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# Strength and Stiffness of Large-Gap Metal-Plate Wood Connections

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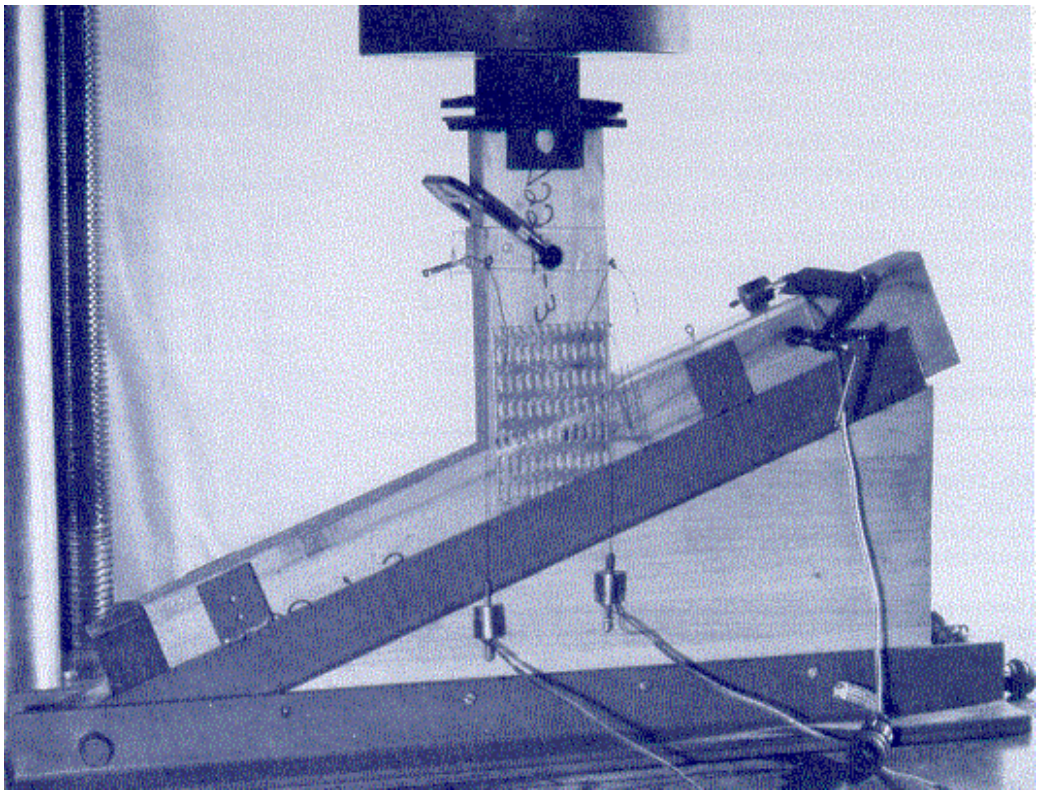

# Abstract

This study was conducted to determine the feasibility of fabricating light-frame trusses using square-cut commodity webs. We developed a simple test to identify different plate types and provide design information on square-cut webs to determine their effect on load capacity of truss connections. Test results suggested that square-end commodity webs are technically feasible. While the combination of bending moment and axial load carried by the joint had a critical effect on buckling load of connectors, it was difficult to simulate actual truss boundary conditions for an individual joint test. Full-scale truss test results and an accurate analytical model to determine boundary conditions will be presented in subsequent reports.

Keywords: Trusses, square-end webs, plate buckling, joint tests, plate orientation, joint gaps

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# Strength and Stiffness of Large-Gap Metal-Plate Wood Connections

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## Introduction

Metal-plate-connected wood trusses are reliable and cost-effective building components for light commercial as well as residential structures. Because of the trend toward mass production in residential construction, relatively minor changes in truss design, fabrication, and erection procedures mean significant cost savings. One proposed change is to replace custom-cut truss webs with square-end webs in standard lengths. This will decrease material handling and machining costs, and the cost of the lumber itself if the webs can be purchased as sawmill shorts. Truss manufacturers must compare these savings to the cost of larger plate sizes, which will be necessary if sawmill shorts are used.

The use of standard web lengths will often result in unequal panel lengths (as defined by divisions of the truss chords) and will create gaps between the ends of the wood members. Both of these results are inconsistent with current industry practice. Unequal panel lengths create higher bending moments in chords and joints. Gaps between wood members mean compressive forces cannot be transferred directly by wood-to-wood bearing. Thus, the entire force must be carried by the connector plates. Metal connector plates at joints with gaps will not receive the lateral support they usually get from teeth embedded into the wood. Therefore, there is a potential for plate buckling under compressive loads.

Square-end webs will have the most significant impact on joint behavior at connections of a single compression web to a truss chord (Fig. 1a). Here the plate must transmit the entire compressive force because this connection has essentially no wood-to-wood bearing area. The plate may have a long unsupported length across the gap between wood members, so there is potential for plate buckling. Single and double web-to-chord tensile connections (Fig. 1b) rely on plate–wood contact area and plate tensile strength to transmit forces. Square-end webs are not likely to affect failure modes or loads as long as plate size is increased to provide sufficient plate-wood contact area. Double web-to-chord connections with one web in tension and the other in compression (Fig. 1c) will include a triangular unsupported plate area with wood on all three sides. While this area may distort under high shear forces parallel to the chord, the support on three sides makes the area less likely to buckle under compressive forces than the single web-chord compression joint.

This study is part of a project that includes truss modeling research at the University of Wisconsin–Madison and full-scale truss tests at the USDA Forest Service, Forest Products Laboratory. Results from both of these efforts will be reported in future papers.

## Literature Review

Industry standards for truss fabrication attempt to minimize gaps at all joints. The U.S. light-frame truss industry quality standard (Truss Plate Institute 1989) states that web members should provide average gap widths no greater than 0.0625 in. (1.59 mm) for 80 percent of the joints. Twenty percent may have average gap widths up to 0.125 in. (3.18 mm). These limitations are based on both aesthetics and engineering design standards.

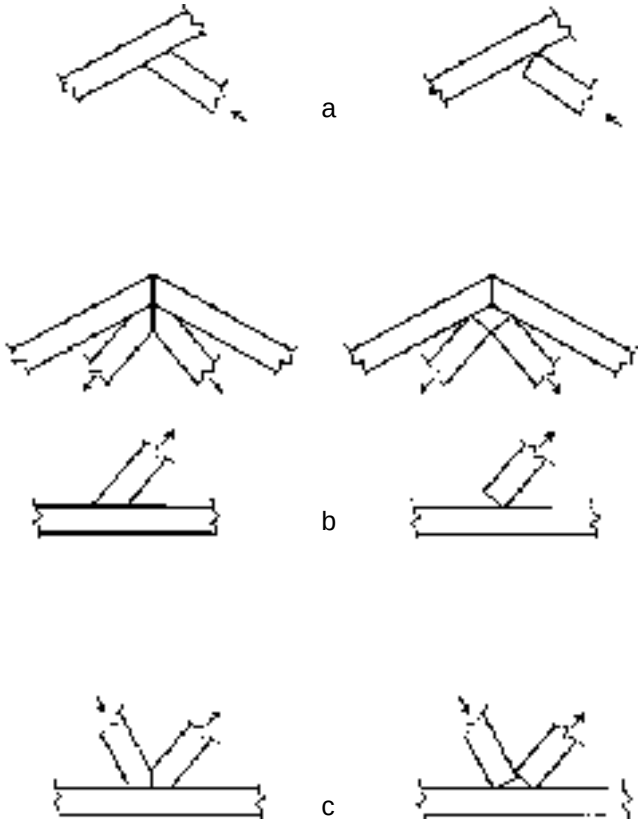


*Figure 1—Typical web-to-chord joints with fitted webs (left) and square-end webs (right). Arrows indicate direction of axial force in web members. Plates are not shown. (a) Single compression web-to-chord joint; (b) single and double tension web-to-chord joint; (c) one tension web and one compression web joined to chord.*

Relevant research in this area is limited. Kirk and others (1989) studied the effect of gap size on axially loaded compression splices. They found that plate buckling can occur at loads less than the reported design load for 20-gauge (0.91-mm-thick) plates with 0.0625-in. (1.59-mm) and 0.125-in. (3.18-mm) gaps, but 16-gauge (1.47-mm) plates did not buckle until the reported design load was exceeded. They concluded that the design of compression joints should not be based solely on plate properties derived from tensile tests. Kocher (1986) investigated the efficiency of unsupported plate areas carrying shear loads in web-to-chord joints of parallel-chord floor trusses. She found that the industry standard in effect at the time was overly conservative and the limitation should be based on the length of an unsupported plate edge rather than the plate's unsupported area. Wolfe (1991) and Gupta (1994) found that joint load eccentricity will significantly reduce axial load capacities that are based on a concentric load test. Square-end web connections accentuate these effects by confining wood-to-wood bearing to a small area, often outside the plate area of the connection. Load eccentricity, therefore, is a more important consideration for square-end web connections than it is for fitted webs. The plated joint computer model developed by Cramer and others (1990) is capable of simulating the behavior of joints under combined axial and bending loads. This model was shown to model plate behavior under tension and bending and includes analogues to account for plate buckling. However, it has not been verified by comparison to tests of full-scale trusses exhibiting plate buckling failure modes.

Truss production data and analytical models support the premise that using commodity square-end webs will provide significant costs savings for the production of light-frame roof trusses in domestic and foreign markets. Test methods as well as strength test data are needed, however, to verify their technical feasibility.

# Objectives

A design procedure for joints with square-end webs cannot be approved until three tasks are completed. First, sufficient joint strength and stiffness must be shown, and the appropriate limit state for design must be identified. Second, a standard test procedure to assess the compressive capacity of plates must be developed. Finally, a conservative strength prediction model must be proposed and verified. This study addresses the first two tasks through three series of tests that focus on individual joint behavior.

1. Simplified plate buckling tests: Peak load was defined under different plate orientations and gap widths to identify the simplest test that might serve as a standard method to determine design properties.

2. Concentric load joint tests: Simulated web-to-chord joints were tested with concentric axial compression (no bending moment) to determine load-displacement behavior prior to plate buckling, buckling load and mode, and postbuckling behavior. Specimens with fitted webs and square-end webs were tested.

3. Eccentric load joint tests: Although accepted simplified truss analysis procedures do not recognize the rotational stiffness of most plated joints, some bending moment is transferred across the joints. To investigate the effect of bending moment along with axial force, joint specimens were tested with eccentric axial loading. Specimens with fitted webs and square-end webs were tested.

# Materials and Methods

## Materials and Specimen Preparation

All test joints were made from 12-ft- (3.6-m-) long, medium density Southern Pine nominal 2- by 4-in. (standard 38- by 89-mm) lumber (2 by 4's) obtained from a local truss fabricator. Neither species, grade, nor specific gravity was meant to be a variable for this study. After the lumber was

conditioned to 12-percent moisture content (MC), each 2 by 4 was cut to give three test specimens. Each specimen was then cut in half, with a 1-in. (25.4-mm) block cut from one of the newly exposed ends to determine MC and specific gravity. The two halves of each specimen were joined with metal plate connectors after inspection verified that there were no defects (knots, wane, high grain angle) in the 3 in. (76.2 mm) closest to the connection.

Plates were obtained from a single manufacturer. The 20-gauge (0.91-mm) plates had a tooth density of 10 teeth/in$^2$ (1.55 teeth/cm$^2$) and a tooth length of approximately 0.30 in. (7.62 mm). The 16-gauge (1.47-mm) plates had 4.8 teeth/in$^2$ (0.74 teeth/cm$^2$) and a tooth length of about 0.38 in. (9.65 mm). Teeth were aligned in the 20-gauge (0.91-mm) plates and staggered in the 16-gauge (1.47-mm) plates. In both cases the teeth had a twist of approximately 40° from root to tip. Joint members were placed in an alignment jig to assure consistent placement of the members and connector plates during pressing. The plates were centered on the 3.5-in. (88.9-mm) face of at least one joint member. All plates were pressed to full embedment with a testing machine and then stored in the conditioning room for a minimum of 14 days before testing.

## Specimen Configurations

The specimen geometry for simplified axial buckling tests is shown in Figure 2a. Both16-gauge (1.47-mm) and 20-gauge (0.91-mm) plates were tested with their major axis parallel and perpendicular to the force in joints with 0.25-in. (6.35-mm) and 1.0-in. (25.4-mm) gaps. As this was a test for behavior of the plate over a gap, the orientation of the wood members in these specimens was not intended to have an effect. The member orientations were chosen to create a test specimen that could be supported on the bed of almost any universal testing machine.

Concentric load joint tests (indicated by the first letter C in the specimen identification) were conducted with specimens in the three configurations shown in Figure 2b. Joint configuration CA, representing a fitted joint with wood-to-wood contact across the width of the joint, served as a control for the study. Configuration CB displayed the effects of axial compression across the unsupported plate area. Configuration CC simulated a square-end web connection, where the connector plate transfers shear as well as a normal force to the chord member. Four variations, including specimen geometry and plate gauge, within each configuration were tested. Geometry variations included gap and member angles ($\theta$ and $\phi$, respectively) and plate gauge and orientation (A or E for plate principal axis pArallel or pErpendicular to load, respectively). Two nonzero gap angles ($\theta$ = 22° and 45°) were planned for the joints, but the joints in one group intended for a 45° gap were mistakenly fabricated with $\theta$ = 33°.

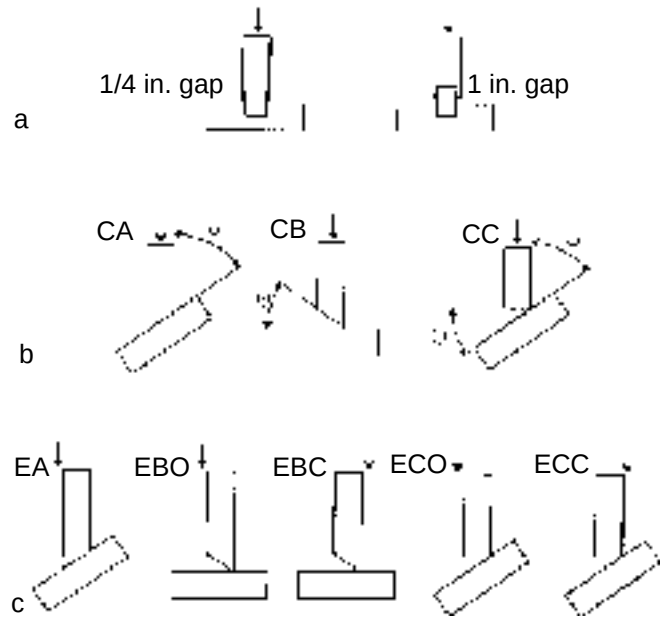


*Figure 2—Specimen configurations for the three series of tests. (a) Buckling tests; (b) concentric load joint tests; (c) eccentric load joint tests. Plates are not shown in parts b and c.*

Eccentric load joint tests (indicated by the first letter E in the specimen identification) were conducted for limited cases using the same three basic configurations described above (Fig. 2c). Half of the 20-gauge (0.91-mm) test samples were loaded eccentrically to the open (EBO, with an O for open) side of the joint and half were loaded eccentrically to the closed (EBC, with a C for closed) side of the joint. The 16-gauge (1.47-mm) samples were all loaded to the closed side of the joint.

## Test Setup and Procedure

All joints were tested using a screw-type test machine operated at a head speed of 0.04 in/min (1.02 mm/min). Joint displacements were measured with four or five linear variable differential transducers (LVDTs) attached to the wood members or plates, and loads were obtained from the test machine's load cell. Loads and displacements were recorded on a personal computer. Most tests were concluded after the load decreased due to plate buckling or some other joint failure, but a few joint tests were continued to investigate joint behavior beyond plate buckling.

A jig, designed for this study, provided support for specimens in all tests. The upper arm of the jig (Fig. 3) was a steel channel that provided bearing and lateral support for the simulated chord member. This support channel was pinned to the lower support arm so it could pivot from 0° (horizontal) to greater than 45° to accommodate a range of joint angles. It was held at the proper angle with a hardwood wedge. The lower arm of the jig consisted of two steel angles joined by a steel plate. A steel dowel, passed through a hole

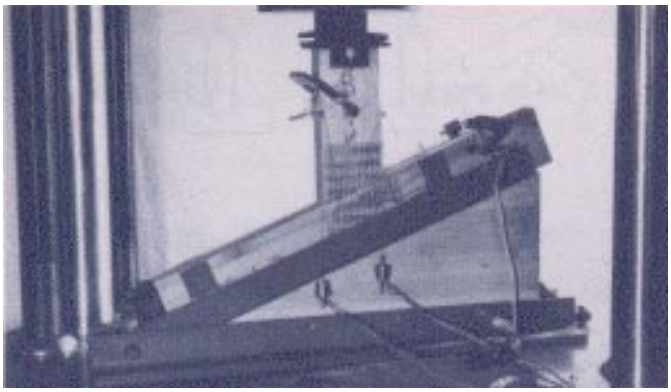

*Figure 3—Steel testing jig fabricated to support specimens subjected to concentric or eccentric compression.*

in this plate and into a hole on the bed of the testing machine, kept the jig and test joint in line with the load head.

Tests were conducted using both concentric and eccentric loads. Concentric loads were applied directly by the test machine load head to the end of the specimen's vertical member. Thin rubber pads were placed between the two surfaces to help distribute the load. Eccentric loads were applied with a loading grip described by Wolfe and others (1991). This grip was attached to the specimen's vertical member using a steel pin that passed through pressed steel shear connectors. The eccentric load was applied with a 2.625-in. (66.8-mm) offset, parallel to the vertical member's centroidal axis.

Joint tests required up to five LVDTs to completely characterize joint deformation. For concentric and eccentric load-joint specimens with a member angle $\phi = 90°$, four LVDTs were used. All four LVDTs spanned the joint and were mounted parallel to the vertical member with one at each corner of the member. Four LVDTs were necessary to detect rotation in the two vertical planes as well as vertical deformation. The resulting measurements included deformations of the plate itself and the plate-wood interfaces at both wood members. A fifth LVDT was used for specimens with $\phi < 90°$ to detect horizontal movement of the vertical member with respect to the chord. This could be used along with the rotation measurement to calculate the amount of shear deformation at the joint. Displacement information was not collected for the simplified buckling tests.

Member specific gravity and MC were determined following ASTM standard D2395 (ASTM 1993) procedures using the blocks cut during joint fabrication.

# Results and Data Analysis

## Moisture Content and Specific Gravity

The average, high, and low specific gravities were 0.51, 0.67, and 0.37. The MC ranged from 6 percent to 15 percent with an average of 12 percent. Although the range of these values is significant, they were not included as variables for the study. The MC and specific gravity affect the overall stiffness of the joints because they affect the stiffness of the plate–wood interface. Specific gravity and MC do not, however, affect the joint load at which plates buckle.

## Simplified Buckling Tests

All specimens failed by plate buckling, characterized by a significant decrease in load as the plate deformed out-of-plane. The average buckling load ranged from 1,420 lb (6.32 kN) for 20-gauge (0.91-mm) plates oriented perpendicular to the load with a 1.0-in. (25.4-mm) gap to 6,640 lb (29.53 kN) for 16-gauge (1.47-mm) plates oriented parallel to the load with a 0.25-in. (6.35-mm) gap (Table 1). There was a large amount of scatter. As gap size increased, buckling load variability appeared to increase for 20-gauge (0.91-mm) plates and decrease for 16-gauge (1.47-mm) plates. Due to the relatively small sample sizes involved, however, this should not be taken as characteristic behavior.

Table 1—Simplified buckling test results

| Group ID | Number of specimens | Gap (in. (mm)) | Plate orientation | Buckling load (lb (kN)) | | |
|---|---|---|---|---|---|---|
| | | | | Average | High | Low |
| 20-gauge (0.91-mm) plates | | | | | | |
| BB20-1 | 5 | 0.25 (6.35) | A | 3,180 (14.14) | 3,660 (16.28) | 2,090 (9.30) |
| BB20-2 | 5 | 0.25 (6.35) | E | 3,550 (15.79) | 4,670 (20.77) | 2,410 (10.72) |
| BB20-3 | 5 | 1.0 (25.4) | A | 1,450 (6.45) | 1,930 (8.58) | 880 (3.91) |
| BB20-4 | 5 | 1.0 (25.4) | E | 1,420 (6.32) | 3,470 (15.43) | 700 (3.11) |
| 16-gauge (1.47-mm) plates | | | | | | |
| BB16-1 | 5 | 0.25 (6.35) | A | 6,640 (29.53) | 8,520 (37.90) | 4,840 (21.53) |
| BB16-2 | 5 | 0.25 (6.35) | E | 3,940 (17.53) | 5,200 (23.13) | 2,160 (9.61) |
| BB16-3 | 5 | 1.0 (25.4) | A | 4,340 (19.30) | 5,000 (22.24) | 3,560 (15.83) |
| BB16-4 | 5 | 1.0 (25.4) | E | 2,770 (12.32) | 3,420 (15.21) | 2,040 (9.07) |

Buckling load was not substantially affected by plate orientation for 20-gauge (0.91-mm) plates, but 16-gauge (1.47-mm) plates supported significantly higher loads when the plates were oriented parallel to the load. This difference is probably a result of the tooth patterns—the 20-gauge (0.91-mm) plates had aligned teeth that created solid steel bands in both the plate major and minor directions. Although these bands had different widths and spacings in the two directions, they were apparently balanced in their ability to support compressive loads. The 16-gauge (1.47-mm) plates had staggered teeth, so there were no continuous bands of steel in the plate minor direction. Therefore, the plate was not as effective when used to carry compressive loads in this direction.

## Concentric Load Joint Tests

Peak load was well defined by plate buckling for all specimens in the CB and CC configurations subjected to concentric compressive loading. Peak loads for specimens in the CA configuration were defined by either plate buckling or tooth slip. Plate deformation due to buckling tended to be more pronounced on the open side of the joints, as expected. When buckling occurred, the plate deformation assumed one of several different forms. In some cases, the plates deformed both away from one another or toward one another while the two wood members remained coplanar. Another observed form was asymmetric: one plate deforming "in" and one deforming "out," again with the wood members remaining coplanar. In most cases, however, the plate deformation was asymmetric, but the vertical wood member was forced slightly out from the original plane of the specimen. This type of buckling mode has serious implications for truss performance, which will be discussed later.

The average peak load ranged from 2,540 to 4,040 lb (11.30 to 17.97 kN) for 20-gauge (0.91-mm) plates with gaps, were higher for 16-gauge (1.47-mm) plates with gaps, and were much higher for fitted joints (Table 2). For each set of equivalent joint designs (CA-1, CB-1, and CC-1, for example), configuration CA was the strongest, as expected. There was an inconsistent difference between configurations CB and CC, with CC-1 averaging about 25 percent higher than CB-1, but CC-2 averaging slightly lower than CB-2. Within each configuration, the pattern of relative strengths was the same: using 20-gauge (0.91-mm) plates oriented parallel to the load with $\theta = 22°$ as the basis for comparison, buckling load was reduced when the gap angle was increased. The highest buckling load within each configuration was achieved with 16-gauge (1.47-mm) plates and $\theta = 45°$. The 16-gauge (1.47-mm) plates also had the smallest amount of variability.

Specific examples of the load-deformation plots are shown because the variability for matched specimens was large enough to make an average plot dissimilar to any individual plot (Fig. 4). These three were not chosen because they represent typical behavior (the high degree of variability renders the term "typical" almost meaningless), but rather to illustrate specific details of joint behavior that might affect truss performance. The displacements shown in this figure are the averages of the four vertical LVDTs, and the data were processed to eliminate the initial stiffening portion of the response. Three differences are notable. First, the initial stiffnesses were not the same because of the amount of plate–wood contact area and the orientation of the plate with respect to the grain in the lower wood members; the different initial stiffness values are not of primary importance for this discussion. Second, the buckling loads varied considerably.

Table 2—Concentric load joint test results.

| | | Joint angles (deg) | | | Peak load (lb (kN)) | | |
|---|---|---|---|---|---|---|---|
| Group D | Number of specimens | Gap ($\theta$) | Members ($\phi$) | Plate orientation | Average | High | Low |
| 20-gauge (0.91-mm) plates | | | | | | | |
| CA-1 | 5 | 0 | 68 | A | 10,500 (46.70) | 12,000 (53.38) | 8,540 (37.99) |
| CA-2 | 5 | 0 | 45 | A | 7,780 (34.60) | 9,040 (40.20) | 6,000 (26.69) |
| CA-3 | 5 | 0 | 68 | E | 8,000 (35.58) | 9,600 (42.70) | 6,980 (31.05) |
| CB-1 | 10 | 22 | 90 | A | 2,930 (13.03) | 4,200 (18.68) | 1,910 (8.50) |
| CB-2 | 5 | 45 | 90 | A | 2,630 (11.70) | 3,140 (13.97) | 1,920 (8.54) |
| CB-3 | 5 | 22 | 90 | E | 3,230 (14.37) | 4,010 (17.84) | 2,180 (9.70) |
| CC-1 | 10 | 22 | 68 | A | 4,040 (17.97) | 5,940 (26.42) | 2,200 (9.79) |
| CC-2 | 5 | 33 | 57 | A | 2,540 (11.30) | 2,700 (12.01) | 2,210 (9.83) |
| CC-3 | 5 | 22 | 68 | E | 3,410 (15.17) | 4,090 (18.19) | 2,810 (12.50) |
| 16-gauge (1.47-mm) plates | | | | | | | |
| CA-4 | 5 | 0 | 45 | E | 11,300 (50.26) | 11,500 (51.15) | 10,900 (48.48) |
| CB-4 | 5 | 45 | 90 | E | 4,870 (21.66) | 5,420 (24.11) | 4,240 (18.86) |
| CC-4 | 5 | 45 | 45 | E | 4,460 (19.84) | 4,850 (21.57) | 4,020 (17.88) |

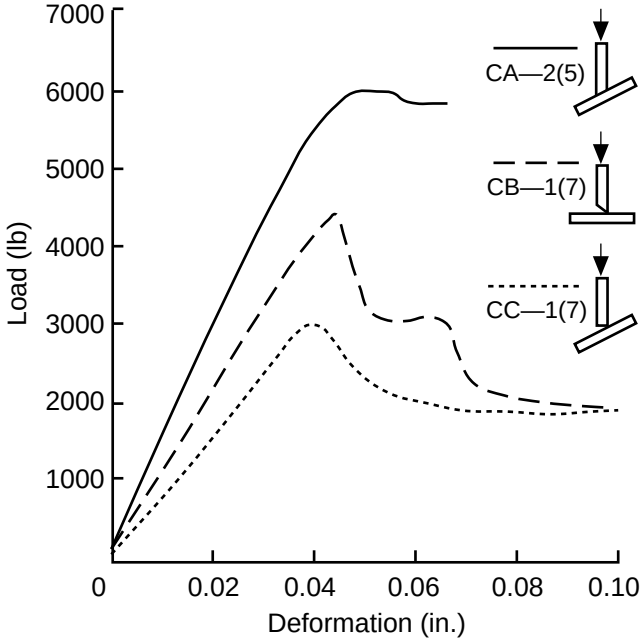


*Figure 4—Measured load–deformation response for concentrically loaded specimens CA-2(5), CB-1(7), and CC-1(7).*

This was due to the different unbraced plate length (ideally equal to zero for CA) and the different shape of the unbraced plate area. Third, the load drop after buckling was more severe in the CB and CC specimens than in the CA specimen; the implications of this for truss performance will be discussed later. After the plates buckled, the load seemed to be supported primarily by wood-to-wood bearing. Specimens in the CB and CC configurations carried only about half the peak load after plate buckling due to the small amount of wood-to-wood contact area in these configurations. Significant wood crushing occurred in these joints as the bearing area increased enough to support even half the peak load. The stepped load drop-off for CC-1(7) was caused by nonsimultaneous buckling of the plates on either side of the joint. The in-plane and out-of-plane joint rotations calculated from differences among the four vertical LVDTs were less than 1° in all cases. Rotation in the CA configuration was toward the low side, or toe, of the joint; for the CB and CC configurations, it was toward the open side of the joint. The horizontal movement of the vertical member in configurations CA and CC was insignificant for all specimens, indicating that the problem configuration CB was intended to avoid need not have been a consideration for concentric load tests.

## Eccentric Load Joint Tests

Plate buckling defined peak load for all eccentric load joint tests with the possible exception of some EA group specimens, which experienced tensile failure at one side of the plates. It is not clear whether the tensile failures were the primary failures or secondary events that occurred after the peak load had been defined by buckling. Plate deformations indicated that buckling occurred primarily at the side of the joint in which the load eccentricity was directed, even when this was the closed side of the joint. The observations of buckling asymmetry and out-of-plane deformation in the concentric load tests applied to this set of tests as well.

The average peak values for eccentric loads were much lower than those for concentric loads in identical joints. The averages ranged from 560 to 1,520 lb (2.49 to 6.76 kN) for all configurations (Table 3). For both the EB and EC groups, buckling occurred at lower loads when the eccentricity was toward the open side of the joint, as expected. A surprising result is that the average peak load for the EA group of specimens (with fitted joints) was lower than that of the EBC group. The eccentricity in the EA group was toward the low side of the joint. Average peak loads were higher for the EB group than the EC group, indicating that the component of loading parallel to the chord member in the EC group may have affected buckling. Deflection data from the LVDT parallel to the chord in the EC group specimens did not confirm, however, that there was a significant force component in this direction. The specimens with 16-gauge (1.47-mm) plates had a lower average buckling load than one group of specimens with 20-gauge (0.91-mm) plates, but the gap angle was larger for the 16-gauge (1.47-mm) plates creating an unsupported plate length almost twice that of the other groups. Also, the 16-gauge (1.47-mm) plates probably would have had higher peak loads if the plates' major axes had been oriented parallel to the load.

Table 3—Eccentric load joint test results

| Group ID | Number of specimens | Joint angles (deg) | | Plate orientation | Peak load (lb (kN)) | | |
|---|---|---|---|---|---|---|---|
| | | Gap ($\theta$) | Members ($\phi$) | | Average | High | Low |
| 20-gauge (6.91 mm) plates | | | | | | | |
| EA | 5 | 0 | 68 | A | 1,200 (5.34) | 1,270 (5.65) | 1,100 (4.89) |
| EBC | 5 | 22 | 90 | A | 1,520 (6.76) | 1,690 (7.52) | 1,330 (5.92) |
| EBO | 5 | 22 | 90 | A | 780 (3.47) | 850 (3.78) | 680 (3.02) |
| ECC | 5 | 22 | 68 | A | 840 (3.74) | 890 (3.96) | 790 (3.51) |
| ECO | 5 | 22 | 68 | A | 560 (2.49) | 640 (2.85) | 530 (2.36) |
| 16-gauge (1.47-mm) plates | | | | | | | |
| EBC-16 | 5 | 45 | 90 | E | 1,230 (5.48) | 1,350 (6.00) | 1,100 (4.89) |

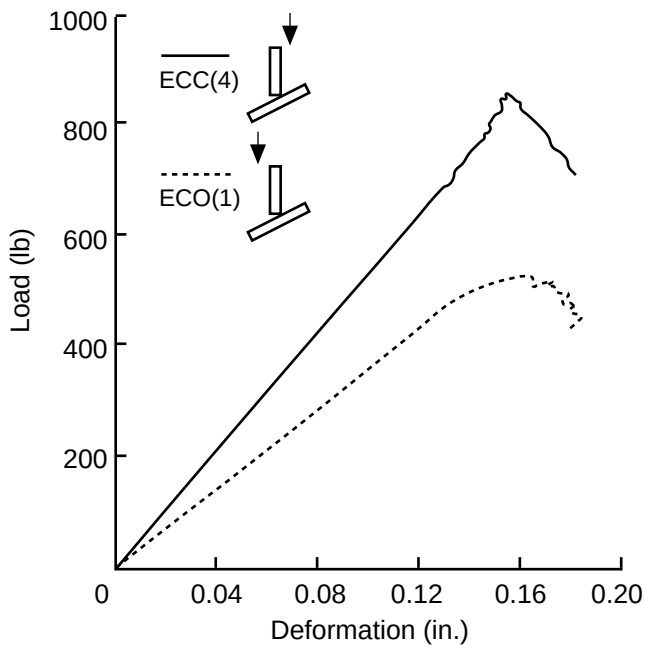


*Figure 5—Measured load–deformation response for eccentrically loaded specimens ECC(4) and ECO(1).*

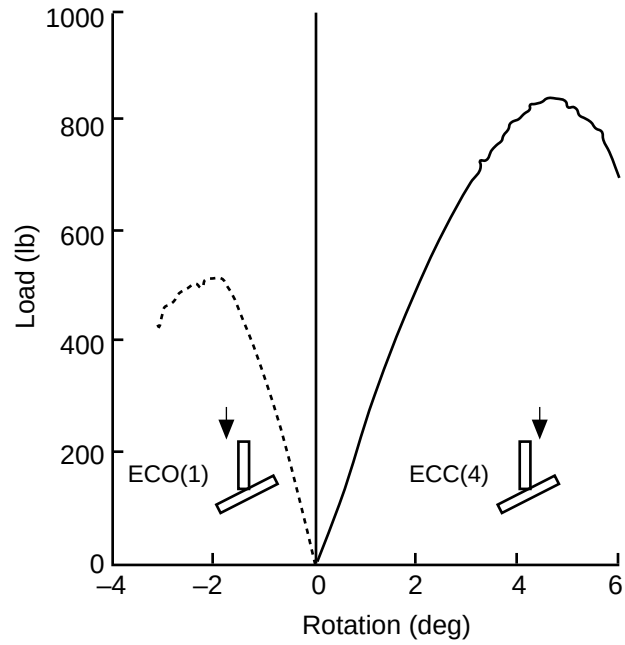


*Figure 6—Measured load–rotation response for eccentrically loaded specimens ECC(4) and ECO(1).*

Sample load-deformation and load-rotation plots are shown in Figures 5 and 6, respectively, for specimens in the EC configuration. Deflections were averaged and processed in the same way as were the concentric test results. The rotations shown are in the plane of the specimen; out-of-plane rotations were insignificant in all cases. All specimens in each configuration had plots similar to these; there was no postbuckling load "rebound" as was observed in some concentric load tests. This was logical for the EB and EC specimens with load eccentricity toward the open side of the joint, but some rebound was expected in the other specimens as the plate could act primarily in tension and the wood-to-wood contact could act in compression. The tensile failures in some EA specimens may have indicated that the joints were beginning to act in this manner, but the tensile strength was exceeded at the extreme edge of the plate before a noticeable rebound occurred. These joints could not carry eccentric compressive loads after the plates buckled.

# Discussion

## Summary of Joint Behavior

Plate buckling, observed in most tests, caused a stiffness loss and subsequent drop in the compressive force carried by the joint. Tests were conducted in displacement control. In this testing method, buckling is a relatively slow, controlled process that is often only discernible by the drop in load; there is no sudden increase in displacement such as when buckling occurs under constant load. The gradual load rebound observed in some concentric load tests was enabled by the displacement control, as the displacement slowly increased until the wood-to-wood bearing stresses rose. With square-end webs, wood crushing must have occurred to develop a large enough wood-to-wood contact area to support a measurable load. Peak loads were significantly reduced when the load was applied with an eccentricity of 2.625 in. (66.5 mm). The load rebound was not observed in any eccentrically loaded specimens.

The variability of peak load was quite large for some groups of nominally identical specimens. Thus, variability appeared to be less for specimens with 16-gauge (1.47-mm) plates than for those with 20-gauge (0.91-mm) plates. Eccentrically loaded specimens also exhibit less variability than concentrically loaded specimens. The difference in variability between specimens with 20-gauge (0.91-mm) and 16-gauge (1.47-mm) plates may have been due to different tooth patterns or a different amount of unintentional plate deformation during pressing and handling of the specimens. The eccentrically loaded specimens may have been less variable because the compressive stress was highest at one extreme edge of the plates. Thus, buckling was always initiated in the same location. For concentrically loaded specimens, the type of deformation resulting from plate buckling was not consistent. In most cases, the vertical member was forced slightly out of the original plane of the specimen when the plates did not buckle symmetrically.

## Implications for Truss Performance

Because of the difference in results between concentric and eccentric load tests, it is uncertain which test results best simulate actual truss conditions. Measurement of forces in wood members is difficult, so experimental information on member forces is limited. Ongoing wood truss modeling

research at the University of Wisconsin–Madison, however, suggests that the two loading cases used in this study probably bracket the range of typical web axial force eccentricities. Peak loads measured for the eccentrically loaded square-end web specimens (EBC and EBO, Table 3) were not significantly higher than web design forces in 25- to 30-ft- (7.5- to 9-m-) span roof trusses. This indicates that these connections might be the limiting component in truss tests. This will be true if the eccentricity at the joints is close to or greater than that used in the tests.

The loading on real trusses is not the steady displacement-controlled loading used in this study. Thus, plate buckling may have a more drastic effect on a truss than it did on our test specimens. If the load cannot be redistributed to other truss members when the plates in a square-end web joint buckle, large deformations must occur before enough wood-to-wood bearing area develops to carry the load. Plates that buckle asymmetrically and force the web member out of the original plane of the truss will cause a serious problem. The resulting out-of-plane forces will eventually force the web completely out of contact with the chord causing the chord to fail. This was observed in some preliminary truss tests.

### Simplified Plate Buckling Test for Design Properties

The simple buckling test used in this study can be used to compare the capacities of various plates, but due to the difficulty of simulating “truss” boundary conditions, these tests alone provide insufficient basis for joint design. Due to the wide range of joint geometries possible, the complex boundary conditions for a plate with teeth embedded into wood, and the tooth punch-out patterns, determining the buckling load for a plate with a triangular unsupported area subjected to bending moment and axial force is a problem. If a reliable and practical method is developed to relate plate buckling load in an actual joint to plate buckling load for uniaxial load and uniform gap, the simplified test developed here could provide the design basis.

## Conclusions and Recommendations

The results of this study indicate that while square-end webs are feasible, truss performance may be negatively affected by several aspects of square-end web joint behavior. The plate buckling loads vary widely for nominally identical joint specimens, and the joints are very sensitive to initial imperfections such as plate deformation from specimen fabrication and handling. The presence of a bending moment at the joints reduces the buckling loads to levels close to typical web design forces in 25- to 30-ft (7.5- to 9-m) span roof trusses. The behavior of the joints after buckling occurs is important: the postbuckling compressive capacity of square-end web joints is much less than that of fitted web joints because significant wood crushing must occur before sufficient wood-to-wood bearing area is developed to support a large load. In some cases, the plates do not buckle symmetrically and the web member may be forced out of the plane of the truss until it completely loses contact with the chord. In summary, square-end web joints can have the strength and stiffness required for use in typical trusses, but design standards should recognize plate buckling as a limit state along with plate pullout and other failure modes.

Attempts to identify a standard joint test in this study gave mixed results. The simplified plate buckling test investigated here will be useful for general comparison of different plate types, but the disparity between its boundary conditions and those in an actual square-end web joint makes it difficult to extract useful design information from this test. Plate buckling loads obtained from the concentric load joint tests could be misleading because the lack of bending moment in these plates gives unrealistically high buckling loads. The eccentric load joint tests provide the best indication of buckling loads in actual trusses. For these results to be applicable to any one truss design, however, the tests would have to be conducted with the proper combination of axial load and bending moment. Regardless of the method chosen for individual joint tests, full-scale truss tests should be used to verify that joint test boundary conditions are reasonable.

Future efforts to optimize the performance of trusses with square-end webs should focus on two goals. The first is to minimize the bending moment at the joint where a square-end compression web meets a chord. This can be accomplished by analyzing various combinations of web lengths and angles using an accurate analysis method. The second goal is to increase plate buckling capacity and decrease the variability. Fabricating plates with stiffeners along one or both edges may accomplish this.

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# Appendix: Standard Tension Tests and Plate Stiffness Parameters

Tensile tests were conducted with the 20-gauge (0.91-mm) plates used in this study. The results of these tests are needed for computer modeling of plated connections as semirigid nonlinear joints.

The plate–wood stiffness equation originally developed by Foschi (1977) uses three experimentally determined parameters: *K* is the initial stiffness (lb/in), P1 is the final stiffness (lb/in), and P0 is the load axis intercept of the final slope (lb):

$$F=(P0 + P1 \cdot \delta \cdot (1 - \exp(-K \cdot \delta/P0)) \qquad \text{(A1)}$$

where *F* is the load (lb) and δ is the slip (in.). All values are usually expressed on a per-tooth basis. It is more appropriate in some modeling situations to set the final stiffness to zero; this puts an upper limit on the force which the plate can resist.

The specimens used for these tests are defined in Canadian Standards Association S347 (CSA 1980) and include four basic joint configurations: AA, AE, EA, EE. The first letter refers to the orientation of the plate's principal axis with respect to the load (A for pArallel, E for pErpendicular); the second letter refers to the orientation of the wood grain with respect to the load. The relevant ASTM standard (ASTM 1989) defines only one joint configuration.

Plates were placed asymmetrically to encourage tooth withdrawal on one side of the joint, called the "target" side, and to minimize deformation at the other side, or "control" side. The degree of asymmetry was not considered to be large enough to neglect slip on the control side of the joint for the EA and EE specimens so wood screws were used to reinforce these joints. All fabricated specimens remained in the conditioning room at least 14 days prior to testing. Ten specimens were tested in each of the four configurations.

Specimens were tested in a screw-type test machine operating at a head speed of 0.04 in/min (1.02 mm/min). The tensile load was transmitted to the specimens with through-bolts with pressed steel shear-plate timber connectors. Deformation was measured with two LVDTs, one on each edge of the wood members; the average of the two was used in the following calculations. After correcting the initial stiffening portion of the data, load data for all specimens in a configuration were averaged at discrete deformation levels, creating an average load-displacement curve for each configuration. The parameters in Equation (A1) were found by a least-squares regression of these average curves, using data up to the peak load.

The parameters for the best overall fit and for the best fit with P1 set to zero are shown in Table A1. These are "per-tooth" values with approximately 10 teeth/in$^2$ (1.55 teeth/cm$^2$) on these plates. Note that the orientation of the wood grain with respect to the load has a much greater effect on the stiffness parameters than the orientation of the plate with respect to the load.

Table A1—Plate–wood stiffness parameters[a]

| Joint configuration | Best overall fit | | | Best fit with P1 = 0 | | |
|---|---|---|---|---|---|---|
| | *K* (lb/in) | P0 (lb) | P1 (lb/in) | *K* (lb/in) | P0 (lb) | P1 (lb/in) |
| AA | 7,200 | 44 | 320 | 6,600 | 54 | 0 |
| AE | 3,900 | 32 | 230 | 3,700 | 46 | 0 |
| EA | 6,600 | 44 | 10 | 6,680 | 44 | 0 |
| EE | 4,100 | 26 | 470 | 3,500 | 37 | 0 |

[a]Values are per tooth; 10 teeth/in$^2$.