moment which causes compression in the purlin flange attached to the clips and standing seam panels
rib type standing seam roof - a panel which has ribs with sloping sides and forms a trapezoidal shaped void at the side lap
sliding clip - a hold down clip which allows the roof panel to move independently of the roof substructure
standing seam roof system - a roof system in which the side laps between the roof panels are arranged in a vertical position above the roof line. The roof panel system is secured to the purlins by means of concealed hold down clips that are attached to the purlins with mechanical fasteners.

3.3 Symbols:

\[ b \] = flange width of the purlin
\[ d \] = depth of the purlin
\[ B \] = purlin spacing
\[ F_y \] = design yield strength
\[ F_{y1} \] = measured yield strength of tested purlin
\[ L \] = span of the purlins tested, center to center of the supports
\[ M_n \] = nominal flexural strength of a fully constrained beam, \( S_e F_y \)
\[ M_{n_{min}} \] = average flexural strength of the thinnest sections tested
\[ M_{n_{max}} \] = average flexural strength of the thickest sections tested
\[ M_{fr} \] = flexural strength of a tested purlin, \( S_e F_{y1} \)
\[ M_{ts} \] = failure moment for the single span purlins tested, \( w_{ts} L^2/8 \)
\[ p_d \] = weight of the specimen (force/area)
\[ p_{ts} \] = failure load (force/area) of the single span system tested
\[ P_L \] = lateral anchorage force in accordance with Section D3.2.1 of the AISI Specification
\[ R_s \] = modification factor from test, \( M_{ts}/M_{fr} \)
\[ R \] = reduction factor computed for nominal purlin properties
\[ R_{tsn} \] = mean minus one standard deviation of the modification factors of the three thinnest purlins tested
\[ R_{tsn} \] = mean minus one standard deviation of the modification factors of the three thickest purlins tested
\[ s \] = tributary width of the purlins tested
\[ S_e \] = section modulus of the effective section
\[ S_{ef} \] = section modulus of the effective section of the tested member using actual dimensions
\[ t \] = purlin thickness
\[ w_{ts} \] = failure load (force/length) of the single span purlins tested

4. Significance

4.1 This test method provides the requirements for evaluating the resisting moment under gravity loads for cold-formed C- and Z-sections used with standing seam roof systems. This procedure is referred to as the “Base Test Method”. The method is the result of extensive testing of various combinations of purlins, standing seam panels, and fastening devices. The tests were conducted over several years, benefiting from the experience
provided by technical and industry experts. This procedure utilizes the results obtained from single span tests to predict the strength of multi-span conditions.

4.2 The Base Test Method shall be permitted to be used to evaluate the gravity loading nominal flexural strength of C- and Z-sections of multi-span, multiple purlin line, standing seam systems, with or without discrete intermediate braces.

4.3 The Base Test Method is applicable to both “rib” or “pan” type standing seam roof panels with “sliding” or “fixed” type clips.

4.4 The Base Test Method shall be conducted using standing seam roof panels, clips, fasteners, insulation, thermal blocks, discrete braces, and purlins as used in the actual standing seam roof system except as noted in Section 4.5.

4.5 Tests conducted with insulation are applicable to identical systems with thinner or no insulation.

5. Apparatus

5.1 A test chamber capable of supporting a positive or negative internal pressure differential is necessary. A rectangular frame shall be constructed of any material with sufficient strength and rigidity to provide the desired pressure differential without collapse. A typical test chamber is shown on Figure 1. Other chamber orientations shall be permitted.

5.2 The length of the chamber shall be determined by the maximum length of the secondary members as required by Section 7.2. The width of the chamber shall be determined by the maximum panel length as required by Section 6.9. Allowance shall be made in the interior chamber dimensions to accommodate structural supports for the secondary members and sufficient clearance on all sides to prevent interference of the chamber wall with the test specimen as it deflects.

5.3 The height of the chamber shall be sufficient to permit assembly of the specimen and to insure adequate clearance at the maximum deflection of the specimen.

5.4 The chamber shall be sealed in a manner to prevent air leakage. All load carrying elements of the specimen or its supports shall transfer the load to the frame support; the specimen, including intermediate brace, shall not be attached to the chamber in any manner that would impede the deflection of the specimen.

5.5 The test chamber shall be sealed against air leakage by applying 6 mil (0.15 mm) maximum thickness polyethylene sheets, large enough to accommodate the system configuration and deflections. The polyethylene shall be located on the high pressure side of the panel with sufficient folds so as not to inhibit the spread of panel ribs under load. Edges of the polyethylene sheets shall be sealed against air leakage with tape or other suitable methods. Polyethylene sheets around the perimeter of the specimen shall be draped so as not to impede deflection or deformation of the specimen.

5.6 When a specimen smaller than the test chamber is tested, other panels and structure shall be installed to complete the coverage of the chamber opening. No attachment shall be made between the test specimen and this supplemental coverage.

5.7 An air pump is necessary to create the pressure differential in the chamber. The pump shall be of sufficient capacity to reach the expected test values required by the applicable specifications.
5.8 The type of air pump being used will determine the method of control. This control shall be able to regulate the pressure differential in the chamber to ± 1 psf (0.05 kPa). This can be accomplished by (a) a variable speed motor on the pump, (b) valving on the pump, or (c) variable size orifices on the chamber. It shall be permitted to use multiple pumps where very large chambers are being used. One pump connection to the chamber is satisfactory.

5.9 A minimum of two pressure differential measuring devices shall be monitored throughout the duration of the test. These devices shall be capable of measuring the pressure differential to ± 1 psf (0.05 kPa).

6. Test Specimens

6.1 Test purlins shall be supported at each end by a steel beam. The beams shall be simply supported and one of the frame end beams shall be sufficiently free to translate laterally to relieve any longitudinal catenary forces in the specimen. Purlins shall be connected to the supporting beams as recommended in the field erection drawings. Figure 1 shows the directional axes that are referred to in this test procedure.

6.2 Panel supporting clips, fasteners, and panels shall be installed as recommended in the field erection drawings.
6.3 Means of providing restraint of purlins at the support shall be as required for use in actual field application, and shall be installed as recommended on the field erection drawings.

6.4 The purlins shall be arranged either with their flanges facing in the same direction or with their flanges opposed. If the test is performed with the purlin flanges opposed, and they are field installed with their flanges facing in the same direction, a diaphragm test must be conducted in accordance with Section 8.7.

6.5 For tests including intermediate discrete point braces, the braces used in the test shall be installed in such a manner so as not to impede the vertical deflection of the specimen.

6.6 A 1 in. x 1 in. (25 mm x 25 mm) continuous angle with a maximum thickness of 1/8 in. (3 mm) or a member of compatible stiffness shall be attached to the underside at each end of the panels to prevent separation of the panels at the ends of the seam. Fasteners shall be placed on both sides of each major rib. If the specimen is arranged with the purlin flanges facing in the same direction, a 3 in. x 3 in. (76 mm x 76 mm) continuous angle with a maximum thickness of 1/4 in. (6 mm) or a member of compatible stiffness shall be permitted to be substituted for the 1 in. x 1 in. (25 mm x 25 mm) angle at the end of the panel, corresponding to the eave of the building using the standard panel to eave fastening system (See Figure 1).

6.7 All transverse panel ends shall be left free to displace vertically under load. When the 3 in. x 3 in. (76 mm x 76 mm) eave angle is used when the purlin flanges face in the same direction, it shall be permitted to be restrained against horizontal deflection at its ends as shown in Figure 1, providing the vertical deflection is left unrestrained.

6.8 Panel joints shall not be taped and no tape shall be used to restrict panel movement.

6.9 Panel length to be used in the test shall be, as a minimum, that length which provides full engagement of the panel to purlin clip and attachment of the 1 in. x 1 in. (25 mm x 25 mm) angle at the panel ends; but a length not greater than that required to achieve zero slope of the panel at the purlin support.

6.10 The spacing of purlins being tested shall not exceed the spacing typically used with the roof system. Results from this test shall be permitted to be used in designing purlins of the same profile that are spaced closer together than the spacing used in the tests.

7 Test Procedure

7.1 A test series shall be conducted for each purlin profile, specified steel grade, and each panel system. Any variation in the characteristics or dimensions of panel or clip constitute a change in panel system. The thickness of insulation used in the test is discussed in Section 4.5. Any change in purlin shape or dimension other than thickness constitutes a change in profile. However, the lip dimension shall be permitted to vary with section thickness consistent with the member design and not constitute a change in profile.

7.2 No fewer than six tests shall be run for each combination of purlin profile and panel system. Three tests shall be conducted with the thinnest purlin of the profile and three tests shall be conducted with the thickest purlin of the profile. All tests shall be conducted using the same purlin span which shall be the same or greater than the span used in actual field conditions.
7.3 The physical and material properties shall be determined in accordance with ASTM A370 using coupons taken from the web area of the failed purlin. Coupons shall not be taken from areas where cold-working stresses could affect the results.

7.4 A pressure differential load shall be applied to the system to produce a positive moment in the system. A positive moment is defined as one which causes compression in the purlin flange attached to the clips and standing seam panels.

7.5 An initial load equal to 5 psf (0.25 kPa) differential pressure in the direction of the test load shall be applied and removed to set the zero readings before actual system loading begins.

7.6 The system shall be loaded to failure and the mode of failure noted. Failure is the point at which the specimen will accept no further loading. The pressure differential at which the system fails shall be recorded as the failure load of the specimen. When the test must be stopped due to a flexural failure of the panel or web crippling of the purlin, it shall be permitted to exclude the test from the test program.

7.7 Vertical deflection measurements shall be taken at the mid-span of both purlins. The deck deflection in the horizontal direction shall be measured at the seam joint nearest the center of the test specimen.

7.8 Deflections and pressures shall be recorded at pressure intervals equal to a maximum of 20 percent of the anticipated failure load.

8. Test Evaluation

8.1 The single span failure load is obtained from the Base Test where a uniform load is applied until failure occurs.

For Z-purlins tested with flanges facing the same direction and with the top flanges of the purlins not restrained by anchorage to a point external to the panel/purlin system:

\[ w_{ts} = (p_{ts} + p_d)s + 2P_L \left( \frac{d}{B} \right) \]

where,

\[ P_L = 0.041 \left( \frac{b^{1.5}}{d^{0.90} t^{0.60}} \right) (p_{ts} + p_d)s \]

For Z-purlins tested with flanges opposed and for C-sections:

\[ w_{ts} = (p_{ts} + p_d)s \]

The expression \( 2P_L(d/B) \) takes into account the effect of the overturning moment on the system due to the anchorage forces, as defined in Section D3.2.1 of the AISI Specification, applied at the top flange of the purlin by the panel and resisted at the bottom flange of the purlin at the support. The expression \( 2P_L(d/B) \) is to be applied only to Z-sections when the purlin flanges are facing in the same direction, but shall not be included in those systems where discrete point braces are used when the braces are restrained from lateral movement.
8.2 From the single span failure load, \( w_{u} \), the maximum single span failure moment \( M_{u} \) is calculated as:

\[
M_{u} = w_{u} L^{2} / 8
\]

8.3 The single span base test moment is the maximum positive moment the system can resist with the purlin size used in the test. The maximum allowable positive moment in the mid-span region of a roof system purlin, simple span or continuous, is limited by the results of this test.

8.4 Using Section C3.1.1(a) of the AISI Specification, the flexural strength of each tested purlin, \( M_{ut} \), of a fully constrained beam is calculated as:

\[
M_{ut} = S_{et} \times F_{yt}
\]

where \( S_{et} \) is the section modulus of the effective section calculated using the measured cross-sectional dimensions and \( F_{yt} \) is the measured yield strength obtained in accordance with Section 7.3.

8.5 The modification factor, \( R_{t} \), is calculated for each purlin tested as:

\[
R_{t} = M_{ut} / M_{nt}
\]

8.6 For purlins of the same profile, specified steel grade and panel system as tested, the reduction factor shall be determined from the following equation:

\[
R = \left( \frac{R_{t_{min}} - R_{t_{max}}}{M_{nt_{max}} - M_{nt_{min}}} \right) \left( M_{n} - \overline{M_{nt_{min}}} \right) + R_{t_{min}} \leq 1.0
\]

where,

- \( R_{t_{min}} \) = mean minus one standard deviation of the modification factors of the three thinnest purlins tested, calculated in accordance with Section 8.5. This value may be greater than 1.0.
- \( R_{t_{max}} \) = mean minus one standard deviation of the modification factors of the three thickest purlins tested, calculated in accordance with Section 8.5. This value may be greater than 1.0.
- \( M_{n} \) = nominal flexural strength of section for which \( R \) is being evaluated \((S_{et} F_{yt})\).
- \( \overline{M_{nt_{min}}} \) = average flexural strength of the thinnest section tested, calculated in accordance with Section 8.4.
- \( \overline{M_{nt_{max}}} \) = average flexural strength of the thickest section tested, calculated in accordance with Section 8.4.

8.7 If the test is performed with the purlins opposed or with an eave member at one or more edges, the diaphragm strength and stiffness of the panel system must be tested unless the purlins are also opposed in actual field usage. The anchorage forces for the system braced in the manner tested shall be calculated in accordance with Section D3.2.1 of the AISI Specification. The diaphragm strength of the panel system must be equal to or greater than the calculated brace force at the failure load of the purlin. The stiffness of the diaphragm must be such that the deflection of the diaphragm is equal to or less than...
the purlin span divided by 360 when subjected to the calculated brace force at the failure load of the purlin.

9. Test Report

9.1 Documentation - The report shall include who performed the test and a brief description of the system being tested.

9.2 The documentation shall include test details with a drawing showing the test fixture and indicating the components and their locations. A written description of the test setup detailing the basic concept, loadings, measurements, and assembly shall be included.

9.3 The report shall include a drawing showing the actual geometry of all specimens including material specifications and test results defining the actual material properties - material thickness, yield strength, tensile strength, and percent elongation.

9.4 The report shall include the test designation, loading increments, displacements, mode of failure, failure load, and specimen included for each test.

9.5 The report shall include a description summarizing the test program results to include specimen type, span, failure moments for the test series, and the supporting calculations.
References


APPENDIX II
WRITERS' BASE TEST METHOD FOR STRUT PURLINS SUPPORTING A STANDING SEAM ROOF SYSTEM

1. Scope

1.1 The purpose of this test is to determine the ability of a particular standing seam roof system to provide lateral and rotational support to the purlins to which it is attached. This applies to direct lateral and torsional bracing for purlins subjected to axial loading.

1.2 This test method applies to an assembly consisting of the standing seam panel, purlins, and attachment devices used in the system being tested. It is not a test for the capacity of the individual components of the assembly.

1.3 Due to the wide variation in types and construction of standing seam roof systems and their attachments, it is not practical to develop a generic method to predict the interaction of a particular standing seam roof system and supporting structure. Therefore, the amount of axial load which the supporting purlins can achieve can vary from the partially braced condition to the unbraced condition for a given system.

1.4 This test method provides the designer with a means of establishing a nominal axial load for simple span or continuous span purlins supporting a standing seam roof system from the results of tests on a single span, two purlin line, sample of the system.

2. Applicable Documents

2.1 ASTM Standards
   A370 - Tensile Test Method for Steel Sheets

2.2 AISI Specification for the Design of Cold-Formed Steel Structural Members, 1996 Edition.

3. Terminology

3.1 ASTM Definition Standards:
   E6 - Definitions of Terms Relating to Methods of Mechanical Testing.
   E380 - Standard for Metric Practice.

3.2 Description of terms specific to this test method:
   fixed clip - a hold down clip which does not allow the roof panel to move independently of the roof substructure
   insulation - glass fiber blanket or rigid board
   lateral - a direction normal to the span of the purlins in the plane of the roof sheets
   thermal block - strips of rigid insulation located directly over the purlin between clips
   pan type standing seam roof - a "U" shaped panel which has vertical sides
   positive moment - a moment which causes compression in the purlin flange attached to the clips and standing seam panels
   rib type standing seam roof - a panel which has ribs with sloping sides and forms a trapezoidal shaped void at the side lap
sliding clip - a hold down clip which allows the roof panel to move independently of the roof substructure

standing seam roof system - a roof system in which the side laps between the roof panels are arranged in a vertical position above the roof line. The roof panel system is secured to the purlins by means of concealed hold down clips that are attached to the purlins with mechanical fasteners.

4. Significance

4.1 This test method provides the requirements for evaluating the nominal axial load capacity for cold-formed C and Z-sections used with standing seam roof systems. This procedure is referred to as the "Base Test Method for Strut Purlins". This procedure utilizes the results obtained from single span tests to predict the capacity of single span and multi-span conditions.

4.2 The test method may be used to evaluate the axial loading capacity for C and Z-sections of simple span or multi-span standing seam systems, with or without discrete intermediate braces.

4.3 The test method is applicable to both "rib" or "pan" type standing seam roof panels with "sliding" or "fixed" type clips.

4.4 The test method must be conducted using standing seam roof panels, clips, fasteners, insulation, thermal blocks, discrete braces, and purlins as used in the actual standing seam roof system except as noted in Section 4.5.

4.5 Tests conducted using insulation are applicable to identical systems with thinner or no insulation.

5. Apparatus

5.1 Test fixtures shall be capable of delivering a concentric axial load to the test specimen. The fixtures may be constructed of any material with sufficient strength and rigidity to apply the concentrated concentric loading.

5.2 The fixtures shall provide for the testing of C and Z-sections of a minimum length of 20 feet.

5.3 A typical test arrangement is shown in Figure 1.

6. Test Specimens

6.1 A single span test arrangement shall be constructed as specified in Section 5. This section describes how the test specimens shall be assembled.

6.2 Test purlins shall be supported at each end of the test frame. The support beams shall be free to translate longitudinally at one end or both ends. Purlins shall be connected to the supporting beams so that the applied load is delivered to the purlins concentrically.

6.3 Panels supporting clips, fasteners, and panels shall be installed as recommended in the building manufacturers building erection drawings.

6.4 Purlins shall be arranged with their flanges facing in opposed directions.
6.5 For tests including intermediate discrete point braces the braces used in the test shall be installed in a manner as not to impede the longitudinal or vertical deflection of the specimen.

6.6 A one inch by one inch (25.4 mm by 25.4 mm) continuous angle with a maximum thickness of 1/8 in. (3.18 mm) may be attached to each end of the panels to prevent separation of the panels at the ends of the seam. Fasteners shall be placed on both sides of each major rib.

6.7 All transverse panel ends shall be left free to displace longitudinally and vertically under load.

6.8 Panel length to be used in the test shall be, as a minimum, that length which provides full engagement of the panel to purlin clip and attachment of the 1 in. by 1 in. (25.4 mm by 25.4 mm) angle at the panel ends.

6.9 The spacing of purlins being tested shall be equal to or greater than the nominal purlin spacing used in actual field conditions.

6.10 Purlins must be placed in the test fixture so that as load is applied any resulting PΔ moments cause compressive stresses in the unsupported flanges of the test specimens. This may require an eccentricity be used in the initial placement of the test specimens in the fixture, or that the decking be attached so that the test specimens produce PΔ moments which cause compressive stresses in the unsupported flanges of the test specimens.

7. Test Procedure

7.1 A test series must be run for each purlin profile and each panel system. Any variation in the characteristics or dimensions of the panel or clip constitutes a change in the panel system. Any change in purlin shape or dimension other than thickness or lip dimension constitutes a change in profile. It is appropriate to test the thickest purlin of the profile being tested.

7.2 No fewer than two tests shall be conducted for each combination of purlin profile and panel system. A third test shall be conducted if the average of the previous two tests exceeds 10% of lower test result. The average value obtained from the lowest readings of two tests shall be used in the evaluation of the nominal axial load capacity.

7.3 The physical and material properties shall be determined in accordance with ASTM A370 using coupons taken from the web area of the failed purlin. Care should be taken to avoid areas where cold-working stresses could affect the results.

7.4 An initial load equal to 10% of the anticipated ultimate concentrated load shall be applied and removed to set to zero readings before actual system loading begins.

7.5 The system shall be loaded to failure and the mode of failure noted. Failure is the point at which the specimen will accept no further loading.

7.6 Vertical deflections shall be taken at the mid-span of both purlins.

7.7 Deflection shall be recorded at load intervals equal to a maximum of 20% of the anticipated failure load.
8.0  Test Evaluation

8.1  The failure load is obtained from the base tests by averaging the results of the two tests conducted.

8.2  The weak axis nominal load capacity for any purlin of the type tested can be calculated based on the smaller stress value obtained from the test or the stress obtained from AISI Specification Section C4.4, Equation C4.4-1 for any section with the same depth and flange width.

9.0  Test Report

9.1  Documentation - The report shall include who performed the test and a brief description of the system being tested.

9.2  The documentation shall include test details with a drawing showing the test fixture and indicating the components and their locations. A written description of the test setup detailing the basic concept, loadings, measurements, and assembly shall be included.

9.3  The report shall include a drawing showing the actual geometry of all specimens including material specifications and test results defining the actual material properties - material thickness, yield strength, tensile strength, and percent elongation.

9.4  The report shall include the test designation, loading increments, displacements, mode of failure, failure load, and specimen included for each test.

9.5  The report shall include a description summarizing the test program results to include specimen type, span failure moments for the test series, and the calculations of the nominal moment.
Fig. 1 Test Arrangement