Exploratory study of truss heel joints constructed with round mortise and tenon joints

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Abstract
Tests were conducted to determine the load capacity and structural behavior of trusses constructed with round mortise and tenon heel joints. The average peak load capacity of trusses constructed with 2-inch diameter tenons averaged 4,000 pounds. Comparable values for 3-inch diameter and 4-inch diameter tenons amounted to 7,225 pounds, and 22,600 pounds, respectively. Peak deflection of the trusses was linear up to about 70 percent of ultimate load. Joint failure usually resulted from fracture of the tenon at its point of entry into the tie beam. Overall, the behavior of the trusses/joints indicated that they merit further investigation for use in light timber frame construction.

The use of round mortise and tenon joints in light timber frame construction provides the means for rapidly assembling house, farm, and light industrial building frames from a limited set of standard precut parts. Such parts can be cut from squares or from small-diameter tree stems. These frames, therefore, provide potential outlets for underutilized domestically grown small-diameter tree stems. In contrast, in lesser-developed countries, they provide the means of producing timber frames from small-diameter stems specifically grown for conversion into frame members. Potentially, therefore, they provide a means of significantly improving quality of life in regions desperately in need of shelter.

The use of round mortise and tenon joints in light timber frame construction has been illustrated in previous papers (Eckelman et al. 2002,
1 2006
). In the constructions reported in those papers, the heel of the rafter, the wall plate, and the tie beam were linked together by a tenon cut on the top end of a corner post or wall stud that had been inserted through mortises cut in these three members (Fig. 1a). This construction allowed for easy assembly of the roof frame system (Eckelman et al. 2002) and provided sufficient strength to resist substantial roof loads (Eckelman et al. 2006). Although the heel joints performed well in the construction reported in the latter study, detailed information concerning their ultimate load capacity and the factors that influence their performance are lacking. Exploratory tests were conducted, accordingly, with small frame bents similar to those shown in Figure 1 to obtain initial estimates of the ultimate load capacities of such joints and to obtain related information concerning their structural behavior.

Purpose
The purpose of the tests was to determine the structural characteristics of one type of round mortise and tenon heel joint used in the construction of bents used in light timber frames under the action of gravity loads. The primary objective of the study was to determine the strength characteristics of joints constructed with 2-inch diameter tenons along with supplementary additional information concerning the characteristics of joints constructed with 3- and 4-inch diameter tenons. A secondary objective was to determine the structural behavior and mode of failure of the joints—in particular, the difference in behavior of trusses with notched rafters (Fig. 1a) compared to trusses with beveled wall plates (Fig. 1b). Also of interest was the effect of wood species on joint strength.

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Materials and methods of construction

Corner posts with 2-inch tenons were constructed of standard and better Douglas-fir (Pseudotsuga menziesii), No. 2 and better eastern pine (Pinus sp), No. 2 southern yellow pine along with ungraded red elm (Ulmus rubra) and white ash (Fraxinus alnus). Corner posts with 3-inch tenons were constructed of treated mixed green No. 2 southern pine timber; corner posts with 4-inch tenons were constructed of ungraded yellow-poplar (Liriodendron tulipifera) and ungraded white ash timber. All material was stored outdoors in a covered shed. Moisture content (MC) at time of testing averaged 12 percent.

Typical configurations of the frames used in the tests are shown in Figure 1. All of the trusses were constructed with a 30° slope. One group of frames was constructed with the rafters notched to mate with the wall plates as shown in Figure 1a, whereas the remaining frames were constructed with beveled wall plates as shown in Figure 1b. Rafter cross section was identical to bottom chord cross section. Other specific details of each truss construction are given in Table 1. The end of each tie beam was reinforced with either one or two cross bolts (Fig. 1) to eliminate any possible failure of the wall or the mortise and thereby ensure tenon failure. Three specimens of each configuration were constructed.

Method of testing

All of the tests were conducted on a screw-powered Riehle universal testing machine. The sill of each frame rested on the bed of the testing machine. The tenons on the lower ends of the corner posts (which frame into the sill) were cut slightly longer than the depth of the sill so that loads applied to the posts were transferred directly to the bed of the testing machine.

Loads were applied to a 2-, 3-, or 4-inch steel cylinder embedded in the peak of each frame (Fig. 1) at a rate of 0.25 inches/minute. Vertical movement of the load head was monitored by a digital gage. Ordinarily, loading was continued until some type of failure occurred that resulted in a substantial release of load.

Results and discussion

Results of the tests are given in Table 1; ultimate load values are shown graphically in Figure 2.

The structural behavior of the joints is such that as loads are applied to the peak, the rafter tends to rotate downward on the inside edge of the corresponding wall plate, Figure 3. At the same time, the bottom face of the rafter exerts an outward force on the tenon, which tends to rotate the wall plate about its lower outside edge. This action tends to cause crushing of the tenon on its inside edge and a bending fracture on its outside edge at its point of entry into the bottom face of the rafter. At the same time, this action also causes (lesser) crushing of the outside edge of the tenon and a bending fracture on its inside edge at its point of entry into the top face of the tie beam. Bending fractures can occur at both of the above locations, but truss failure usually results from fracture of the tenon at its point of entry into the top surface of the tie beam. Longitudinal shear failures can also occur in the upper portion of the tenon.

The ultimate load capacities of trusses constructed of various species with 2-inch tenons can be seen in Figure 2. Overall, the highest values were obtained with Douglas-fir. Values
Table 1. — Truss construction and head support details along with peak loads and SDs.

<table>
<thead>
<tr>
<th>Member Dim.</th>
<th>Tenon species &amp; grade</th>
<th>Tenon diam.</th>
<th>Wall plate config.</th>
<th>Avg. ultimate load (lb)</th>
<th>SD</th>
<th>5% load (lb)</th>
<th>5% slope (in/lb)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3.5 by 3.5</td>
<td>E Pine&lt;sup&gt;a&lt;/sup&gt;</td>
<td>2</td>
<td>Beveled</td>
<td>2,975</td>
<td>550</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>3.5 by 3.5</td>
<td>D-fir</td>
<td>2</td>
<td>Beveled</td>
<td>4,923</td>
<td>313</td>
<td>4,200</td>
<td>2.05 × 10&lt;sup&gt;-4&lt;/sup&gt;</td>
</tr>
<tr>
<td>Rad 4.5</td>
<td>Red Elm</td>
<td>2</td>
<td>Beveled</td>
<td>4,667</td>
<td>1,159</td>
<td>2,700</td>
<td>3.79 × 10&lt;sup&gt;-4&lt;/sup&gt;</td>
</tr>
<tr>
<td>2.5 by 3.5</td>
<td>White ash</td>
<td>2</td>
<td>Beveled</td>
<td>4,280</td>
<td>650</td>
<td>3,400</td>
<td>2.67 × 10&lt;sup&gt;-4&lt;/sup&gt;</td>
</tr>
<tr>
<td>3.5 by 3.5</td>
<td>Spruce&lt;sup&gt;b&lt;/sup&gt;</td>
<td>2</td>
<td>Beveled</td>
<td>3,650</td>
<td>568</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>3.5 by 3.5</td>
<td>SPine</td>
<td>2</td>
<td>Notched</td>
<td>3,890</td>
<td>75</td>
<td>--</td>
<td>--</td>
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<tr>
<td>3.5</td>
<td>SPine</td>
<td>2</td>
<td>Notched</td>
<td>3,814</td>
<td>501</td>
<td>2,760</td>
<td>2.81 × 10&lt;sup&gt;-4&lt;/sup&gt;</td>
</tr>
<tr>
<td>5.4 by 5.4</td>
<td>SPine</td>
<td>3</td>
<td>Beveled</td>
<td>7,272</td>
<td>588</td>
<td>4,760</td>
<td>1.52 × 10&lt;sup&gt;-4&lt;/sup&gt;</td>
</tr>
<tr>
<td>8 by 8</td>
<td>Y-poplar&lt;sup&gt;c&lt;/sup&gt;</td>
<td>4</td>
<td>Beveled</td>
<td>20,333</td>
<td>2,060</td>
<td>14,500</td>
<td>8.0 × 10&lt;sup&gt;-4&lt;/sup&gt;</td>
</tr>
<tr>
<td>8 by 8</td>
<td>White Ash</td>
<td>4</td>
<td>Beveled</td>
<td>25,000</td>
<td>na</td>
<td>19,000</td>
<td>7.3 × 10&lt;sup&gt;-4&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

<sup>a</sup>Eastern pine.
<sup>b</sup>Southern yellow pine.
<sup>c</sup>Yellow-poplar.

yellow pine constructions with square wall plate plates (notched rafters) was only 6.6 percent greater than that of similar constructions with beveled wall plates (unnotched rafters).

The ultimate load capacity of the 3-inch diameter southern yellow pine tenons was essentially twice that of comparable 2-inch diameter tenons. The bending moment capacity of a 3-inch tenon would be expected to be 27/8, or 3.4, times that of a 2-inch tenon; however, the moment arm (a function of "h," Fig. 3) for the 3-inch tenons was about 1.6 times greater than for the 2-inch tenons. Thus the estimated increase in load capacity (compared to 2-inch tenons) amounts to about 3.4/1.6, or 2.1. In comparing these values, however, it should be noted that all the 3-inch tenons had trash failures resulting from the inclusion of juvenile wood in the tenons.

The ultimate load capacity of the 4-inch diameter yellow-poplar tenons amounted to 20,333 pounds, or 5.2 times that of the 2-inch southern pine tenons. Likewise, the ultimate load capacity of the 4-inch white ash tenons was 25,000 pounds, or 5.5 times that of the 2-inch white ash tenons. The bending moment capacity of the 4-inch tenons would be expected to be 8 times that of the 2-inch tenons; the moment arm, however, was 6.4/3.5, or 1.8 times, as great so that the expected increase in capacity amounted to 8/1.8, or 4.4 times that of the 2-inch tenons. The woods used in constructing the 2-inch and 4-inch tenons were of different species, but the comparisons do indicate that the depth of the wall plate must be considered in estimating the load capacities of connections with larger tenons. Changes in design may help improve performance. In some constructions, for example, it may be possible to use a two-piece wall plate that consists of a thin beveled plate located above the tie beam and a rectangular plate beneath the beam, Figure 4.

In addition to ultimate load, a "5 percent deviation" load capacity was also determined. In this procedure, curves were first plotted for the average values of each set of joint displacement data. Straight lines were then fitted to the data corresponding to the central linear portions of the load-displacement curves. The load at which a test displacement curve deviated from its corresponding linear fitted curve by 5 percent was then regarded as the 5 percent deviation load for the joint. The 5 percent deviation loads along with the slope of the fitted curves are given in Table 1. In the case of the Douglas-fir, red elm, white ash, and southern yellow-poplar 2-inch

Figure 2. — Graph of test results.

obtained with the red elm tree stems were 95 percent as great, whereas values obtained with white ash and yellow pine timbers were 92 percent and 74 percent as great, respectively. Inclusion of juvenile wood in the tenons likely accounts for the low values obtained with the southern yellow pine. Finally, values for mixed eastern pine species were about 60 percent as great. Based on modulus of rupture (MOR), only the white ash had lower than expected values. Notching of rafters vs. beveling of wall plates had little effect on load capacity. Specifically, the ultimate load capacity of the southern

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**Figure 2.** — Graph of test results.
tenons, the 5 percent yield load averaged 73 percent of ultimate load capacity. The 5 percent load capacity of the 3-inch southern yellow pine tenons, however, averaged only 66 percent as great as the ultimate. Finally, the average 5 percent load capacity of the 4-inch tenons amounted to 74 percent of the average ultimate load capacity. These values, though based on an arbitrarily selected deflection value, do provide an indication of the load capacity of the joint at the onset of yielding.

Given these estimates, it is useful to consider the ultimate uniform roof loads that the joints of these trusses could carry. In terms of equivalent roof loads, a uniform line load, w, with
magnitude \( w = F/L \), acting along the entire length, \( L \), of a truss with 30° slope generates an axial rafter force only 62.5 percent as great as that produced by an equivalent peak load (Fig. 5a). Thus, for purposes of analysis, the joints of trusses subjected to uniform loads would be expected to carry a total uniform load, \( wL \), that is 60 percent greater than a corresponding peak load.

Assuming a 20-foot length as the upper limit for small-diameter logs (Wolfe et al. 2005) used as rafters, then for a 30° slope and an overhang of little more than a foot, a half-span of 16 feet appears reasonable—for a total span of 32 feet. Based on the 5 percent deviation load results given in Table 1, as a first approximation, the joints of a 32-foot truss with 2-inch diameter Douglas-fir tenons would be expected to carry a uniform vertical line load of about \( (4,200/0.625)/32 \), or 210 pound/ft. It should be noted, however, that uniform loads produce a rotation of the ends of the member in addition to the rotation caused by deflection of the peak. For a 4 by 4-inch Douglas-fir member, this rotation would add about 6 percent to that resulting from the peak deflection. The exact effect of this action on ultimate joint strength is not known, but, if for purposes of discussion, a straight line reduction in strength is assumed, the joints would be expected to carry uniform loads that are 57 percent rather than 60 percent greater than corresponding peak loads, i.e., 198 rather than 210 pound/ft. Thus, for a roof system with a 2-foot rafter spacing (and a corresponding projected roof area of 64 ft²), the joints of a 32-foot truss would be expected to carry a uniform vertical load of 198/2, or, 99 psf; likewise, for a 4-foot spacing, the joints would be expected to carry a load of 49.5 psf. Similarly, for a 32-foot truss with 4-inch diameter yellow-poplar tenons, the joints would be expected to carry a uniform vertical line load of \( (14,500 \text{ pound/ft})/32 \), or, 684 pound/ft. For a 12-foot truss spacing, this amounts to a uniform vertical load of 56.6 psf. The comparable value for white ash tenons is 74.5 psf.

**Conclusions**

Results of the study indicate that the joints of trusses constructed with round mortise and tenon joints of the type shown in Figure 1 have sufficient load capacity and regular behavior to be useful in light timber frame constructions. Nonlinear yielding of the truss joints occurs at about 70 percent of ultimate load. Truss failures normally result from fracture of the tenon at its point of entry into the top surface of the tie beam. Even with eventual allowances for necessary safety factors, it appears that trusses constructed with such joints could be designed to provide useful levels of strength. Thickness of the wall plate substantially affects the ultimate moment capacity of the tenon. Use of a two-piece wall plate, which shortens the distance from the top surface of the tie beam to the underside of its corresponding rafter, may provide added load capacity. Finally, it is to be noted that the results of the study provide estimates only of the ultimate load values and loads at 5 percent deviation that were obtained with this type of joint. Determination of safety factors and other truss design characteristics are clearly beyond the scope of this investigation.

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