TIMBER FRAMING

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Load-Bearing Housings

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On the front cover, Tom Nehil, chair of the Technical Activities Committee of the Guild's Timber Frame Engineering Council, load-testing housings in a red oak beam. Self-portrait Tom Nehil. On the back cover, James David Whidden, 1962–2011, perched on collar beams to work on roof frame of Alexander Knight House, constructed in mid-17th-century style in Ipswich, Mass. Beneath in loft, Ipswich architect Mathew Cummings, bemused, pondering his plans. Photo Cynda Warren Joyce.

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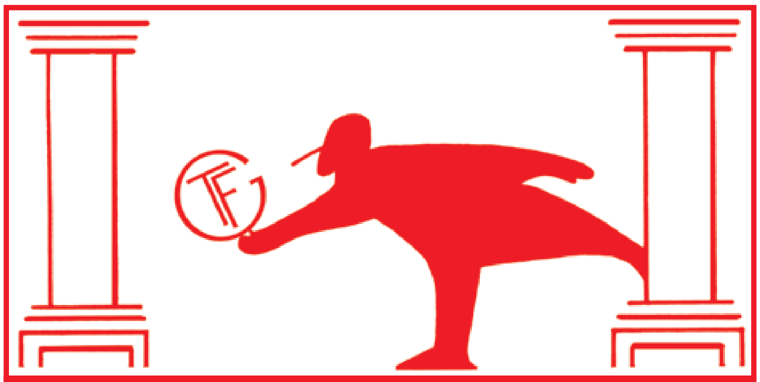
TIMBER FRAMING, Journal of the Timber Framers Guild, appears in March, June, September and December. The journal is written by its readers and pays for interesting articles by experienced and novice writers alike.



Notes & Comment

Whither the Guild?

IN its 28-year history, the Guild has sustained important administrative turns. Formed in 1985 as an enthusiastic 165-person club, and taking its original bylaws from another artisans' organization that happened to be run by a neighbor of one of the Guild's founders, the first board tried to figure out where to drive this new car, which had no onboard navigation at all, and soon defined our direction as the education of our members and the public. Within four years the group had grown to 600 members and organized six conferences. Under the leadership of another of its founders, it undertook an elaborate public project with a partner organization, engaging the efforts of hundreds of Guild members. Over the next quarter century the Guild grew, to a maximum membership of 1900 in 2006, undertook some 75 public projects with partners of all kinds and held 50 general conferences, equally divided between continental east and west, as well as 22 specialized conferences since 1990 focused on historical subjects.

Under Articles 7 and 8 of the bylaws, the "property, affairs and concerns of the Guild shall be vested" in an elected board of nine directors, and the "treasurer shall serve as financial officer of the organization and chair of the finance committee." Each director serves for three years, and three of the nine seats are up for annual election. Thus each year the membership can reconstitute the board in part while preserving continuity. As the Guild gained speed, boards meeting monthly by phone and occasionally face-to-face noticed that they could not keep up with the Guild's work and also find time to do their own. An executive secretary kept the records in a home office and answered the phone.

In 1991, after an amendment of the bylaws to make it possible, the board sought out and hired a salaried executive director to take on a well-defined administrative burden (if not responsibility for policy and finance, which remained with the board itself) and to generate his own salary in new revenue. The next elected board fired that executive director in less than a year, dissatisfied with performance and alarmed at the cost of the salary. The administrative burden then returned to a wary board for the next seven years.

In 1995, the decision to hive off the Timber Frame Business Council (see Joel McCarty, "Topics: Ontogeny or Phylogeny?" TF 38), ultimately with its own charter, relieved the increasingly difficult problem of the Guild attempting to serve its educational mission, properly that of a 501(c)(3) organization, while simultaneously promoting (if badly, and that was most of the problem) the business interests of its large company members, properly the work of a 501(c)(6). *Large* is a purely relative term, of course.

By 1998, pressure to find an executive director had again built to a decisive level. The board of the day advertised the position, interviewed five applicants and discovered in long deliberations that no one applicant seemed to fit the job description completely. The board thus hired *two* co-executive directors, Will Beemer and Joel McCarty, with equal status but with different roles. (See Curtis Milton, "Guild Notes & Comment: A New Executive Directorate," TF 50.) As the arrangement sorted out in practice, Will Beemer's home office in Massachusetts managed the conferences and membership and kept the books, Joel McCarty's in New Hampshire acquired and administered projects and represented the Guild to the world. Both co-executive directors were charter Guild members and had already served on the board for at least six years. They knew the subject matter and proved capable at their respective roles. The arrangement worked pretty well for a decade, as did, apparently, and for about the same period, the American economy.

In 2009, another turning point. What had been brewing in the economy for some time in volcanic fits earned a proper name, the Great Recession. On the Guild side, at the same time, Will Beemer announced his forthcoming retirement as a co-executive director. The Guild board of the day, not committed to a dual executive directorate, began a search and ultimately accepted Joel McCarty's proposal that he become sole executive director in 2010, continue his duties and reorganize matters to account for the functions that had been performed at the administrative office of Will Beemer.

The reorganization led to the establishment of a long-desired storefront office for the Guild, in Alstead, New Hampshire, as well as to the outsourcing of bookkeeping and of conference development. In the economy and in the Guild, hard times took hold, reflecting two or three years of decline. Treating it as a reserve fund, the leadership dissolved and spent down the Guild's invested scholarship fund, originally established with major donations in memory of deceased Guild members and over the years steadily augmented by annual member checkoff donations and investment gains. A separate apprentice-training fund, investments by journeyworkers dedicated to a distinct program, was commingled with the Guild's general fund and drawn down substantially for current expenses, as was much of the Guild's Engineering Council's dedicated current account. The Guild's four regular streams of income—membership dues, publications advertising, conferences and projects—all declined over several years, the latter two declines masked, respectively, by generous auction donations at loss-making conferences and by substantial income from the major Poland project in 2011.

In the fall of 2012, a partly reconstituted board, kissed not by Prince Charming but by Prince Responsibility, awoke energetically to realize that the Guild, having been adequately managed when it was relatively easy to do so, was now mismanaged and floundering financially. The question was not how to fulfill the Guild's mission through this or that workshop, or this or that conference, but how to keep it alive at all.

In *Scantlings* 176 (February), Guild President Randy Churchill announced abruptly to the membership that the board would shortly "replace" Executive Director Joel McCarty, while at the same time recognizing that the board itself (in failing to oversee affairs closely for some time, as it appeared to observers) had been "part of the problem" that required the severance, and was now "engaged in a deep-drilling organization assessment and board effectiveness review." Joel McCarty, using Guild magic and the considerable charm of his own personality as often as he used a reliable administrative style, had generated skeptics as well as admirers over the years. A definitive figure in the Guild from the first days, "whose fingerprints are on every Guild project," as one dismayed supporter remarked of him at the time of the surprise severance, and the "irreverent, witty, compassionate . . . shepherd for the soul of our community," as another observed, Joel McCarty's persona will not be replaced, though someone else (or something else) may run, for better or worse, the Guild's affairs.

FOR many Guild members who live in the East, "AMC" has always meant the Appalachian Mountain Club. Henceforth in a conversation about the Guild it will likely mean an association management company. The *Prairie Home Companion* faux radio sponsor, the Federated Association of Organizations, turns out to be not so implausible—the Association Management Company Institute (amcinstitute.org) is the real thing that the board consulted to find a manager and administrative services. And Mike Nizankiewicz, the expert whom the board hired in March to direct the Guild temporarily and to assess Guild structure and function over several months, and ultimately to advise the board on restructuring, comes to us from Transition Management Consulting (transitionceo.com) with "38 years of association executive leadership and organization development experience."

At the Burlington 2013 conference concluded a few weeks ago on the pleasant Vermont campus of Champlain College—mostly 19th-century-style buildings in brick, copper and dark green, ranged along leafy streets giving out over the city toward Lake Champlain and the summer haze of the distant Adirondacks—the members' meeting drew a larger crowd than usual. Mike Beganyi, a director who joined the board last fall, took the stage alone to announce that he stood in for Guild President Randy Churchill, who had resigned two days earlier for personal reasons (another surprise). In a performance whose like had never been seen at a member meeting, Mike then delivered an uninterruptable state-of-the-Guild address illustrated with chart after chart, list after list, for more than three-quarters of an hour, analyzing the Guild's activities and comparative performance over seven years, before inviting questions for the rest of the board, who now took the stage with him.

One of these questions revealed that the executive committee has firmly decided that the Guild should be managed by a hired company, at least for a time, and was then in a process of choosing one from among several whose representatives had come to the conference for interviews. Should a management company be chosen, the Alstead office will close and all Guild adminstration will go to the contractor, including the provision of an executive director.

While the board is convinced that an outside company managing numerous associations can do much better for us than we can do for ourselves—first of all to correct our financial position and then to centralize fundraising, finance, record keeping, conference adminstration, project management, publicity, grant writing, web development, and so on—the audience at the members' meeting largely was not pleased to hear of the decision.

WE are at a new turning point. To its members, the Guild is a unique aggregation of people, ideas and history. To a professional association person, all associations are associations and the Guild too is an association. Some who came to the charter conference in 1985 to join the Guild supposed it would be devoted to discovering the secrets of the past and renewing a craft then in decline, and they were dismayed to find that other Guild members were more interested in the difference between polystyrene and isocyanurate panels than between the English tying joint and the American dropped tie. Still others, who cared—who had to care—about business success because they had employees' families to support as well as their own, expected to take something else again from the Guild. All of these people have been accommodated over time by evolutionary rearrangement without loss of character. Following a face-to-face board meeting in Chicago at the end of June, an emailed bulletin from the Guild president proposed that Guild culture can remain intact while professional administrators manage its affairs: "The future executive director will not be the face of the Guild: projecting the personality of our group will fall to . . . others among our friends and colleagues who coordinate our programs." Can it really be done?

—Ken Rower

Capacity of Load-Bearing Housings

ONE long-standing challenge in timber frame design has been to assess the load-bearing capacity of joist and beam housings cut into the sides of receiving beams. In particular, for a given joist load, there has always been a question how much material in the receiving beam should be left below the joist bearing. There are no engineering guidelines to permit quantitative assessment of the capacity of a given notched bearing configuration in a given species. For investigators of old frames to be repurposed, and designers of engineered frames with such joinery, this lack of a mechanics-based design method leaves us guessing.

The *National Design Specification for Wood Construction* (*NDS*) does not address partial-width notches (as engineers call these housings), whether loaded or not. In addition, the shear provisions relating to bolts loaded perpendicular to grain (Article 3.4.3.3) indicate that the safe load-carrying capacity of a beam is greatly diminished as the position of the bolts approaches the bottom surface, the *loaded face*, of the timber. By inference, load-bearing housings would be subject to similar limitations. If applied, however, the reduction in load-carrying capacity provided in *NDS* Equation 3.4-6, which controls at housings within a distance of five times the beam depth from the supports at the end of a beam, would effectively prohibit load-bearing housings in timber framing. Yet such housings have been used successfully for hundreds of years.

Section 2.3.4 of the Timber Frame Engineering Council's *Standard for Design of Timber Frame Structures and Commentary, TFEC 1-2010*, sets certain limits on the proportioning of partial-width notches on the sides and compression-tension faces (usually upper and lower) of flexural members and provides guidelines for analysis of members containing such notches. But it does not quantitatively address the load-carrying capacity of housings cut in the side of members and loaded perpendicular to grain:

> Partial-width notches on the lateral faces of bending members that extend below mid-depth of the member from the compression face are common in timber framing, for example where joists frame into the sides of beams. In such instances, bearing of the joist on the bottom surface of the notch may induce tension stress perpendicular to the grain. While rules of thumb exist for the minimum distance from the loaded surface of the notch to the tension face of the member, no testing has been performed or analytical models developed to define the relationships between notch dimensions, clear distance to the tension face, mechanical properties of the wood (in particular tensile strength perpendicular to the grain), and safe load-carrying capacity of the notch. It is recommended that, whenever possible, load-bearing notches not extend below mid-depth of the member.

The recommendation runs counter to much traditional practice that (to the authors' knowledge) has not resulted in performance problems. Because no numerical analysis could effectively predict the postfracture behavior of load-bearing housings, we undertook a series of load tests.

Test setup and loading protocol Three species of timber were used for the initial screening tests: Eastern white pine, red pine and red oak. All timbers were 8x8, boxed heart, and green at the time of testing (moisture content at the surface of the timbers was 40 percent and ranged to more than 100 percent at the pith). Using Northeastern Lumber Manufacturers Association (NELMA) rules, we graded the white pine as No. 2, while the red pine and red oak we judged to be No. 1.

The housings were notched in 2 in. from each side of a beam leaving 4 in. of solid section in the center, and made 4 in. long measured parallel to the length of the beam (Fig. 1). The depth of material left intact below the housings varied from 2 in. to 4 in., to examine resulting changes in capacity and behavior.

Seasoned white oak blocks were inserted into the housings to load the bottom surface and held ½ in. out from the back (the interior face) to represent realistic loading conditions expected to result from construction tolerances, shrinkage of the receiving beam and rotation (lifting) at their ends when the joists sag under load—all of which move effective bearing away from the back of the housing. A hydraulic jack positioned at the intersection of centerline of beam and centerline of housings provided the force. Load transferred from the jack to the two loading blocks via a 6x6 white oak crossbeam.

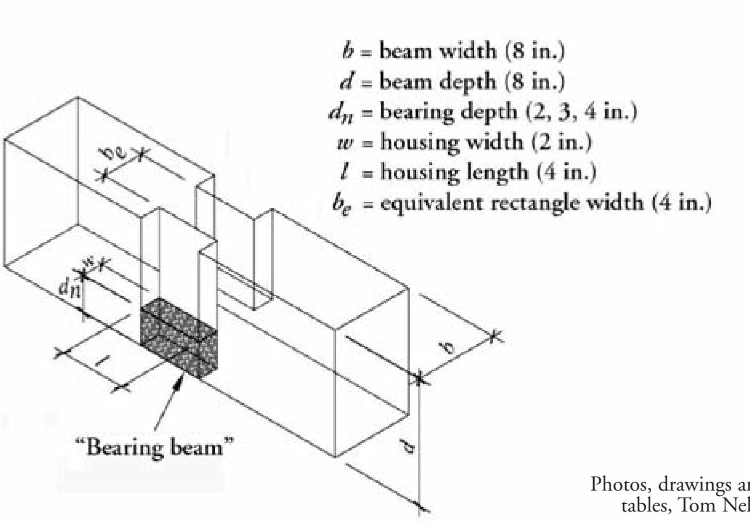
Specimens were clamped to a reaction beam with the hydraulic jack inserted asymmetrically between two clamps spaced 4 ft. apart, or 4 ft. 8 in. apart for the 4-in.-bearing-depth tests (Fig. 2). For the 4-ft. interval, clamps were set 1 ft. 4 in. and 2 ft. 8 in. respectively from the centerlines of the housings. In this way, shear would be higher on one side of the housing than the other, approximating the variation in shear that occurs at the last load-bearing housing adjacent to a post or other support for the beam. The unequally offset clamps also induced a higher shear for a given bending moment than a centerpoint loading would generate. Finally, at their minimum distances of approximately five bearing depths ($5d_n$) from the housings, we thought the clamps would not likely influence local behavior at the housings.

As the focus of this testing was to determine the behavior of the load-bearing housing up to and including failure, we did not study deflection of the beam and the effect of housings on beam behavior. To develop load-deformation curves, we elected to measure separation of the "bearing beam" section from the rest of the timber as demarcated by horizontal fractures that developed parallel with the bearing surface at the face of the beam. Reference pins driven into the beam before testing approximately 1 in. above and below the bearing surface on either side of the housing enabled fracture-width measurement uninfluenced by crushing of fibers at the bearing surface.

In testing, incremental load increases varied with wood species (500 lbs. typically for Eastern white pine, up to 2000 lbs. for red oak). The load increased over a relatively short time at each step, approximately 10 to 15 seconds. Once the next level was achieved, the load was held constant until deflection readings stabilized. Thus the duration of any load test varied with housing depth and species, ranging from about 40 minutes to four hours. (Testing did not follow the procedures of ASTM D1761, *Standard Test Methods for Mechanical Fasteners in Wood*.)

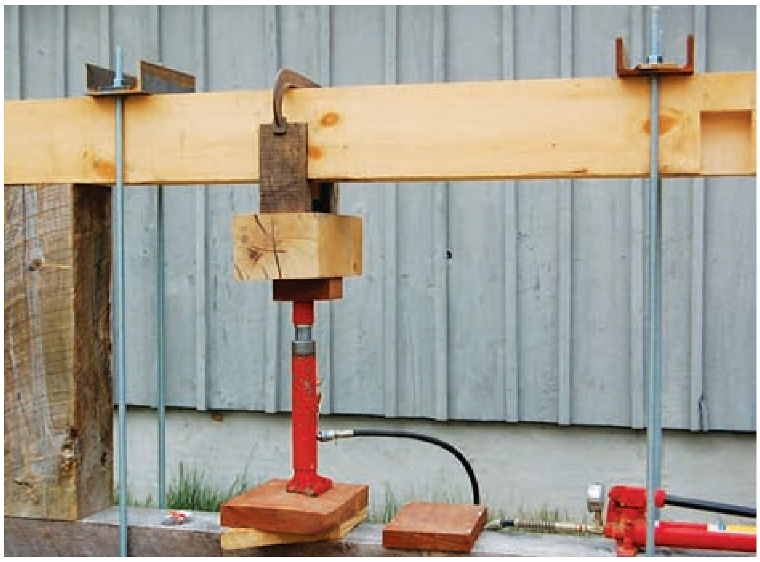
After peak load was reached, load would fall off a certain percentage of the maximum but would not drop to zero. Additional load-point displacement was applied to observe the postpeak behavior, examine ductility and watch for any secondary increases in load-carrying capacity. Testing typically terminated when the gap between the bearing surface of the housing and the body of the beam reached ¾ in. as measured at the exterior surface of the specimen.

Findings Table 1 at right shows the average maximum test load for each of the three different species and housing depths. This simplified summary shows the basic trends as a function of bearing

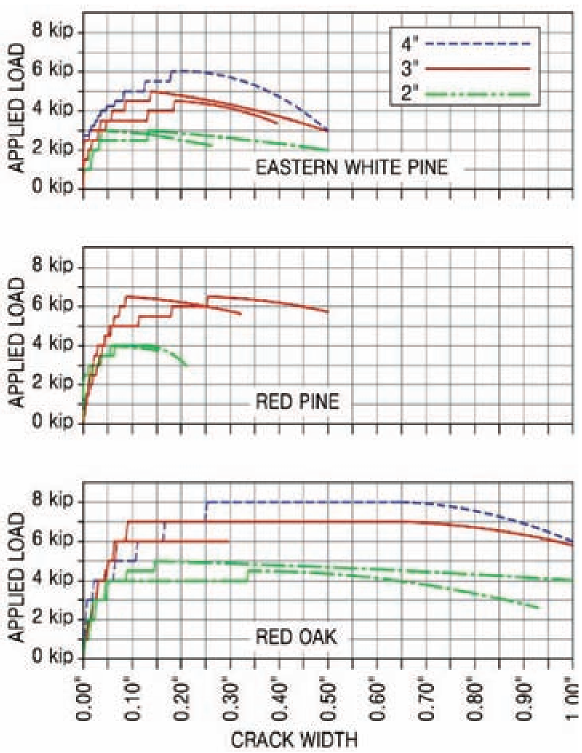


Photos, drawings and tables, Tom Nehil

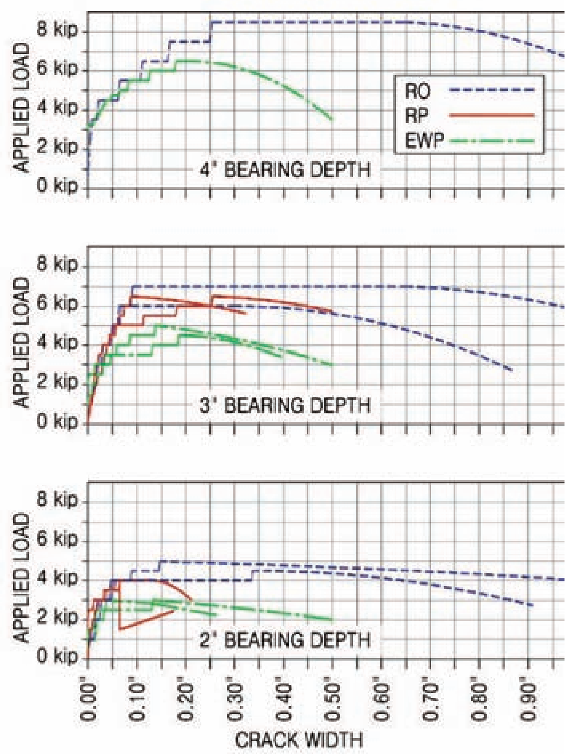
1 Notation, terminology and dimensions.



2 Load-test arrangement with white pine specimen.



3 Load deformation curves by species for bearing depths tested.



4 Load deformation curves by bearing depth for species tested.

Table 1 – Average Maximum Test Values (Pounds per Housing, i.e., Half of Total Applied Load)

Bearing Depth Species	2 in.	3 in.	4 in.
Eastern White Pine	3000	4750	6000
Red Pine	4000	6500	not tested
Red Oak	5000	6500	8000

beam depth d_n and the specific gravity of the species. Figs. 3 and 4 provide graphic summaries of test load vs. fracture width.

As the load-deformation curves illustrate, the housings did not fail in a brittle manner—the behavior did not remain linearly elastic up to peak load followed by complete loss of load-carrying capacity. Rather, in all cases an indication that peak load was being approached was evidenced by a change from near-linearly elastic behavior to a marked increase in fracture width and length at the face of the beams with increments in load.

The mode of failure at peak load varied primarily as a function of species. In Eastern white pine, the weakest of the species tested, the failure was marked by a sudden rupture of the fibers in the bearing beam in the bending regions, or in some cases by a shear failure where the fracture extending from the bearing surface ran out opportunistically along sloping grain to the tension face of the beam. Observed fractures always followed the grain of the timber, and so slope of grain was an important factor locally for the white pine. By contrast, the red oak specimens, the strongest of the species tested, never failed in a shear or flexural rupture. As the width and length of the fractures extending from the bearing surface increased past some critical point, the bearing beam eventually became too flexible to support the load, and a steady decrease in load-carrying capacity was observed with increasing deformation.

The red pine proved intermediate in behavior between white pine and red oak. Because the specimen was straight grained and red pine is somewhat stronger than white, its performance more closely resembled that of the red oak. Note that the curves shown for red pine suggest a less ductile behavior than the red oak, but that is an artifact of the test setup. Multiple tests were run on the same timber and, in the case of the red pine specimen, fractures from the housing being tested grew in length until they intersected adjacent housings, which then changed the behavior of the bearing beam. Absent these nearby housings, the red pine behavior likely would have appeared more ductile.

Fracture configuration The characteristic fracture that developed as the tests proceeded comprised a compound surface, both a horizontal extension from the bearing surface, parallel to the length of the beam, and a diagonal sloping surface extending from the bearing surface at the back of the housing down and toward the center of the beam (Fig. 5).

The load applied to the bearing surface of the housing induces tension perpendicular to grain on the horizontal surface extending outward from the bearing surface into the surrounding wood, and it induces rolling shear as well as tension perpendicular to grain on the vertical plane below the back of the housing. (Since the load is eccentric to the centroid of the beam, the load induces rotation of the bearing beam, which generates the latter tension.) This combination produces diagonal tension perpendicular to grain that causes the fracture surface at the back of the housing to angle downward toward the tension face of the beam.

As the width of the fracture increased, the direction of the horizontal fracture surface tended to change from parallel to the length of the beam to sloping toward the tension face. This characteristic form was repeated in virtually every test performed. The general form illustrated in Fig. 5 was typical of the 2-in. and 3-in. bearing depths, while the 4-in.-bearing-depth specimens tended to fracture straight across between housings. The fracture shown represents a snapshot during the testing. The actual shape and length of the fracture at peak load varied with species and bearing depth (Figs. 6–8).

Analysis The test results presented in Figs. 3 and 4 make evident several aspects of load-bearing housing capacity and behavior. The deeper the bearing depth (within the range tested here), the greater the load-carrying capacity. The strength exhibited was approximately linearly proportional to the depth, not a geometric function of the depth, indicating that the flexural strength and stiffness of the so-called bearing beam were not primary factors in determining housing strength.

The stronger the wood (as measured by tensile strength parallel to grain, tensile strength perpendicular to grain, and shear strength parallel to grain), the higher the load-bearing capacity. The strength was not directly proportional to the nominal specific gravity of the species as listed in the *NDS*—that is, the housing capacity did not increase from Eastern white pine to red oak as much as the increase in specific gravity between the two species.

The failure mechanism is more complicated than just a splitting perpendicular to grain, as shown by the fact that load capacity continues to increase after the initial development of the fracture plane. The geometry of the fracture plane suggests that tension perpendicular to grain, rolling shear, shear parallel to grain, and flexure all play a role, with different aspects being primary at different stages of deformation and fracture lengthening. Tension perpendicular to grain may initially be the stiffest and most directly engaged "spring" in the mix of resistance mechanisms, but it is also the weakest of the multiple mechanisms at work and is first to drop out. The absence of abrupt failure with the first appearance of fractures at the sides of the housings suggests that there are stronger though more flexible springs in the system that carry the load after failure of the stiffest spring.

It appears that even at practical working load levels where the fractures, if any, have not extended very far, it is inappropriate to think of the material below the bearing surface as a beam supported at each end by tension. It more appropriately could be considered a thick plate supported on three edges with the strength and stiffness of the support varying around the edges.

Estimating allowable loads Applying customary factors to get from test results to working stresses (and keenly aware that we haven't enough samples for statistical reliability), we find an "allowable" load in the linearly elastic region (essentially unfractured behavior) of the housing. For example, dividing the 3-in. red pine average maximum load capacity of 6500 pounds by the necessary factors produces a working load of about 2000 pounds, which can be seen in Figs. 3 and 4 to be in the elastic portion of the curve.

Table 2 shows the peak test values converted to implied allowable loads (ASD) by dividing the average peak test load by a load duration factor C_D of 1.5, a conversion factor K_F of 3.32 and a resistance factor ϕ of 0.65, and provides a comparison between these values and the allowable load that would result from applying *NDS* Equations 3.4-6 and 3.4-7 to the beam shear capacity and treating the housing bearing depth d_n as equivalent to d_e used in those equations. The maximum shear V_{max} produced in the beam in these tests was equal to two-thirds of the total load applied due to the third-point loading used in the test setup. That is,

$$V_{\max} = \tfrac{2}{3} \times 2P = \tfrac{4}{3} \times P$$

where P is the load applied at each housing in pounds. Then

$$P_{\text{allowable}} = \tfrac{3}{4} \times V_r' = \tfrac{3}{4} \times \tfrac{2}{3}\,[F_v' \times b \times d_e][d_e \div d]^2 \quad \text{(per 3.4-6)}$$

$$P_{\text{allowable}} = \tfrac{3}{4} \times V_r' = \tfrac{3}{4} \times \tfrac{2}{3}\,[F_v' \times b \times d_e] \quad \text{(per 3.4-7)}$$

where V_r' is the adjusted design shear strength in pounds and F_v' is the adjusted allowable shear design stress, taking into account all modifying factors (such as load duration, moisture conditions and so on), in pounds per square inch.

This comparison shows that Equation 3.4-6 clearly does not relate to the capacities that were demonstrated in these tests. Equation 3.4-7 on the other hand shows good agreement with the test results, even though the *NDS* indicates it should not be applied to connections closer than five times the beam depth to the support. The comparison suggests that Equation 3.4-7 might be a reasonable starting point for a mechanics-based capacity formula in a design standard, and it is consistent with the finding that the load-bearing capacity of a housing appears to be a linear function of the bearing depth. Also, allowable shear is a value readily obtained from the tables in the *NDS Supplement* (as opposed to tension perpendicular to grain and rolling shear) and thus is a desirable property to use in a design equation to predict safe capacity of load-bearing housings.

Recommendations for additional study For the three species tested, considerably more data will be required to understand the potential variability of peak load results for the same geometries. It will also be appropriate to test additional species used today in timber framing, such as Douglas fir, white oak, Southern yellow pine and yellow poplar.

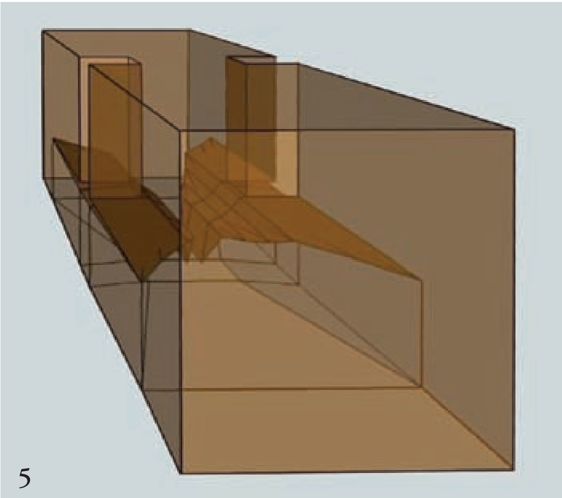
A broader range of geometries, reflecting plausible applications, should be studied to verify whether the dimension of the housing parallel to the length of the beam has any appreciable effect on capacity. Another relevant issue to be clarified is whether and how the ratio of the bearing depth to the overall beam depth affects capacity. The relationship between bearing depth and overall beam depth is obscure. For purposes of evaluating load-bearing housings, the most applicable range of $d_n \div d$ (based on practice) is from 0.15 to 0.5 for timbers ranging from 8 to 14 in. deep—that is, the depth of beam below the housing might range from about one-sixth to half of the entire beam depth—with the 0.15 ratio relevant to 10-in. and deeper timbers, larger ratios to beams shallower than 10 in.

Determining how the width of the bearing surface, measured perpendicular to the side of the beam, affects the housing capacity would be of value in guiding development of predictive models. In the field, that dimension can vary from a practical minimum of (say) ¾ in. up to the full width of the beam in through-mortises.

During testing, the housings demonstrated varying degrees of

Table 2 – Comparison of Test Values (Converted to Allowable) with NDS Equations (3.4-6) and (3.4-7) for Each Bearing Depth and Species, Pounds per Housing

Species	Eastern White Pine			Red Pine			Red Oak		
Bearing Depth	Test Allow.	Eq. 3.4-6	Eq. 3.4-7	Test Allow.	Eq. 3.4-6	Eq. 3.4-7	Test Allow.	Eq. 3.4-6	Eq. 3.4-7
2 in.	926	63	1000	1235	65	1040	1543	78	1240
3 in.	1466	211	1500	2006	219	1560	2006	262	1860
4 in.	1852	500	2000		520	2080	2469	620	2480



5 Fracture surface is compound, coplanar with bearing but also downward on diagonal toward beam center.

6 Cross-section of white pine specimen at housing after testing. Fracture extends along length of beam from bearing surface, also along growth ring down from bearing surface toward tension face of beam.

7, 8 Below left, terminal condition of white pine 3-in.-bearing test, with complete rupture on one side displacing thrust block clamp. Below right, top view.

ductility (or toughness), suggesting that higher-strength woods have both higher load-bearing capacity and increased toughness (Figs. 6–8). Whether this behavior is a species-dependent characteristic or can be related to specific gravity remains to be explored.

Since housings of the type tested typically occur in multiples along the length of a beam, testing is needed specifically to examine what effect spacing of the housings has on peak capacity for a given bearing depth, and to examine how cumulative shear in a beam affects the capacity of load-bearing housings closest to the end supports of a given beam.

We need to develop a better understanding of the failure mechanisms and process. Good agreement of test values with predicted capacity based on the 2012 *NDS* Equation 3.4-7 for split-ring connectors, shear-plate connectors, bolts and lag screws was found for this data set, but preferably we would develop a design formula conceptually better related to the fracture plane that appears characteristic of the failure mechanism. While the testing was limited and relatively unsophisticated, the results do provide a preliminary indication of the capacity of load-bearing housings taken in isolation, and of their behavior as the limit state is reached. Once a reasonable database of test results across a variety of geometries, species and spacings is available, the next task will be to take up numerical modeling so that reliable predictive design equations can be developed that apply across a broad range of geometries and species for use in timber frame design codes. The goal is to develop a capacity-based design formula for incorporation into our timber framing standard *TFEC-1* and the American Wood Council's *Timber Frame Design Standard*, under development.

—Thomas E. Nehil, PE, and Benjamin I. Trojniak, EIT

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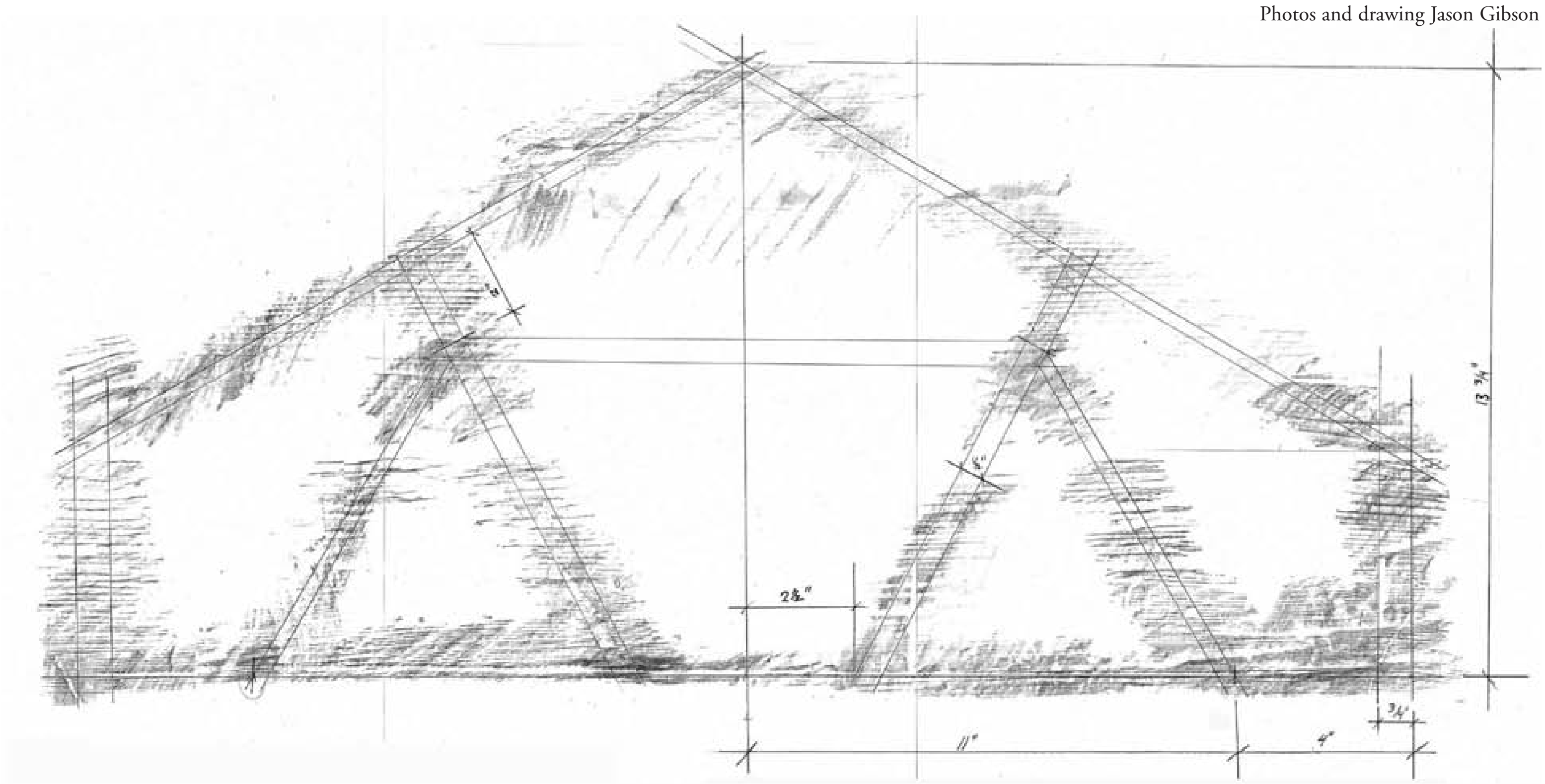
A Swing-Beam Barn Working Drawing

Photos and drawing Jason Gibson



A couple of years ago, we moved our shop from the vicinity of Kingston, Ontario, to a new location about 50 miles away. This involved converting a barn into a workshop, moving tools and people, and transporting our inventory of barn frames. During the move, my wife and I firmly agreed not to take *any more* barns into inventory, as our core business is new frames.

We were almost finished with this large move for our small company when I received a call from a client in our original neighborhood. He wanted a barn removed from his property as it was no longer in use and its roof would need repair soon. I weakened and went back to take a look. Standing up to my waist in old hay, ancient farm machinery and a variety of wildlife, I called my wife and said, “You have to see this one.” This barn had a five-bent, 30x48-ft. canted purlin post white pine frame in good condition. Those in a similar business and carrying the same defective gene know there was no option but to dismantle the building and bring it home.

Facing page at top, white pine swing beam in mid-19th-century Ontario 30x48-ft. repurposed barn, showing scaled section of roof frame drawn by builder. Facing page bottom, rubbing of drawing, corrected for effect of check, with author's superimposed measurements and added lines to show purlin post tie.

At right, rafters removed, barn frame being dismantled at original location near Kingston, Ontario. Note purlin post tie in gable end.

Below, frame cleaned, modified and reerected in Perth, Ontario, finished as show house for timber framing company. Swing beam measures 10x17 at midpoint, 10x10 at ends, and tie beam 10x12. Note doubled and twinned 4x4 braces from tie beam and swing beam, respectively, to wall post.

Jason Gibson

We decided to reerect this building right away as a show house for our company's work. (Visiting firemen are welcome too.) We brought the timbers home, washed them, made drawings for a structural review, reerected the frame and fitted it out as a house.

Throughout the dismantling, shipping and cleaning, we never noticed an original drawing on the swing-beam bent. When the reassembled frame was standing on the new subfloor, one of our framers pointed it out. For me this was a great find. I have always wondered how the layout was done for the canted purlin post and what sort of plans were used in the mid-1800s.

The drawing is scratched into the swing beam with an awl or knife, a scale drawing at one inch to the foot, and done accurately. The timbers appear to be drawn not as actual but as sized: although the canted purlin posts are hewn to 7 in., they are represented by ½ in. on the plan and in fact sized to 6 in.

As far as I can tell, the drawing sequence went thus:

1. The 30-ft. dimension was set, then the roof pitch drawn at 7 in 12 and a 4-in. depth drawn for the rafter.

2. The upper face of the canted purlin post was laid out on the top of the tie beam at 2 ft. 6 in. from centerline and angled to hit the midspan of the rafter.

3. The upper face of the strut was laid out 2 ft. down the canted purlin post from the top and 4 ft. in from the outside wall on the tie.

I do not see any arcs struck to create this layout, and the dimensions cited are calculated directly from the scale drawing. My guess is that the rafter length was taken from the scaled drawing, as well as the lengths and angles of the roof members. This drawing is a nice window into the working world of the past. —Jason Gibson

Jason Gibson (jgibson@ripnet.com) operates Gibson Timber Frames in Perth, Ontario. For a discussion of swing-beam barns, see TF 103.

The Alexander Knight House


1

Photos Cynda Warren Joyce

AS a child I lived in the Stow House, an impressive 1696 framed colonial saltbox in Stow, Massachusetts, with its exposed timbers, a narrow winding staircase beside a massive stone chimney, and wide pine floors. My father, who had restored the house, passed on to me a great appreciation for these buildings of hewn timbers. Lately I have participated in building a plausible small house in that style while using traditional tools and techniques, the Alexander Knight House in Ipswich (Fig. 1), the coastal town in Massachusetts where more first-period (1625–1725) houses remain than anywhere else in the country.

The idea for the Knight House project developed with the help of Ipswich architect Mathew Cummings and craftsman James Whidden (see photo back cover), who had come to Ipswich from Ashburnham in central Massachusetts to work on historic houses. The new building was to be a re-creation of a one-room, English-style timber-framed house built in 1657, using as nearly as possible the tools, materials and construction methods of the first period.

Alexander Knight, an innkeeper in Chelmsford, England, immigrated to Ipswich in the Massachusetts Bay Colony in 1635 and quickly acquired significant land, but his fortunes had declined as soon as 1641. The town appears to have granted him a small lot and house in 1657. Susan Nelson, an architectural historian and an expert on historic preservation in Ipswich, found this reference in the Town Register of April 1657: ". . . secure a house to be built for Alexander Knight of 16 foote long & twelve foote wyde & 7 or 8 foote stud upon his ground & to pervyd thatching & other things nesasary for it."

Whidden's and Cummings's work in modern times allowed them a unique view into the past as the skeletons of old Ipswich houses were revealed during restoration. Sound determinations regarding local techniques and materials could be made from this experience.

The Ipswich Museum agreed to host the project on the grounds of its 1677 Whipple House property and, in 2009, the Alexander Knight House team was formed of entrepreneurs and several skilled craftsmen who shared a keen interest in the project. In addition to Whidden, Cummings and Nelson, they included Richard Irons, restoration mason; Tim Chouinard, hardscape designer and builder; and myself, Cynda Warren Joyce, visual artist. Later, in 2012, Matt Diana, a housewright, joined the team.

The team used traditional tools throughout the construction process, providing ongoing demonstrations at the museum site, accompanied by lots of discussion. Sitework began by digging the cellar hole by hand, with shovels and volunteers. Tim Chouinard built the fieldstone foundation and Richard Irons the hearth, both using local materials.

1 Facing page, Alexander Knight House, nearly finished siding and thatching, a plausible 1657 construction on the grounds of the 1677 Whipple House in Ipswich, Massachusetts. Matt Diana shown preparing to set donated leaded-glass window in opening.

2 Alan Ganong trimming white pine log at Ledyard, Connecticut, sash sawmill, to produce siding and flooring for Knight House. Carriage stops before cut is complete and board must be pried off.

3 Plate assembled to post on blocks in shop, showing teazle tenon and half-dovetail housing for tie beam in English tying joint.

4 English tying joint in oak, signature assembly of first-period New England houses.

5 Matt Diana, left, and Jesse Brown assembling tying joint during raising. Dovetail flare is on far side of tie beam. Crushing on near face likely resulted from previous test assembly.

The carpentry began by selecting white oak for the frame. Early New England builders chose white oak for its strength, rot resistance and nearer resemblance to English brown oak than other New England oaks. While white oak was abundant in 1657 in Ipswich, it is relatively scarce today. The best trees for hewing grew in the deep forest, without lower branches, thereby eliminating knots for considerable lengths of timber. After a tree was felled, it would be cut to length and hewn where it lay into the major framing timbers—sills, posts and principal rafters. (In our case, but for one that became a timber, we did not fell the necessary trees.)

Jim Whidden, Matt Diana and Jesse Brown hewed and prepared the timbers. To minimize waste and effort, logs of appropriate diameter for each timber were needed; some wane was acceptable (and even desired for authenticity). Whidden acquired white oak locally that was too dry and too large, and thus difficult to work with. These logs went to working 19th-century sash sawmills to be converted into scantlings, the smaller members of the frame. (Powered by water, sash mills feature blades stretched in a sliding wooden frame or *sash*, crank-driven to travel up and down, and were in wide use from the 17th through the 19th centuries. Records exist of one granted in Ipswich as early as 1649.) Green logs of a smaller diameter were required for the main members of our little frame. Fortunately, I found white oak in my own firewood and sought out the producer in central Massachusetts, who then became the source for our white oak logs.

The finish floors and siding required wide, clear white pine boards appropriately sawn at a sash mill. In 1657, 24-in.-dia. and larger clear pine would have been plentiful (while the king's pine, over 36 in., was sent to England). Suitable 12-ft.-long white pine logs were donated from Maine, and additional logs 16 ft. and longer were purchased in Connecticut from a managed forest.

Whidden delivered many logs to the sash sawmills (in Ledyard, Connecticut, as seen in Fig. 2; Derry, New Hampshire; and Sturbridge, Massachusetts) to prepare the scantlings and boards. The large pine was sent on logging trucks. We worked closely with the sawyers who cut both the white oak and the white pine for the job.

Whidden used scribe rule to lay out the joinery. The layout, cutting and trial fitting of the main frame was done at his shop (Fig. 3). The raising took place in September 2010, with many hands to help (Figs. 4, 5). Volunteer Dick Chapman fashioned white oak trunnels as needed during the raising. The raising crew set the principal rafters by the end of the day. Temporary siding and flooring served as we awaited finish materials. Since much secondary framing remained to do for the walls, roof and chimney, work continued through the following summer and fall. We each cut a few mortises, sawed a few studs or fashioned a tenon in the ever-hardening oak.

6 Thatchers Michael French of Plimoth Plantation and Lorin Johnson, a friend of the Knight House team, place bundled rolls at chimney base to hold next layer. Bundled rolls surround chimney and roof perimeter.

7 Using giant needle, Justin Keegan of Plimoth Plantation weaves hemp cord back and forth through thatch down to roof lath to secure transverse spar.

8 Justin Keegan affixes first course of final layer of dressed bundles. Final treatment of thatch remains to be determined at and near ridge (Fig. 1), with several styles under consideration.

The roof system and chimney framing were nearly finished when, sadly and unexpectedly in late November 2011, our housewright and inspiration Jim Whidden died at the age of 49, leaving a large void and much work still to be accomplished. The team and his friends and family were determined to finish the project, and many donations were made in his memory.

Matt Diana, who had been employed by Whidden for several years and shared his passion for traditional building, decided to take up the reins. He had previously worked on the Knight House and volunteered as our new housewright for the remainder of the work. He finished the roof and chimney framing before winter came again. Matt Diana also framed the windows and door and rived lath of white oak for the roof-thatching and the wood-framed chimney.

We met with artisans from Plimoth Plantation, who came to Ipswich and showed us how to harvest thatch from the Ipswich marshes and prepare it for the roof. In the spring of 2012, the team collaborated with Plimoth Plantation's Michael French and Justin Keegan, who agreed to thatch the roof (Figs. 6–8). Along with their expertise, instruction and considerable time, they supplied appropriate tools and additional necessary materials. We prepared long bundles of reed and tied them around the edges of the roof to hold the thin *fleeking*, a mat of tall grasses, then a thick layer of wet tangled hay. Another row of long bundles around the edges held the finished courses begun at the bottom, laid and dressed and worked up to the ridge. Spars held the dressed bundles in place, woven onto the riven lath using a large needle and cord. Thatching in progress often brought traffic to a stop along South Main Street in Ipswich.

Meanwhile, the chimney frame was fitted with riven white oak lath in preparation for plastering with a daub of local clay, straw and sand. Mason Jeremy R. Brown applied the daub to the wet interior of the chimney, hauling buckets to the top and working his way down (Figs. 9 and 10), the daub having been mixed in frames by barefooted volunteers directed by Richard Irons (Fig. 11, left). In 1657, home fires would have burned year-round on a broad, deep hearth, for cooking and to heat water for washing.

9 Jeremy R. Brown applies daub to framed, heavily lathed interior of spacious wood-framed chimney wetted down in Fig. 10, starting at top.

10 Chimney framing integrated with end wall of house. Note collar beam, studs, end rafters. Pulley wheel and rope allowed mason to haul daub bucket to top to start job, then to any point below.

11 Richard Irons, left, oversaw mixing of local clay, straw and sand into daub for chimney. Daub mixer Kai Colombo, center, a restoration glass artist, donated a leaded-glass window to the project.

The wide Eastern white pine boards for finish work eventually arrived from the mills, to be used for the exterior bevel siding and interior flooring after decay-prone sapwood was removed. Early houses had plank doors of clinch-nailed, shiplapped pine boards, which Diana built and finished with reproduction hardware. Salvaged wrought-iron nails, along with hardware forged by George Ivan Paré, of George Forge in Rhode Island, and Alex Bandazian, a blacksmith at Plimoth Plantation, can be seen throughout the house. In 1657, an oilcloth or shutter normally covered an opening in a house of this size; glass, a luxury at the time, was imported and therefore very expensive. But an important donation to the Knight House arrived from restoration glass artist Kai Colombo (seen treading daub in Fig. 11, center): a diamond-paned, leaded-glass window. The window was fitted to the house despite the fact that historically it was not appropriate for such a modest dwelling. Perhaps it might have been given to Knight by a wealthy benevolent neighbor?

The team will complete the Alexander Knight House later this year and formally donate it to the Ipswich Museum. The house will then become a permanent exhibit on the grounds of the Whipple House, offering visitors a chance to see how an ordinary person lived in the Massachusetts Bay Colony.

Using Colonial timber frame construction methods and related authentic processes to build the Alexander Knight House, I met people who showed uncommon commitment and freely shared their knowledge. Along with our timber framers and carpenters, the sawyers, thatchers, masons, stonemasons and blacksmiths all displayed passion for their work, and the countless volunteers' and contributors' interest and enthusiasm were a delight to behold. Although I too possess passion for the things I do, I will be forever changed by the experience of working with these men and women, the builders of our world. —Cynda Warren Joyce

Cynda Warren Joyce (cwjdesigns.com) is an artist and designer living in Ashburnham, Massachusetts. More information on the Alexander Knight House is available at ipswichknighthouse.org.

Wooden Houses of Istanbul

HANIM EFENDI," confided the shop owner in Kadiköy to the lady who had come in seeking directions, "the Istanbul that you knew no longer exists." Sadly it was true, the city of 60 years ago, known by long-established families of Istanbul, had been overrun by progress, life, and the ebb and flow of 10,000,000 more people trying to make a living. But if some of the old houses could be preserved, perhaps some small part of what had been known long ago could still exist as modest visual testimony to a nearly vanished culture, to what was treasured and how things used to be.

Istanbul, the only city in the world that stands astride two continents, is the second largest city of Europe, with an official population of 13,500,000 in 2012, according to the Turkish Statistical Institute. For over 2000 years it has been the crossroads of caesars, emperors and sultans. Twenty-five synagogues, dozens of churches and hundreds of mosques indicate the multicultural complexion of the city and pay homage to the past.

Dividing the European and Asian shores of the city is the Bosphorus Strait, through which a river of taxable Black Sea commerce flows on its journey to the Mediterranean and beyond. One of the first settlements of the historical city center was Lygos, founded in the 13th century BC on the peninsula known as Sarayburnu (Nose of the Palace), where stand Topkapi Palace and the Hagia Sophia Museum and the Sultan Ahmet Mosque. Sarayburnu, also known as Palace (or Seraglio) Point, throughout the centuries has been the historical and imperial city center.

After visiting Istanbul several times over the past 37 years, I have discovered small, delightful gems in unexpected places. Exploring the city and peeling back its layers is an adventure and treasure hunt. There are many examples of monumental architecture in Istanbul but until recently 500 years of domestic architecture was on the brink of extinction. This valuable heritage embodied in Ottoman timber-framed houses has been recognized in the nick of time. These houses, modest in size and limited in number, have become a cause célèbre as legitimate candidates for restoration, to promote urban revival and oppose gentrification of the city core.

The simple historical Ottoman wood-framed house in Istanbul used modest-size timbers for the second and additional floors, set on first-floor walls of stone or other masonry. The design was "mandated by an imperial edict in 1509, when a severe earthquake caused destruction in the predominantly stone housing stock" (Gülkan and Langenbach 2004) and had something in common with North American houses during our period of rapid westward expansion. They were the tract houses of their time, three or four stories high and about 20 ft. wide, built close together in the crowded city. They were simple to build and easy to replace when destroyed by frequent fires and the occasional earthquake. Logs of oak, beech, pine, walnut and linden were imported through the Black Sea for manufacture into millwork and ornament. Abundant local yellow pine was sawn into structural posts, beams and studs, as well as siding and flooring. Steam-powered sawmills appeared during the mid-1850s. The efficient mills provided large volumes of standard-sized, affordable lumber required for efficient house-building in the growing city.

Walls were studded but with 6x6 corner posts and 3x6 secondary posts on 3- or 4-ft. centers. These vertical members joined horizontal members with lap joints, notches or butt joints fastened by square iron boat nails. Window and door frame elements were 2x6 and 3x6. Plates both top and bottom were 4x6 and 6x6. Diagonal braces, sometimes full wall height, were 6x6 (shown in modern rendering, Fig. 1, and elevation, Fig. 5).

The third floor starts on the top plate with 3x8 or 3x10 floor joists. Timber connections to the plates included simple notches, modest size mortise and tenon joints and butt joints reinforced on each side with blocks fastened to the plates (Fig. 2).

Roof structures were simple. As a general rule, the moderate climate of the city means few big snow loads. In addition, the short spans provided by interior support walls kept roof trusses to a fairly basic form with small members. Roofs were gabled or hipped with rafters extending to the corners of the building and beyond in typically 3-ft. overhangs.

The builders were not purists and, like many house-builders today, they used a variety of techniques to complete the frame as efficiently or as economically as possible. As metal fasteners became more readily available, nails, straps, staples, and drift pins were freely used. On the roof, typical clay tiles were fastened to 1x2 skip sheathing or 1x3 battens over the roof trusses.

Construction styles In the *Hımış* half-timber style, wall cavities were empty or filled with brick, sticks, adobe or rocks, according to materials available and regional style and precedent. Bricks used for infill might be laid and roughly mortared in alternating courses angled at about 30 degrees (Fig. 2), while interior and exterior cladding covered them from view. Or they might be laid neatly and carefully mortared to fill a decoratively subdivided timber framed wall (Fig. 3). In the *Dizeme* half-timber style, short boards nailed in provided the infill, looking in part like close studding but also fitted horizontally above windows (Fig. 4). *Bağdadi* construction (Fig. 6), not really half-timber, called for a matrix of small timbers infilled by sticks, branches and trim-ends, all covered inside and outside by lime mortar (although the term *Bağdadi* is also used sometimes to describe walls with no infill at all).

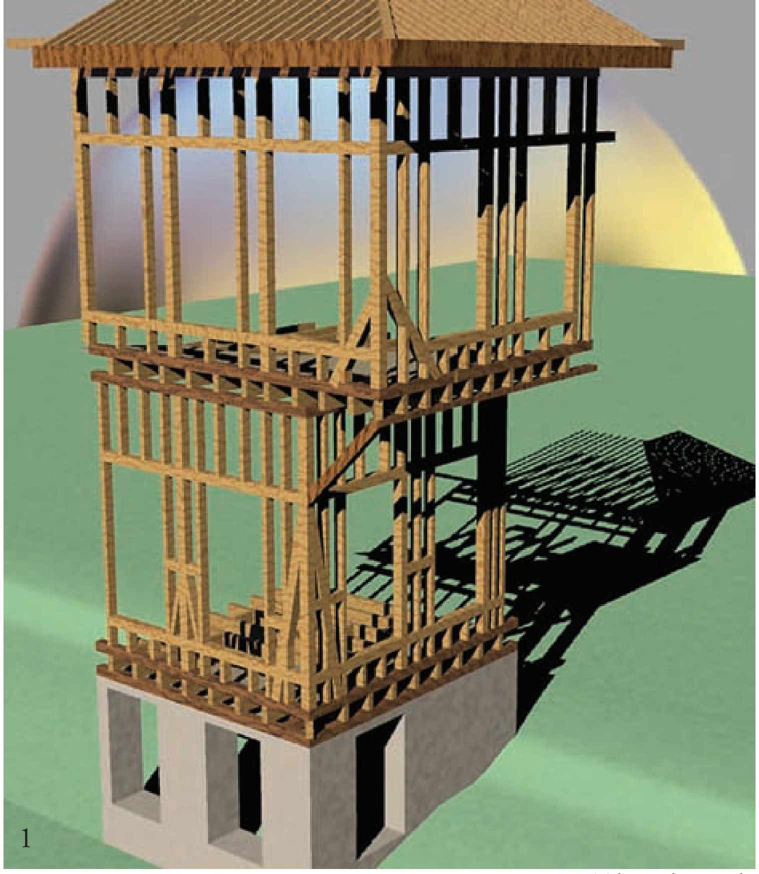
In *Hatil* construction, however, wood plays mostly a binding role in a masonry wall. Horizontal timbers lie in courses up to 5 ft. apart, and openings are further framed by wooden jambs (Fig. 7).

Reinforced concrete building failures in the Kocaeli earthquake of 1999 and the Düzce earthquake of 1992 led some to reconsider traditional Turkish wood construction. "Many of the ancient traditional timber framed houses remained intact, only a few were heavily damaged. However, reinforced concrete buildings presented a high level of damage," reported Stephen Tobriner of the University of California in 2000. In a paper written for the United Nations, Tobriner concluded that during earthquakes historic Turkish timber frame construction was equal to and often better than the reinforced concrete buildings that then stood.

Dizeme-style construction is thought by some to be best during earthquakes, and Gülkan and Langenbach (2004) observe that all *Hımış*-framed buildings enjoy the advantage of ductility compared with reinforced concrete.

Nevertheless, in occasional earthquakes and recurring fires, all examples of original Istanbul wooden houses dating from the 1500s have been destroyed. Most surviving examples of wooden houses date from the 1780s to the early 1900s. There are only about 250 wooden houses, in various states of repair, left in all of Istanbul.

Interior walls of finished frames might be covered with lath and smooth plaster, exterior walls with lath covered with adobe or plaster, or siding boards. Starting around 1895, Istanbul wood-framed houses were left with no infill of any kind, merely empty wall cavities between exterior and interior finish. Exterior wall covering changed over time from plaster to wood siding, using square-edged boards about 1x8, later evolving to shiplap siding with flush, beaded or V-joints (Fig. 8).



Tulay Cobancaoğlu

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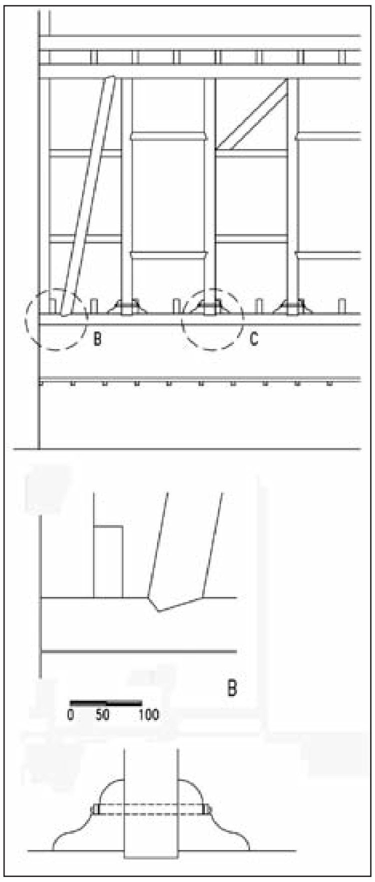
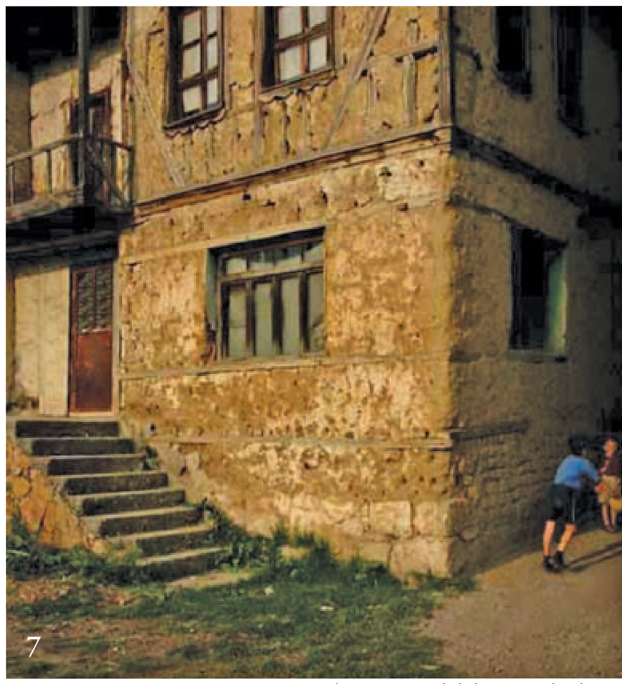


5 Tulay Cobancaoğlu



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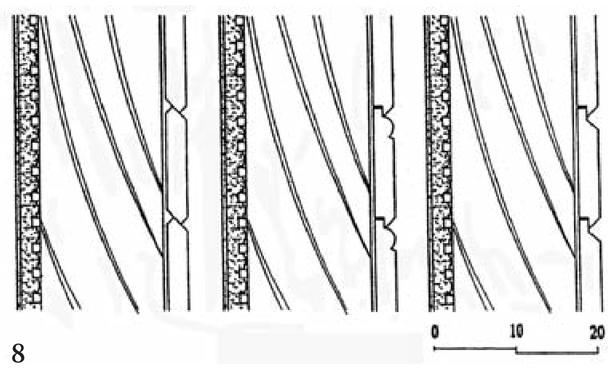

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1 *Hımış* construction model, here 19th-century Istanbul domestic timber framing.

2 Rough brick infill for utilitarian or covered *Hımış*-style wall.

3 Fine brick infill for exposed *Hımış*-style wall.

4 *Dizeme* construction, named for its lightly nailed wood infill of short lengths.

5 *Hımış* frame details in elevation.

6 *Bağdadi* construction in Gölçük, with rough wooden infill, covered in service.

7 *Hatil* construction for first story of building in Bayirköy, *Hımış* story above.

8 Exterior siding patterns, with plaster to inside of cavity wall.

The oldest is the Divanhane, a waterfront house built in 1699 on the Asian side of the city near the south entrance of the Bosphorus. It was a high-end structure built as part of a series of traditional buildings on a large estate, the Amcazade Yalısı, and hangs over the water with typical low window sills to provide views and abundant sea breezes during the hot summers. Partial and temporary restoration in 1908 and 1947 addressed badly deteriorated foundation posts. Additional recent work has been more extensive. Located at the edge of the lot and perched on a built-up foundation wall, the house cantilevers out 6 to 8 ft. above the Bosphorus, supported by 12-ft. diagonal braces from the outside edge of the building back to the base of the foundation (Figs. 9–10). Not surprising, these diagonal piles have been an ongoing maintenance headache for restoration projects.

The largest historic wood-framed structure in Europe today is on the Princes' Islands, a short trip from the Golden Horn ferry terminal in downtown Istanbul. The Prinkipo Palace Hotel, built in 1899 and today also called the Rum Orphanage, was designed by the architect Alexandre Vallaury and set on the large island among summer mansions of the well-to-do Jewish, Armenian and Greek city dwellers who escaped the city heat for several months of the year. (Vallaury also designed several government and commercial buildings that stand today, notably the recently renovated Pera Palace Hotel, 1895, located north of the Golden Horn in Little Europe.) Prinkipo Palace's square footage of the main sections plus extensions and auxiliary spaces totals about 190,000 sq. ft. The multiple cascading cantilevered overhangs are a notable design feature on some sections of the structure (Fig. 11).

Prinkipo Palace, as well as being a hotel, was designed as a resort and casino for eastbound travelers in transit through Istanbul leaving from Haydarpasha Station on the Baghdad Railway east to Iraq or the Hejaz Railway south to Damascus and the Arabian provinces of the Ottoman Empire. On completion of the building, however, the Sultan refused to give his final approval for its use as a resort casino. So the structure was later donated to the local Greek Orthodox Church and operated as an orphanage for 50 years. It has sat empty and unused since the early 1960s. There is discussion of restoration.

Luxury mansions Magnificent historic wooden mansions survive in various stages of restoration today (Fig. 12). Built between 1890 and the early 1900s, these iconic estates, embassies and compounds were constructed for Ottoman officials, expatriates and wealthy Armenian and Greek businessmen. Many stand in poor repair, but there has been interest in restoration over the last 10 years, sometimes paid for by subdivision and building modern townhouses on part of the former lots. A good example is the restored Ragip mansion, designed and built in 1890 by the Austrian architect August Carl Friedrich Jasmund (Fig. 13). It sits on the waterfront half of the original lot. The front four or five acres close to the busy street have been redeveloped as modern upscale housing. Upgrading and historic restoration of the original house close to the water have been completed.

One of the biggest problems of wood-framed construction in crowded urban environments is fire, which occurred with regular frequency over the centuries, destroying the oldest Ottoman houses. After 1865, new wooden structures built close together were generally banned unless a firewall was included between them. The Yıldız house (Fig. 14) is a duplicate of what used to be in this neighborhood. On either side are the concrete and masonry block apartments that replaced wooden houses starting in the 1940s and 1950s.

Destruction by fire continues to this day. In January a serious fire destroyed a timbered roof of the Feriye palaces on the Bosphorus at Ortaköy, a borough just north of Beşiktaş. The palaces were built in 1871 by the Ottoman-Armenian architect Sarkis Balyan, whose other work includes the Yıldız, Ciragan and five other palaces, 15 mansions and many public projects.

Wealthy homeowners opted for stone whenever possible for their waterfront estates in the suburbs. New construction in the Sarayburnu, and across the Golden Horn in Beyoglu, Pera and north into Little Europe, used masonry—blocks, brick, stone or reinforced concrete. From Pera on the Golden Horn uphill toward Taksim Square in the European business district were the trading houses, banks and foreign embassies, which remain. These were built primarily of masonry walls with timber-trussed roofs. After 1900 steel beams were commonly used even in houses.

The future For about 600 years until the early 1900s, wood-framed houses were the predominant residential structures in Istanbul. From the 1940s onward, the typical fate of wooden houses was to be replaced by concrete apartment blocks. By 1980 the last of these wooden houses were on the verge of extinction. Conflicting forces of population explosion, infrastructure projects near historic areas and the economic push to build high-rises have put many historic areas at risk. The pendulum has swung back somewhat toward conservation, with influential academics, government officials and forward-thinking businessmen advocating it in recent years. Perhaps the trash, demolish and burn philosophy of urban planning is a thing of the past. The International Council of Monuments and Sites in Paris and the Istanbul organization Koruma Uygulama Denetim Müdürlüğü are in the forefront of conservation of the architectural and archaeological heritage of Istanbul, the latter with an apprenticeship program for traditional millwork, cabinetmaking and carpentry, which so far has repaired 55 houses. The value of the remaining few hundred wooden houses has been recognized. What would it be like for members of the Timber Framers Guild to participate in a restoration project in Istanbul? —Bruce Lindsay

Bruce Lindsay (brucelindsay@shaw.ca), operates Evergreen Specialties in Vancouver, British Columbia. He was materially assisted in the preparation of this article by Professor Tulay Cobancaoğlu and others at the Mimar Sinan Fine Arts University in Istanbul.

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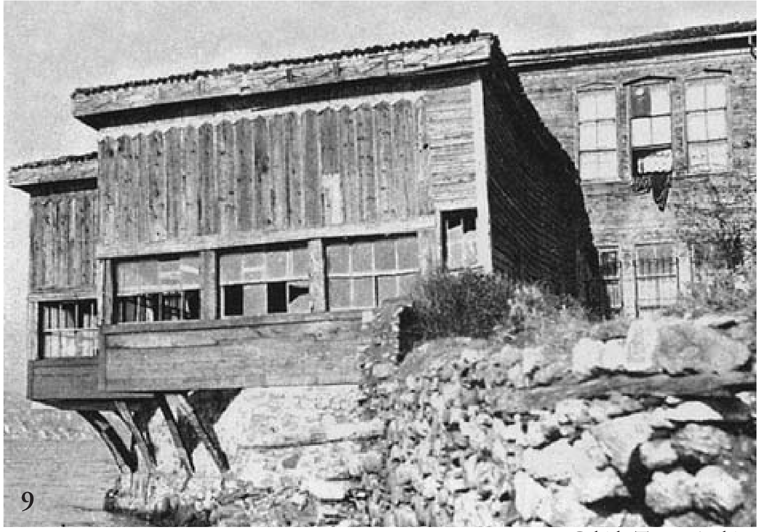

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Bernd Seeland

Emine Erdoğmuş, World Monuments Fund

Above and below, Bruce Lindsay

9 Archival photo of divanhane at Amcazade Yalısı, 1699, Istanbul's oldest wood house.

10 Basswood 1:20 scale model of Amcazade Yalısı at Karlsruhe University, showing interior vaulting and dome framed independently of roof.

11 Prinkipo Palace Hotel, 1899 (also called Rum Orphanage), thought to be Europe's largest framed historic structure.

12 Wood mansion built at turn of 20th century for the well-to-do, now candidate for restoration.

13 Ragip waterfront mansion, 1890, architect-designed, now restored on subdivided lot.

14 Yıldız house, a replica of what was once typical housing.

Structural Screw Technology

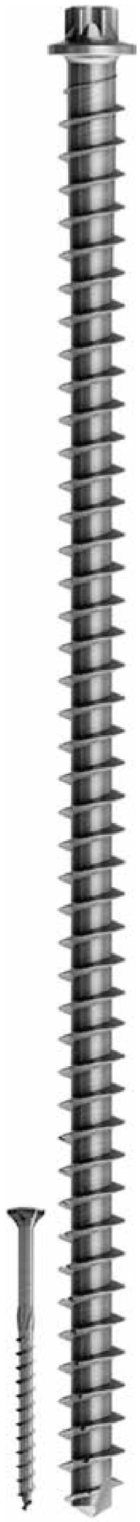
CAN you imagine a world without screws? The invention of the screw thread was an exceptional achievement and we benefit from it significantly today, though we can neither name the inventor confidently nor determine the moment when the screw thread first appeared. (Archimedes, 287–212 BC, is often mentioned as the inventor and, earlier, Archytas of Tarentum, 428–350 BC.) Possibly the idea of the screw came to someone's mind when observing forms and behavior in nature. Falling, spinning maple seeds and snail shells are two of numerous suggestive examples from nature.

Instead of squeezing or driving one object into another, it is easier to turn it in with a consistent rotating motion. Primitive screws may have been made by coiling grasses and cords obliquely around cylinders of wood or metal, according to Volker Benad-Wagenhoff in *Schrauben und Gewinde* (Thorbecke, 1992), a treatise on screws and threads. Screw threads in wood developed in very early times for agricultural presses and in metal by the late Middle Ages. Mass production of threads in metal began during the 18th century, and threads were standardized by the end of the 19th. Rolled threads (cold-forged, not cut) appeared in the late 19th century and, with improvements, came to dominate in the 20th. Our fasteners have evolved further in the 21st century and now include engineered structural screws. The functional principle of the screw—a continuous inclined plane wrapped around a cylinder or cone—remains the same.

The fastener industry has evolved right along with the engineered-timber industry and its development of high-performance products such as structural-composite lumber and cross-laminated timber. Manufacturing issues of sustainability and environmental concern are addressed by the implementation of energy-management systems and ISO50001 certification of manufacturers (in Europe, so far). To keep up with ever-larger available timber sections, screw manufacturing technology must provide ever-larger screws in a variety of diameters, head shapes and thread shapes. Structural screw diameters range from ¼ in. (6mm) to 9⁄16 in. (14mm), and lengths range from 2 to 59 in. (Fig. 1). Screws smaller than ¼-in. dia. are not commonly used for structural applications.

Engineered structural screws fall into two main categories: partial thread and full thread.

Partial thread Partial-thread screws (Fig. 2) can be supplied with different heads such as washer, countersunk and hex, and with rough threads (large thread spacing and steep thread pitch) to increase drive-in speed. Structural partial-thread screws are typically available in diameters from ¼ in. to ½ in. and in lengths from 2 in. to 40 in. Most have a cut-point or "self-tapping" tip to reduce wood splitting, but for fast and aggressive bite into the wood, some have threads rolled onto the tip (Fig. 2a). The threads engage with the wood fibers right away and the screw can be driven quickly.



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Washer-head screws pull timber members tight together and often eliminate the need for assembly clamps. The increased bearing area under the large head distributes compression stresses and reduces wood crushing, and hence draws timber members together without sinking the head deeply into the timber. With an increase in fastener diameter, an increase in head diameter naturally follows to accommodate higher loads and ensure sufficient draw capacity for larger timbers.

Countersunk-head screws are designed to be driven flush with the timber surface or to be counterbored if desired. The head is typically equipped with milling pockets and cutting edges to reduce wood tearout for a clean finish at the surface. For steel-to-wood connections, a countersunk predrilled hole is required in the steel plate to accommodate the screw head in the usual way for a flush finish.

Hex-head screws are engineered for wood-to-wood and steel-to-wood connections. In all-wood connections, the screw serves as a multipurpose fastener without features tailored to a special application. In steel-to-wood connections, however, a thick reinforced and tapered shoulder under the screw head (Fig. 2, at right) centers the screw and yields tight connections with reduced initial connection slip. The hex head provides a decorative finish when steel plate connections are used as an architectural element and allows socket driving when assembling steel elements.

Many engineered screws are equipped with a so-called shank cutter right above the threads (Fig. 2), designed to enlarge the core hole and reduce necessary drive-in torque through friction reduction (or clearance) at the screw shank. (To help further, modern screws are supplied with a friction-reduction coating on the entire screw.) Considering that available partial-thread screws can have unthreaded shanks up to 34 in. (860mm) long, a shank cutter becomes an essential feature on long screws. In addition to friction reduction, the enlarged core hole also allows the wood to settle freely in log or timber assemblies subject to shrinkage, or to shrink around the screw shaft locally. This allowance is of particular importance whenever green wood is used and shrinkage is expected through the service life of the structure.

To allow for shrinkage and corresponding settling, it's important that screw threads are embedded only in one timber (Fig. 3). Equally important, the screw can pull the timbers tight only when threads are embedded in just one connection member. Threads embedded in both members may drive members apart, immediately if in an unclamped assembly, or, even if initially clamped, after shrinkage of one or both members.

Full thread Full-thread screws (Fig. 4) are used predominantly in engineered connections in kiln-dried wood with moisture content 12–15 percent, where high forces are to be transmitted between timber members. Full-thread screws are also used to reinforce or repair wood across the grain. They are typically available with a nearly cylindrical head or a countersunk head.

A so-called cylindrical head (in fact slightly tapered) is relatively small in diameter and easy to sink into the wood. The small hole can be plugged with a grain-matched plug and the fastener fully concealed. (A fully concealed fastener can achieve fire ratings of several hours under sufficient wood cover. Having the fastener fully embedded into the wood with a certain wood cover thickness may also protect it from direct contact with corrosive atmospheres such as in roof structures over swimming pools.) Because the small head is hardly visible in wood, this style is favored for upgrades in existing structures or to meet concealed connection requirements. Full-thread wood screws with cylindrical heads are in general not suitable for steel-to-wood connections.

Countersunk-head full-thread screws are suitable for multipurpose use in wood-to-wood and steel-to-wood connections. The

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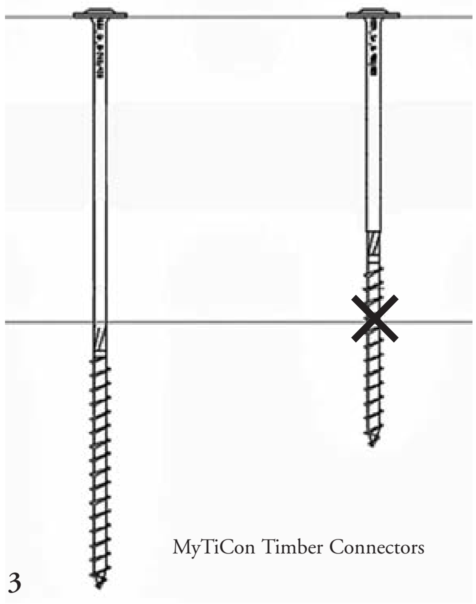

MyTiCon Timber Connectors

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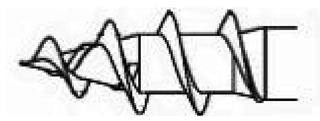

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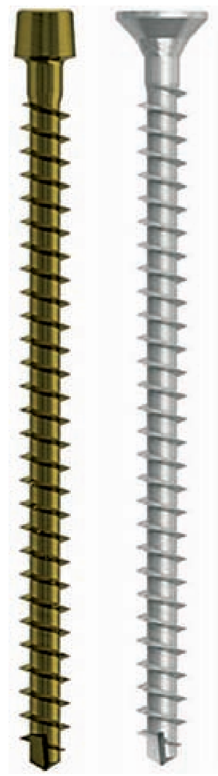

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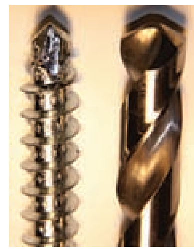

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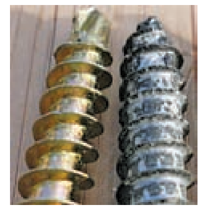

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MyTiCon Timber Connectors

Illustrations SWG Schraubenwerk Gaisbach Gmbh except where noted

1 12-in. partial-thread and 59-in. full-thread screws compared.

2 Partial-thread screws with (left to right) washer, countersunk and hex heads. Note shank cutters above threads and countersink cutter. Threads rolled onto tip can provide quick engagement (2a).

3 In assemblies, threads must embed in one member only (left) lest members be driven apart by threads during assembly or seasoning.

4 Full-thread screws with "cylindrical" and countersunk heads.

5 Robust hexalobular head for long and large full-thread screws.

6 Screw tip functions roughly as drill bit, minimizes splitting.

7 Smaller core diameter of newer screw on left yields reduced volume and broader threads, both advantageous.

countersunk head provides a flush finish in any wood-to-wood connection and, in combination with a countersunk predrilled hole, also can be used in steel-to-wood connections.

Hexalobular screw heads (Fig. 5) are designed for extremely high-torque transmission required to drive the biggest full-thread screws. The large amount of thread generates high friction forces that must be overcome at the screw head to create forward motion. This special head shape, driven by an inverted-Torx socket (and used also in inverted form on small screws), is found on full-thread screws exceeding ⅜-in. dia. and 40-in. length.

Most full-thread screws are equipped with a "self-tapping" tip that functions like a drill bit (Figs. 6, 7). The tip is thread-free and hence requires more pressure and effort to get the screw started, but provides better predrilling equivalence and reduces splitting. Also, fastener spacing as well as end- and edge-distances may be reduced.

Reinforcement The principle of structural screw wood reinforcement can be understood by analogy to concrete reinforcement. Concrete is weak in tension and therefore reinforced with steel wherever tension stresses are to be transmitted. Wood is weak in tension perpendicular to grain (splitting) and longitudinal to grain (shear failure), but reinforcements are not yet commonly applied. The full-thread screw embedded in wood can be compared to a steel rebar enclosed in concrete. Just as the rough-textured surface of the rod is gripped by the concrete, the threads of the screw are continuously bonded to the wood and efficiently transfer tensile stresses. The wood is now reinforced and can transmit high stresses perpendicular to the grain. This reinforcing technology can also be used in checked or cracked timbers where fissure size has become a structural concern. Full-thread screws can be used efficiently as well to repair glulam beam delaminations, following the same mechanical principles.

Full-thread screw reinforcement may be of assistance in traditional timber framing joinery conservation. Joinery in historic structures that has suffered damage from accident or long-term overload, and which would otherwise require costly repair or replacement, may be satisfactorily reinforced by full-thread screws, thus retaining original fabric.

A key advantage for all structural screws, however, is that predrilling with a bit to make a pilot hole for the screw body is no longer needed. In conjunction with the special screw tip design to reduce splitting, a further, less obvious, feature assists direct insertion. A special hardening procedure yields high steel strength, allowing for a smaller core diameter on the screw. With a smaller core diameter, the volume of the screw decreases and the wood sustains smaller splitting stresses when the screw is driven in. In addition, a smaller core diameter of the structural screw yields broader thread wings and therefore more bite into the wood fiber, ultimately yielding higher withdrawal resistance (Fig. 7).

One might suppose structural capacity is reduced by a smaller core diameter, but this is not the case because engineered structural screws are heat-treated and hardened to a high strength level while maintaining flexibility. Manufacturing standards in Europe require the screw to be able to bend up to 45 degrees without breakage. Structural screw breakage during installation, or later breakage in service, is thus rare. —Max Closen

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Reflections on Load Capacity of Historic Covered Bridges

IN nearly 40 years of work on timber buildings and bridges, I have repeatedly encountered in historic covered bridges the mystery of apparent reserve capacity for live load. Routine analytical evaluations using current specifications indicate these bridges should have fallen down long ago, yet they continue to support vehicles with no apparent distress. Why?

While most of our remaining covered bridges were built after the basics of engineering analysis had been established in the middle of the 19th century, no standard design specifications were available to the builders of these structures. Extant bridges may have survived for a variety of reasons, but not because they were built in accordance with modern design practices.

Standardization of timber specifications did not commence in earnest until the late 1930s (with a subsequent big push during World War II), after almost all of the remaining covered bridges had been built. Builders could size bridges based on past experience or instincts without the need for numbers to document the dimensions needed. They had access to great-quality old-growth timber and were astute enough to place the best pieces where they would be exposed to the highest forces.

Heated exchanges occur in public meetings about repair or replacement of covered bridges. Bridge lovers want no noticeable modification of these wonderful old structures and challenge the engineer to find a way to avoid any proposed changes. The engineer, saddled with responsibility for public safety, has to be able to document such safety in keeping with the current standard of care. A lack of appreciation of the complexity of the structure and the nature of timber is a major part of the problem.

If we examine how timber design specifications were developed, we may be able to shed some light on why historic covered bridges seem to have more capacity than analytical evaluations indicate they should.

Wood vs. lumber vs. timber Wood is the material of the tree and is used here in reference to small pieces prepared for testing. In the US, lumber is sawn wood in specified dimensions, and much of the specification and historical development to be cited is technically aimed at lumber elements. Generally speaking, timber is large sawn or hewn elements. (Wooden-bridge engineers habitually identify themselves as "timber engineers," not "wood engineers," a nuance not intended to confuse.)

Although this discussion is limited to timber elements without regard to their connections, it must be recognized that the structural capacity of a historic timber structure is almost invariably controlled by the connections. The analytical review of connections, however, is a world unto itself whose inclusion would not illuminate our particular question.

Material background Like metal, timber reacts to loading in a generally predictable linear elastic manner up to a certain point, after which the relationship of stress to strain becomes nonlinear as the element is loaded to failure. Unlike metal, however, timber is anisotropic, with significantly different stress/strain properties depending on the direction of loading with respect to grain. Timber variation—most notably the extreme difference between cross-grain and parallel-to-grain cell structure—affects its strength, as do the interruptions of knots and the slope of grain to loaded surfaces. Variations in density and moisture are also significant factors.

Testing and material variability One of the first challenges when developing design standards is to confront the variability of the material. The chosen method of wood technologists was an extensive testing program of small clear specimens of the species in question, roughly 2 in. x 2 in. x 30 in. long with straight grain and no knots or other apparent imperfections, which served as the starting point for predicted strength. Each species for which a design parameter would be developed had to have a sufficient number of samples and tests to produce a statistically reliable value.

Inasmuch as this effort aimed to establish values for design of new structures, another question must then be confronted. In a large number of specimens, one might expect a plot of the results to follow a normal distribution curve (bell curve) of strength values. What value should then be selected from the normal distribution curve to serve as the basis of design? The mean? The 25th percentile? The 10th percentile?

The 5th percentile, commonly known as the 5 percent exclusion value, was adopted. That is, if 100 tests were performed on small clear specimens, the desired value would be found on the normal distribution curve where 95 percent of the test results were higher. It would be adopted as the basis for the strength of the group being tested. In other words, statistically one would expect that 95 out of 100 elements would have greater strength than the value used for sizing of elements. Does that not seem appropriately conservative?

I have delved into the history of this decision because it seems a part of this exercise that may justify a difference between design of new versus evaluation of existing elements. The 5 percent exclusion value is a refinement of earlier work by John Newlin, chief of the Timber Mechanics Division at the Department of Agriculture's Forest Products Laboratory (FPL) from 1910 to 1939. Newlin recognized the need to account for "within-species variability." To do so, he chose to multiply the mean of the tests by 75 percent.

This approach produces results quite close to the 5 percent exclusion value for most species of wood. I find this more visually clear—25 percent less than the mean value—and still seemingly conservative. History shows that the 5 percent exclusion value has worked quite well for nearly a century. But keep in mind that this procedure was the basis for design, not evaluation.

Next, a related question: How confident must we be that the normal distribution curve adequately represents the true strength of the group? 100 percent? 90 percent? 75 percent? A number of curves could be used to represent the test results. A "normal" distribution curve (a bell curve) was settled on by the industry, and then that required a decision as to how closely one demanded the curve to represent the tests. (This part of the work is aimed at how many tests are required for an average—2, 5, 35, 137?) The number of tested specimens for a given grouping was selected so that the normal distribution curve would represent a 75 percent confidence level in the results. A higher confidence level was considered unnecessary in combination with the 5 percent exclusion value, and there was a practical limit as to the number of tests that could be funded.

Hence design of new structures and the corresponding sizing of elements would be based on a 5 percent exclusion value with a 75 percent confidence level.

But notice that the stress/strain response of timber is substantially affected by the rate of loading in the test machine. It can "absorb" relatively large sudden loads without permanent deforma-

Phil Pierce

Hamden Bridge, Delaware County, New York, 128-ft.-span Long truss built over Delaware River in 1859 and propped midspan by pier in 1940s. During rehabilitation in 2000, pier was removed and bottom chords replaced by glulam because of theoretical weakness of chord splices, even if in good condition, in original design.

tion, but it will gradually creep under long-term load. Hence, the FPL decided early on that design values would be adjusted to what is termed *normal* loading—a period of load duration equivalent to 10 years. Adjustments for load duration of other than 10 years would be necessary in a subsequent phase of design.

These testing protocols and results are described and contained in American Society for Testing Materials (ASTM) specifications, notably D2555 *Standard Practice for Establishing Clear Wood Strength Values* and D245 *Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber*. More advanced work includes a commonly-cited extensive series of "in-grade tests": D2915 S*tandard Practice for Sampling and Data-Analysis for Structural Wood and Wood-Based Products* and D1990 *Standard Practice for Establishing Allowable Properties for Visually Graded Dimension Lumber from In-Grade Tests of Full-Size Specimens*.

The most widely adopted and cited tabulations of reference design values are provided in the *National Design Specification for Wood Construction* (*NDS*), promulgated and published by the American Wood Council, most recently in the 2012 edition. The 1991 edition of the *NDS* provides a meaty explanation of these matters, which I have simplified while retaining the important steps.

Design methodology Like metal and concrete specifications, early timber design specifications were based on an allowable-stress methodology derived from a stress-strain curve and statistical analysis, with appropriate reduction by a factor of safety to produce an acceptable or "allowable" stress for comparison to predicted (calculated) actual stresses.

These published allowable stresses, already reduced by the factor of safety, are commonly referred to as *reference design values*. As an example, the value for allowable tension can be thought of as equivalent to 55 percent of yield stress of a steel element—a value readily recognized by bridge engineers more familiar with steel than timber.

When working with steel, we commonly think of a single value as "the" factor of safety for a given stress—the 55 percent of yield in tension equates to a value of 1.82 as the factor of safety. But it is important to note that in timber the factor of safety as developed via statistical process is nothing near consistent.

In general, the average factor of safety in timber is on the order of 2.5. But because of the variability of wood, the factor may be larger or smaller for a given element. The procedures for establishing reference values per ASTM D245 and D2555, cited earlier, indicate that for 99 out of 100 pieces, the factor will be greater than 1.25, and for 1 out of 100, the factor will exceed 5. Such indeterminacy is very different from design in steel.

(The safety factor in wood is no simple matter. For a thorough analytic explanation of its development, see Lyman W. Wood, "Factor of Safety in Design of Timber Structures," *Transactions of the American Society of Civil Engineers*, Vol. 125, No. 1, pp. 1033–45. For a practical understanding of safety factors in wood, see fpl.fs.fed.us/documnts/fpltn/fpltn-222.pdf for the Forest Products Laboratory's *Technical Note 222*.)

What about "Load and Resistance Factor Design" methodology, now in vogue? Should we not be talking about its format conversion factors and resistance factors to enable sizing of elements? Perhaps so, but LRFD methodology still relies on strength values from small, clear specimen tests, with the 5 percent exclusion value and 75 percent confidence level built in. (And some timber engineers believe that there are kinks to be worked out in calibrating values between the two methodologies.)

Predicted stresses While focused on discussing development of allowable stresses, we became separated from the other part of the work, the prediction of actual stresses in service. Now we have to determine forces and corresponding stresses for the various types of loading that can be applied to the structure, and in which combinations, with their corresponding probability of occurrence.

The guidelines for which loads, and in which combinations they are to be applied to bridge structures in the United States, follow those published by the American Association of State Highway and Transportation Officials (AASHTO). In general, AASHTO has adopted the reference design stresses and adjustment factors of *NDS* with some minor tweaking.

Another diversion is necessary as part of predicting stresses. Recall that timber tends to accept short-term loads without damage but will creep with long-term loads. This phenomenon is accounted for in timber specifications via the load-duration factor, incorporated in stress evaluation according to the duration of the specific group of loads being considered. The factor for the individual group is associated with the shortest duration of load because if it were associated with the longest duration (dead load), the factor would always be the same. (Yes, this is weird and confusing—there is nothing really like it in steel or concrete analysis.) Load-duration groups could include, for example, the following:

Dead load only. AASHTO assumes dead load to be permanent throughout the life of the structure and assigns a load-duration factor, C_D, of 0.9 (this reduces the allowable to account for long-term creep).

Dead load + vehicular live load. AASHTO assigns a value equivalent to a total of 10 years of accumulated design loading over the life of the structure for a C_D of 1.00 (since the reference values are already given for an assumed load duration of 10 years).

Dead load + wind load. AASHTO assigns a value equivalent to a total of 10 minutes of full design wind force over the life of the structure with a corresponding C_D of 1.6—greater than one, recognizing the ability of wood to absorb relatively short bursts of loading.

Proceeding through the various combinations of loads with application of corresponding C_D, one arrives at the highest predicted stress to compare against the allowable selected from the tables with appropriate adjustments. (As an interesting aside, we might note that timber's ability to absorb large, quick loading eliminates the need for the "impact provision" multiplier of vehicular live loading required in steel or concrete bridge design.)

Member sizing Now that we have briefly reviewed the basis of wood design values as the statistically adjusted performance of small clear specimens of a given species, and explained some considerations in predicting stresses, we proceed to sizing of elements.

As we have seen, timber elements contain a variety of so-called defects—variations from clear straight grain—that reduce the capacity of the element from that implied by allowable stresses derived from small clear specimens. For example, a knot represents a major interruption to the flow of stress/strain along the path of an element. Reduction factors account for the effect of such deficiency. The slope of grain of an element is another key defect: if it's out of tolerance, it may warrant a reduction factor. Other defects include shakes and splits (forms of fiber separation), wane and other features of a natural material. More or relatively larger defects require greater reduction of allowable stress.

Reduction in capacity is made evident to designers by timber grading, with each grade—Select Structural, No. 1, No. 2 and the like—associated with a different set of allowable stresses. Identifying the grade via a "strength ratio" (the more the defects, the lower the value) is a multiplier of the results of small clear specimen tests.

So, we go to the *NDS* tables knowing the wood species we intend to use and select a structural grade that we intend to specify in our design. We then obtain the reference design values for bending, compression, shear, et al. Next, to size an element, we account for issues that can reduce the reference design value—e.g., moisture content or load-duration factor. We proceed to size the element accordingly so that the stresses are acceptable. But compared to what?

Restate the problem Comparisons of predicted stresses of extant wooden covered bridges against those of the *NDS*/AASHTO allowables routinely indicate overstress, i.e., lack of sufficient capacity of the bridge. In many cases, structure performance demonstrates more capacity than indicated by the standard allowables during evaluation. This leads to conflict over the need for element replacement or reinforcement.

What's wrong with our evaluation? Let's start with determination of predicted loads/stresses. It's important to recognize that dead load of covered bridges is much higher as a proportion to total load than is typical of modern steel or concrete bridges. The unit weight of timber elements varies per species, moisture content and preservative treatment. AASHTO specifies a density of 50 lbs. per cu. ft. (pcf) for design of new timber bridges, based on timber elements with high moisture content and creosote preservative. In-service unit weight of extant covered bridges is usually much less—often less than 30 pcf. If taken into account, this in-service weight would make a big difference to the calculation of reserve capacity for live loading. (Use of site-specific unit weights for extant covered bridges is accepted by AASHTO.)

Now suppose we consider capacity, the allowable stress side of the comparison. The determination of capacity of an extant wooden bridge begins with timber species. A wood scientist can readily confirm species based on small samples (the size of a pencil). Then, what's the grade of the timbers? This is not so easy a question because not all surfaces of all timbers can be seen, but with limitations understood it can be tackled by a certified lumber grader to identify size and distribution of knots, slope of grain, etc. When examining elements in an extant structure, the best that can be done is to identify the highest grade that can be assigned to the element, based on what is visible (the unseen material could be better or worse).

Another limitation is that each element has its own defects and therefore possibly its own grade, but it would be impractical to assign different grades to each element. Finally, a structure with relatively more hidden surface is more difficult—a Town lattice truss with its high proportion of mating surfaces, for example, would be more difficult to evaluate than any other truss configuration. (Removing the bridge siding for this exercise is probably not going to be performed for practical or economic reasons.) It is then up to the engineer to choose an appropriate grade—a daunting decision that depends on confidence and circumstance.

Given species and grade for the given element, we now go to the *NDS* tabulation and find the reference design values. We identify all appropriate adjustment factors via *NDS* with AASHTO overrides as appropriate and come up with the allowable stress to compare to the predicted actual stress for the various group-loading combinations. Does the extant bridge have the live-load capacity we were looking for or expecting? Probably not. What we have done so far is the easy part.

Now what? Are there other factors related to loads or stresses that can be tweaked for covered bridges with hope of gaining the capacity that seems hidden? Well, recall that AASHTO specifies use of a load-duration factor C_D = 1.0 based on an assumed 10-year total duration of design vehicular loading. That seems suspect. A total of 10 years of vehicle load on a single element? Recall that this spec is to represent the accumulated total time. The passage of a vehicle over the bridge probably takes seconds. How might that accumulate to 10 years (315,360,000 seconds)? And the value is to reflect the accumulation of passages of the "design" vehicle, the heaviest plausible, not the accumulation of all vehicle passages: passages of other weights do not count. Something is odd here.

For extant covered bridges, it would seem reasonable to calculate a revised value of this load amplification factor based on actual, estimated or hypothetical traffic information, or at least more rational values than the arbitrary value of 10 years used by AASHTO.

Forest Products Laboratory Research Paper RP-487, "Statistical Considerations in Duration of Load Research," uses a certain equation to develop duration-of-load factors for various types of loads —e.g., two months for snow load C_D = 1.15; one day for wind load C_D = 1.33. (See fpl.fs.fed.us/documnts/fplrp/fplrp487.pdf.)

The formula for the duration of load factor is

$$108.4 \div (60X)^{0.04635} + 18.3$$

where X is the total number of minutes for which the given load has been applied over the life of the structure, and the numerical values are based on best-fit from research.

There are 5,256,000 minutes in 10 years. For the 10-year C_D values in the tables, the formula then yields

$$108.4 \div (60 \times 5{,}256{,}000)^{0.04635} + 18.3 = 62.1$$

(Those who may use this equation to verify the C_D for loads and combinations cited earlier will find the formula for "permanent" loads—that is, dead load—does not lead to 0.9. Apparently the value of 0.9 was selected around the time of World War II as modern timber specifications were being developed.)

Suppose our extant bridge was built in 1880 and has been crossed by heavy loads equivalent to our intended design vehicle on average 10 times per day since it was first built, with an average duration to the loaded element of one second during the passage of the vehicle. That yields a total loading duration of

$$133 \times 365 \times 10 \text{ passages} \times 1 \text{ second} \div 60 = 8091 \text{ minutes} = 5.62 \text{ days}$$

Less than six *days*, not 10 years as AASHTO would suggest. Using the 8091 minutes instead of 10 years, the formula yields a value of

$$108.4 \div (60 \times 8091 \text{ minutes})^{0.04635} + 18.3 = 77.4$$

compared to the normal loading value of 62.1, indicating a load-duration factor of $77.4 \div 62.1 = 1.24$, or 24 percent less live load than when using AASHTO's generic value of 1.00. Maybe we are unwilling to go this far and we instead assume twice as many occurrences or 20 per day. That leads to a value of 75.5 for the equation—or a C_D value of 1.21—still 21 percent less than the generic value.

While this result may not represent a lot of savings, it's fair to explore the concept in a real-life evaluation. A younger covered bridge would probably have a larger C_D due to many fewer passages of vehicles, which indicates more capacity. This exploration assumes the same live-load force during each of the passages throughout the life of the bridge, whereas the 1800s did not necessarily have today's design loads. Certainly the weight of individual vehicles crossing the bridge varies with the vehicle.

This is a sticky issue. We are evaluating the effect of a specific weight of vehicle over a specific period of time and attempting to identify the total number of minutes of those passages. We could also be evaluating the results of a vehicle weighing less, but with more passages per day for comparison. There is no easy way to consolidate this topic into something truly black and white and widely accepted.

To complicate matters even more, since covered bridges can have snow on top of the roof while vehicles pass through the bridge, we have to consider a group-load combination of dead-plus-live-plus-snow at its own load-duration factor as well as its own probability-of-occurrence group-loading factor. Snow loads are not contained in AASHTO for modern design since we now use snow plows to push snow off uncovered bridges.

Are we lost yet? Hope not. Let's assume that our "refined" predicted actual stresses still don't properly cover real conditions.

How else to account for that extra strength? What about the extreme variability of that factor of safety noted above? Should we consider something else in our evaluation of a historic covered bridge? Is it the old-growth timber that we hear so much about, which must be stronger than modern timber? There is no doubt that old-growth timber was much more dense but, while density is an important aspect of the strength of timber, the reference design values provided in *NDS* have allowances for density built into specific grades based on empirical information. There are no readily available means of adjusting values of extant material to account for specific density.

Also, it's true that old-growth timber had many fewer knots (long story), but grading in the field is the limit of our means to evaluate a belief that old material is somehow stronger. We are trying to find extra strength that we can document in accordance with the standard of care of our times. We are not advocating that timber engineers go back to making our own specifications as did the 19th-century builders.

What about load testing? Strain gauges are commonly used to measure deflection or other movement of metal elements and sometimes concrete. Can we use strain gauges on timber? Hidden defects of larger bridge elements probably obviate strain measurements as indicative of actual stress. How do I know that I am measuring a legitimate "average" stress in an element, or even a realistic maximum stress? And what about the connections?

If we measure actual strains in an element and predict a capacity, can we say with any certainty that the joints have a similar or higher factor of safety? I think not. What we *can* do with strain measurements is to compare relative load sharing. For instance, the distribution of forces around a termination of a chord element of a Town lattice truss can be evaluated by strains with some degree of confidence.

What about deflection measurements? Flexural elements can be tested practically and with some confidence based on deflection, but not trusses, invariably the structural heart of a historic covered bridge. Deflections of timber trusses are extremely small, and the required accuracy of measurement makes reliance on the method suspect as well. For example, I have used various means to measure the deflection of a few historic bridges and found midspan deflections under a 15-ton vehicle load to be less than one-half inch. Such values are hard to replicate, and associating such small deflections with predictions of the capacity of the bridge is difficult.

What about our force analysis? I have not addressed the means and methods used to determine forces for this evaluation. Regardless of whether we use a simplified hand-analysis based on pinned-joint representation of truss behavior, or a computer program based on frame behavior (which should be very thoughtfully prepared), or some more advanced finite element representation, it's traditional that the analysis of the trusses be performed on a two-dimensional basis representing a single truss (without consideration of the deformation of the structure as a consequence of loading). Does truss analysis adequately account for the behavior of the structure as a whole?

Should we expand the analysis into a full three-dimensional representation of the structure, complete with floor system, bottom lateral bracing system (if one exists), overhead bracing system, maybe even the rafters, roofing and siding? But positing some sort of box to account for observed supplemental bracing and strengthening is not a reliable structural representation for support of vehicular live loads, because clearly these structures move and shift forces among the various available load paths, especially at joints, in ways we cannot model, probably not even fathom.

Is it possible that the timber deck may represent a potential benefit as additional "bottom chord" material? This theory obviously demands confidence in the deck acting compositely with the truss, in which case a physical attachment between truss and floor is required to account for horizontal shear load sharing. This could be more readily evident in a Town lattice truss with closely spaced floor beams than in a queenpost truss with widely spaced floor beams.

Last chance—there must be something! Let's look yet again at that 5 percent exclusion value. It was selected for the purpose of sizing structures or elements, and it has proven to yield structures that stand up to loads quite well. But is it too conservative for evaluation of extant covered bridges?

If an existing element was one of those with a low initial capacity because of some defect or poor overall quality, there is a good chance that it has already failed and been replaced with one of much higher capacity than indicated by our 5 percent exclusion value. On the other hand, if we believe that the element is one with higher capacity, then how do we justify raising the bar (numerically increasing the exclusion value)?

One can develop a tabulation to compare the increased basic reference stress from an increase in exclusion. If we consider just this effect, and limit ourselves to Coastal Douglas fir as a species, we find that for a 20 percent exclusion (or at 86 percent of the mean), we gain about 23 percent in strength. At a 30 percent exclusion (or 91 percent of the mean), we gain about 32 percent. Similar findings can be shown for any other species, based on their test data.

So what are appropriate reasons to modify the exclusion value to account for within-species variability?

Should the value be at all a function of age?

Should it be a function of element location within the bridge? An element with a lower design stress level may have been subjected to many fewer instances of high overstress, and hence may be worthy of less caution, perhaps allowing a higher exclusion value. If the element is one with a higher design stress, it probably has had many more instances of even higher overstress, thereby dipping into that reserve capacity more frequently (and thereby being more prone to failure sooner than later), so we should be more careful in that situation, and a numerically *lower* exclusion limit probably would be appropriate.

Should the value be a function of bridge location? A bridge that has survived on a more heavily traveled road might have more inherent capacity than one on a lightly traveled road, thereby potentially justifying a higher exclusion rate, or it may be on the verge of its capacity, while a bridge on a lightly traveled road may not have seen many heavy loads and could have ample reserve (or little reserve).

Is there finally a reason to consider modifying the exclusion value to account for what's not included in the myriad of other modification factors? I am not advocating for a specific value but for thoughtful consideration of this factor when faced with the implied need to replace elements of historic covered bridges. It may be that accepting a higher exclusion rule would support retention of elements that appear to be serving well, regardless of statistical implications of weakness.

Where does this leave us? Well, not with an answer, but with food for thought and perhaps a hunger to continue this exercise. I remain convinced that the 5 percent exclusion value in the setting of allowable stresses is the most suspect element in our evaluation of a historic covered bridge's capacity.

Anything else? It remains vital that we always strive for sensible weight limitations on extant historic covered bridges—lower than eight tons whenever possible. Three tons is a common limitation when alternate routes are readily available. Covered bridges were not built for large modern vehicles and should not be expected to support them. Allowing heavier vehicles to use these precious structures only hastens their demise. We should not be looking for hidden capacity to support unnecessarily heavy loads. —Phillip Pierce

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