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Experimental Assessment of Wood Trusses With Square-End Webs

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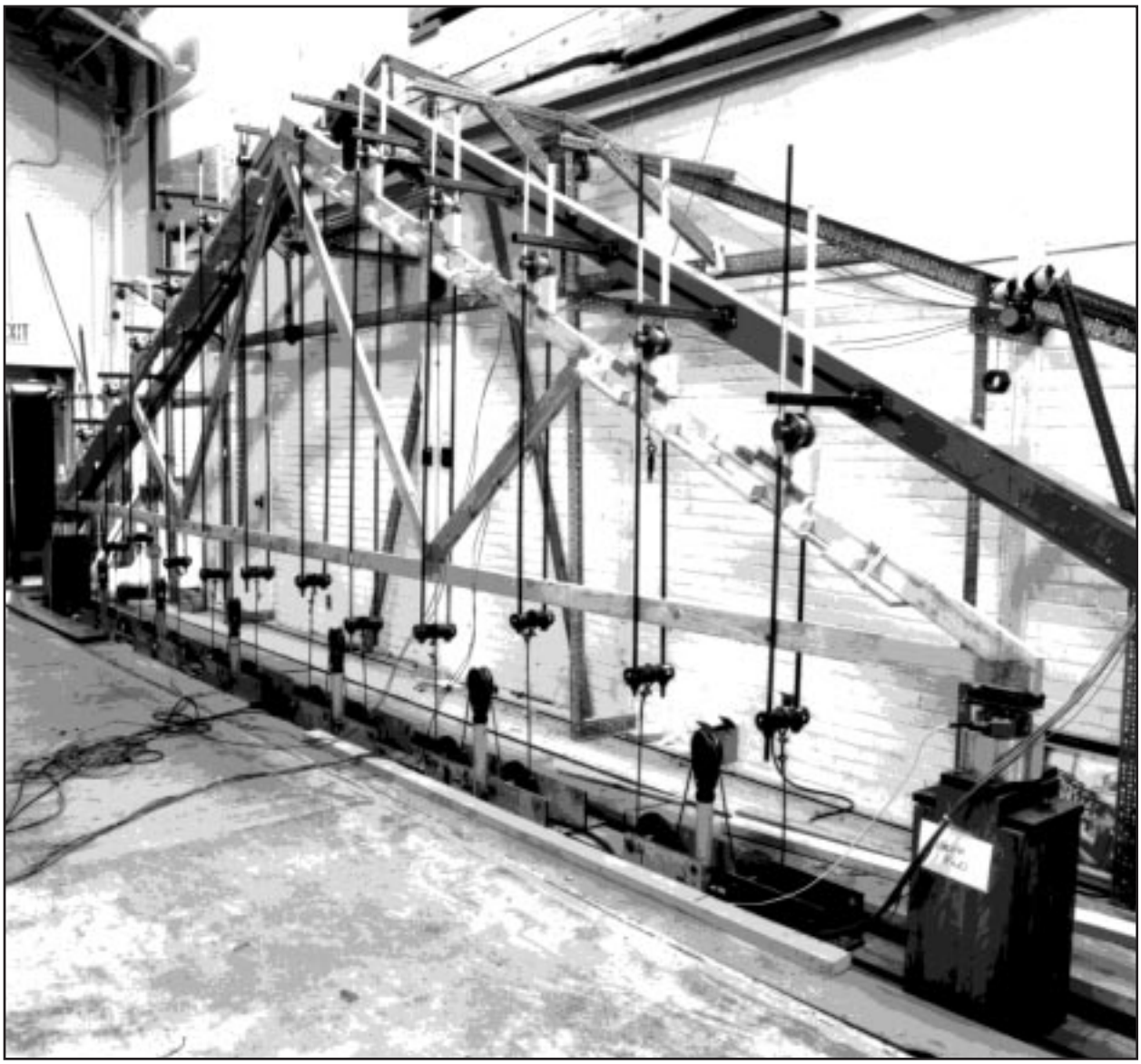

Abstract

A return to square-end truss webs may provide an economic advantage by reducing machining, handling, and waste disposal costs. The objectives of this study were to (1) provide a preliminary assessment of the structural feasibility of using square-end webs and (2) establish a database for a truss model that is capable of predicting square-end web connection failures and evaluating their effect on truss stiffness and strength. Nine trusses of a Fink configuration were tested to evaluate the effects of square-end web connections on truss stiffness and strength. The square-end webs had no significant effect on truss stiffness. Low strength values measured for the square-end web trusses were partially due to insufficient plate area on tension web connections rather than buckling of plates at compression webs. Results suggest that square-end web joints are structurally feasible and deserve further attention, even though their effects on truss stiffness and strength vary with truss configuration and load distribution.

Keywords: Truss, joint behavior, square-end webs, strength, stiffness, member force

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Introduction

For the past 40 years, developments in both computer and sawing technologies have influenced the shift away from the use of square-end web trusses to dual-end fitted webs. Precision-fit joints give the appearance of quality fabrication and optimize compression-joint load capacity by taking advantage of wood-to-wood contact area. Although appearance is an important consideration from a marketing perspective, it is not certain that the additional joint strength is always worth the extra cost. Square-end webs have the economic advantages of requiring less expensive equipment, less handling, and generating less waste than fitted webs. Strength differences can be compensated by use of larger connector plates.

Square-end webs provide an alternative for conventional light-frame truss fabrication. As a result of the relatively low levels of member stress, it is possible to design the metal-plate connections so that web forces are transferred without the benefit of wood-to-wood bearing. For webs stressed in tension, this is simply a matter of increasing connector plate size to give the proper number of teeth in the web and chord members. For compression webs, however, connector buckling resistance and consequences must be considered in the truss design. If the metal-plate connector can be proven reliable in carrying web-chord interaction forces, it may be reasonable to consider the economic advantages of buying square-end “commodity” webs, mill “shorts,” or mass producing webs using a “bunk” cutter.

The economic advantage of square-end webs compared with fitted webs is strongly dependent on material resource, labor cost, and marketing. Increasing concern for waste reduction and efficient use of materials weighs in favor of square-end webs. In developing countries where labor costs are low relative to equipment costs, the square-end web would have a definite fabrication cost advantage over fitted webs, but it may not be a large enough advantage to counter the aesthetic appeal of fitted joints in all applications.

To evaluate the feasibility of producing and using commodity square-end webs for light-frame truss fabrication, the tools necessary to assess the cost of fabricating reliable connections must be developed. These tools include a structural performance database for square-end web connections, a full-scale square-end web truss, and an analytical model capable of assessing possible buckling failures and their effect on truss load capacity. This paper discusses the design, fabrication, and testing of light-frame trusses to provide a basis for assessing the effect of square-end webs on truss stiffness and strength.

Literature Review

Prior to the introduction of the metal-plate connector by Carol Sanford in 1952 (Meeks 1979), trusses constructed using wood gussets often contained square-end web members. Unlike metal-plate connectors, wood gussets usually have sufficient stiffness and cross section to carry compressive loads from truss webs, without approaching a critical buckling load.

Few, if any, publications discuss the effects of square-end webs on truss stiffness and strength. Publications dealing with fabrication and testing of square-end web trusses (Midwest Plan Service 1981; Jones and Percival 1959; Percival 1973, 1975; APA 1964) discuss design considerations and test performance of plywood gusseted trusses. Plywood gussets normally have a large enough radius of gyration to resist buckling across gaps, resulting from the square-end web connections on common truss configurations. Failures of these trusses commonly occurred in the high stress regions, such as the heel connection or the top-chord end panel, rather than in the web-chord connections.

The Truss Plate Institute (TPI) wood truss design specification allows wood-to-wood bearing stress to account for transferring as much as half the force between compression web

and chord members (TPI 1985)[1]. Without the advantage of wood-to-wood bearing, a larger plate, capable of transferring 100 percent of the web force, would be required. This would increase the degree of end fixity on the webs, resulting in an increase in the moment carried by the web.

To experimentally quantify the effects of increasing web-end fixity requires some means of measuring the moment and axial force on the members joined by the connector. Inlow and others (1988) reported limited success with using strain gauges on truss members as a means of monitoring effects of changes in joint fixity.

Metal-plate connections on parallel-chord trusses were instrumented with strain gauges in an attempt to locate a single point that could be considered an analog "joint." Stress shifts as a result of gap closure, tooth withdrawal, and plate distortion made it difficult to locate a single point. Strain gauges placed directly on the wood gave linear load–strain response generally consistent with computer analog estimates based on design modulus of elasticity (MOE) values. However, some variations in member strain for different trusses of the same configuration and at the same load were attributed to variations in wood fiber MOE between early and late wood.

King and Wheat (1987) tested three replications of each of three parallel-chord truss configurations to compare analytical and measured estimates of member stress distribution. Strain gauges were placed on select members of each truss to give estimates of member stress distribution. After testing each truss under uniform load and collecting information on truss deflection and member strains, the truss members were cut out for determination of member properties necessary to convert member strain measurements to member stresses. To alleviate some problems with spring and summer wood variations, King and Wheat used 1.2-in.- (30-mm-) long gauges with polyester backing. The relatively low-resistance (120 Ω) gauges were wired as half a Wheatstone bridge with a compensating gauge on an unstressed piece of wood. Results obtained from King and Wheat's study provided a basis for judging the efficacy of various analog assumptions used to estimate truss member forces and deflections.

Several problems are associated with the use of strain gauges on wood. Some have to do with the attachment, calibration, and heat dissipation. These problems can be reduced or eliminated by the use of strain gauge extensometers. These can be designed to be calibrated prior to mounting on the wood and removed for reuse when the test is completed. Loferski and others (1989) described the fabrication of a simple clip-on extensometer fabricated by placing strain gauges on either side of a curved piece of spring steel. By placing two strain gauges on top and two on the bottom of the spring steel, in the form of a Wheatstone bridge circuit, Loferski reported a resolution as low as 5 microstrain.

The effect of square-end web connections on truss stiffness and strength is likely to vary with plate configuration. In a study by Stahl and others (1994), the effects of square-end webs on plate connection performance were evaluated by testing individual joints under a variety of load conditions and comparing their performance with fitted joints. The plate-buckling failure mode is difficult to predict and capacity may be highly variable, depending on joint angle, plate configuration, and boundary conditions. In some cases, light gauge (20 gauge, 0.04-in.- (1-mm-) thick) plate steel, commonly used for light-frame trusses, appeared to give a sufficient strength–safety margin. Depending on the boundary conditions and load eccentricity, it may be necessary to use heavier gauge plates or even custom-made buckling resistant plates to meet truss design requirements. To make a valid assessment of the value of individual joint tests, a comparison must be made of simulated and actual truss joint boundary conditions.

Objectives and Scope

This paper discusses the truss testing phase of a three-part study that includes an evaluation of square-end web joints (Stahl and others 1994) and development of an analytical model to simulate the effects of plate buckling on truss deflection and member stresses (Stahl and others, in press).

This study had two primary objectives: (1) provide a preliminary assessment of the technical feasibility of square-end webs and (2) establish a verification database for a truss model that is capable of predicting square-end web connection failures and evaluating their effect on truss stiffness and strength. The technical feasibility assessment is based on a comparison of square-end webs and fitted web truss behavior. The verification database required an assessment of truss member axial and bending stresses as the trusses were loaded to failure.

Tests were planned to permit comparative evaluation of full-scale trusses having three different joint configurations. Trusses fabricated using fitted webs, connected using 20-gauge plates, provided a basis for the evaluation of those fabricated using square-end webs. The square-end web trusses were fabricated using two plate types. Three square-end web trusses were fabricated using light-gauge (20-gauge) metal-plate connectors, and three were fabricated using heavy-gauge (16 gauge, 0.06-in.- (1.5-mm-) thick) metal-plate connectors. All trusses were tested to failure using a uniformly distributed load over a 28-ft (8.5-m) span.

[1] ANSI/TPI [in preparation]. National design standard for metal-plate-connected wood truss construction. Truss Plate Institute, Madison, WI.

Materials and Methods

Joint behavior and member stresses, evaluated in full-scale trusses as they were loaded to failure, provided a basis to evaluate the efficacy of individual joint tests reported by Stahl and others (1994) as well as computer model predictions of square-end web effects on truss capacity (Stahl and others, in press). Chord and web member strains were monitored to permit member force evaluations. Deflection measurements provided a comparative assessment of truss stiffness; truss reaction forces provided a measure of load amplitude and symmetry. Truss failure sequence observations provided added verification for predicting critically stressed members.

Materials

Test trusses were fabricated using Southern Pine lumber. The material was obtained from a Wisconsin truss fabricator in three lengths: twenty 16-ft (4.9-m) pieces graded No. 2D were used for top chords; twenty 14-ft (4.3-m) pieces and twenty 12-ft (3.7-m) pieces graded No. 2 were used for bottom chords and webs, respectively.

The metal-plate connectors used in this study (App. A) were obtained from an industry cooperator. All plates were made using galvanized steel. In all cases, joints other than web-to-chord connections were made using 20-gauge plates. Web-to-chord connections were made using either 20- or 16-gauge plates. In all cases, plates were sized to provide the required plate area following established design standards (TPI 1985). For fitted-web trusses, established design standards permit 50 percent of the component of web force normal to the surface of the chord to be carried by the bearing area. This reduces the size of the required plate connector. For the square-end web trusses, 3-in.- (76-mm-) wide plates were used, and plate area was designed to carry the full axial force in the web.

Trusses

This study involved one basic truss configuration (Fig. 1). This 28-ft (8.5-m) span Fink design with a 6 in 12 top-chord slope was proposed by industry cooperators. It was designed to allow standard 4- and 8-ft (1.2- and 2.4-m) square-end webs to be substituted for fitted webs. This resulted in a slight modification of conventional design practice in that the top-chord web connection was not located midway between head and peak. The top-chord panel adjacent to the peak was 33 in. (838 mm) longer than that adjacent to the heel.

These trusses were designed to carry a live load of 30 lb/ft^2 (1.4 kPa) and dead loads of 7 lb/ft^2 (335 Pa) top chord and 10 lb/ft^2 (478 Pa) bottom chord when spaced 24 in. (610 mm) on center. This gave a total design load of 2,630 lb (11.7 kN).

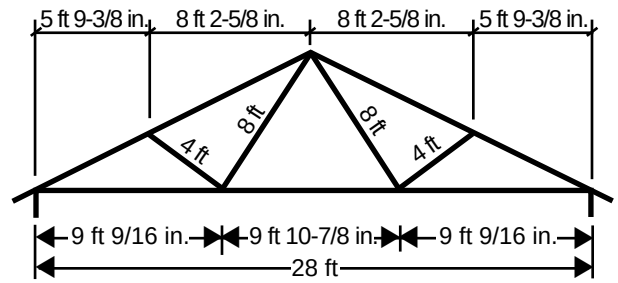

Figure 1—6/12 Fink truss configuration used for test trusses.

Test truss variations focused on the web-to-chord connections. Three trusses were tested in each of three following connection categories:

- F: Conventional Fitted web
- L: Square-end web connected using Light-gauge (0.04-in.; 1-mm) plates
- H: Square-end web connected using Heavy-gauge (0.06-in.; 1.5-mm) plates on the compression-web connection to the top chord

Methods

Prior to truss fabrication, MOE of each of the 60 pieces of lumber was determined using a commercially available dynamic MOE measuring device. The lumber was sorted according to MOE and split into three MOE classes. Within each connection category (F, L, H), one truss was fabricated from lumber in each MOE class, giving three truss stiffness classifications (high, medium, low).

Trusses were tested using a simulated uniformly distributed load. Although the design total load of 2,600 lb (11.5 kN) included a bottom-chord dead load of 20 lb/ft (292 N/m), all loads were applied to the top chord. The loads were applied at 12 discrete points spaced 25 in. (635 mm) on center over the span of the truss, beginning 30.5 in. (775 mm) from each end.

The test rack used to test the trusses is shown in Figure 2. This rack was designed to limit horizontal components of applied load by using rolling cylinder load heads bearing on the horizontal surface of wedge-shaped distribution blocks. This detail permitted the truss to expand horizontally when vertical load was applied, without inducing a horizontal force component on the chord. At midspan, loads were applied on either side of the truss peak through fixed load heads and rectangular distribution blocks to restrain in-plane horizontal translation of the truss. Loading frames, used to transfer load to the loading cylinders, were held in plane by steel alignment arms anchored to a wall of the Engineering Mechanics Laboratory at the USDA Forest Service, Forest Products Laboratory (FPL). The test rack is discussed in detail in Appendix B.

Figure 2—Truss test frame set-up to test 28-ft (8.5-m) span Fink trusses under simulated uniform load. Individual truss load points were 25 in. (635 mm) apart. Each was distributed to the top chord through a 12-in.- (305-mm-) long wedge-shaped block.

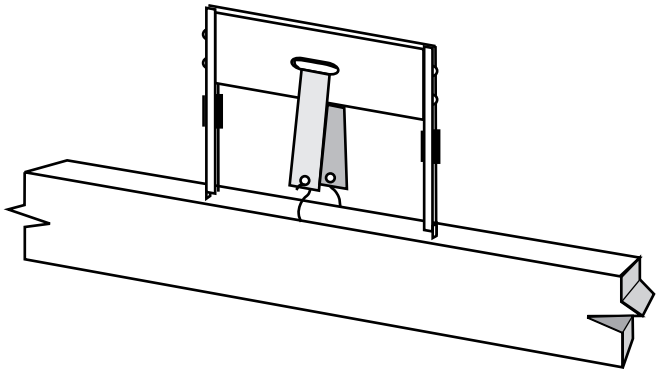

Figure 3—Portal frame extensometer.

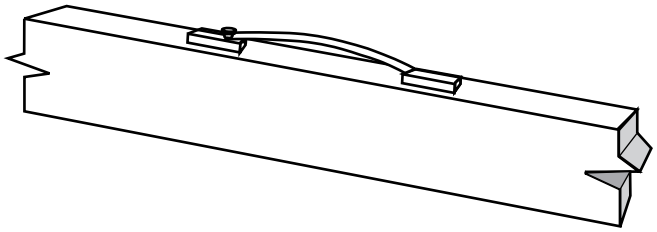

Figure 4—Spring-steel extensometer.

Top chords were fitted with buckling restraint saddles to prevent out-of-plane deflection. These saddles comprised 10-in.- (254-mm-) wide plywood plates with 4.25-in.- (108-mm-) long blocks screwed and glued to the surface to serve as lateral restraints on either side of a 1.5-in.- (38.1-mm-) wide slot. The saddles were held on to the bottom side of the chord members, between web connections, with elastic bands (Fig. 2).

Data Collection

For each truss test, 17 data channels were scanned at 1-s intervals. Output from these channels was recorded on a floppy disk. Truss reactions were recorded using two 10,000 lb (44.5 kN) capacity load cells. Output from these load cells was used to control the loading rate and holding periods. Seven linear rotational potentiometers (LRPs) were used to measure vertical truss deflections at each of the five truss node points and the reaction points. The LRPs were mounted on a frame above the truss. Nylon fishing line connected the truss node point to a tracking wheel in front of the LRP, which turned as the truss deflected. The remaining 8 channels monitored the clip-on extensometers used to measure axial strain on the top and bottom edges of the compression webs and top-chord panels adjacent to the peak. These measurements were used to calculate axial and bending stresses in these members.

Two types of clip-on extensometers were used to measure member strain (App. C). One clip-on extensometer had the configuration of a portal frame (Fig. 3), with strain gauges on the surfaces of the frame arms. The other extensometer comprised a single piece of light-gauge spring steel, with strain gauges on top and bottom surfaces, held between reaction blocks attached to the surface of the wood (Fig. 4). Extensometers were located on the top and bottom edges of top chords and compression webs. On the top chords, extensometers were centered on a line 24 in. (610 mm) above the intersection of compression web and chord centroidal axes. On the compression webs, extensometers were centered on a line 18 in. (457 mm) below the bottom edge of the top chord on the web centroidal axis.

The portal gauges did exhibit some advantages in terms of resolution, consistent gauge length, strain measurement on the surface of the wood, and a linear load-displacement curve. The portal gauges had a measurement accuracy of 0.0001 in. (0.0025 mm) compared with 0.001 in. (0.025 mm) for the spring gauges. Arms of the portal gauges were separated by 2 in. (51 mm), and a template was used to slightly indent the surface of the wood to provide a seat for the gauge arms and ensure consistent gauge length. The spring-steel gauge was held in place between two reaction blocks by its own reaction force. Gauge length for the spring-steel clips was measured as the distance between the screw holes used to fasten the reaction blocks in place; these measurements varied slightly with each application. Holding the spring-steel gauge above the surface of the wood in the reaction blocks required an interpolation of measured strains to estimate the value at the surface of the wood. Finally, although the spring

gauges gave repeatable readings, their output voltage compared with displacement curves was slightly nonlinear and had to be calibrated using a quadratic equation.

Details of the extensometers are given in Appendix C and in a report by Loferski and others (1989).

Test Procedure

Each truss was tested in three stages. The first stage was simply a preload to tighten the system. The second stage was a load to the total design load, and the third stage was load to failure. In each case, the hydraulic pressure driving the loading rams was controlled by a hand-operated servo valve on the hydraulic pump. To maintain some control on loading speed, 3-min holds were used at half-design load increments.

The first stage load was 1,300 lb (5.8 kN), about half the design load. Beginning with a dead load of approximately 480 lb (2.1 kN), an additional 820 lb (3.6 kN) was applied slowly, held for less than 1 min, and released.

The second stage was a two-step load to full-design load. This included two 3-min holds: one at half design load and one at design load. After the last hold, the load was released and the truss was given a 5-min recovery before being loaded again.

The third stage was the load to failure. In this case, load was slowly applied in 1,300-lb (5.8 kN) increments to the point of failure. As the truss was loaded, observers noted obvious signs of stress, such as noises, plate distortions, or members bending out of plane.

Compressive MOE

To improve accuracy of the member force determinations, compressive MOE values were determined for lumber samples cut at the gauge location after the trusses were tested. As a result of the short gauge length of the extensometers and potential variability of MOE along the length of a piece of wood, the full-span MOE values determined for each piece of lumber were not considered sufficiently precise to convert the measured strain to a member stress. Therefore, test blocks, 8 in. (203 mm) long, were cut at each gauge location. The blocks were carefully cut to ensure that the end cuts were at 90° angles to the longitudinal axis of the block. Slight grooves were placed in the wide faces of each block, 2 in. (51 mm) on either side of the centerline. Brackets, which held the linear variable differential transducers (LVDTs) that measured displacement, were held in place on knife edges in these grooves. The blocks were tested in compression parallel to the grain to a maximum load of 5,000 lb (22.3 kN) to obtain a load deformation plot. Figure 5 shows the test set up.

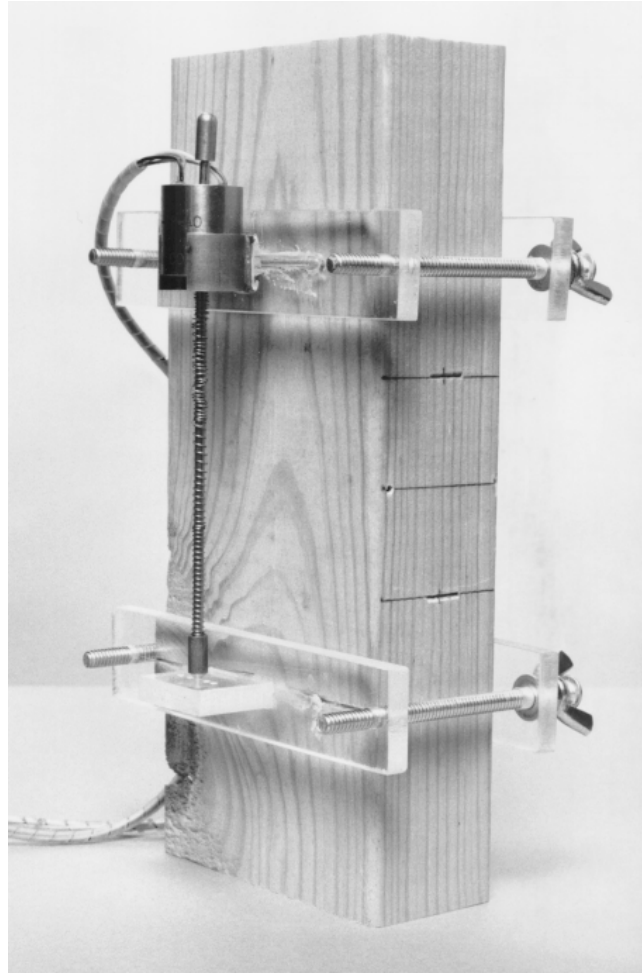

Figure 5—Compressive MOE test block configuration.

Results

Information recorded in this study is intended to provide a basis for comparing the structural capacity of square-end web trusses with that of conventional fitted web trusses. Results include an observation of lumber and connection properties, an evaluation of truss fabrication quality, and data from which to judge the effects of web connections on stress distribution within the trusses and on truss failure mode.

Lumber

The dynamic bending and compressive MOE values were in close agreement, on the average. Compressive and bending MOE ratios ranged from 0.84 to 1.25, with an average of 0.98. Average bending MOE values were 1.5×10^6 lb/in^2 (10.4 GPa) for the No. 2D material and 1.38×10^6 lb/in^2 (9.5 GPa) for the No. 2 material. Table 1 lists the MOE values determined for each truss member; Figure 6 is a comparison of compressive and bending MOE values.

Table 1—Measured MOE for each truss member. Short-column sections were cut from top chord and web members to determine compressive MOE values at extensometer locations

	Member MOE flatwise bending (short-column compression) ($\times 10^6$ lb/in^2)[a]					
	Top chord		Bottom chord		Web	
Truss	South	North	South	North	South	North
F1	1.67 (1.63)	1.93 (1.55)	1.5	2.00	1.48 (1.54)	1.89 (1.87)
F2	1.47 (1.31)	1.50 (1.25)	1.34	1.40	1.33 (1.30)	1.39 (1.30)
F3	1.32 (1.33)	1.38 (1.63)	1.20	1.27	1.19 (1.28)	1.25 (1.39)
L1	1.63 (1.44)	1.89 (2.20)	1.47	1.77	1.46 (1.43)	1.76 (1.65)
L2	1.43 (1.70)	1.49 (1.70)	1.32	1.38	1.30 (1.04)	1.37 (1.50)
L3	1.26 (1.01)	1.35 (1.49)	1.18	1.25	1.11 (1.23)	1.24 (1.19)
H1	1.56 (1.70)	1.83 (1.90)	1.42	1.66	1.41 (1.31)	1.57 (1.37)
H2	1.41 (1.36)	1.49 (1.54)	1.30	1.37	1.28 (1.30)	1.36 (1.35)
H3	1.18 (1.07)	1.34 (1.19)	1.05	1.23	1.01 (0.96)	1.22 (1.20)

[a] 1 lb/in^2 = 6,894 Pa.

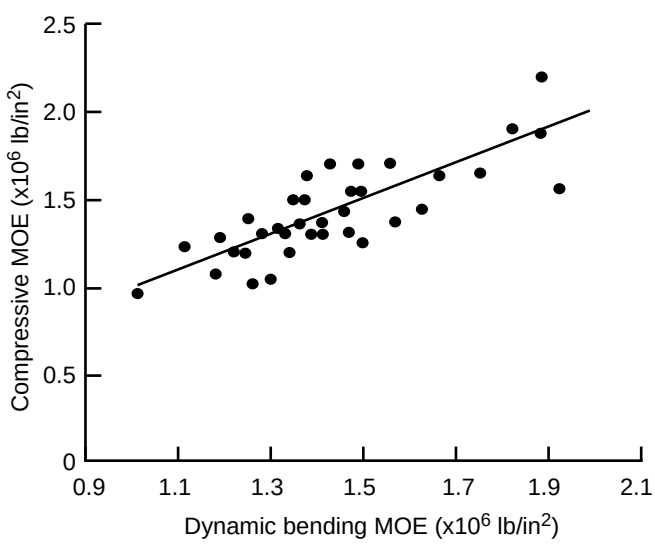

Figure 6—Compressive MOE measured for chords and compression web members compared with dynamic bending MOE.

Trusses

Although all plate connectors were designed to give the correct plate-contact area at each joint, plate placement problems seemed to be more prevalent in the square-end web trusses. Of the six square-end web trusses tested, five had peak plates placed to give greater contact on the webs than on the chords. One of these also had a badly misplaced heel plate. Of the three fitted web trusses, one had a peak plate misplaced in favor of greater contact on the webs and a 0.24-in. (6-mm) gap between chord members at the peak. In one case, truss F2, a web member was twisted, resulting in a poorly embedded plate at the top-chord joint.

Load Symmetry

Truss reactions supported the premise that load was symmetrically distributed to the north and south chords. As the trusses were loaded from dead load to 1,000 lb (4.4 kN), several trusses exhibited reaction loads that varied by as much as 20 percent from the north to south reactions. However, beyond 1,000 lb (4.4 kN), the reactions converged to values that varied by less than 6 percent.

Nodal displacement measurements also support symmetric load distribution. For six of the nine trusses, nodal displacements measured on the north and south sides of the peak were in close agreement. Trusses L2, L3, and H1 had bottom-chord node displacement measurements that were smaller on the north than on the south, although their top-chord displacements were essentially the same. Greater displacements measured for the top-chord nodes than the bottom-chord nodes for these trusses, combined with the north and south deviations, suggest that LRPs appeared to be a problem at these two locations.

Load-Displacement Characteristics

Node displacements measured using the LRPs, driven by nylon fishing line, introduced discrepancies that required correction for the analysis of truss deflections. Figure 7 shows an extreme example of these discrepancies; the average load-displacement curve for the two bottom-chord nodes was compared with that for the two top-chord nodes. Note that the top-chord nodes, which are closer to the reactions, had a greater displacement than the bottom-chord nodes and the hysteresis, apparent for both plots, was worse for the bottom chord than for the top. These trends, characteristic of all the truss tests, were attributed to line flexibility and LRP rotational resistance. As the trusses began to deflect, the force on the nylon line served to stretch the line to the point that the line force exceeded the static friction in the LRP and inertia of the counter weight used to hold it taut. Beyond that point, displacement increased as a linear function of applied load. Because the lines that attached the bottom-chord nodes to the LRPs were about three times as long as those attached to the top-chord nodes, the amount of stretch in these lines was

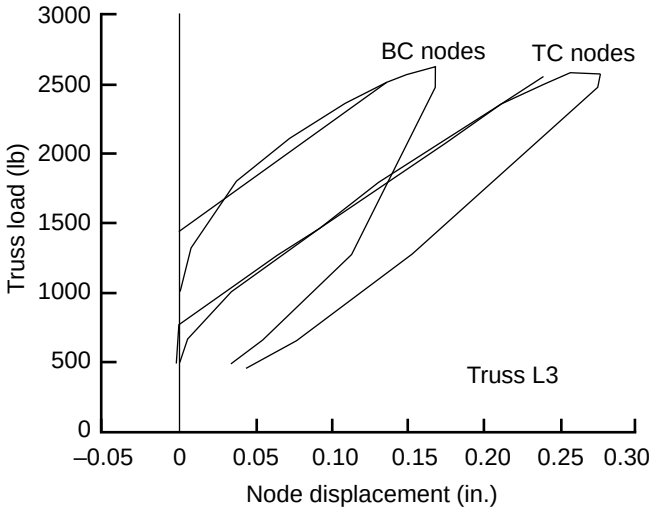

Figure 7—Load deformation curves for truss L3 showing hysteresis typical of the LRP deflection measurements. These plots represent average curves for the north and south nodal displacements. In most cases, north and south measurements were coincident; however, for trusses L2, L3, and H1, the south bottom-chord node displacement was greater than the north bottom-chord displacement.

expected to be about three times as great if the counter weight and rotational resistance were the same for all LRPs.

Unfortunately, there was some variation in LRP rotational resistance, counter weight, and/or line MOE. For a number of trusses (L2, L3, H1), some discrepancy was also noticed between the north and south bottom-chord displacements that did not appear to be justified on the basis of member MOE variation and measured truss reactions.

Member Strains

Truss member strain readings exhibited characteristics that suggested possible load asymmetry. However, this was not supported by the reaction and nodal displacement measurements. Although most strain gauge readings taken up to design load appeared to be linear and within expected ranges, readings occasionally showed one or two members to be stressed much more than could be predicted on the basis of static analysis and measured reactions. Member MOE values determined from the short blocks taken from the compression webs and top chords averaged 94 percent of those obtained by the long-span bending test, so local MOE variation was ruled out as the cause of this phenomena. The problem appeared mostly in readings from the spring-steel extensometers and was attributed to a combination of electronic noise in the system and possible slight misadjustment in the reaction blocks.

Insufficient voltage gain limited the resolution of the four spring-steel clip-on extensometers and made “noise” in the system more problematic for these readings. The problem with these devices was the use of 90° angle rosette gauges rather than four individual gauges oriented parallel to the spring steel. Thus, the voltage gain from a given displacement was not as great as that obtained from the portal gauges that had four gauges oriented parallel to the principal stress.

As a result of better resolution, portal gauges were used to measure web strains after the first two tests. For the first two tests, portal gauges were used on the south compression web and top chord, and spring-steel extensometers were placed on the north compression web and top chord. However, the increased strain levels in the chords were less likely to be drowned out by the inherent noise in these gauges, so gauge locations were changed to obtain greater sensitivity on the webs.

Truss Failures

Maximum loads ranged from 3,400 to 6,400 lb (15 to 28 kN) (Table 2). Although the fitted-web trusses averaged 1,000 lb (4.4 kN) greater maximum load than the square-end web trusses, it was not obvious that the difference was due to the compression web-joint configurations. The fitted-web trusses exhibited greater quality fabrication in terms of chord quality and plate placement. Four of the six square-end web truss failures were due to misplaced plates and the other two failures were due to critical knots in chord members.

Three truss failures were attributed to wood failure. Trusses F3 and L2 had bending failures in the top chord and truss H3 failed in the bottom chord, 2 ft (610 mm) from the north heel joint. Truss F3 failed as a result of bending stress in the middle of the upper panel of the top chord close to the load. Truss L2 failure was associated with a knot on the tension edge. Truss H3 had a misplaced peak plate similar to those observed in trusses H1 and H2. However, by reinforcing this connection with wood screws through the plate into the top chords, the failure was forced to occur in the bottom chord, close to the north end heel connection. This failure was associated with a spike knot that occupied at least 50 percent of the cross section at the point of failure.

Compression Web Connections

The light-gauge plates used on the compression webs at the top-chord connection (trusses L1, L2, L3) displayed varying degrees of buckling. Although some plate deformation was apparent during the truss loading, plate buckling occurred as part of the failure event and could not be clearly defined as cause or effect. Post-failure inspection showed that for the L1 truss, both compression web chord connections had buckled plates. For truss L2, slight buckling of these plates occurred along the upper wood-bearing edge but no buckling along the gap edge. Truss L3 also showed signs of plate buckling, but not as pronounced as L1. None of the H category trusses showed any sign of buckling of the compression web connector plate.

Table 2—Stiffness of the truss lumber was compared with truss stiffness and strength. Truss stiffness is the slope of the average load displacement for the two top-chord nodes and the peak. The truss design load in each case is 2,640 lb (11.7 kN).

Truss	Member MOE class	Truss stiffness (lb/in) [a] Stage 1	Stage 2	Truss maximum load (lb)[b]	Truss failure mode
Conventional fitted-web trusses					
F1	a	13,700	11,600	6,100	Heel joint failure
F2	b	10,400	9,420	6,400	Heel joint failure
F3	c	9,440	8,650	5,200	Top-chord bending failure
Average		11,200	9,890	5,890	
Square-end webs with light-gauge plates					
L1	a	12,500	13,200	5,200	Peak plate pulled from top chords
L2	b	9,430	9,920	5,100	Wood failure associated with spike knot on top chord, upper panel
L3	c	7,600	8,070	3,400	Misplaced heel plate pulled out
Average		9,840	10,380	4,600	
Square-end webs with heavy-gauge plates					
H1	a	11,800	12,700	5,100	Misplaced peak plate pulled from top chords
H2	b	10,400	10,100	4,700	Misplaced peak plate pulled from top chords
H3	c	8,710	8,930	4,800	Wood failure associated with knot in bottom chord close to north end heel.
Average	10,300	10,600	4,900		

[a] 1 lb/in = 1.751×10^2 N/m.
[b] 1 lb = 4.45 N

Analysis of Results

Truss test results were analyzed to determine the effects of the square-end web connections on global stiffness and strength of the trusses as well as on member stresses. Results show that for the truss and load configuration considered in this study, the square-end webs had a minor effect on truss strength and stiffness compared with effects of lumber stiffness and plate-holding capacity. Larger plates used for the square-end web connections appeared to transfer greater moments into the webs, but the resulting greater web stresses did not appear to significantly affect truss strength or stiffness.

Truss Stiffness

Within the design load range, lumber stiffness classification appeared to have a greater effect on truss stiffness than the web-chord connections. For purposes of comparison, truss stiffness values given in Table 2 were determined as the slope of a load-deflection plot, representing the average measured displacements of the peak and two top-chord nodes. As a result of the discrepancies associated with stretch in the nylon line, we did not use bottom-chord node displacements to evaluate truss stiffness.

Truss stiffness was assessed as the slope of a straight line fit to the linear portion of the average load-displacement curve. The displacement intercept of this fitted line along an axis through the truss dead load (500 lb (2.2 kN)) gave a measure of the displacement masked by stretch in the line. Line stretch values ranged from 0.02 to 0.07 in. (0.5 to 1.8 mm) for the top-chord nodes, but the LRP was within 24 in. (610 mm) of the node. Load and deflection data for each truss are given in Appendix D.

Truss stiffness values (Table 2) varied more with stiffness classification (high, medium, low) than with web-connection category. In comparing values for the load to design (stage 1), it appears that it may be necessary to use a heavier gauge plate as well as a larger one to mimic the performance of a fitted-web truss. However, on the second loading (stage 2), the square-end web trusses became stiffer and the fitted-web truss became less stiff. Given the sample sizes tested, a significant difference in truss stiffness for the three web-joint categories cannot be concluded.

Truss Strength

Trusses exhibited a range of maximum loads. As shown in Table 2, all fitted-web trusses and one square-end web truss (L1) had maximum loads greater than or equal to twice their design value of 2,600 lb (11.5 kN). In general, low strength values of the square-end web trusses were attributed to the quality of construction rather than gaps in the compression web connections. The lowest strength truss (L3) failed at 3,400 lb (15 kN) because of a misplaced heel plate, and three others failed at the tension web connection at the peak. The remaining two square-end web truss failures occurred at knots: one grade-limiting spike knot in the end panel of the lower chord and one inside edge knot at mid-panel of the top chord.

The nonsymmetric placement of the top-chord node may have provided some benefit to the square-end web trusses. A load uniformly distributed along the top chord placed greater load on the longer upper panel than on the lower panel. This caused moment at the web top-chord connection to rotate the top chord in a direction to open the joint gap, placing most of the connector plate in tension rather than compression. Compressive force in the plate was confined to the "closed" side of the joint where the unsupported plate length was at a minimum. The square-end web trusses fabricated using the 20-gauge plates exhibited only slight buckling of the upper edge of the compression web-chord connection, and this did not appear to have a significant effect on load capacity. The 16-gauge plates did not show signs of buckling. If moment at the web-chord connection had been reversed, plate buckling may have had a stronger influence on failure and maximum load.

Member Stresses

Truss member stresses were determined on the basis of measured strain × MOE. Axial strains were evaluated as the average of the strain measurements taken on the upper and lower edges of the chord. Bending strains were assessed as half the difference between the measured edge surface strains.

In all cases, MOE values used to convert strain readings to member force and moment were those obtained from the short-column stiffness tests (Table 1). Member MOE values determined on the basis of the short-column tests were close to the values measured for full-span bending.

Bending strain at the surface of the wood was measured directly with the portal gauges but not with the spring gauges. For the spring-steel extensometers, the strain-force conversion was adjusted for gauge location with respect to the wood surface. The reaction blocks held the ends of the springs 0.44 in. (11 mm) above the surface. Assuming a linear strain displacement with distance from the neutral axis of the member, the strain displacement at the surface of the nominal 2- by 4-in. (standard 38- by 89-mm) members was calculated to be 0.8 times that measured 0.44 in. (11 mm) above the surface.

Figures 8 and 9 show axial force and moment values as a function of truss load; Table 3 lists values determined at design load. In all cases, member forces and moments varied linearly with truss load up to the design load. Within this range, web axial forces increased at a rate ranging from 20 to 40 percent of that for the chords. Test results showed no marked difference in member axial forces between fitted and square-end web configurations. However, web moments increased with plate size and thickness, despite the large gaps in the square-end web connections.

In some cases, the extensometer readings suggested a load asymmetry not consistent with measured reactions or displacements. For trusses F2, F3, L2, and H2, differences between north and south chord forces suggested load asymmetry. However, the average ratio of north to south truss reactions ranged between 0.98 and 1.016 for all test trusses in the first load stage to 2,600 lb (11.5 kN). Member axial force ratios for these four trusses ranged from 0.63 to 1.9, thus load asymmetry was not the cause of the difference in chord forces.

An evaluation of the change in member force with change in truss load suggests that gauge calibration error was the cause of the discrepancy. The deviations in chord forces were proportional to truss load, making it highly unlikely that they were due to electronic noise. For seven out of the nine trusses, the change in member force with truss load or slope of the force/load curve varied by less than 15 percent from the south to the north side of the truss. For trusses F2 and H2, this variation was more than 70 percent. The problem may have been due to a setup error that affected the extensometer calibration.

In addition to providing some assessment of member stresses, the strain gauge extensometers allowed assessment of load distribution. For most trusses, the strain readings adjusted to give member forces and moments indicated linearity and symmetry of chord and web member force prior to design load. Five of the nine trusses exhibited noticeable nonlinearity in member moment compared with truss load, indicating some degree of member force redistribution prior to maximum load.

Member moments varied much the same as axial forces. For the light-gauge square-end trusses, web moments decreased as the truss load approached the maximum value. This was accompanied by increasing chord bending moment. The web members in the fitted-web trusses never picked up any significant moment. For the heavy-gauge trusses (H2 and H3), the webs carried almost as much moment as the chords.

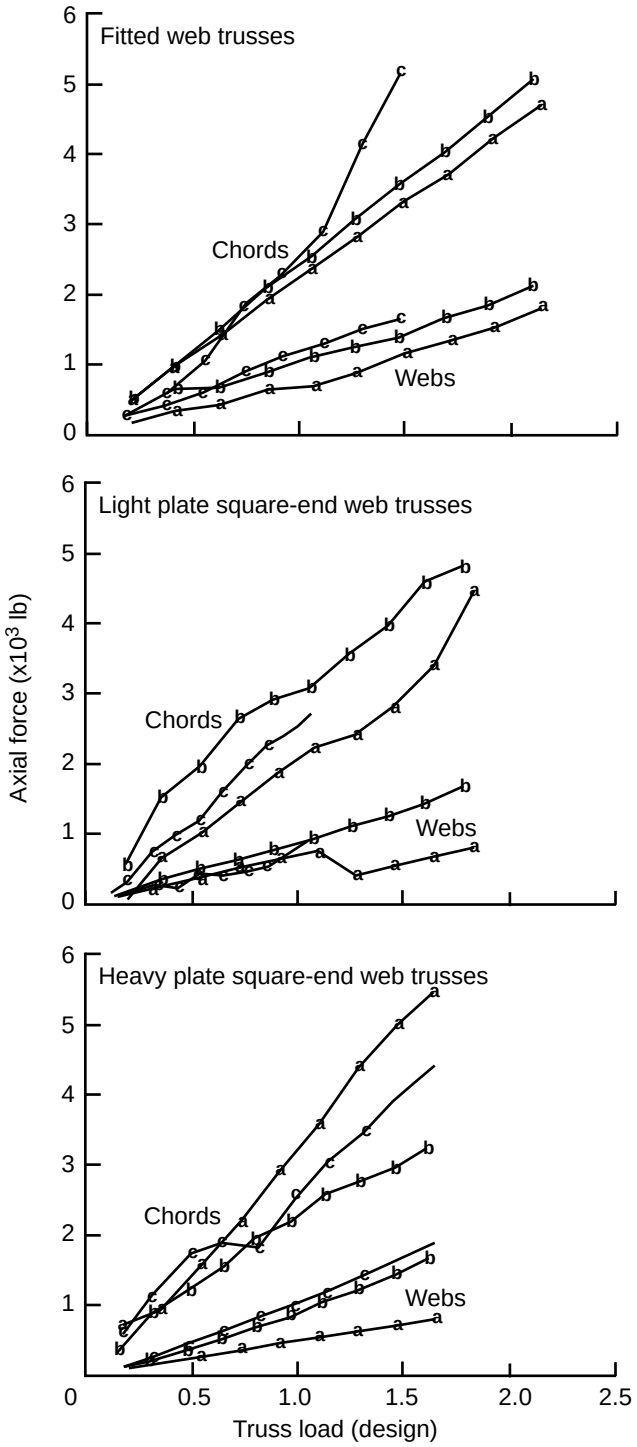

Figure 8—Member axial forces for the three truss categories tested. These plots show average axial force for the chord and web members of the three stiffness classes (high, medium, low) within each truss category.

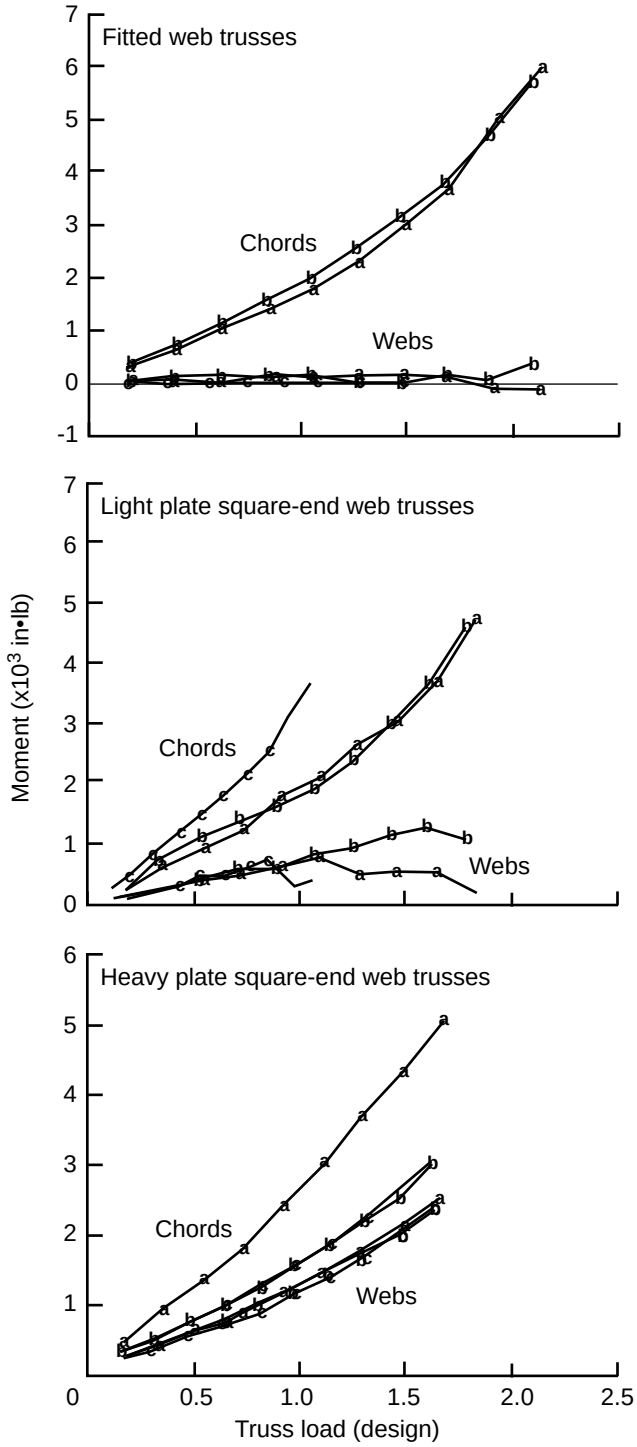

Figure 9—Member moments for the three truss categories tested, showing average moments determined for the chord and web members of the three stiffness classes (high, medium, low) within each truss category.

Table 3—Axial compression forces and bending moments at design load for the chords (SC is south chord; NC is north chord) and webs (SW is southwest web; NW is northwest web)

	Axial force (lb)[a]				Moment (lb·in)[b]			
Truss	SC	NC	SW	NW	SC	NC	SW	NW
F1	2,330	2,130	810	560	1,800	1,600	340	70
F2	1,890	2,960	830	1,300	2,200	2,200	140	160
F3	3,290	1,730	1,930	420	60	60	70	0
L1	1,980	2,250	760	730	2,100	2,100	790	740
L2	3,480	2,640	870	950	2,090	2,090	720	820
L3	2,570	2,570	970	770	3,300	3,300	470	1,230
H1	3,330	3,110	410	730	3,210	3,210	1,350	1,260
H2	2,890	1,700	1,010	910	1,900	1,900	1,250	1,130
H3	2,390	2,940	850	1,270	1,640	1,640	1,660	1,530

[a] lb = 4.45 N.
[b] lb·in = 1.13 N·m.

Discussion

Truss stiffness was not significantly affected by the presence of the square-end webs. Stiffness variations as a result of lumber MOE variations overshadowed any variation caused by the webs or their connections to the truss chords. Average load-deflection curves for the three truss groups (Fig. 10) suggested that the square-end webs had little affect on truss behavior up to the point of failure.

The low strength measured for square-end web trusses was indirectly related to the web-chord connections. Four of the six square-end web trusses failed because of insufficient plate area. Only one truss exhibited significant plate buckling and that was not noted until after the truss had failed, so it was uncertain whether that was a cause or an effect of the truss failure.

Behavior of the compression-web to top-chord joints in these truss tests emphasized the importance of boundary conditions to proper joint evaluation. Joint buckling was limited in the truss tests because of the magnitude and direction of load eccentricity. For the joints that did buckle, the plates buckled outward on both sides of the joint, keeping the web in line with the chord, opposed to the web moving out of the plane of the truss as a result of the plates both buckling in the same direction. The latter buckling mode was not uncommon for the individual joint tests described by Stahl and others (1994) and may have been more common in the tests if the truss and load configurations had been conducive to buckling failures.

Strain gauge extensometers provided useful information related to member forces and moments. The portal gauges used in this study had a number of advantages over the spring-steel gauges. However, with some improvements to the design and fabrication of the spring-steel gauges, their measurement accuracy would be adequate for this application. Portal gauge advantages included direct surface measurement, consistent gauge length, linear voltage–strain relationships and better resolution than the spring clips. Further refinement of these tools will enable more meaningful model verification for nonlinear analysis of trussed components.

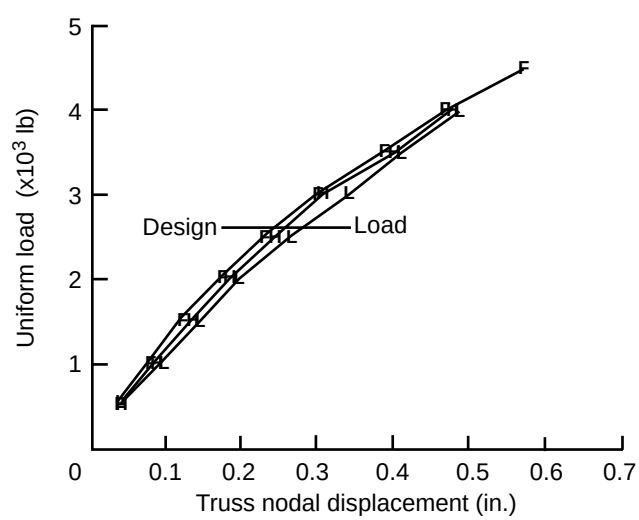

Figure 10—Average load deformation curves for the three test groups.

Conclusions

Results of the nine truss tests conducted in this study provide a limited basis for judging the technical feasibility of using square-end webs for light-frame residential roof trusses. Results support further study of the economic benefits in addition to truss configurations and load conditions more conducive to joint compression failures.

For the truss configuration tested, we note the following:

- Truss stiffness was more sensitive to the MOE of the constituent lumber than to the existence of square-end webs, regardless of the plate gauge used.
- Member axial forces were not affected by the presence of the square-end webs.
- The increase in plate size and gauge resulted in greater moment transfer from truss chord to the web for the square-end web trusses than for the fitted web trusses.
- Strength deficiencies of the square-end web trusses appeared to be caused more by insufficient plate area on tension connections than by buckling limitations for compression joints.
- Plate buckling at the compression web-to-chord joint appeared to be a secondary effect rather than a cause of truss failure.

Although these results suggest that advantages of the larger plates offset disadvantages of the square-end webs, it is important to note that the connector plates ability to transfer moment is partially due to the joint moment direction. For the test trusses, joint moment at the compression web connection placed the major portion of the plates unsupported area in tension, thus avoiding failure caused by plate buckling. Other truss designs may reverse this situation, thus an expanded testing program is required before square-end webs are accepted for general use.

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Appendix A—Metal-Plate Connectors

The metal-plate connectors used to fabricate the trusses were obtained from TrusWal Systems Corp.,[2] a commercial plate manufacturer. The plate connectors had aligned teeth, 0.31 in. (8 mm) long in rows 0.38 in. (9.6 mm) apart. Within each row, teeth were spaced just over 0.25 in. (6 mm) apart, giving a tooth density of 10/in^2 (0.006 m^2). These plates were tested on Southern Pine to derive the joint parameters given in Table A1. These parameters were determined by testing standard test joint configurations described in the CSA S347–M1980 standard method for evaluation of truss plates used in lumber joints (CSA 1980). Equation (A1) was then fit to the average load-displacement curves for these tests using a least squares nonlinear regression. These data are discussed in detail by Stahl and others (1994).

$$F = (P_0 + P_1 \times \delta)\ [1 - \exp(-K \times \delta / P_0)] \qquad \text{(A1)}$$

where

F is the connection load to cause the displacement δ.

K an approximation of the initial slope of the load-displacement curve.

P_1 the slope of the F–δ curve beyond the elastic limit.

P_0 the load intercept of the tangent to the F–δ curve which has slope P_1.

Table A1—Plate-wood stiffness parameters[a]

Direction of joint testing[b]	Best fit			Best fit $P_1 = 0$	
	K (lb·in)	P_0 (lb)	P_1 (lb·in)	K (lb·in)	P_0 (lb)
AA	7,200	44	320	6,600	54
AE	3,900	32	230	3,700	46
EA	6,600	44	10	6,675	44
EE	4,100	26	470	3,500	37

[a]Determined using CSA S347-M1980 (CSA 1980); 1 lb·in = 11.3 N·m; 1 lb = 4.45 N.

[b]AA is parallel to the wood and parallel to the machine direction of the plate connector. AE is parallel to the wood and perpendicular to the machine direction of the plate connector. EA is perpendicular to the wood and parallel to the machine direction of the plate connector. EE is perpendicular to the wood and perpendicular to the machine direction of the plate connector.

[2] The use of trade or firm names in this publication is for reader information and does not imply endorsement by the U.S. Department of Agriculture of any product or service.

Appendix B—Truss Test Rack

Test boundary conditions for trusses should be designed to simulate those imposed in service. If a pitched-chord truss is tested using "simple beam supports," having one reaction pinned and one on rollers, any lateral movement or "spread" of the truss will accumulate at the roller end. A gravity load applied in this situation rides along with the lateral movement and maintains a strictly vertical orientation. However, controlled loads (loading rams, cable and pulley systems) often react against a fixed point of reference. If the load points on the truss move with the truss relative to the fixed reaction points, a horizontal component of load develops at the expense of the vertical component. In most instances, this horizontal component acts to resist spread, making the truss appear to be stiffer; it may also force unexpected failures.

This problem can be alleviated in several ways. The most obvious is to use gravity loading. Safety and time considerations often eliminate this option, especially for trusses being loaded to failure. A second option is to place rollers at both reactions and fix the center of the truss against horizontal movement. This will give a 50-percent reduction in the maximum accumulated lateral displacement, but will not solve the problem of horizontal force components contributing to truss stiffness. Because the versatility of a gravity force that permits the load to move with the truss cannot be truly simulated without acquiring a horizontal component, the load position and orientation must remain constant as the truss moves.

The FPL truss test rack was designed to minimize lateral displacement relative to the reactions and keep the load position and orientation constant with respect to the fixed reference. Roller supports were used at both reactions, and rolling loads were applied to wedge-shaped blocks fastened to the truss top chord to minimize transference of horizontal force components as the truss deflected.

The truss test frame used for the square-end web truss evaluations is shown in Figure B1. The frame included a double steel I-beam base, which supported six hydraulic loading units. Loads were designed to pull down on the truss by having the hydraulic rams (Fig. B2) react against the I-beam supports to place a tensile load on the pull-down cable. Each ram controlled two loading frames (Fig. B3), which transferred the downward load to the top chord. A tubular steel alignment frame was used to support the lateral alignment arms used to hold the truss in a vertical orientation.

Each loading unit consisted of a 10,000-lb (44.5-kN) loading ram, two 8-in. (203-mm) diameter pulleys, and 12 ft (3.7 m) of 0.38-in.- (9.6-mm-) diameter steel cable. The cable passed over a pulley mounted on top of the loading ram and under

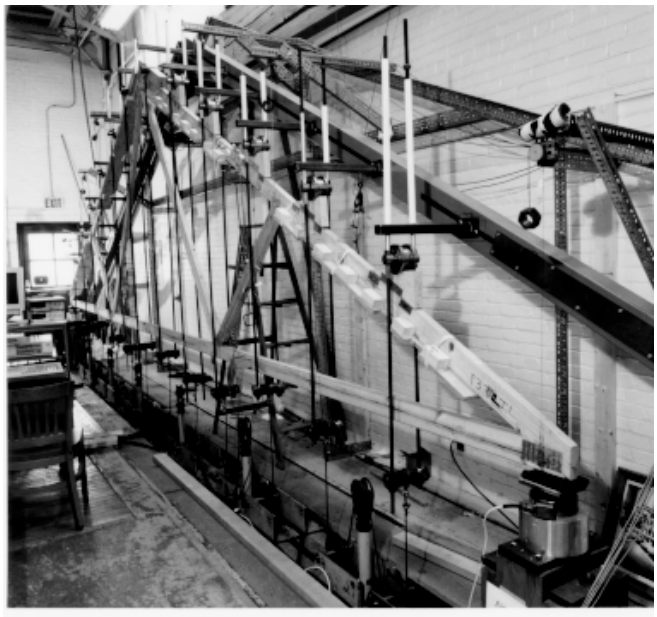

Figure B1—Test rack with an square-end web truss mounted for testing. Adjustable alignment arms hold the loading frames in a vertical orientation which in-turn keep the truss oriented vertically while applying the load to the top chord. Rolling load heads allow the truss to move laterally, minimizing horizontal force components. Linear rotational potentiometers were used to measure node point displacements, load cells at the reactions measured truss load, and strain gauges on web and chord members measured member strains.

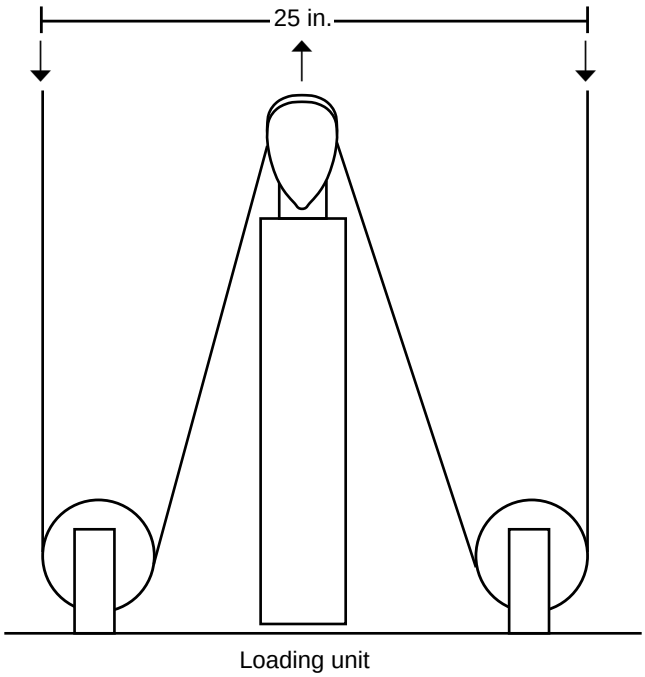

Figure B2—Loading unit comprised a 10,000- (44.5-kN) capacity loading ram with a 10-in. (254-mm) stroke three pulleys and a 0.38-in. (9.6-mm) steel cable. The cable passed over the pulley attached to the top of the ram and under the side pulleys so that an upward force of the ram was translated to a downward force on the truss.

the two 8-in. (203-mm) pulleys placed on either side of the loading ram. The ends of the cable were attached to loading frames so that as the ram extended upward, the cable pulled down on the loading frames, applying a concentrated load to a point along the top chord of the truss.

The loading frames were designed to apply a vertical load with minimal lateral restraint. Each loading frame comprised two parallel 0.63-in. (16-mm) threaded rods held 4 in. (102 mm) apart by spacer bars at the top and bottom. The bottom spacer was a 3-in.- (76-mm-) diameter pipe that also served as the cable attachment. For 10 of the 12 loading frames, the top spacer was part of a rolling loading mechanism. This loading mechanism was a 2-in.- (51-mm-) diameter steel shaft that served as the rod spacer. Ring bearings mounted on the shaft supported an outer cylinder, which was free to rotate. When pulled vertically onto a horizontal surface, the mechanism applies a vertical load with little resistance to any in-plane horizontal movement. By placing a wedge-shaped load-distribution block between the loading mechanism and the top chord of the truss, a vertical load could be applied with minimal restraint to horizontal thrust.

Two of the loading frames had top spacers with no rolling mechanism. These were used to apply load at points adjacent to the truss peak where their restraint on horizontal movement of the truss had minimal effect on truss “spread” between supports.

In addition to transferring load from the loading rams to the top chord of the truss, the 12 loading frames served as a barrier to flying debris, keeping large pieces from flying off the truss at the point of failure.

The alignment rack had a slot cut in a wide surface to permit steel alignment arms to be attached and positioned for equal spacing of the load frames. These arms served as guides and supports for the loading frames. The steel rods on the loading frames passed through steel sleeves mounted on the alignment arms. This served to maintain vertical alignment of the frames as load was applied to the truss.

Reaction blocks were spaced 27.5 ft (8.4 m) apart. Truss reaction points were supported on roller bearing plates that permitted lateral movement. These plates were in turn supported on 10,000-lb (44.5-kN) load cells that measured truss reaction forces.

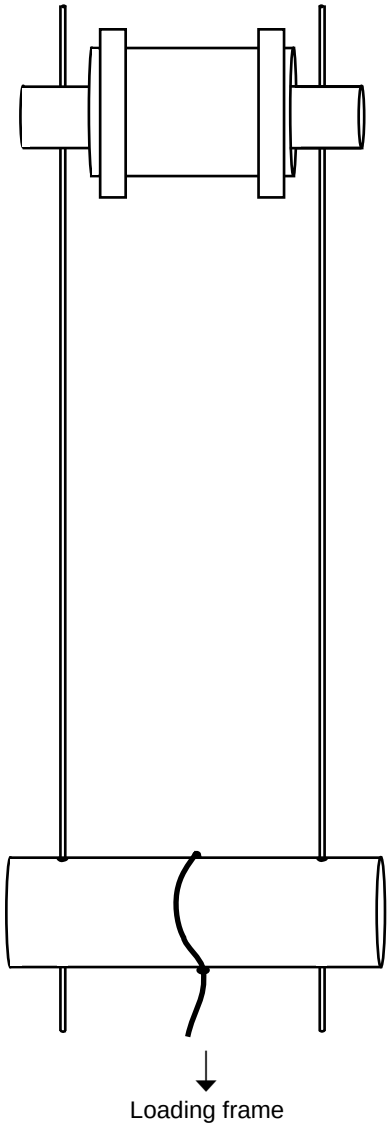

Figure B3—Loading frame served to translate a vertical force from the cable under the truss to a vertical downward on the top chord with no horizontal force component.

Nine Fink-type trusses were tested to failure in this rack as part of the square-end web study. Although the Fink configuration is not nearly as problematic as a scissors truss, in terms of spreading, the prototype testing rack performed as desired. The trusses showed little sign of out-of-plane buckling or bending during the test. Longitudinal displacements up to 0.05 in. (1.3 mm) were observed at the reactions with no binding of loading frames. This suggested that the roller cylinder load heads also performed their intended function. Truss reaction forces and deflection measurements attest to loading symmetry that varied by less than 6 percent from one end to the other.

Appendix C—Strain Gauge Extensometers

As a result of the difficulties encountered in using strain gauges applied directly to wood, it is often beneficial to use them in the form of clip-on extensometers. Extensometers are normally constructed and applied to the surface of the wood so that elongation of surface fibers of the wood will cause strain in metallic elements of the extensometer. Strain gauges attached to these elements then exhibit a change in resistance, which is linearly proportional to the surface strain in the wood.

Two types of clip gauges were used in this study. One was a portal frame (Fig. C1), with the gauges located on the legs of the frame. The other was a spring-steel gauge similar to that described by Loferski and others (1989) (Fig. C2). The portal gauge consisted of two phosphor bronze arms 0.05 in. thick by 0.38 in. wide and 1.5 in. long (1.3 by 9.6 by 38.1 mm) with a 2-in.- (51-mm-) long aluminum cross member. The

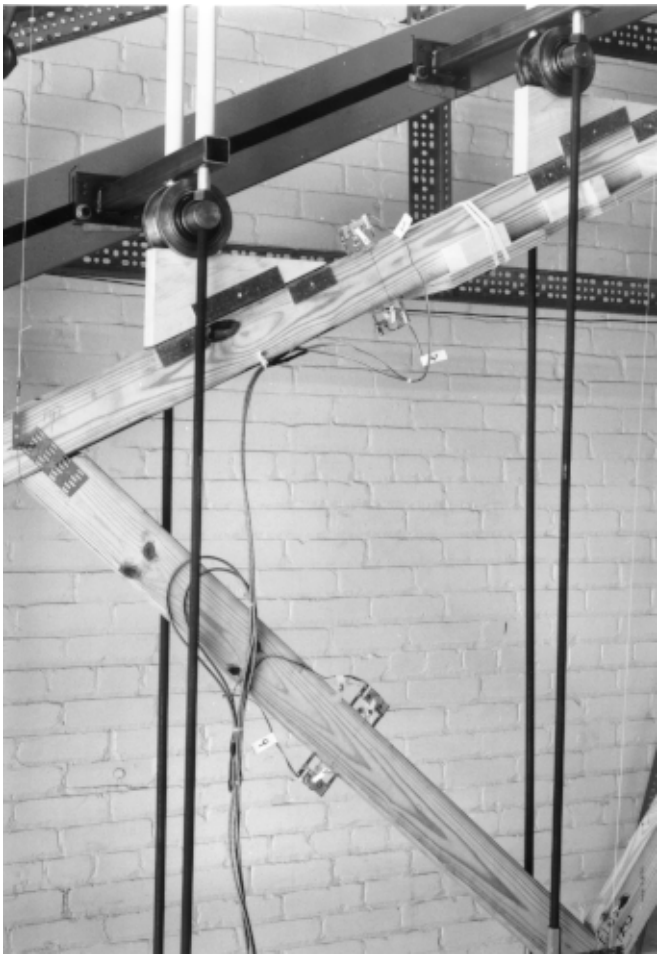

Figure C1—Portal frame clip-on extensometer with the gauges located on the legs of the frame.

Figure C2—Spring-steel clip-on extensometers. Strain gauges on the top and bottom surfaces of the arched section gave a fairly linear voltage response with displacement of the ends. These sections were compression fit between two reaction blocks that moved toward each other on the compression surface and away from each other on the tension surface.

arms were attached to the ends of the cross member giving the extensometer a gauge length of 2 in. (51 mm). Strain gauges glued to the inner and outer surfaces of each arm were hooked up in a Wheatstone bridge configuration so that the tensile and compressive strains on opposing surfaces gave additional voltage as the two arms are either spread apart or forced together.

The spring-steel clip-on extensometers were made using 2-in.- (51-mm-) long sections of steel taken from a clock main spring. Strain gauges on the top and bottom surfaces of the arched section gave a fairly linear voltage response with displacement of the ends. These sections were compression fit between two reaction blocks that moved toward each other on the compression surface and away from each other on the tension surface.

Each extensometer had four individual 350-Ω strain gauges in a full Wheatstone bridge circuit. The output from these clip gauges gave a resolution of less than 50 microstrain. Thus, with wood MOE in the range 1.2 to 1.5×10^6 lb/in^2 (8.3 to 10.4 GPa), member stresses were detected in the range 60 to 75×10^6 lb/in^2 (413 to 517 GPa) or 300 to 400 lb (1.3 to 1.8 kN). This corresponds to the web force at design load on the trusses.

Appendix D—Truss Deflections and Member Forces

Truss deflections, reported in Table D1, were measured using seven linear rotational potentiometers mounted above the truss and connected to points centered over the truss reactions and each member intersection point. The combination of tortional resistance of the LRP shaft and flexibility of the 10-lb (44 N) test line used to connect the LRPs to the reference points on the truss caused load-deflection traces to exhibit an initial nonlinearity. Deflections of the truss did not register until tension in the line overcame the resistance of the LRP and the counter weight. This nonlinearity was removed by fitting a line to the linear portion of the load-deflection curve between 600- and 1,600-lb (2.7- and 7.1-kN) load beyond the equipment dead load. The deflection intercept derived from a least-squares linear regression was used as a measure of the stretch in the line. Readings shown in Table D1 represent measured deflection with the effect of line stretch removed.

Truss member forces and moments were measured using clip-on strain-gauge extensometers. These were attached to the narrow faces (top and bottom) of the top chords, 24 in. (610 mm) above the intersection of the chord and compression web centroidal axes, and the compression webs, 18 in. (457 mm) from the bottom edge of the top chord. Strain measurements were converted to axial force and moments using compressive MOE values determined from tests of 8-in.- (203-mm-) long blocks removed from the trusses after the truss test was completed. Table D2 gives axial forces and moments taken at 10 uniform load increments from the test dead load to failure. Loads are expressed as the ratio of the truss design total load. Member forces and moments are lb (N) and lb·in (N·m), respectively. Member forces are given only for the second load stage where the truss was loaded to failure.

Table D1—Truss node point deflections

Load (lb)	Truss node deflection (in.)[a] South top	South bottom	Peak	North bottom	North top
Truss 1 to design load					
500	0.029		0.032		0.095
1,000	0.066		0.077		0.056
1,500	0.091		0.104		0.038
2,000	0.134		0.145		0.02
Truss 1 to failure					
500	0.034	0.03	0.036	0.035	0.033
1,000	0.075	0.045	0.076	0.068	0.056
1,500	0.114	0.079	0.118	0.103	0.093
2,000	0.151	0.117	0.156	0.147	0.13
2,500	0.196	0.165	0.214	0.19	0.173
3,000	0.249	0.226	0.257	0.256	0.219
3,500	0.305	0.3	0.313	0.322	0.272
4,000	0.375	0.383	0.381	0.398	0.334
4,500	0.497	0.482	0.463	0.483	0.408
5,000	0.604	0.676	0.618	0.643	0.529
Truss 2 to design load					
500	0.047	0.047	0.052	0.055	0.046
1,000	0.093	0.063	0.092	0.081	0.086
1,500	0.14	0.108	0.149	0.132	0.133
2,000	0.2	0.17	0.211	0.196	0.192
Truss 2 to failure					
500	0.043	0.027	0.041	0	0.036
1,000	0.077	0.038	0.066	0.057	0.062
1,500	0.115	0.068	0.106	0.099	0.098
2,000	0.167	0.121	0.17	0.155	0.15
2,500	0.2	0.157	0.207	0.195	0.185
3,000	0.293	0.262	0.302	0.3	0.28
3,500	0.369	0.357	0.385	0.391	0.366
4,000	0.453	0.459	0.483	0.497	0.464
4,500	0.522	0.54	0.554	0.576	0.531
5,000	0.642	0.693	0.692	0.726	0.668
5,500	0.843	0.94	0.953	0.98	0.919
Truss 3 to design					
500	0.056	0.067	0.058	0.071	0.059
1,000	0.113	0.134	0.121	0.114	0.115
1,500	0.176	0.206	0.186	0.182	0.178
2,000	0.248	0.295	0.272	0.268	0.258
Truss 3 to failure					
500	0.047	0.057	0.052	0.053	0.046
1,000	0.098	0.113	0.109	0.082	0.097
1,500	0.15	0.173	0.165	0.137	0.146
2,000	0.222	0.258	0.238	0.216	0.213
2,500	0.29	0.339	0.315	0.29	0.281
3,000	0.36	0.434	0.4	0.38	0.36
3,500	0.47	0.572	0.524	0.506	0.474
4,000	0.567	0.697	0.63	0.617	0.572
4,500	0.708	0.887	0.795	0.791	0.725
Truss 4 to design					
500	0.033	0.042	0.029	0.042	0.033
1,000	0.075	0.061	0.071	0.068	0.073
1,500	0.115	0.1	0.105	0.104	0.111
2,000	0.168	0.159	0.154	0.161	0.165
Truss 4 to failure					
500	0.024	0.02	0.022	0.018	0.02
1,000	0.062	0.024	0.056	0.022	0.054
1,500	0.089	0.047	0.085	0.044	0.082
2,000	0.141	0.095	0.133	0.091	0.131
2,500	0.194	0.15	0.177	0.145	0.178
3,000	0.26	0.23	0.239	0.221	0.241
3,500	0.423	0.411	0.389	0.384	0.371
4,000	0.48	0.482	0.441	0.449	0.431
4,500	0.585	0.6	0.538	0.568	0.524

Load (lb)	Truss node deflection (in.) South top	South bottom	Peak	North bottom	North top
Truss 5 to design					
500	0.045	0.048	0.047	0.003	0.049
1,000	0.097	0.075	0.103	0.005	0.105
1,500	0.147	0.118	0.152	0.004	0.156
Truss 5 to failure					
500	0.046	0.049	0.047	0.004	0.049
1,000	0.102	0.071	0.097	0.003	0.102
1,500	0.15	0.119	0.145	0.01	0.152
2,000	0.201	0.172	0.201	0.05	0.208
2,500	0.262	0.236	0.262	0.101	0.268
3,000	0.328	0.307	0.33	0.173	0.338
3,500	0.427	0.418	0.43	0.275	0.441
4,000	0.52	0.527	0.527	0.377	0.532
Truss 6 to design					
500	0.057	0.063	0.23	0.012	0.06
1,000	0.119	0.097	0.526	0.01	0.121
1,500	0.18	0.16	0.6	0.023	0.189
Truss 6 to failure					
500	0.058	0.061	0.05	0.016	0.054
1,000	0.135	0.103	0.126	0.014	0.126
1,500	0.195	0.156	0.177	0.026	0.181
2,000	0.262	0.223	0.248	0.086	0.247
2,500	0.398	0.332	0.336	0.166	0.341
Truss 7 to design					
500	0.035	0.038	0.034		0.039
1,000	0.08	0.058	0.074		0.087
1,500	0.117	0.085	0.108		0.126
Truss 7 to failure					
500	0.039	0.036	0.035	0	0.04
1,000	0.081	0.053	0.07	0.002	0.082
1,500	0.122	0.086	0.105	0.002	0.122
2,000	0.166	0.135	0.147	0.017	0.169
2,500	0.214	0.134	0.191	0.048	0.219
3,000	0.261	0.235	0.233	0.093	0.268
3,500	0.342	0.333	0.309	0.183	0.36
4,000	0.389	0.392	0.356	0.24	0.412
Truss 8 to design					
500	0.043	0.047	0.054	0.047	0.047
1,000	0.089	0.093	0.102	0.098	0.095
1,500	0.137	0.143	0.158	0.151	0.145
2,000	0.206	0.212	0.221	0.227	0.216
Truss 8 to failure					
500	0.04	0.037	0.046	0.042	0.038
1,000	0.086	0.081	0.093	0.092	0.088
1,500	0.133	0.126	0.142	0.142	0.134
2,000	0.186	0.184	0.197	0.197	0.184
2,500	0.245	0.25	0.25	0.263	0.244
3,000	0.305	0.321	0.31	0.334	0.309
3,500	0.398	0.442	0.411	0.443	0.409
4,000	0.492	0.542	0.502	0.545	0.495
Truss 9 to design					
500	0.055	0.063	0.057	0.062	0.061
1,000	0.11	0.121	0.112	0.121	0.117
1,500	0.167	0.184	0.172	0.184	0.184
2,000	0.231	0.255	0.236	0.258	0.252
Truss 9 to failure					
500	0.043	0.047	0.043	0.047	0.043
1,000	0.104	0.119	0.097	0.113	0.102
1,500	0.157	0.174	0.151	0.17	0.156
2,000	0.22	0.238	0.207	0.235	0.216
2,500	0.283	0.311	0.27	0.307	0.282
3,000	0.358	0.39	0.339	0.389	0.355
3,500	0.452	0.518	0.447	0.52	0.473
4,000	0.535	0.624	0.53	0.633	0.559

[a] 1 in. = 25.4 mm

Table D2—Forces and moments on top chords and compression webs determined using strain gauge extensometers

	Member forces (lb)[a]				Member moments (lb·in)[b]			
	Chords		Webs		Chords		Webs	
Load[c]	South	North	South	North	South	North	South	North
Truss 1								
0.21	526	486	178	97	330	311	77	-24
0.43	1,027	984	356	375	722	631	156	6
0.64	1,546	1,378	525	370	1,114	978	222	28
0.86	2,001	1,874	704	621	1,521	1,341	298	69
1.07	2,492	2,262	865	523	1,937	1,726	363	64
1.28	2,964	2,681	1,042	747	2,528	2,166	413	48
1.5	3,466	3,184	1,266	1,047	3,360	2,730	453	28
1.71	3,875	3,576	1,434	1,266	4,171	3,255	448	0
1.93	4,406	4,088	1,576	1,490	6,108	4,041	-117	-1
2.14	4,818	4,508	1,765	1,886	7,266	4,828	-19	-150
Truss 2								
0.21	428	542	160	150	270	427	38	-10
0.42	815	1,221	349	952	558	849	65	191
0.63	1,237	1,789	529	871	956	1,325	97	91
0.84	1,619	2,526	701	1045	1,286	1,905	112	106
1.05	1,976	3,092	870	1,383	1,694	2,290	150	173
1.26	2,341	3,788	1,066	1,502	2,198	2,907	153	55
1.47	2,672	4,441	1,256	1,515	2,763	3,579	142	-40
1.68	3,002	5,037	1,464	1,910	3,411	4,099	91	290
1.89	3,254	5,810	1,795	1,845	4,628	4,839	-526	640
2.1	3,534	6,609	2,084	2,217	5,501	5,941	-577	1,448
Truss 3								
0.18	292	315	307	42	54	9	10	4
0.37	710	448	670	135	69	-13	24	2
0.55	1,307	896	1,037	223	82	6	37	2
0.74	2,197	1,590	1,426	310	138	56	50	0
0.92	2,907	1,556	1,810	388	144	55	65	0
1.11	3,814	1,978	2,105	475	165	68	75	0
1.29	5,287	2,975	2,370	589	236	91	84	1
1.47	6,685	3,694	2,624	689	293	115	94	0
1.66	8,984	7,067	2,794	831	367	230	104	1
1.84	11,744	9,162	2,648	1,182	477	295	99	0
Truss 4								
0.18	-160	183	133	183	73	424	138	93
0.36	610	814	321	321	696	684	277	267
0.55	828	1,253	425	412	817	1,057	412	370
0.73	1,154	1,859	563	542	1,145	1,394	554	512
0.91	1,884	1,976	696	683	1,687	1,920	701	665
1.09	2,070	2,527	828	775	1,945	2,271	881	819
1.27	1,903	2,932	416	407	2,611	2,777	377	641
1.45	1,765	3,933	578	605	2,950	3,342	375	789
1.64	2,305	4,538	698	758	3,536	3,904	328	868
1.82	3,412	5,568	830	913	4,549	5,112	315	228
Truss 5								
0.18	676	416	109	219	174	364	110	108
0.35	2,146	974	309	373	841	679	233	260
0.53	2,617	1,335	442	571	1,172	1,089	357	386
0.71	3,553	1,898	583	683	1,504	1,434	474	545
0.88	3,487	2,394	767	849	1,522	1,803	609	694
1.06	3,471	2,769	925	1,005	1,718	2,234	770	886
1.24	4,040	3,141	1,080	1,173	2,105	2,765	948	1,061
1.42	4,489	3,549	1,221	1,330	2,491	3,464	1,160	1,266
1.59	5,164	4,075	1,417	1,529	3,156	4,155	1,217	1,450
1.77	5,087	4,570	1,771	1,713	4,243	5,095	923	1,308

Table D2—Forces and moments on top chords and compression webs determined using strain gauge extensometers—con.

	Member forces (lb)[a]				Member moments (lb·in)[b]			
	Chords		Webs		Chords		Webs	
Load[c]	South	North	South	North	South	North	South	North
Truss 6								
0.11	104	233	77	109	129	392	46	123
0.21	309	427	169	183	442	636	120	206
0.32	764	731	251	273	768	919	193	302
0.43	1,067	957	255	292	1,051	1,317	272	413
0.53	1,214	1,175	339	420	1,329	1,664	356	531
0.64	1,682	1,598	412	476	1,708	1,987	436	676
0.75	2,034	1,942	494	558	2,062	2,306	521	827
0.85	2,250	2,349	584	595	2,321	2,725	526	1,000
0.96	2,427	2,492	931	717	3,391	3,128	-492	1,163
1.07	2,820	2,695	1,035	851	4,049	3,599	-419	1,352
Truss 7								
0.18	866	634	96	135	410	531	234	202
0.37	1,010	1,011	190	293	774	1,066	444	415
0.55	1,732	1,546	268	424	1,182	1,581	690	627
0.74	2,319	2,165	335	550	1,496	2,156	936	862
0.92	3,042	2,853	379	678	1,955	2,930	1,219	1,146
1.11	3,726	3,475	452	811	2,502	3,603	1,540	1,423
1.29	4,609	4,288	525	898	3,075	4,401	1,861	1,718
1.48	5,077	5,014	570	1,017	3,535	5,172	2,242	2,012
1.66	5,520	5,560	631	1,188	4,051	6,036	2,650	2,416
1.85	6,487	6,393	720	1,295	4,728	7,093	3,031	2,786
Truss 8								
0.16	544	238	137	143	213	331	204	176
0.32	1,240	646	271	242	418	577	380	338
0.49	1,632	930	425	389	612	855	582	516
0.65	2,100	1,136	587	549	832	1,159	777	702
0.81	2,467	1,409	790	705	1,039	1,498	984	879
0.97	2,788	1,659	980	878	1,267	1,836	1,208	1,093
1.13	3,321	1,882	1,156	1,046	1,512	2,173	1,437	1,299
1.29	3,548	2,082	1,361	1,222	1,809	2,571	1,720	1,561
1.46	3,651	2,344	1,558	1,430	2,026	3,078	2,077	1,814
1.62	4,059	2,523	1,788	1,703	2,414	3,653	2,464	2,191
Truss 9								
0.17	705	666	124	167	231	179	217	215
0.33	1,318	1,012	275	346	310	519	461	450
0.5	1,914	1,589	415	533	552	725	742	681
0.66	1,947	2,008	551	772	540	1,013	1,004	924
0.83	1,447	2,223	692	1,099	377	1,415	1,304	1233
0.99	2,377	2,911	843	1,259	738	1,629	1,645	1,512
1.16	2,679	3,434	1,001	1,502	961	1,888	1,966	1,851
1.33	3,126	3,910	1,163	1,772	1,200	2,265	2,298	2,237
1.49	3,416	4,674	1,366	2,065	1,464	2,734	2,783	2,738
1.66	3,652	5,288	1,558	2,347	1,808	3,125	3,182	3,218

[a] 1 lb = 4.45 N.

[b] 1 lb·in = 1.13 N · m

[c] Load is the ratio of the truss design total load.