BEHAVIOR OF METAL-PLATE-CONNECTED TRUSSES WITH SQUARE-END WEBS

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ABSTRACT

Metal-plate-connected trusses are presently fabricated with webs cut to produce tight joints and approximately equal panel lengths on the top and bottom chords. This paper evaluates the structural feasibility of replacing these custom-cut webs with webs that have square end cuts and come in a few standard "commodity" lengths. A computer model was developed to simulate the behavior of trusses with square-end webs and its performance was verified by comparison with data from full-scale truss tests. The model and the test data showed that moments in chords and webs were affected by the presence of square-end webs, raising the possibility that higher grade chords and/or webs could be required. The square ends themselves have less effect on member forces when the web length is such that panel lengths are approximately equal. Although plate buckling did not directly cause failure of any of the trusses tested or modeled, we believe that plate buckling should be treated as a truss failure mode for design purposes. Given this limitation and the narrow scope of this study, we believe that the use of square-end webs is feasible from a structural viewpoint.

Metal-plate-connected wood trusses have become tremendously important structural components in the residential and low-rise commercial building industries, primarily due to their low fabrication and erection costs. In most applications, wood trusses are not stock items; fabrication costs are low despite the fact that trusses are custom designed and custom fabricated for each building project. This paper evaluates the structural implications of a proposal to reduce truss fabrication costs by eliminating custom cutting of web members. Currently, webs are cut to ensure a tight fit between web and chord members and to create approximately equal panel lengths, so any change in truss span or pitch requires webs with different length and end details. If the use of square-end webs (SEWs) in a few commodity lengths can be justified structurally, truss manufacturers could purchase graded webs as sawmill "shorts" in several lengths and avoid all in-house cutting of webs.

The use of SEWs has implications for performance of wood members as well as metal connection plates. The unequal panel lengths created by commodity-length webs affect the distribution of bending moments in chords. An SEW leaves a gap between the web end and the edge of the chord, creating a region of the metal plate that is not laterally supported by wood and, therefore, may be susceptible to buckling under compressive stress.

In order to evaluate these and other more subtle effects, researchers at the USDA Forest Products Laboratory (FPL) and the University of Wisconsin-Madison (UW) have combined a program of testing individual joints and full-scale trusses with a computer model that simulates truss behavior up to failure. This paper focuses on the computer model as a tool to improved understanding of the effect of SEWs on truss behavior; details of the joint and truss testing have been reported elsewhere (10,11). A truss design typical of roof trusses in the manufactured housing industry provides an example for evaluating truss performance **(Fig. 1).** Working with a single example truss is intended to point out the promise and potential problems with SEWs. Of course, a more complete testing program would be required

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Forest Prod. J. 46(6):78-84.

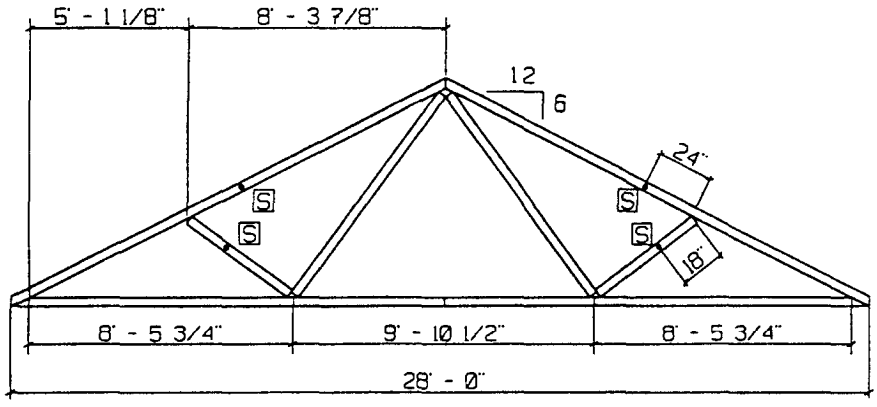

Figure 1. — Roof truss with commodity length (4 ft. and 8 ft.) square-end webs. Strain gauge locations noted by "S."

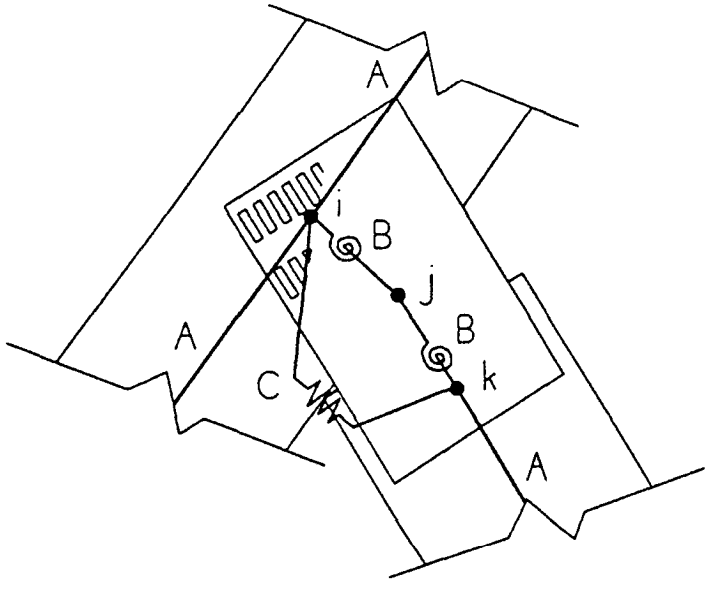

Figure 2. — Analog for the connection of a single square-end compression web to a chord. A = wood element, B = wood-plate interface element, C = wood-wood contact element, i = chord node, j = plate node, and k = web node.

before SEWs are adopted as acceptable fabrication details. The value of a verified truss analysis model is clear in this regard because it provides the capability to estimate the behavior of any truss and identify designs that might be problematic.

In this paper we will establish that a computer model can accurately model trusses with SEWs. The model was used to explore the effect of SEWs and commodity-length webs on truss behavior, with results presented here. Measures of truss performance evaluated include deflections, member axial forces and bending moments, and failure modes.

Background

The current industry practice is to cut web ends to minimize gaps between members (9). Although there is no requirement that panel lengths be equal, the common practice is to make them approximately equal. The current industry "design model" (5) allows web-chord joints to be modeled as pins, and plates (other than heel plates) to be sized based only on the axial force at each plate-wood interface. Plates can be designed for 1/2 the axial force at compressive joints, with the remaining force assumed to be carried by wood-to-wood contact.

Joint tests have shown that plates can buckle when joints with gaps are subjected to axial compression with no bending moment (7), and that the presence of moment reduces the axial force required to cause buckling (10). A large amount of variability in test results of the latter report indicates that buckling may be sensitive to joint fabrication imperfections such as initial out-of-plane deformation.

Semi-rigid and nonlinear behavior of plated joints is well documented, and several researchers have developed truss analysis models that include the complex behavior of the wood-plate interface as well as eccentricity of member centerlines (3). The SAWTEF truss analysis program developed at the UW (2) has been shown to predict truss deflections, member axial forces, and member bending moments more accurately than the industry's design model (3).

Evaluation of Three Models for Truss Analysis

Truss models

Three truss analysis procedures were investigated. The first was the SAWTEF program, a nonlinear finite element program for analysis of metal-plate-connected wood trusses. The joint analog in SAWTEF includes eccentricity of member centerlines and the nonlinear semi-rigid behavior of the plate-to-wood interface, but it does not include the possibility of plates buckling, or forces generated through bearing of adjacent wood members (3). The second analysis procedure evaluated consisted of the same program with a special joint analog added to simulate the special behavior of SEW connections. The third analysis procedure was a matrix analysis using the wood truss industry's recommended design model (5). In this model, webs and chords are pinned at their ends but chords are continuous through web-chord joints, and member centerlines are assumed to be coincident at all joints.

The behavior of an SEW connection differs from that of a standard connection because the plate in an SEW connection may buckle under a lower axial force, and after buckling, wood-to-wood contact is responsible for most of the force transferred between wood members (10). The connection of a single compression web to a chord is the joint most critically affected by SEWs, because the gap created by an SEW at this connection leaves one edge of the plate unsupported, creat-

TABLE 1. — Test truss forces (at strain gauge locations) and displacements at design load.[a]

Truss no.	Chord axial force	Chord bending moment	Web axial force	Web bending moment	Displacement at TC/web joint	Displacement at peak
	(lb.)	(in.-lb.)	(lb.)	(in.-lb.)	(in.)	
1	2,250	2,100	730	740	.20	.19
2	2,640	2,090	950	820	.29	.28
3	2,570	3,300	770	1,250	- -*[b]	- -*
Avg.	2,490	2,500	820	940	.25	.24

[a] From (11).
[b] * = displacement data recording for truss no. 3 ended before design load was reached.

ing an ideal buckling site. Contrast this with the connection of two webs to a chord, where although the gap created by SEWS does leave the central area of the plate unsupported, all three edges of this area are supported. The SEW analog developed for this study can simulate plate buckling and the wood-to-wood contact that occurs after the plate buckles. This analog, illustrated in **Figure 2,** includes finite elements representing the interface between the plate and the web, the interface between the plate and the chord, and the direct (wood-to-wood) contact between the web and the chord. The SEW analog was installed in SAWTEF only at connections of a single compression web to a chord; other SEW connections are modeled by the standard SAWTEF analog with plate sizes adjusted to provide the same plate-wood contact area as in the SEW connections.

Each of the plate-wood interface elements in the SEW joint analog is indicated by a rotational spring in **Figure 2** (B), but these elements actually consist of translational springs in the horizontal and vertical directions as well as the rotational spring. The stiffness of each spring is based on the plated area and nonlinear stiffness characteristics determined in standard plate tests (3,10). The nonlinear stiffness of the wood-to-wood contact element, indicated in **Figure 2** by a translational spring normal to the edge of the chord (C), is based on the geometry of this contact as a comer of the web is pressed into the edge of the chord. The initial stiffness of this spring is zero, as there is no contact area until a finite displacement causes local crushing of the web or chord. Once contact is established and there is embedment of the comer of the web into the edge of the chord, contact stress is modeled with a nonlinear function (similar to that used for plate stiffness in (6)):

$$\sigma = S\left[1 - \exp\left(\frac{-k_f\,\delta}{S}\right)\right] \qquad [1]$$

where:

S = perpendicular-to-grain crushing strength of wood

s = contact stress at any point in the contact area

d = embedment at this point

k_f = a foundation modulus

The crushing strength is a function of the contact area as suggested by the National Design Specification (8):

$$S = S_0\left[\frac{L + 0.375}{L}\right] \qquad [2]$$

where:

L = total length along the chord edge of the contact area (in.)

S_0 = an upper limit for the crushing strength

We chose values for k_f (57,000 psi/in.) and S_0 (1,200 psi) to provide the best fit of Equations **[1]** and **[2]** to experimental data for local crushing (4).

Plate buckling is evaluated by partially following the method of Foschi (6), who treated the plate as a series of individual steel strands with uniform length and determined the buckling load for each length experimentally. The latter part of Foschi's method is not practical in this study because the unsupported lengths of the strands vary with the angle between web and chord members, so a more analytical approach was used. The plate was divided into a number of equal-width strands, the length and effective length of each was determined, and the Euler buckling load of each was calculated. When the Euler buckling load was greater than half the yield load, an interpolation formula was used:

$$Pc = Py\left[1 - \frac{Py}{4\pi^2 EI}(kL)^2\right] \qquad [3]$$

where:

P_c and P_y = the strand's buckling load and yield load

I = the moment of inertia for bending out of the plane of the plate

kL = the strand's effective length

This equation, commonly used in the design of steel columns (1), creates a smooth transition of critical load between the Euler formula for longer strands and the yield load for very short strands. The effective length of a strand is its unsupported length between the wood members multiplied by a factor whose value is 1 when the gap between wood members is rectangular, but is less than 1 when the unsupported area is triangular or trapezoidal. This factor accounts for the partial support given to longer strands by neighboring shorter strands; it was developed to give the best match with measured buckling loads in a series of tests of individual joints (10). The actual force in each strand is a function of the compressive force and the moment in the plate. The plate is assumed to buckle when the first strand buckles. A buckled plate is simulated by reducing the stiffnesses of the springs in the plate/wood interface element to very low values.

TRUSS TESTS

Three replicates of the truss shown in **Figure 1** were tested at the FPL (11). The truss design, including lumber species and grade, and sizes and locations for 20-gauge plates, was provided by the sponsor of this work. Modulus of elasticity (MOE) for all lumber was measured with a dynamic MOE tester and the trusses were manufactured by a local fabricator. The trusses were supported in a wall-mounted testing rack and loaded at 12 points along the top chord to simulate a uniform load. The load was increased in steps to failure. Strain-gauged clips were

TABLE 2. — Model truss forces (at strain gauge locations) and percent error at design load.

Model	Truss no.	Chord axial Force	Chord axial Percent error	Chord bending Moment	Chord bending Percent error	Web axial Force	Web axial Percent error	Web bending Moment	Web bending Percent error
		(lb.)		(in.-lb.)		(lb.)		(in.-lb.)	
SAWTEF	1	2,010	-11	1,960	-6.7	749	2.6	627	-15
	2	2,010	-24	1,980	-5.3	749	-21	685	-16
	3	2,020	-21	2,010	-39	749	-2.7	741	-41
	Avg.	2,010	-19	1,980	-21	749	-8.7	684	-27
SAWTEF with SEW joint analog	1	2,010	-11	2,390	14	743	1.8	301	-59
	2	2,010	-24	2,410	15	747	-21	304	-63
	3	2,010	-22	2,440	-26	751	-2.5	311	-75
	Avg.	2,010	-19	2,410	-3.6	747	-8.9	305	-68
Design model[a]	1	2,170	-3.6	942	-55	898	23	0	-100
	2	2,170	-18	939	-55	898	-5.5	0	-100
	3	2,170	-16	939	-72	898	17	0	-100
	Avg.	2,170	-13	940	-62	898	9.5	0	-100

[a] Design model from (5).

TABLE 3. — Model truss displacements and percent error at design load.[a]

Model	Truss no.	Top chord/web joint Displacement	Top chord/web joint Percent error	Peak Displacement	Peak Percent error
		(in.)		(in.)	
SAWTEF	1	.23	15	.22	16
	2	.26	-10	.26	-7.1
	Avg.	.25	0	.24	0
SAWTEF with SEW joint analog	1	.33	65	.22	16
	2	.35	21	.26	-7.1
	Avg.	.34	36	.24	0
Design model[a]	1	.18	-10	.17	-11
	2	.21	-28	.21	-25
	Avg.	.20	-20	.19	-21

[a] Design model from (5).

used to measure wood strain at locations indicated in **Figure 1.** After the truss tests, an 8-inch block was cut from the lumber at each strain gauge location and tested to determine the local MOE, which was used with strain measurements to calculate the web and chord bending moment and axial force. A detailed discussion of the methods and results of these tests is presented by Wolfe et al. (11).

Partial results of the truss tests are shown in **Table 1.** The forces and bending moments in the table are all at the strain gauge locations noted in Figure 1. The axial forces in the webs and chords are all compressive, the chord bending moments all cause compression on the top edge of the chords, and web moments cause compression on the edge of the web closer to the truss centerline. Note that there is some variation in the values: the chord moment and web moment in truss 3 and the web axial force in truss 2 are much higher than the other two readings. All of the values reported in **Table 1** are for the same side of the trusses (the trusses are not exactly symmetric because MOE was not constant among members).

Comparison of Analyses and Test Results

The goal of this comparison is to determine which of the three truss analysis models is best able to simulate actual truss deflections and member forces. To this end, the inputs given to the programs match the properties of the test trusses as closely as practical. Each truss member was assigned its measured MOE (11) and the plate stiffness parameters for the SAWTEF programs were measured for the plates (10). One characteristic of the trusses that was not included in the models was the misplacement of plates on some joints; this proved to be important in determining failure modes for the tested trusses. The models were based on the design drawings from the truss manufacturers, and plate misplacements were not documented with sufficient accuracy to be included. The load distribution on the models matched the loading in the tests, and the models were analyzed at steps of 1/8 of the total design load. The SAWTEF programs predict ultimate failure of a truss, so these programs continued increasing the load until failure of a member or connection was indicated. The design model does not require a nonlinear analysis, so one analysis was made using a simple structural analysis program and the results were scaled for comparison at other load steps.

The models were all analyzed with nodes at the locations of the strain gauges in the test trusses to allow simple comparison of forces; analysis forces at design load are given in **Table 2.** These forces have the same sign conventions as the corresponding quantities in **Table 1.** The percent error column is the analysis output minus the test reading divided by the test reading, times 100. Note that the design model predicts member axial forces with approximately the same accuracy as SAWTEF and SAWTEF with the SEW joint analog. The lowest error in chord bending moment is from SAWTEF with the SEW joint, and the lowest error in web moment is given by the standard SAWTEF model. Displacements from the analyses given in **Table 3** correspond to the displacements from the tests in Table 1. The average error from the standard SAWTEF model is zero. The SAWTEF with SEW joint model predicts that the effect of plate buckling

on truss displacement is localized to the top chord near the buckled plate, as indicated by the larger displacement values for this situation in **Table 3.** This localized deflection was not verified by the test data where only one of the three trusses showed clear signs of plate buckling.

Results for truss 1 are shown in graphical format in **Figures 3** to **5.** Axial forces in the web and top chord as a function of truss load are shown in **Figure 3.** When viewed in this format, all three models appear to be close to the test results, although the web force in the test truss dropped sharply at about 1.1 times design load and none of the models predicted this. **Figure 4** shows top chord and web bending moments. Note that the design model moments are not close to the other models and test results; the design model predicts zero moment in webs, compared to moments on the order of 1,000 in.-lb. in the tests and SAWTEF models. The test web moment drops at about 1.1 times design load; SAWTEF with SEW predicts a similar drop occurring when the model buckled the web-chord connection plates at about 0.75 times design load. There is a corresponding rise in chord moment at the same time in SAWTEF with SEW, but none in the test truss. **Figure 5** shows a plot of truss load versus peak deflection. Nonlinearity near and above design load was measured in the test and predicted by the SAWTEF models, and clearly contrasts the linear behavior predicted by the industry's design model. Note, however, that the design model produces fairly accurate deflection predictions up to design load, its intended range of use.

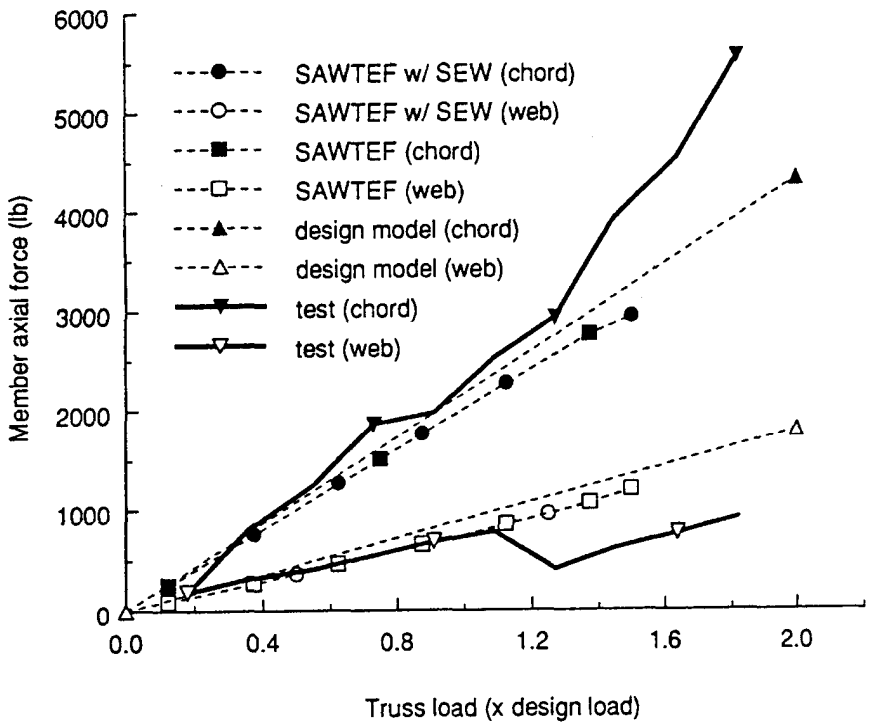

Figure 3. — Truss 1 north side axial forces.

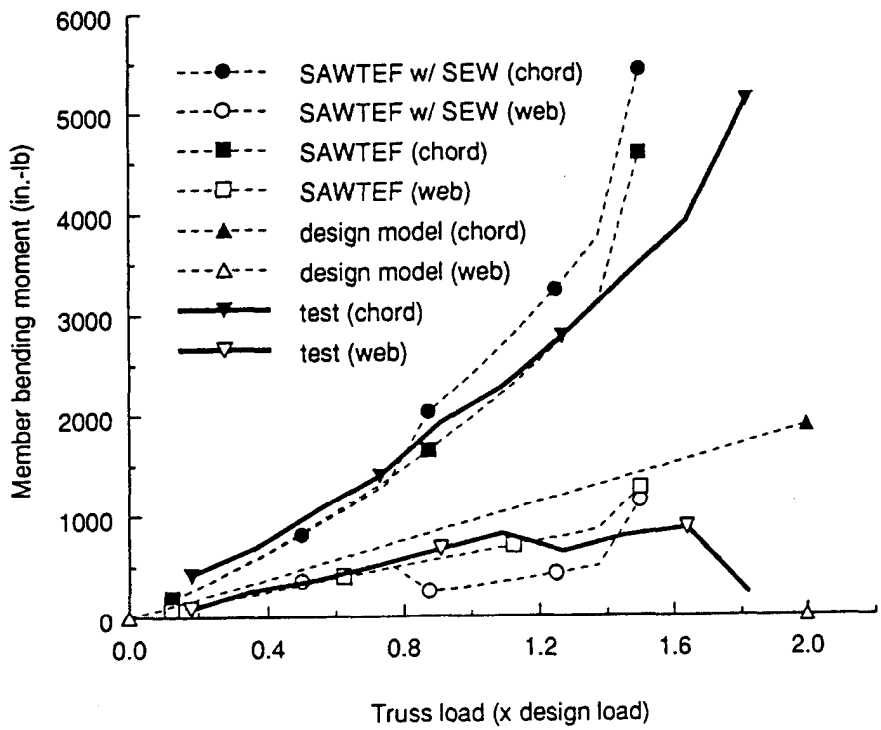

Figure 4. — Truss 1 north side bending moments.

SAWTEF with the SEW joint analog predicts web-chord plate buckling at 0.7 to 0.8 times design load for all three trusses. Only one test truss showed clear signs of plate buckling; the drop in web moment noted above for truss 1. This is a disappointing comparison; the plate buckling routine was calibrated with data from earlier joint tests but the comparison with truss tests suggests it is not very accurate. For this truss design, tests indicated that plate buckling does not have an important influence on truss failure. As a result, the standard SAWTEF model without the special joint analog for buckling at SEW connections best simulates the behavior of these trusses. There are likely other truss designs and loading conditions for which the effects of plate buckling are magnified, and the standard SAWTEF model might be accurate only until a plate buckles.

Failure modes varied for the tested trusses, with one peak plate failure, failure of a poorly placed heel plate, and bending-compression failure of a top chord at a large knot (11). All three SAWTEF predictions were for failure at the peak plates, but recall that plate mis-

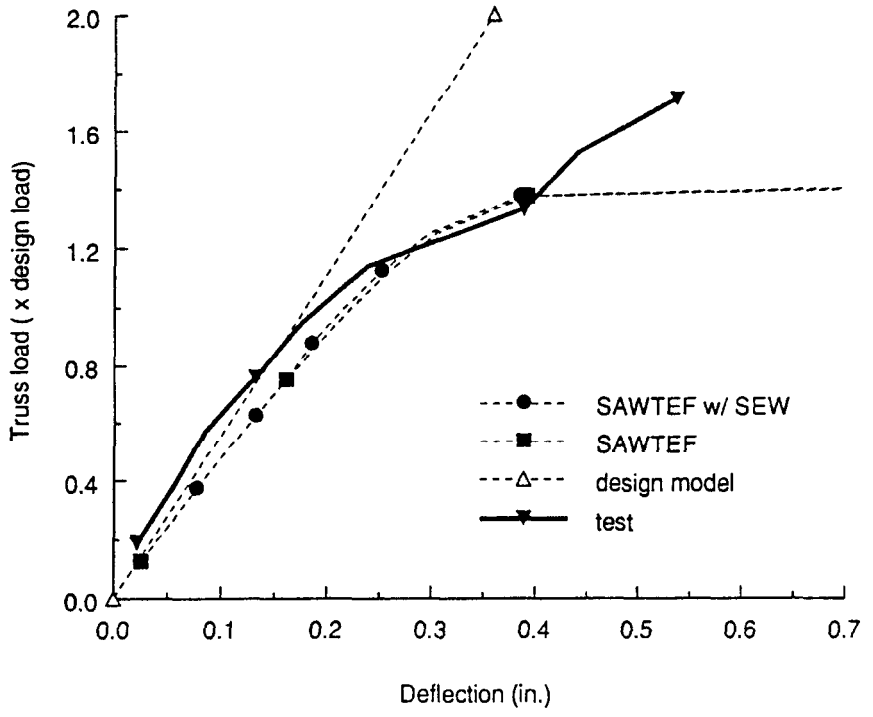

Figure 5. — Truss 1 peak deflection.

TABLE 4. — Comparison of truss member combined stress index (CSI) at design load.

Truss type	Analasis	Top chord CSI			Compression web CSI
		Inner panel	Panel point	Outer panel	
Normal design	SAWTEF	.59	.62	.59	.03
Custom length	SAWTEF	.60	.63	.59	.06
Square-end web	SAWTEF	1.2	.76	.12	.34

placements in the trusses were not included in the models.

USING SAWTEF TO ASSESS SQUARE-END AND COMMODITY-LENGTH WEBS

THREE TRUSSES

Three variations of the truss in **Figure 1** were analyzed with the SAWTEF model to separately evaluate the effects of SEWs and commodity-length webs. A "normal" design (NO) has the same overall dimensions as the truss in **Figure 1,** but the webs divide each top chord into two equal lengths and the bottom chord into three equal lengths. The web ends are cut to eliminate gaps. A "custom-length" design (CL) uses the same web lengths and panel dimensions as NO, but the webs have square ends so there are gaps at their ends. The third truss is the one analyzed previously, which is shown in Figure 1, and has commodity-length square-end webs (SQ). The loading is the same as that used by the truss manufacturer in designing this truss: 30 psf snow load, 7 psf roof dead load, and 10 psf bottom chord dead load, with trusses spaced at 24 inches on center. All chord members are No. 1 southern pine and all webs are No. 3 southern pine.

RESULTS AND DISCUSSION

The most significant difference between predicted behavior of the three truss configurations is the difference in top chord bending moment when the web layout is changed to use commodity-length webs. **Table 4** shows the combined stress index (CSI) for the top chords and compression webs of the trusses as calculated from SAWTEF output. (CSI is a measure of the combined effects of axial force and bending moment, defined in (8).) Values for inner panel and outer panel are at locations of maximum positive moment in the chord, near the middle of each panel. Calculation of CSI at other than a panel point requires an estimate or calculation of the buckling length of the wood member. The wood design specification requires that these values be chosen according to "good engineering practice" (8). The actual length of the wood member was used for the webs and 0.8 times the distance between panel points was used for the chords. The chosen values imply that the webs can buckle as columns with pinned ends and that the chords can buckle as columns pinned at one end and fixed at the other (where the chords are continuous across the panel point). We believe that this simple estimate of effective length is appropriate because more rigorous computation of buckling load is influenced by interpretation of boundary conditions and imperfections in the structure. The resulting differences in effective length and member capacity are small.

While the two top chord panel lengths are approximately equal in trusses NO and CL, the ratio of the longer to the shorter panel in the top chord of truss SQ is over 1.5. This imbalance creates a much higher bending moment in the inner panel of truss SQ's top chord than at any other location in any of the three trusses and results in a CSI greater than 1.0, indicating a deficient design.

At design load, the compression web axial forces are approximately equal for the three truss designs, but the bending moment transferred to the web from the top chord varies significantly. In trusses NO and CL, the panels to either side of this joint have approximately equal lengths whose moments roughly balance one another at the web-chord joint. The metal plate at this joint in NO is so small (being designed to carry one-half the axial force in the web and no moment) that it has little rotational stiffness, and it transfers about 54 in.-lb. to the web. The metal plate at the same joint in CL is larger because it must transfer all of the web's compressive force, and as a result it has more rotational stiffness and transfers 140 in.-lb. to the web. In tress SQ, the combination of unequal panel lengths and the large plate results in 970 in.-lb. being transferred to the web. The difference in moment transfer to the webs is

evident in the web member CSIs given in **Table 4.** Although none is close to the critical value of 1.0, the large change in CSI from the normal layout (truss NO) to truss SQ indicates that web stresses deserve closer attention if square-end commodity-length webs are used. Any truss analysis that models web ends as pinned connections will predict zero moment in web members and, therefore, cannot predict the larger CSI in truss SQ.

Conclusions

SAWTEF's predictions of member forces and truss deflections are better overall than those of the design model and of SAWTEF with the SEW joint analog. This observation must be qualified, however, because the comparison of test data and analyses was limited to a single truss design. The SEW joint analog's predictions of truss load to cause buckling and the effect buckling has on truss behavior cannot be verified by the data. Based on these results, SAWTEF is the most attractive option for modeling trusses with SEWs.

The larger plates needed at SEW joints permit significant bending moments to be transferred from chords to webs. Truss designs with longer compression webs may require higher grade webs or more web bracing than would be required with typical small plates at angle-cut web ends. Any truss analysis that assumes web ends have pinned connections cannot predict these moments. Graded commodity-length webs produce unequal panel lengths, creating higher chord bending moments than those occurring when the same length chord is divided into equal panels. Higher grade chord members may be required in some cases. Using SEWs with custom lengths to produce approximately equal panel lengths has less effect on member forces than using SEWs in commodity lengths.

Plate buckling does not have a significant influence on the behavior or failure of the tested trusses, although it may occur at lower load levels or have a more serious impact in other truss designs or loadings. If SEWs are used, we recommend treating plate buckling as a truss design limit comparable to plate pullout or failure of a wood member.

The proposal to eliminate custom web cutting in metal-plate-connected wood trusses is reasonable from a structural point of view. The design process should ensure that plates do not buckle at truss service loads. Larger plates and, in some cases, higher grade chords and webs will be required. If the cost savings are still significant given these limitations, the truss industry should proceed with the additional testing and analysis needed for implementation of this concept.

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