Performance tests of small barn frame constructed with round mortise and tenon joints

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Abstract
Structural tests were conducted to determine the strength and deflection characteristics of an 8 by 10 foot light-timber barn frame constructed with 3.5-inch square Douglas-fir members and round mortise and tenon joints. Structural analyses of the frame were also conducted using finite element methods and predicted deflections compared with test values. Results of the tests indicated that the roof and frame system could withstand positive and negative roof loads of 10,000 pounds, or 125 psf. Diagonal bracing or shear resistant siding likely would be needed to resist high lateral loads, however. Relatively close agreement was obtained between predicted and test deflections, after taking the semi-rigid behavior of the joints into account. Overall, results of the study indicate that high strength, ease and simplicity of construction, and the fact that members can be machined from small-diameter tree stems makes round mortise and tenon light timber frame construction well-suited for relieving both long- and short-term shelter problems, particularly in less-developed countries.

The use of round mortise and tenon joints in light-timber frame construction provides the opportunity for high-speed processing of standardized components that can easily be assembled to form a wide variety of modular frame constructions with a multitude of potential end uses. These frames provide potential outlets for square or round timbers cut from small-diameter tree stems and can thus lead to better utilization of underutilized timber resources. These frames also provide the means for constructing frames from local resources in less developed regions of the world and can thus lead to significant improvements in quality of life. In addition, the quick and easy assembly characteristics of these frames make them potentially useful for disaster relief in many parts of the world. In this respect, it is anticipated that these frames can be constructed to have high resistance to wind and other racking forces.

The construction of a small backyard barn frame using round mortise and tenon joints and squared members fabricated from small-diameter yellow-poplar stems was discussed in a previous paper (Eckelman et al. 2002). The purpose of that study was to investigate the feasibility of round mortise and tenon light-timber frame construction from a machining and assembly viewpoint. Parallel cooperative research with similar structures is also underway at the Institute of Technology of Costa Rica. Marketing of such structures was treated by Morgan (2003). This method of construction has also been incorporated into commercially available structures (Round-Peg 2005). In addition, the strength of small-diameter timbers as well as the strength of members cut from them has been addressed (Wolfe 2000, Wolfe and Moseley 2000, Wolfe and Murphy 2005). This paper provides additional information about the structural behavior and load-carrying characteristics of this type of construction.

Purpose
The primary purpose of the study was to obtain information concerning the strength and deflection characteristics of a small light-timber frame constructed with squared timbers and round mortise and tenon joints; the secondary purpose was to determine whether the frame deflections could be estimated with reasonable accuracy through structural analysis.

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The specific objectives of the study were to 1) determine the load-carrying capacity of the frame both with respect to positive and negative roof loads and determine the resistance of the frame to side loads; 2) obtain information concerning the deflection characteristics of the frame; and 3) determine if the deflections of the frame at selected points could be predicted through structural analysis.

Frame design, materials, and construction

A three-dimensional drawing of the frame is given in Figure 1. Overall, the frame measured about 8 feet wide by 10 feet long by 10 feet tall with 1-foot side overhangs and a 30-degree roof slope. Frame members were constructed of 3.5-by 3.5-inch Douglas-fir (Pseudotsuga menziesii) posts obtained from a local lumberyard. In general, all of these timbers were sound but had been deemed unfit for public sale because of large splits and member twist. Essentially all of the timbers contained boxed heart. All material was kept outdoors in a covered shed until needed for testing. Moisture content, as determined by the ovendrying method, averaged 12 percent.

Tenons for the frame were cut with a 2-inch-(internal) diameter by 12-inch-long deep hole saw. Excess material was removed from around the base of the tenons in order to leave a uniform rectangular shoulder on the tenon. Nominal diameter of the tenons at the time of machining was 2 inches. Mortises were drilled into the members with Forstner bits that produced nominal mortise diameters of 2-1/16 inches.

Construction of the frame itself began with the insertion of the floor joist tenons into the corresponding mortises in the sills (Fig. 1). End sills were then put in place and the corner-post tenons inserted into the lapped corner-sill mortises. The wall studs were then added by inserting their tenons into corresponding mortises in the side and end sills. Next, the front and back tie beams were slipped into place over the tops of the corner-post tenons and the wall-stud tenons and seated on the shoulders of the tenons. Side wall plates were then slipped into place over the ends of the sidewall-stud tenons and the corner-post tenons. The ridge beam support columns, or king posts, were next inserted into place. Then the rafters were slipped into place over the tops of the corner-post and wall-stud tenons while the rafter tenons simultaneously were slipped into mortises in the corresponding half ridge beam. It should be noted that the ridge beam was constructed of two parts (Fig. 2) that were joined together with 3/8-inch-diameter lag screws spaced about 36 inches apart after the frame was assembled. Use of a two-part ridge beam eliminated the need for a unique larger ridge timber, simplified machining of the beam, and facilitated assembly of the roof frame. Finally, the facia boards were added.

Vertical and horizontal load tests

Three types of vertical load tests were conducted: downward vertical load tests on the ridge beam, downward vertical load tests on the rafters, and upward vertical load tests on the rafters. In addition, two types of horizontal load tests were conducted: horizontal end load tests and horizontal side load tests on frames with and without cross bracing.

Vertical load tests on ridge beam

Tests were conducted with the loads applied vertically to the ridge beam in order to obtain information concerning the overall deflection characteristics and strength of the frame along with information concerning the performance of various components. Factors of particular importance were the effects of the king posts and the facia boards on vertical deflection of the ridge beam and horizontal deflection of the wall plates. Also of interest was the effect of intermediate tie beams on deflections along with the effect of reinforcement of the wall plates with 3.5-inch-square beams (clamped at the ends and center to the inside faces of the wall plates) on ridge and wall plate deflections. Tests were conducted, accordingly, with the frame configured with and without king posts and with and without facia boards in various combinations, as listed in Table 1.

In these tests, five loads were applied to the ridge beam by means of 4-inch air cylinders, i.e., a load was applied at each of the five points midway between the rafters. The barn framework rested on a foundation of cross beams located under the sills. The cylinders were attached to brackets that, in turn, were bolted to the foundation cross beams at their mid-length points. The rod ends of the cylinders were attached to the ridge beam by means of chains that were looped over the beam and secured with clevises. Loads were monitored by means of an additional 4-inch cylinder attached to the load heads of a uni-
Vertical load tests on rafters

In conducting the downward vertical load test on rafters, five loads were applied along the length of the roof frame on each side (10 loads total), along a line parallel to and midway between the ridge beam and the wall plate. These loads were applied to nominal 2- by 4-inch purlins laid flat (one on either side of the roof); these purlins were attached to each rafter with a lag screw. The cylinders were attached to brackets that were bolted to the foundation cross beams as in the ridge beam tests but at the quarter points of the beams. The rod ends of the cylinders were attached to the purlins by means of chains that were looped over the purlins (midway between rafters) and secured with clevises. Deflections of the ridge beam were measured in these tests also, as described previously.

In tests with intermediate tie beams, two tie beams (identical to those in the end frames) were added to the two centrally located side wall studs. These tie beams were removed in all other tests.

Upward vertical load tests on rafters were conducted in a manner similar to the downward load tests except that the chains were replaced by extension rods. One end of an extension rod was rigidly attached to the rod end of a cylinder, whereas the free, or top, end of the rod was attached to a bracket-type clevis mounted on the underside of the purlins. Use of these extension rods, which effectively extended the length of the cylinder rods by 76 inches, allowed the air cylinders to push upward on the purlins.

In these uplift tests, the king posts in the end frames were replaced with queen posts (Fig. 2b). This was done for two reasons. First, the king-post tenon and the end-rafter tenons intersect in the ridge beam. Thus, the king-post tenon must be shortened to accommodate the rafter tenons so that little relish is left on the tenon to resist withdrawal forces. Secondly, queenpost construction is the normal frame construction when a vent or window is located immediately below the ridge beam in the gable end, and thus its resistance to uplift forces needed to be investigated. (It should be noted that gable studs were purposely omitted so that the carrying capacity of the queen posts acting alone could be evaluated.) In making this change, the king post was removed and a queen post attached to an end rafter and the corresponding tie beam 1 foot on either side of the original king-post position.

Finally, all of the joints subjected to tensile forces were cross pinned with 1/2-inch steel pins.

Horizontal load tests

Loads were applied to the frame in the horizontal side load tests by means of two air cylinders. These cylinders were attached to brackets that were bolted to the wall sill on one side of the frame – one at each end of the sill – adjacent to the corner posts. Chains were attached to the rod ends of the cylinders and then looped over the wall plate on the opposite wall frame and secured with clevises. Thus, loads were applied to the top plate, adjacent to each corner post, along a line running from the centerline of the wall plate to the centerline of the opposite wall sill. The horizontal component of the applied force was used in subsequent stiffness calculations. Horizontal side sway deflections were measured at the top outside edge of the wall plate at each end of the plate as shown in Figure 3a.

Two sets of tests were conducted to determine the effect of cross bracing on side sway resistance. In the first test, the end frames on one side of the frame were reinforced with a single cross brace (Fig. 4a). In the second test, an additional cross brace was added in the opening adjacent to the first (Fig. 4b). Loads were applied so that the braces were loaded in compression. Measurements were taken in these tests in the same manner as in the previous side sway tests.

The horizontal end-load tests were conducted in essentially the same manner as the side sway tests except that the cylinders were attached to an end sill and the chains to a tie beam on the opposite wall. Also, deflections were measured at the top edge of the tie beam at each end of the beam (Fig. 3b). The frame was not braced in these tests.

Semi-rigid joint connection factors

Owing to the reduced cross section of the tenons compared to that of the members on which the tenons are cut, round
mortise and tenon joints must be considered as semi rigid. Thus, correction factors are needed to make the ordinary exact methods of structural analysis applicable to frames with round mortise and tenon joints. The rotation that occurs in semi-rigid joints is commonly described by the semi-rigid connection factor $Z$, which, in turn, is defined by the expression:

$$Z = \frac{\phi}{M}$$

where $\phi$ = angle change resulting from the semi-rigid behavior of the connection, radians; $M$ = bending moment acting on the joint, in-lb (Lothers 1960).

In-plane and out-of-plane rotation factors were determined using a test set-up similar to that shown in Figure 5. Rate of loading was 0.1 in/ min. Values of the rotation connection factors were determined by means of the expression:

$$Z = \frac{(d_1 + d_2) / x}{F \times L}$$

where $d_1$ and $d_2$ = absolute values of the dial gage deflections, inch; $x$ = distance between gages, inch; $F$ = vertical load applied to the rail, pound; $L$ = distance from load application point to the side wall of the mortise member, inch. Calculated values of the connection factors are given in Table 2.

Individual member tests

The average E-value (modulus of elasticity) for the Douglas-fir members used in the frame, based on static bending tests of five members, was $1.8 \times 10^6$ psi with a standard deviation of $4.0 \times 10^5$ psi. These tests were conducted using centerpoint loading with a span length of 78 inches.

The mortises cut into the ridge rafters, top plates, and sills along with the bevel cut on the ridge rafters and top plates would be expected to substantially affect the EI (stiffness) values of these members and thus the accuracy of the structural analyses of the frame. Load vs. deflection tests were carried out, accordingly, on the wall plates and ridge rafters to determine the EI values of the wall plates and ridge rafters in directions both parallel to and perpendicular to the axes of the mortises. The average EI values for the beams amounted to $7.5 \times 10^6$ lb-in$^2$ for loads applied parallel to the mortise orientation and $14.3 \times 10^6$ lb-in$^2$ for loads applied perpendicular to the mortise orientation. Similarly, the average EI value for the facia boards with the loads applied perpendicular to the mortise axes was $22.1 \times 10^6$ lb-in$^2$, and $18.6 \times 10^6$ lb-in$^2$ with the loads applied parallel to the axes.

Results and discussion

Vertical load tests on ridge beams

Results of the vertical load tests on the ridge beams are given in Table 1. Gross vertical deflection of the ridge beam at midspan relative to the end sills – when the frame was configured without king posts and without facia boards under a load of 500 pounds/cylinder (2,500 lb total) – was 1.328 inches. The corresponding horizontal deflection of each wall plate at midspan was 0.860 inch. The analogous deflection of the ridge beam when the king posts were added was 1.016 inches with a corresponding horizontal wall plate deflection of 0.781 inch. Thus, use of king posts decreased the vertical deflection of the ridge beam by 23.5 percent and the horizontal deflection of the wall plate by 23.4 percent.
Likewise, when the frame was configured with king posts but without facia boards and the wall plates were reinforced with 3.5-inch square beams that were clamped at the ends and center to the inside faces of the wall plates, the midspan deflection of the ridge beam amounted to 0.938 inch and the horizontal wall plate deflection to 0.625 inch. Thus, reinforcing the wall plates as described, decreased ridge beam deflection by 7.7 percent and wall plate deflection by 20 percent. Similarly, the ridge beam deflection of the frame configured without king posts or facia boards but with the wall plates reinforced with 3.5-inch beams amounted to 1.016 inches with a corresponding horizontal wall plate deflection of 0.624 inch. Thus, reinforcement of the wall beams decreased the analogous ridge beam deflection by 23.5 percent and the horizontal wall plate deflection by 27.4 percent.

Deflection of the ridge beam when the frame was configured with facia boards and king posts averaged 0.686 inch with a horizontal wall plate deflection of 0.422 inch. Thus, use of the facia boards reduced ridge deflection by 32.6 percent and wall plate deflection by 46 percent.

Finally, both when the frame was configured with and without king posts but with two intermediate tie beams, midspan ridge beam deflection for both cases averaged 0.188 inch while midspan wall plate deflection was essentially eliminated. Thus, use of intermediate tie beams effectively reinforced both the ridge beam and the wall plates.

**Vertical load tests on rafters**

Deflection of the ridge beam at midspan when the frame was configured with king posts and facia boards under the action of ten 500-pound vertical loads acting downward on the purlins amounted to 0.750 inch; corresponding wall plate deflection amounted to 0.438 inch. At the 1,000-pound load level, the corresponding deflections were 1.5 inches and 0.938 inch, respectively.

When the frame was configured without king posts but with facia boards, deflection of the ridge beam at midspan amounted to 0.688 inch; the corresponding wall plate deflection amounted to 0.5 inch. At the 10,000-pound load level, horizontal wall plate deflection amounted to 0.719 inch (ridge deflections were not measured at this load level). In terms of floor area, this loading compares to an equivalent uniform roof load of 10,000/80, or 125 psf. Thus, this construction presumably could be used for substantially larger spans. If the width of the frame were increased to 24 feet, for example, the roof still would be able to carry a load of 41 psf; or the roof could carry a load of 31 psf over a 32-foot span. If the length of the structure were increased, however, intermediate tie beams would be needed to reinforce the wall plates.

In the vertical uplift tests, the frame was configured with queen posts and facia boards. A vertical uplift load of 1,000 pounds per cylinder was applied to the purlins, for a total load of 10,000 pounds. Again, in terms of floor area, this amounts to an equivalent vertical uplift loading of 125 psf. Although the frame carried the load without damage, it should be noted that this loading produces large perpendicular to grain forces on the ridge beam and on the sills. Because in practice these members could have longitudinal drying splits that would reduce their resistance to this type of loading, the ridge beams, in particular, should be reinforced to resist loading perpendicular to the grain in applications where the roof will be subjected to high uplift forces.

**Side and end load tests**

Based on the results of three test loadings, the average side sway deflection of the frame, when measured as shown in Figure 3a, under the action of a horizontal 300-pound load applied at each end, as shown in Figure 3a, was 2.073 inches. With one brace in each end frame, the comparable deflection was 1.555 inches for a reduction in deflection of 25 percent. With two braces in each end frame, the comparable deflection was 0.959 inch for a reduction in deflection of 53.7 percent. Thus, the corner braces can be seen to effectively reduce deflection. The maximum load applied to the frame without braces was limited to 300 pounds to ensure that no damage was done to the frame. In the case of the frame with one corner brace in each end frame, two horizontal loads of about 500 pounds each were applied without damage to the frame. Finally, two horizontal loads of about 725 pounds each were applied to the frame with two end braces. For a 7-by-10-foot side wall, this amounts to a comparable uniform load of about 40 psf.

The end sway deflection of the frame when measured as shown in Figure 3b under the action of two 100-pound loads applied as shown in Figure 3b amounted to 0.784 inch. The maximum load applied to each side frame was 323 pounds (646 total frame load) with an accompanying deflection of 2.5 inches.

Both the end and side sway results indicate that the frame has limited resistance to horizontal forces so that interior bracing would likely be required unless the frame has adequate shear-resistant siding or sheathing. Some improvement could be obtained if larger sills were used so that the wall-stud to sill joints could be constructed with 3-inch rather than 2-inch tenons.

**Frame analysis: Estimated vs. test results**

An end-frame of the three-dimensional model used to analyze the framework is shown in Figure 6. This frame was repeated six times with longitudinal members inserted between joints to form the complete frame. Note that the ridge beam was treated as two individual beams. Internal frame members were then “disconnected” as desired to form various frame configurations; internal tie beams could be added or removed as desired, for example. Joint numbers are enclosed in parentheses, whereas member numbers are enclosed in parentheses.
Estimated deflection of the ridge beam when the frame was configured with king posts but without facia boards was 0.870 inch. Horizontal deflection of the wall plate at midspan averaged 0.509 inch. Thus, the estimated vertical deflection was 1 percent less than the test value and the estimated horizontal value 1.8 percent greater than the test value. The axial forces acting on the ends of the tie beams amounted to 342 pounds, whereas the moment acting on the center section of the wall plate and ridge beams amounted to 4,154 in-lb and 5,207 in-lb, respectively. Likewise, the comparable moment acting on the facia boards amounted to 7,003 in-lb. Thus, as expected, except for the axial force, these values do not differ greatly from those obtained for five 500-pound loads applied to the ridge beam.

Estimated deflection of the ridge beam at midspan when the frame was configured without king posts but with facia boards under the action of ten 500-pound loads applied to the rafters at mid length was 0.757 inch. Horizontal deflection of the wall plate at midspan averaged 0.509 inch. Thus, the estimated vertical deflection was 1 percent greater than the test value and the estimated horizontal value 1.8 percent greater than the test value. The axial forces acting on the ends of the tie beams increased to 886 pounds, but the bending moments increased only slightly.

Sidesway.—Analyses of the frame indicated that the principal resistance to side sway forces is provided by the side wall stud to rafter joints, corner post to rafter joints, end wall stud and corner post to tie beam joints, and end wall stud and corner post to sill joints. In these analyses, the center end wall stud to tie beam joint was treated as pinned because it has a short tenon, to allow for the king-post tenon. Similarly, the corner post to end sill joints were treated as pinned since the mortise for the tenon is cut in the lap joint formed by the end and side sills. Under these conditions, the estimated sidesway deflection amounts to 0.635 inch vs. the measured deflection of 0.66 inch. Thus, the estimated value was 3.8 percent less than the test value.

End to end.—Structural analyses of the frame indicated that the principal resistance to end sway forces is provided by the side wall stud and corner post to wall plate joints, and the side wall stud and corner post to sill joints. Again, the corner post to end sill joints were treated as pinned since the mortise for the tenon is cut in the lap joint formed by the end and side sills. Under these conditions, the estimated end sway deflection amounted to 0.715 inch vs. the measured deflection of 0.784 inch. Thus, the estimated value was 8.8 percent less than the test value.

Conclusions

Results of the tests indicate that an 8- by 10-foot lightweight frame constructed with 2-inch-diameter round mortise and tenon joints, fabricated from readily manufactured standardized parts, can withstand positive and negative roof loads...
of at least 125 psf. Presumably, therefore, similar frames could be constructed with wider spans, but at reduced roof load capacities. For longer structures, however, intermediate tie beams would be needed to support the wall plates.

Bracing is needed to resist high lateral loads unless the walls are clad with shear-resistant siding or the frame is braced internally. Use of corner bracing was shown to substantially reduce sideways deflection of the frame, but the need for bracing in relation to the use of siding or sheathing and the most efficient type of cross bracing for the frame requires additional study.

Comparison of test deflection results with predicted results indicate that the deflection behavior of the frame can be reasonably predicted provided the semi-rigid behavior of the joints is taken into account. Presumably, therefore, the behavior of other frames can be predicted provided that Z-factors for the various joints are known. This is a particularly important outcome since numerous modular constructions are possible with this construction system.

Overall, results of the study indicate that round mortise and tenon joints provide a means of constructing sturdy easy-to-assemble frames from a relatively few standardized part sizes. Since these members can be machined from small-diameter tree stems, these frames could provide an outlet for what is now a largely unused domestic wood resource. Furthermore, the ease and simplicity of construction of the frames coupled with the fact that the members can be machined from small rapid-growth tree stems makes this construction well suited for relieving house, farm, and light industrial building frame needs in less developed regions. Finally, the simplicity of construction also makes this type of construction well suited for relieving both long- and short-term shelter problems in disaster areas.

Literature cited