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Capacity Prediction of Open-Web Steel Joists Partially Braced by a Standing Seam Roof

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Abstract

A new strength prediction approach is presented for open web joists partially braced by a standing seam roof. The approach employs the existing AISC column curve to calculate top chord flexural buckling capacity using the critical elastic buckling load including standing seam roof bracing stiffness. Recently derived buckling load equations are presented that account for lateral stiffness provided by the roof and the parabolically varying axial load from a uniform vertical pressure along the span. A new hybrid experimental-computational protocol is introduced for approximating standing seam roof lateral stiffness for systems without and with intermediate bridging. The strength prediction approach is demonstrated to be accurate for a small set of experiments, however a larger scale validation effort is still needed.

1. Introduction

Open-web steel joists are a staple of modern metal building construction. These joists typically span over 40 ft. between primary portal frames, supporting a roof skin that serves as the barrier to rain, snow, and wind. A common type of metal building roofing system supported by open-web joists is the standing seam roof (Fig. 1). Metal clips are through-fastened along the joist top chord and then the two edges of a metal roof panel are plastically folded with a special seaming machine around the clips, forming a watertight seal.

The standing seam clips are designed with a sliding mechanism to accommodate roof elongation and contraction from changes in temperature while still providing vertical support to the roof. Because the clips are through-fastened to the top chord, they can provide some lateral bracing if the roof panels are properly anchored to the eave and ridge. The roof panels can also envelop or "hug" the top chord under gravity loads, providing additional lateral restraint. A common question asked by joist manufacturers and metal building engineers is "how much top chord lateral support is provided by a standing seam roof and can this lateral bracing be counted on in the flexural design of a joist?"

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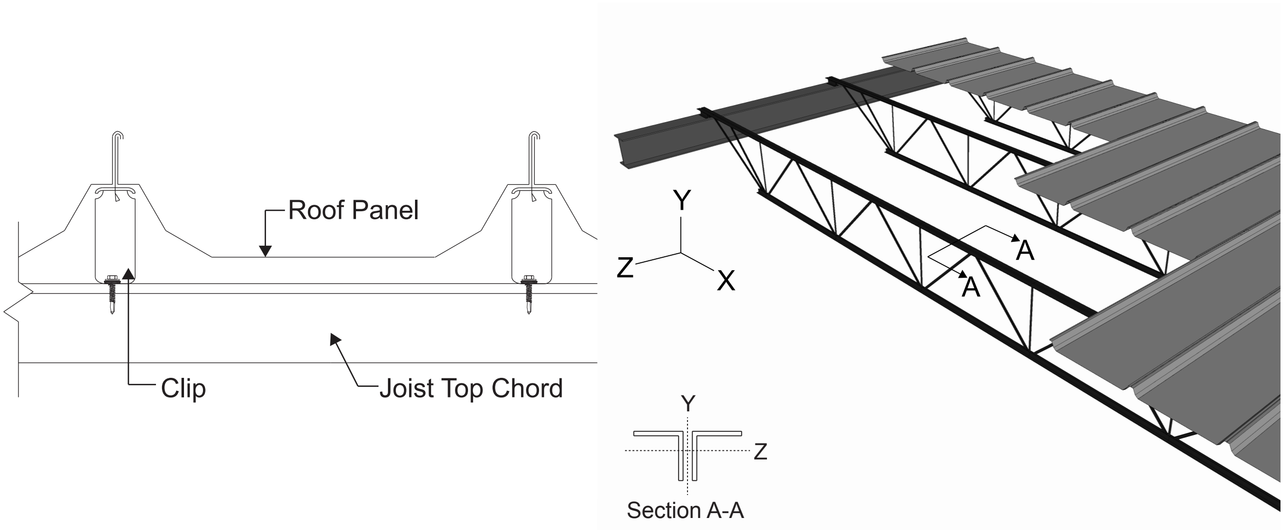


Figure 1: Standing seam roof supported by an open-web joist

There is clear experimental evidence that a standing seam roof can provide partial lateral restraint to a joist top chord. Holland and Murray (1983) evaluated the influence of standing seam clip type, top chord size, and bridging location on joist capacity using a vacuum chamber. Similar tests by Sherman and Fisher (1997) demonstrated that joist capacity decreased with increasing clip height because as clip height increases the lateral stiffness of the clip decreased and the offset of the roof from the top chord reduced the “hugging” effect.

Sherman and Fisher emphasize in their paper that the standing seam panel must be soundly connected to stiff eave and ridge members to ensure that the lateral bracing forces have a load path to the primary framing. They measured these bracing forces in their tests on a two-joist system and used the experimental results to validate engineering expressions for predicting required roof panel capacity that were motivated by Winter (1958). Sherman and Fisher point out that for multiple joists the required bracing force provided by the roof is cumulative and that for sloped roofs the gravity load in the direction of the bracing forces should also be considered in the total force demand.

One of the only existing analytical capacity prediction methods for joists braced by a standing seam roof was developed by Hodge (1986), a master’s student advised by Professor Theodore Galambos. Hodge and Galambos treated the joist top chord as a column with an initial geometric imperfection, discrete springs at the clip locations (hugging was not considered), discrete supports at the bridging lines, and a parabolically varying axial load. Second order elastic-plastic analysis of the column (joist top chord) was conducted by solving simultaneous slope deflection equations describing moment equilibrium at nodes along the column, where the node locations were at each clip (spring) location, bridging line, and end support. Hodge compared his capacity predictions to the Holland and Murray (1983) tests, however the validation was deemed unsatisfactory because of the lack of sweep imperfection measurements in the experiments. Also, Hodge mentions that the Fortran program used to solve the second-order analysis was quite slow.

This paper builds on Hodge and Galambos's analytical model to provide a general strength prediction method for open-web steel joists braced by a standing seam roof. The prediction method utilized classical stability solutions, modern structural analysis tools, and existing code equations. Specifically, the existing AISC column curve is used to predict top chord capacity using closed formed equations for the critical elastic buckling load of the top chord including roof stiffness and bridging. A protocol for approximating the roof stiffness is outlined that uses vacuum chamber test data and second order elastic analysis in a structural analysis program (e.g., MASTAN2, SAP2000, RISA2D). The strength prediction method is verified with a small set of recent experiments. More validation is needed though and the authors are hopeful that others will conduct similar tests to evaluate the accuracy of the following strength prediction approach.

2. Strength Prediction Method

The proposed strength prediction method is for a top chord lateral flexural buckling limit state of an open-web joist partially restrained by a standing seam roof loaded with a uniformed distributed gravity load. The joist top chord is assumed to behave as a column experiencing flexural buckling deformation under a parabolically varying axial load without (Fig. 2a) or with discrete supports (Fig. 2b) at the bridging lines and at the ends of the member. The top chord is a singly symmetric member if the two angles are assumed to act together (they are typically welded together every 24 in.) and therefore flexural-torsional buckling comprises two of the three buckling modes when solving the classical cubic buckling equation (Chajes 1974). However, in this prediction method, only flexural buckling is considered because the diagonal and vertical web stems in combination with the roof are assumed to suppress torsional deformation.

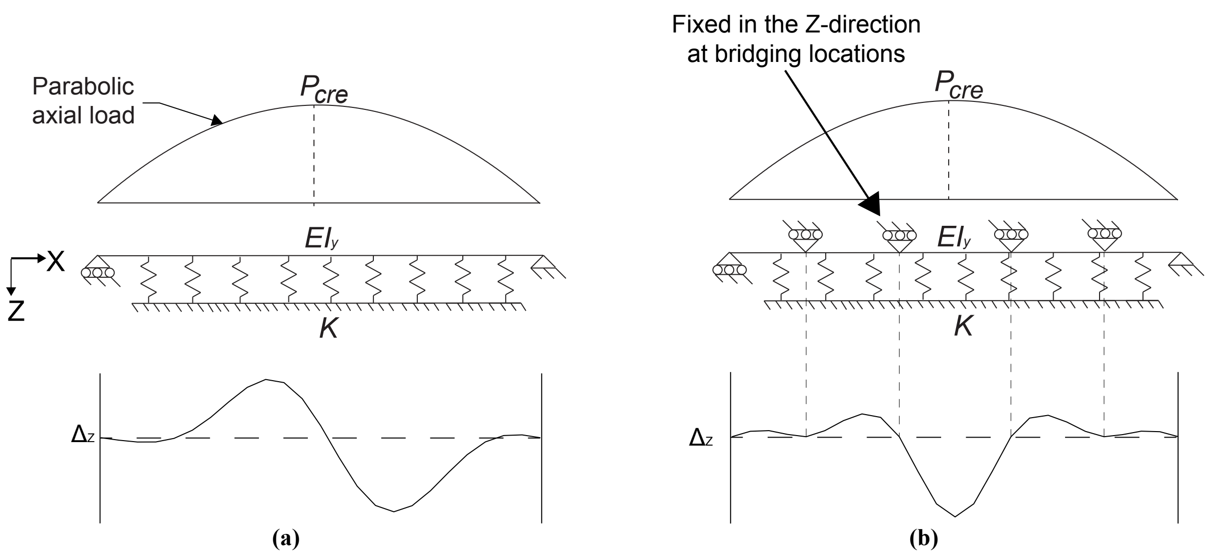


Figure 2: Top chord boundary conditions, loading, and lateral support (a) without and (b) with bridging lines

A uniformly distributed lateral spring with magnitude K (force/length/length) simulates a smeared effect of clip stiffness and hugging along the top chord. This is different from the Hodge (1986) treatment where only the clip stiffness at discrete locations was considered in the prediction.

For an open-web joist partially braced by a standing seam roof and failing by lateral flexural buckling of the top chord (Fig. 2), the ultimate capacity, P_n, is approximated with the AISC column curve equations, reformatted such that the global buckling slenderness, kL/r, is represented instead as $\lambda_c=(P_y/P_{cre})^{0.5}$

$$P_n = \left(0.658^{\lambda_c^2}\right)P_y \text{ for } \lambda_c \leq 1.5$$

$$P_n = \left(\frac{0.877}{\lambda_c^2}\right)P_y \text{ for } \lambda_c > 1.5 \qquad (1)$$

where $P_y=AF_y$, A is the gross cross-sectional area of the top chord, F_y is the top chord steel yield stress, and P_{cre} is the critical elastic buckling load of the joist top chord. The column curve represented in Eq. (1) is used in both hot-rolled steel (AISC) and cold-formed steel (AISI) codes, and therefore this proposed strength prediction method is applicable to top chord angles made from either material. The calculation of P_{cre} can be performed with an elastic eigen-buckling analysis in a structural analysis program, for example, MASTAN2 (MASTAN2/version 3.3) or SAP2000 (SAP2000/Advanced version 14.0.0) with similar models shown in Fig. 2.

Approximate hand methods are also available to calculate P_{cre}. A closed form solution for the case of a joist without bridging was recently derived from numerical studies (Cronin 2012)

$$\frac{P_{cre}L^2}{EI_Y} = 0.1702\left(\frac{KL^4}{EI_Y}\right) + 20.00 \quad \text{for} \quad \frac{KL^4}{EI_Y} \leq 311$$

$$\frac{P_{cre}L^2}{EI_Y} = 0.0302\left(\frac{KL^4}{EI_Y}\right) + 63.55 \quad \text{for} \quad 311 < \frac{KL^4}{EI_Y} \leq 3874$$

$$\frac{P_{cre}L^2}{EI_Y} = 0.0119\left(\frac{KL^4}{EI_Y}\right) + 134.47 \quad \text{for} \quad 3874 < \frac{KL^4}{EI_Y} \leq 16960$$

$$\frac{P_{cre}L^2}{EI_Y} = 0.00626\left(\frac{KL^4}{EI_Y}\right) + 230.03 \quad \text{for} \quad 16960 < \frac{KL^4}{EI_Y} \leq 50000$$

$$\frac{P_{cre}L^2}{EI_Y} = 0.00427\left(\frac{KL^4}{EI_Y}\right) + 329.67 \quad \text{for} \quad 50000 < \frac{KL^4}{EI_Y} \leq 80000$$

$$\frac{P_{cre}L^2}{EI_Y} = 0.00295\left(\frac{KL^4}{EI_Y}\right) + 435.00 \quad \text{for} \quad 80000 < \frac{KL^4}{EI_Y} \leq 200000$$

$$\frac{P_{cre}L^2}{EI_Y} = 0.00165\left(\frac{KL^4}{EI_Y}\right) + 685.80 \quad \text{for} \quad 200000 < \frac{KL^4}{EI_Y} \leq 700000 \qquad (2)$$

where L is the joist span. Equation (2) is a linear fit to the slightly nonlinear exact solution shown in Fig. 3.

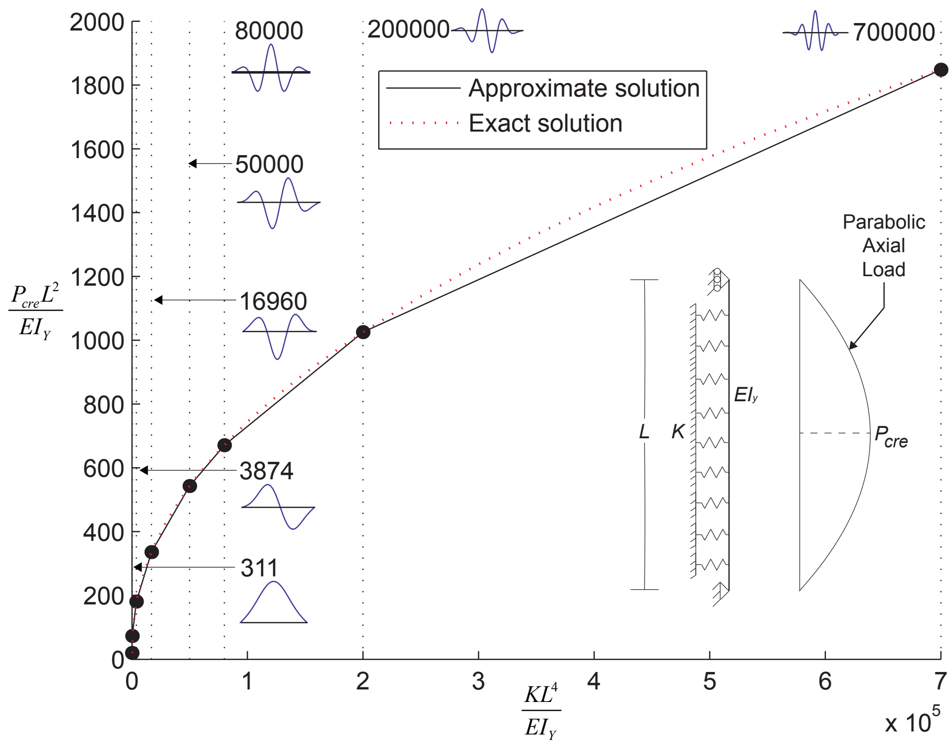


Figure 3: Flexural column buckling on an elastic foundation with a parabolically varying axial load

When intermediate bridging lines brace the top chord, then P_{cre} can be approximated by assuming the top chord acts as a pinned-pinned column on an elastic foundation between bridging lines, where the length of the column, L, is the distance between the bridging lines nearest midspan

$$\frac{P_{cre}L^2}{EI_Y} = 9.87 + \frac{KL^3}{9.87EI_Y} \quad \text{for} \quad \frac{KL^3}{EI_Y} \leq 389$$

$$\frac{P_{cre}L^2}{EI_Y} = 39.48 + \frac{KL^3}{39.48EI_Y} \quad \text{for} \quad 389 < \frac{KL^3}{EI_Y} \leq 3507$$

$$\frac{P_{cre}L^2}{EI_Y} = 88.83 + \frac{KL^3}{88.83EI_Y} \quad \text{for} \quad 3507 < \frac{KL^3}{EI_Y} \leq 14030$$

$$\frac{P_{cre}L^2}{EI_Y} = 157.9 + \frac{KL^3}{157.9EI_Y} \quad \text{for} \quad 14030 < \frac{KL^3}{EI_Y} \leq 38960$$

$$\frac{P_{cre}L^2}{EI_Y} = 246.7 + \frac{KL^3}{246.7EI_Y} \quad \text{for} \quad 39860 < \frac{KL^3}{\pi^4 EI_Y} \leq 87670 \qquad (3)$$

Equation (3) represents the classical solution for a column with a constant axial load on an elastic foundation (Chen and Lui 1987) shown in Fig. 4. The axial force in the joist will be varying with the moment between bridging lines, however for typical bridging spacings it is reasonable and conservative to assume that the axial load is constant for this case.

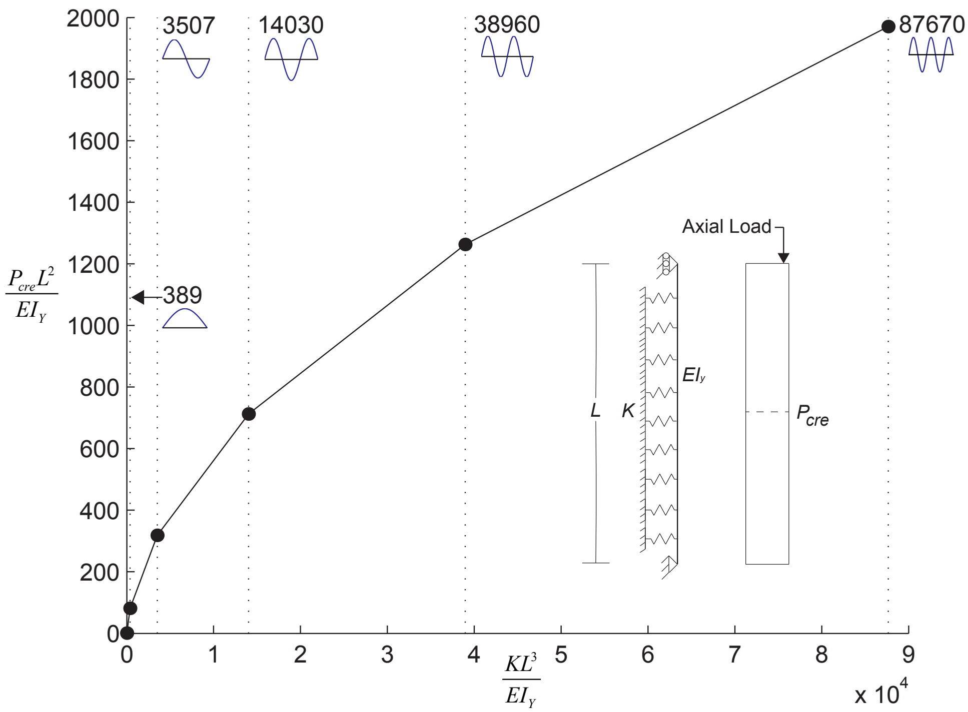


Figure 4: Flexural column buckling on an elastic foundation with a constant axial load

Once P_{cre} is used in Eq. (1) to calculate P_n for the top chord, joist capacity can be converted to a distributed load, w_n, with units of force per length

$$w_n = \frac{8P_n d}{L^2} \tag{4}$$

where d is the distance between centroids of the top and bottom joist chords.

There are several inherent assumptions in this strength prediction approach that may or may not be valid depending upon the joist type, standing seam roof details, and building boundary conditions. Catenary tension in the top chord is ignored which is most likely a conservative assumption. Sudden lateral slip of the standing seam roof clips when the roof is under load is neglected. Downward local bending of the top chord caused by concentrated forces at the clip locations is ignored. A P-M interaction equation could be employed if there is concern that the web stem spacing is too large. The standing seam roof is assumed to be able to provide the distributed spring stiffness K and to have the capacity to carry the associated bracing forces. A procedure for approximating K and the bracing force demand with a standard vacuum chamber test and computer analysis is presented in the next section.

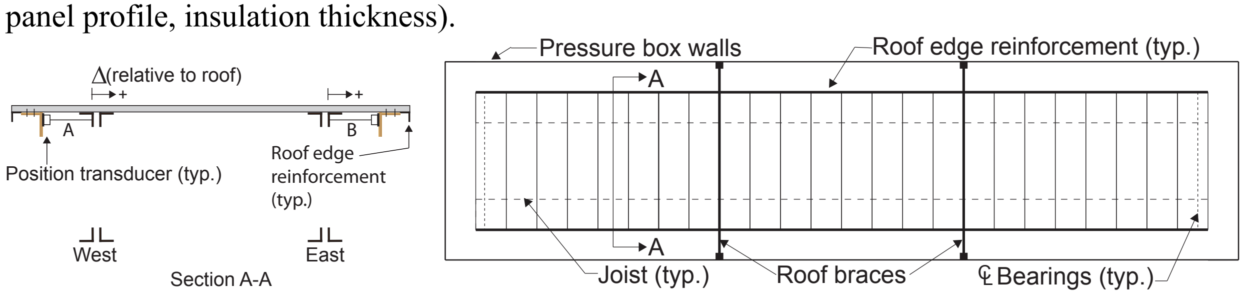
3. Roof Stiffness

The lateral stiffness provided by the standing seam roof to the top chord is influenced by many variables including clip type, clip height, the presence of insulation between the panel and the joist, panel profile and gage. The failure pressure also plays an important role because for roofs with a lower capacity joist, envelopment of the top chord will be minimal, while for a stronger joist top chord hugging may be the dominant contributor to lateral stiffness.

The procedure presented in the following paragraphs for approximating K employs experimental data from a vacuum box test. The procedure is implemented by measuring the lateral displacement of the joist top chord relative to the roof at multiple points along the span, and then a structural analysis program is used to solve for K that results in the best match between the displaced shape in the structural analysis and the measured displaced shape of the joist.

The first step is to start with a vacuum box test and measure the displacement of the joists relative to the standing seam roof near failure as shown in Fig. 5a. To be consistent with the assumptions in the prediction method that the roof eave and rafter boundary conditions are rigid, the roof edges should be reinforced with through-fastened angles and also braced at intermediate points along the span, for example, as shown in Fig. 5b. The initial sweep imperfection shape and magnitudes along the span should be measured and recorded. It is recommended that three tests to failure be performed to ensure that statistical variations can be averaged in the final determination of K.

Once the experiments are complete, a computer model is constructed where the top chord is simulated as a pinned-pinned column with distributed springs along its length (similar to Fig. 2). The measured imperfection shape and magnitudes are imposed on the initial geometry of the model (so if three tests were performed then there would be three models), and a parabolically varying axial load is applied. A second order elastic analysis of the top chord including the imperfections and measured joist top chord section properties can then be run multiple times while varying the spring stiffness K. The roof stiffness K that minimizes the difference between the predicted and measured displacements from the experiment is averaged between the three tests and then used in Eq. (2), Eq. (3) or in an eigen-buckling analysis to approximate P_{cre}. The total bracing force demand can also be approximated by adding up all the spring forces in the model. It is essential when employing the joist strength prediction method described herein that K be experimentally determined for each combination of roof system variables (e.g., clip height, panel profile, insulation thickness).



(a) (b)

Figure 5: Vacuum box experiment details: (a) measurement of joist top chord lateral displacement relative to roof, and (b) proposed roof bracing details

4. Test-to-Predicted Comparison

The prediction method is compared to tested values from a recent experimental program for joists with and without bridging (Fehr 2012). The roof stiffness K was solved iteratively with elastic second order analysis and then used to predict joist capacity for roof systems without and with bridging. The test results are compared to predictions in Table 1, where the top chord critical elastic buckling load, P_{cre}, was calculated with Eq. (2) for the tests without bridging and with eigen-buckling frame analysis in MASTAN2 for the cases with bridging.

The test-to-predicted mean and COV for the results in Table 1 are 1.08 and 0.12 respectively, demonstrating the prediction method is viable for the conditions considered. One caveat is that the favorable statistics may be biased because K was derived from the same tests used to compare with the predictions. Nonetheless, the approximated roof stiffness K increases on average with decreasing clip height and increasing failure pressure, both trends that are consistent with engineering intuition and with results from Sherman and Fisher (1996).

Joist capacity for the tests with bridging are also calculated using the simplified method for P_{cre} in Eq. (3). The capacity P_n in Table 2 is more conservative than using an eigen-buckling analysis in MASTAN2 (compare to Tests 1-4 in Table 1) however the strength prediction approach is still viable for the cases considered. The conservatism comes from the assumption that the top chord is a column with warping free boundary conditions at the bridging locations.

Table 1: As-tested and predicted roof stiffness and joist capacity

Test	Joist Type	Clip Height (in.)	Insulation Thickness (in.)	Thermal Blocks	Bridging Locations, x= (ft.)	Failure Pressure (psf)	P_{test} (kips)	F_y (ksi)	K (kips/in./in.)	P_y (kips)	P_{cre} (kips)	Number of predicted half-waves	λ_c	P_n (kips)	P_{test}/P_n
1	24K4	3-5/16	2	Yes	9.6, 19.2, 28.8, 38.4	24.2	20.0	58.0	0.0015	47.6	27.2	5	1.32	22.9	0.87
2	24K4	3-5/16	2	Yes	9.6, 19.2, 28.8, 38.5	30.4	25.1	58.7	0.0016	48.5	27.9	5	1.32	23.4	1.07
3	24K4	3-5/16	2	Yes	19.2, 28.8	29.7	24.6	58.5	0.0032	47.1	26.7	4	1.33	22.5	1.09
4	24K4	3-5/16	2	Yes	19.2, 28.9	30.8	25.1	57.8	0.0065	45.7	35.2	3	1.14	26.5	0.95
5	24K4	3-5/16	2	Yes	N/A	30.0	24.9	57.5	0.0034	45.1	26.5	2	1.30	22.1	1.12
6	24K4	3-5/16	2	Yes	N/A	29.2	24.2	58.5	0.0025	46.6	23.1	3	1.42	20.2	1.20
7	24K4	3-5/16	6	Yes	N/A	29.0	23.9	58.5	0.0026	46.9	23.2	3	1.42	20.4	1.18
8	24K4	3-5/16	6	Yes	N/A	32.4	26.8	59.4	0.0032	48.9	26.0	4	1.37	22.8	1.17
9	24K4	3-5/16	2	No	N/A	27.3	22.7	59.3	0.0063	48.6	35.6	2	1.17	27.5	0.83
10	24K4	3-5/16	2	No	N/A	26.8	22.0	59.2	0.0032	48.0	25.8	3	1.36	22.7	0.97
11	24K4	2-1/4	2	No	N/A	24.4	19.9	58.7	0.0025	47.7	23.0	2	1.44	20.1	0.99
12	24K4	2-1/4	2	No	N/A	38.0	31.0	59.2	0.0058	42.8	31.2	5	1.17	24.1	1.29
13	24K4	3-3/4	2	Yes	N/A	23.5	19.5	57.6	0.0027	45.9	24.0	2	1.38	21.0	0.93
14	24K4	3-3/4	2	Yes	N/A	23.8	19.8	57.3	0.0019	45.5	20.3	3	1.50	17.8	1.11
15	24K8	3-5/16	2	Yes	N/A	58.2	49.0	56.9	0.0083	70.9	62.3	3	1.07	44.0	1.11
16	24K8	3-5/16	2	Yes	N/A	52.4	43.5	58.8	0.0040	72.8	44.2	3	1.28	36.5	1.19
17	24K12	3-5/16	2	Yes	N/A	76.1	64.1	56.2	0.0062	105.9	69.4	2	1.24	55.9	1.15
18	24K12	3-5/16	2	Yes	N/A	79.0	67.0	58.2	0.0073	108.5	74.3	3	1.21	58.9	1.14

Table 2: As-tested and predicted 24K4 joist capacity including bridging, P_{cre} calculated with Eq. (3)

Test	Joist Type	Clip Height (in.)	Insulation Thickness (in.)	Thermal Blocks	Bridging Locations, x= (ft.)	Failure Pressure (psf)	P_{test} (kips)	F_y (ksi)	K (kips/in./in.)	P_y (kips)	P_{cre} (kips)	Number of predicted half-waves	λ_c	P_n (kips)	P_{test}/P_n
1	24K4	3-5/16	2	Yes	9.6, 19.2, 28.8, 38.4	24.2	20.0	58.0	0.0015	47.6	23.2	1	1.43	20.3	0.98
2	24K4	3-5/16	2	Yes	9.6, 19.2, 28.8, 38.5	30.4	25.1	58.7	0.0016	48.5	23.7	1	1.43	20.8	1.21
3	24K4	3-5/16	2	Yes	19.2, 28.8	29.7	24.6	58.5	0.0032	47.1	24.7	1	1.38	21.7	1.13
4	24K4	3-5/16	2	Yes	19.2, 28.9	30.8	25.1	57.8	0.0065	45.7	23.8	1	1.39	20.9	1.20

5. Concluding Remarks

A new strength prediction approach is presented for open web joists partially braced by a standing seam roof. The approach employs the AISC (AISI) column curve to calculate top chord flexural buckling capacity based on the top chord's critical elastic buckling load. Recently derived buckling load equations are summarized that account for lateral stiffness provided by the roof and the parabolically varying axial load from a uniform vertical pressure along the span. A new hybrid experimental-computational protocol is introduced for approximating standing seam roof lateral stiffness. The strength prediction approach was verified with a small set of experiments. Additional experimental verification is needed to fully validate the approach as a general prediction method for open web joists braced by a standing seam roof.

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