TIMBER FRAMING

JOURNAL OF THE TIMBER FRAMERS GUILD

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On the front cover, detail of cherry and stained tulip-poplar roof frame over 150-sq.-ft. pavilion in northeast Ohio, built by David Yasenchack of Kingsville. On the back cover, end view with builder and client enjoying a mid-summer's day. The four tapered white oak posts were converted from a single quartered log. Only one design sketch was required, with the owners, nearing 80, requesting the builder simply to enlarge the initial plan "to give us room to dance." Photos by Dee Riley.

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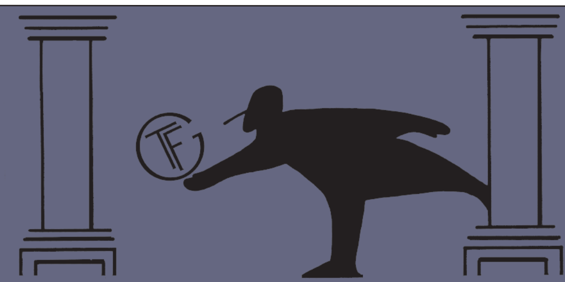
1985



GUILD NOTES & COMMENT
Minneapolis

BEFORE the Guild settled firmly in the mid-1990s into its pattern of annual eastern and western conferences, usually fairly near or at the coasts, board discussions often proposed regional meetings as a means to get widely separated members together more frequently with less effort and travel expense. In 1988, midwesterners gathered twice, in Ohio in February and in Wisconsin in October; our northern neighbors the Canadians got together twice in Ontario, in 1990 and 1991; and southerners *intended* to get together in 1992.

For almost 20 years, the idea of regional meetings lay dormant, until 2009, when Whit and Gabel Holder put together a southeastern meeting in Georgia. Catching fire, last year the idea brought a second southeastern meeting, in North Carolina, and a northwestern meeting in Oregon. This year will see the third southeastern, the second northwestern and the first northeastern meetings.

They will have something to live up to, to equal the pleasures of the first North Central meeting, which together with the Guild board's annual face-to-face meeting, brought me to Minnesota at the end of January. Clark Bremer, newly elected to the board, turned over his capacious Northern Lights Timber Framing shop in Minneapolis's Warehouse District to a hundred of us for a weekend of demos, illustrated presentations, beam-busting, a lively members' meeting and good food brought right into the shop. Imitating the pattern if not the size of the national conferences, the gathering offered a rich slide show (see three pages overleaf) and a refreshing sort of trade show, almost exclusively old hand tools. A contingent of Minneapolis author and architect Dale Mulfinger's architecture students visited us in connection with his lecture on

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LETTERS

To the Editor:

I read with interest the scarf-busting article in TF 98, and of course to see which of the joints performed better, but afterward I thought, "Who would ever subject a scarf joint to such a bending load?" I have used scarfs on sills with foundation support and on top plates either over a brace or at quarter-span where tension and compression forces are neutral. I would have thought the main forces were outward thrust or rolling action, such as a post in a hammer-beam truss might apply to a sill or a rafter to a top plate. A scarf repair to a post would be subject to compression and some bending, a scarf in a bridge timber perhaps to tension. So, the article, while it described what I'm sure was a lot of fun for all involved, kind of missed the boat for me.

Higgs Murphy
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Lisa Sasser

Clark Bremer as MC, and joiners deep into the mortising contest at the North Central regional Guild meeting, Minneapolis, January 29–30.

Minnesota's Edwin Lundie (who designed timber frames right through the 20th century), bringing along their stick models of Lundie's frame designs for our inspection and comment.

To remind us of our origins, Northern Lights roughed out 2x8 mortises in pine beams for 32 contestants to trim full and square in a timed three minutes. When the chips ceased to fly, the judge performed two simple tests. Would the 2-in. blade of a framing square push easily into the mortise crosswise to full depth at any point, and was there any daylight showing at the top when a tri-square set to tenon depth was registered against each edge of the mortise? "I have never met a mortise that was too *big,*" remarked the judge in praising the two joiners whose crisp mortises passed the tests.

Members' meetings at national conferences often disappoint with small participation, even if held with no competition from other events. Not so in Minneapolis, with people leaping into speech all over the room, delighted to be heard. Mostly what they said amounted to "We're here! We make and fix timber frames! Come see us more often!" In conference dynamics, there's evidently great virtue to a group's staying together for all events in the same accommodating room.

IF the first North Central regional Guild meeting was a joy, the 24th annual board of directors' face-to-face meeting, which unfolded over the next two days in a rented house 12 miles away in Minnetonka, could not be so light-hearted.

A kind of retreat, the annual F2F (as it's called) substantially augments the board's monthly teleconferences and encourages self-examination and long-term thinking. At this meeting, given a year's results after a transition from a dual executive directorate with a historically successful division of responsibilities, to a single executive director promising to do everything, there was plenty of short-term thinking as well. Two offices, one devoted to Guild, conference and publication administration, the other to project development and relations with the public, became one—four people became two—and the practical consequences are still being worked out a year later.

Guild membership is a worry. After 20 years of growth to surpass 1900 members in mid-2006, it declined over four years to about 1400 in 2010. The Guild finished 2010 with its operating budget of $648,000 some 5 percent in the red, and the year-end balance sheet shows net equity (the bottom line) reduced by a third, if remaining well in the black. Conference attendance in 2010 and consequent revenues were down substantially, outweighing greater project revenue than expected.

No doubt these downward trends were much assisted by the Great Recession that began in 2007, grew acute in 2008 and 2009, and hardly eased in 2010. But the question whether the Guild serves its members well inevitably arises when contemplating such trends. "Regional conferences and apprenticeship are the only growth areas for the Guild these days. All else is shrinking," said one director. "Is the Guild's educational work—conferences, publications, projects—relevant to many timber framers? No!" said another.

Should the Guild then make a conscious effort to grow its membership, and, in a somewhat separate question, to "sell" projects and conferences using modern marketing methods? Is the question "How do we keep ourselves in existence?"—which one director posed in so many words—even appropriate for an educational service organization? Perhaps the organization, which certainly grew out of fraternal impulses, should simply reflect the waxing and waning preferences of those who discover and join it. Perhaps 1400 is the *right* number.

But supposing the Guild does take steps to preserve or enlarge itself, whom should it aim to serve? Under the bylaws, Guild membership is open to all. Whatever the membership level, historically we have had about half professional members (timber framers or workers in allied trades) and half associate members (all the rest). We have evidence that there are many more practicing timber framers in North America than Guild members, and beyond them a good many do-it-yourselfers interested in the craft who might join or rejoin.

A position developed in the meeting that if the Guild refocused its programs more sharply on the professionals, everyone else interested in timber framing would just naturally come along. In a simultaneous insight, several directors proposed that the apprenticeship training curriculum designed by Will Beemer and others, now well launched (and downloadable from the Guild homepage), would provide a convenient template for this idea. Under this theory, each Guild event would serve some part of the curriculum. Conference presentations and associated workshops would be designed first and foremost to serve the curriculum, which would actually make programming easier, at least for a while. Of course many curriculum subjects are not reducible to 90-minute (or even 8-hour) stretches, but those that are will provide a ready menu for conference programmers. Projects and rendezvous would seem to be easier venues for many curriculum subjects, and there the beauty of the plan is that it supplies an organizing principle to each event. As one director remarked, "Better to have a definite list than everything in the world." —Ken Rower

Photos Deane Hillbrand

Minneapolis Meeting Slide Show Selections

At right, white-pine faux trusses for 21-ft.-square room in 5000-sq.-ft. house in Finland, Minnesota. Trusses, built by Mark Sherman of Woodland Builders in Duluth, were originally designed to be fully joined and craned in. Sudden shrinkage of construction schedule required they be framed in place instead. All joinery housed 1 in., and hidden rods compress the joints. Paired arched members are cut from single timbers.

Facing page at left, lakeside retreat for jazz musician's family in Clam Lake, Wisconsin, designed and built by Deane Hillbrand of Sturgeon Lake, Minnesota, and framed of reclaimed Douglas fir timbers and black ash logs. Challenge was to work with natural and curved shapes "in a straight construction world." About 2000 sq. ft. on three levels.

Joe Taatjes

Facing page at left, barn frame near Black Duck, Minnesota, north of Bemidji. Architectural design (not shown) by Katherine Hillbrand of Sala Architects in Minneapolis, framing by Deane Hillbrand. (They are husband and wife). Plan measures 37x72 ft.; main frame members are ash logs while rafters and joists are red pine. One bay of finished barn will be devoted to domestic purposes, with small kitchen and bath.

At right, pergola for outdoor seating at a café in Luck, Wisconsin, built by Brook Waalen (also of Luck) with white oak posts and white pine tie beams, rafters and purlins. Roof covering is fabric, in place only for the hottest months. Since structure is entirely open to weather half the year, roof structure is bolted together to keep water-catching mortises to a minimum. Post-tops are tarred and flashed and every joint, mortised or not, is caulked.

Stephanie Lundeen

Jeffrey Plakke

Log and timber addition to log house near Eagle River, in Michigan's Upper Peninsula. Design by Mark Johnson and Associates in Pleasant Ridge, General Contractor James Martin of Eagle River, log work by Mark Salo of Dodgeville and Kenneth Svenson of Calumet. Each segment of arc in the smaller photo is 8 degrees on a 50-ft. radius, with joints calling for template work; northern white cedar log on outside of room (larger photo) is naturally curved. Windows give on Lake Superior. Room enclosed serves as formal dining room and connection to large addition, measures about 13 ft. wide at widest point and 21 ft. long. Remaining logs are Eastern white pine, trusses, trim and windows Douglas fir. Photographed in 2009.

Dale Kittleson

Above, completed barn 40x56 ft. in Decorah, Iowa. At left, detail of house frame nearby. Barn design by client and Wild Rose Timberworks in Decorah. Frame and siding Eastern white pine from Wisconsin. Client raises cattle and loft area is for hay storage. Main floor is used for equipment storage and workshop but has also hosted large formal dinner parties. Wild Rose's Dale Kittleson reports: "We lose barns every year because farming has shifted away from the barns' original purposes (loose hay above to feed animals below). The client wanted a traditional monitor style barn, built on a slab, framed and finished in a style respecting the local historic vernacular while still meeting his specific modern needs."

Basic Beam Sizing

MUCH has changed in the engineering world since the original of this piece, "Sizing Roughsawn Joists and Beams," was written in 1982, in the relatively early days of the timber frame revival (Levin 1982). Pencil and paper calculations have been largely supplanted, first by calculator and then by computer. Today there are many for-the-purpose software choices running on a variety of devices, from the PC down to the smartphone. (Yes, Virginia, there are iPhone apps for beam sizing.)

It's easy to get distracted in the maze of technology and lose sight of the fundamental point of grasping first principles to provide a sound and broad foundation for the practice of any craft. Modern sailors are still sent out to train in wooden sailing ships and take sun shots with sextants before moving on to GPS, steel hulls and steam turbines. Likewise apprentice timber framers first wield handsaw, mallet and chisel before picking up Skilsaw, router and chain mortiser. So too, in design as well as execution, we look to come to grips with basic structural laws and their application before picking up the power tools.

To first questions, then. *What is a beam?* Structurally speaking, it is an elongated structural element spanning between supports and loaded transversely to its axis. By contrast, a post or column would be an element loaded along its axis. Some members act in hybrid fashion, being loaded both transversely and axially (i.e., across and along the grain). These, unsurprisingly, are termed *beam-columns* in the structural lexicon. Sloped roof rafters are a prime example. A loaded beam acts in bending, a post in compression (and sometimes in tension). A beam-column does both.

Why focus on beams? First of all, most of the sticks we deal with as timber framers—60 percent or better on average—are beams of one sort or another, loaded entirely or primarily in bending. Second, the action of beams in bending is easily understood both intuitively and mathematically, the behavior of columns less so.

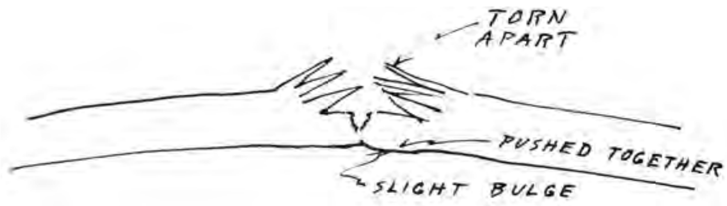
Timber framers typically get a bye in column sizing. To be big enough to receive incoming girts and tie beams, most timber posts are large enough in section to carry axial loads well in excess of what we ask of them over ordinary residential floor-to-floor vertical distances. (This is an observation, not a license to forgo structural review.)

To keep matters simple, we further restrict our scope here to full-section rectangular members, for the most part leaving aside the complexities of connections, fasteners and reduced timber sections, not to mention whole frame analysis.

As always, an intuitive grasp of the subject is a useful (and perhaps necessary) precursor to rigorous mathematical analysis. Recognizing this principle, at least one author and publisher have collaborated to produce sister texts covering the same subject matter with parallel chapter structure, one volume taking an intuitive, nonmathematical approach, the other presenting a full formulaic treatment (Salvadori and Heller 1975, Salvadori and Levy 1981). Following this lead, we'll try to systematize what most of us learned back when we first crossed a stream on a fallen log or bent a branch in our hands. (And there's a glossary on page 11.)

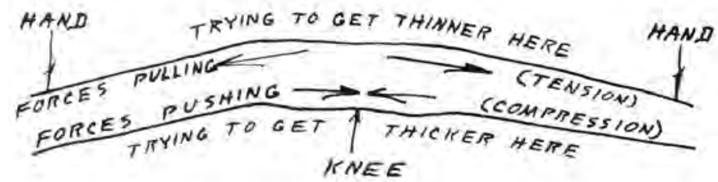
ONE of the best intuitive introductions to the subject comes from engineer Rex Roberts in his book *Your Engineered House* (1964), a classic from *Whole Earth Catalog* days:

> Start to break a stick across your knee. Stop just after it pops and before you have pulled it apart. The broken ends will now look something like this:



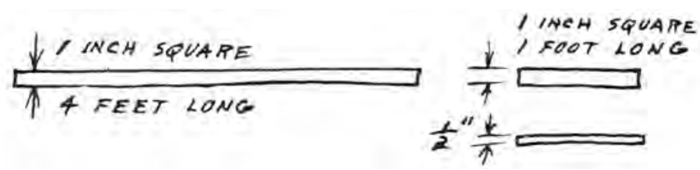
> Back up and do it again, this time looking at the bent stick just before it breaks.

Drawings Rex Roberts



> The forces pulling apart are called tension. The forces pushing together are called compression. The stick that you broke is a beam. It represents, among other things, the floor of a house. Turning the sketches upside down, the side on which the grand piano [would sit] is called the compression side, the other side the tension side.

The author comments, "You are now one-quarter of the way toward being a structural engineer" (perhaps a slight exaggeration), and then compares three square sticks cut from the same plank to 1x1x48, 1x1x12 and ½x½x12.



Breaking the short ½-in.-square stick over your knee is easy, likewise the long 1-in.-square stick. Busting the short 1-in.-square stick is next to impossible. Resistance to failure in bending is what engineers call strength. If you can measure the force applied to the sticks, you'll find that the short ½-in. stick and the long 1-in. stick break under the same load—they are equally strong. But it takes four times as much force to break the 1x1x12.

Experiment with sticks of varying lengths and thickness and you will learn that strength increases in direct proportion to the width and the square of the depth of the bending member—also that strength falls off in direct proportion to the length of the stick.

Taking another step along this path, let's see if we can quantify bending in simple beams. We'll look at two of the most common loading situations: a load uniformly distributed over the length of the beam (as in a joist or purlin), and a single point load at midspan (a girt carrying a summer beam or prick post).

For our uniform load case, we have floor joists 36 in. on center spanning 12 ft. 6 in. or (to keep length units uniform) 150 in., and carrying a 40 lbs. per sq. ft. (psf) live load. In this first runthrough, we'll neglect the dead-load contributions of the flooring and the as-yet-unknown weight of the joist itself. The live line load per joist in pounds per inch (lb/in) works out to

$$w = \frac{40\,psf \times 36in}{144in^2 / ft^2} = 10.0lb / in$$

So, for the span of L = 150 in., the total load is 1500 lbs.

$W = -10.0lb/in \times 150in = -1500lb$

By convention, we assign a minus sign to the load since the force is acting in the downward or negative Y direction.

Given the uniform symmetric loading, the upward reactions at either end of the joist are

$$R_a = \frac{1500lb}{2} = 750lb = R_b$$

Knowing the load and corresponding reactions, we can compute the live load bending moment in the beam by using a drawing called a free body diagram. In our static, two-dimensional system, for a member to be in equilibrium (at rest), all the forces and moments acting on it must sum to zero.

Recall here that a moment is a force acting over a distance and that it imparts torque to the body it acts upon. Consider a 200-lb. timber framer changing a flat tire on his pickup. If he stands on the end of a 1-ft.-long lug wrench, he applies a moment of 200 ft.-lb. (foot-pounds) or 2400 in.-lb. to the lug nut.

So in our system, a force on an object causes it to translate, i.e., to move in the *XY* plane, and a moment causes the object of its affections to rotate in the *XY* plane. Restating the conditions for static equilibrium, for said object to remain at rest, all forces and all moments acting on it must cancel out, satisfying the following three equations:

1. $\sum F_x = 0$
2. $\sum F_y = 0$
3. $\sum M_z = 0$

There is no horizontal component of force acting on our floor joist, so the condition of Equation 1 is met. Likewise Equation 2, since the downward force of the floor load

$W = -1500lb$

is exactly canceled out by the upward reactions

$R_a + R_b = 750lb + 750lb = 1500lb$

There being no external moment imposed on the joist, we have to get a bit tricky in our assessment of Equation 3. The rules of static equilibrium decree that for an object to be at rest, any part of it is also at rest and must satisfy the three equations. So let's isolate the right half of our joist and consider the sum of the moments acting on it at midspan. Note that by convention we take moments acting in the clockwise direction as positive, those acting counter-clockwise as negative.

First, there is the righthand support reaction

$R_b = 750lb$

acting over a distance of 75 in., yielding a moment

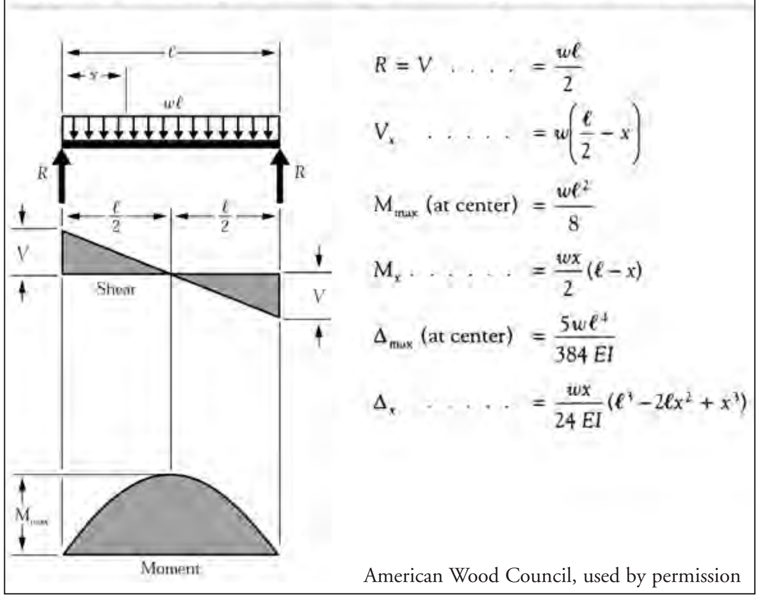
$$M_b = \frac{wL}{2} \times \frac{L}{2} = \frac{wL^2}{4} = 750lb \times 75in = -56{,}250in \cdot lb$$

Second, the righthand half of the distributed floor load

$F_{bc} = 10lb/in \times 75in = 750lb$

acts as if it were a 750-lb. point load placed at its center of force, 37.5 in. from midspan, so

Figure 1 Simple Beam – Uniformly Distributed Load



American Wood Council, used by permission

$$M_{bc} = \frac{wL}{2} \times \frac{L}{4} = \frac{wL^2}{8} = 750lb \times 37.5in = 28{,}125in \cdot lb$$

We know from Rex Roberts's demonstration that the joist feels a bending moment at midspan, and we plug it into Equation 3 as an unknown, M_c :

$\sum M_b + M_{bc} + M_c = 0$ and

$M_c = -M_{bc} - M_b$

$M_c = -28{,}125in \cdot lb - (-56{,}250in \cdot lb) = 28{,}125in \cdot lb$

or, stated algebraically,

$$M_c = -\frac{wL^2}{8} + \frac{wL^2}{4} = \frac{wL^2}{8}$$

In the process of working out Equation 3, we have not only computed the maximum bending moment in the joist at midspan, we have also derived the mathematical expression for maximum bending moment in uniformly loaded beams

$$M = \frac{wL^2}{8}$$

As it happens, many common beam loadings are documented in a standard format giving load, shear and moment diagrams plus matching formulas for shear, moment and deflection along the beam and at local maximums. One easily available collection, *Beam Design Formulas with Shear and Moment Diagrams*, is published by the American Wood Council and available online as Design Aid No. 6 (www.awc.org/pdf/DA6-BeamFormulas.pdf). Reviewing Figure 1 of this document, "Simple Beam–Uniformly Distributed Load" (shown above), we see that

$R = V = \frac{wL}{2}$ and $M_{max} = \frac{wL^2}{8}$

and that the formula for maximum deflection at midspan is

$$\Delta_{max} = \frac{5wL^4}{384EI}$$

where *E* is the *Modulus of Elasticity*, that is, the quantifier of the stiffness of the material, measured in psi, and *I* is the *Moment of Inertia* of the beam section, the quantifier of the stiffness of the beam section, in units of in^4. For rectangular timbers,

$$I = \frac{bd^3}{12}$$

so the Moment of Inertia of a 5x7 beam would be

$$I = \frac{5 \times 7^3}{12} = 142.9 in^4$$

Turning to our example in hand, to choose a joist size we must assess its adequacy in shear, bending and deflection. This is done by computing the individual stresses in the timber resulting from the design loads and comparing them to allowable values for the species and grade of timber under consideration. Design values are found in the *Supplement* to the *National Design Specification for Wood Construction* (*NDS*), also published by the American Wood Council.

Harking back to the introduction to this piece, the calculations that follow fall precisely in the the area where automation has largely displaced hand work. Still, grounding and experience in the fundamentals of beam engineering seem appropriate and wise even if in daily practice you employ a spreadsheet or custom beam-sizing software. Let's run the exercise using a 5x7-in. No. 2 & Better Eastern white pine for our joist.

Bending We know from prior calculations that maximum joist bending moment under live load of 40 psf is 28,125 in.-lb. To get from the bending moment to extreme fiber stress in the joist, we divide the moment by the beam Section Modulus

$$F_b = \frac{M}{S}$$

The section modulus

$$S = \frac{bd^2}{6} = \frac{5in \times (7in)^2}{6} = 40.83 in^3 \qquad \text{so}$$

$$F_b = \frac{M}{S} = \frac{28{,}125 in \cdot lb}{40.83 in^3} = 688.78 psi$$

Checking the *NDS Supplement* Table 4D, we find that allowable F_b for No. 2 Eastern white pine is only 575 psi. So it looks like our 5x7 is not strong enough. Going with a 5x8 (S = 53.33 psi) decreases F_b safely below the allowable value.

$$F_b = \frac{M}{S} = \frac{28{,}125 in \cdot lb}{53.33 in^3} = 527.34 psi$$

So our 5x8 looks good under live load.

But what about combined live plus dead load? In this case, the dead load includes the weight of the timber itself, plus the weight of the swath of floor which the joist carries.

$$\text{Joist dead load } = \frac{5 \times 8 \times 12}{12^3 in^3 / ft^3} = 0.28 ft^3 / ft$$

And at 30 lbs. per cu. ft., the joist weighs

$$0.28 ft^3 / ft \times 30 lb / ft^3 = 8.33 lb / ft = 0.69 lb / in$$

Taking the weight of the flooring (including subfloor, finish floor, sound or thermal insulation, fasteners, etc.) as 10 lbs. per sq. ft., and given the 36-in.-on-center joist spacing, the flooring contributes

$$10 lb / ft^2 \times 3 ft = 30 lb / linft = 2.5 lb / in$$

So, to the live line load w = –10 lb/in, we add –0.69 lb/in for joist dead load and –2.5 lb/in for floor dead load, for a total of

$$w = \left[-10 lb / in\right] + \left[-0.69 lb / in\right] + \left[-2.5 lb / in\right] = -13.19 lb / in$$

And, plugging this combined line load into the moment formula

$$M = \frac{wL^2}{8} = \frac{13.19 \times 150^2}{8} = 37{,}097 in \cdot lb \qquad \text{and}$$

$$F_b = \frac{M}{S} = \frac{37{,}097 in \cdot lb}{53.33 in^3} = 695.57 psi$$

Looks like the combined load induces bending stress greater than the allowable 575 psi. Remedies including upgrading to No. 1 pine (F_b = 875 psi) or going with a stronger timber. No. 1 white pine timber being difficult or impossible to obtain, we look at the second alternative. Bending in a 6x8 (S = 64 cu. in.)

$$F_b = \frac{M}{S} = \frac{37{,}097 in \cdot lb}{64 in^3} = 579.64 psi$$

is still a tad over the line, while a 5x9 (S = 67.5 cu. in.)

$$F_b = \frac{M}{S} = \frac{37{,}097 in \cdot lb}{67.5 in^3} = 549.58 psi$$

puts us back in safe country.

So, having found a candidate timber sufficient in bending, we can check it for live load deflection using the formula given above.

Deflection Modulus of Elasticity (E) for No. 2 EWP is 900,000 psi and

$$I = \frac{bd^3}{12} = \frac{5in \times (9in)^3}{12} = 303.75 in^4$$

For L = 150 in. and w = –10 lb/in (omitting units for clarity),

$$\Delta = \frac{5wL^4}{384EI} = \frac{5 \times \left[-10\right] \times 150^4}{384 \times 900{,}000 \times 303.75} = -0.24 in$$

Checking this deflection against the acceptable standard of L/360 for live load deflection,

$$\frac{L}{360} = \frac{150 in}{360} = 0.42 in$$

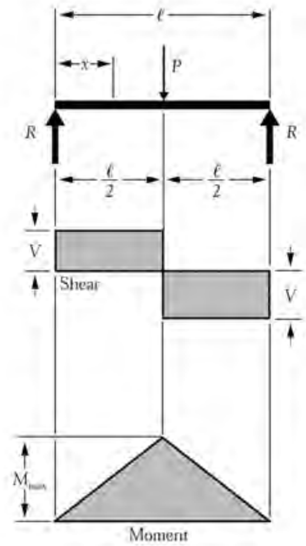
We find predicted deflection being well below L/360, so our 5x9 passes the live load deflection test with flying colors. And, looking at combined load deflection,

$$\Delta = \frac{5 \times \left[-13.19\right] \times 150^4}{384 \times 900{,}000 \times 303.75} = -0.32 in$$

Still under the L/360 threshold and comfortably below the less stringent L/240 standard often used for combined loading:

$$\frac{L}{240} = \frac{150 in}{240} = 0.63 in$$

Figure 7 Simple Beam – Concentrated Load at Center



$R = V \quad \ldots\ldots\ldots = \frac{P}{2}$

M_{max} (at point of load) $= \frac{P\ell}{4}$

$M_x \left(\text{when } x < \frac{\ell}{2}\right) = \frac{Px}{2}$

Δ_{max} (at point of load) $= \frac{P\ell^3}{48EI}$

$\Delta_x \left(\text{when } x < \frac{\ell}{2}\right) = \frac{Px}{48EI}(3\ell^2 - 4x^2)$

American Wood Council, used by permission

Shear We must also check joist shear stress. In a uniformly loaded beam, shear (V) increases from zero at midspan to maximum values at the ends of the beams directly over the supports. As we know, the shear force at the beam ends is equal in magnitude (and opposite in direction) to our reactions

$$V_a = -R_a = -750 = -R_b = V_b$$

In rectangular beams, horizontal shear stress (f_v) is found using the following formula

$$f_v = \frac{3V}{2A}$$

where A is the cross-sectional area of timber. So, for our 5x9,

$$f_v = \frac{3 \times 750lb}{2 \times 5in \times 9in} = 25.0\,psi$$

This is the shear stress value under live load. As before, we also need to look at the combined load shear, by using the combined load value of w to arrive at an augmented shear load:

$$V' = w' \times \frac{L}{2} = -13.19lb/in \times [\frac{150}{2}]in = 989.25lb \qquad \text{and}$$

$$f_v = \frac{3 \times 989.25lb}{2 \times 5in \times 9in} = 33\,psi$$

A look in the *NDS Supplement* at the design values for Eastern white pine shows that allowable shear stress for No. 2 timbers is 125 psi, putting us far below the limit. Shear tends to be an issue only in short, stiff, heavily loaded timbers, especially when they are notched at their ends (a question of joinery design, and so beyond the scope of this article).

Concentrated Load Probably the most common (and among the most stressful) point loadings that timber framers encounter is a single concentrated load at midspan of a beam. For an example, let's stay with our 150-in. span, this time carrying a midspan point load of 750 lbs.

We find from a variety of sources, including *Beam Design Formulas with Shear and Moment Diagrams* Figure 7 (shown above right), that maximum bending moment for a single centered point load is equal to

$$M = \frac{PL}{4}$$

where M is the moment, P is the magnitude of the load (here in pounds) and L is the unsupported span (in inches).

$$M = \frac{PL}{4} = \frac{750lb \times 150in}{4} = 28{,}125in \cdot lb$$

Sticking with a 5x9 EWP beam,

$$S = \frac{bd^2}{6} = 67.5in^3 \qquad \text{and}$$

$$F_b = \frac{M}{S} = \frac{28{,}125in \cdot lb}{67.5in^3} = 416.67\,psi$$

Ample margin here, looks like it's worth checking bending stress for smaller sections. For a 5x8,

$$F_b = \frac{M}{S} = \frac{28{,}125in \cdot lb}{53.33in^3} = 527.34\,psi$$

Still good. And for a 5x7,

$$F_b = \frac{M}{S} = \frac{28{,}125in \cdot lb}{40.83in^3} = 688.78\,psi$$

So, looks like yes to the 5x8, no to the 5x7. But we need to remember to account for dead load. Moments at given points along the beam are algebraically addable, so, looking at the midspan moment under uniform dead load, we recall that

$$w = -0.69lb/in + -2.5lb/in = -3.19lb/in \qquad \text{and}$$

$$M_{DL} = \frac{wL^2}{8} = \frac{3.19 \times 150^2}{8} = 8972in \cdot lb$$

So

$$M = M_{LL} + M_{DL} = 28{,}125 + 8972 = 37{,}097in \cdot lb$$

and, for the 5x8 beam,

$$F_b = \frac{M}{S} = \frac{37{,}097in \cdot lb}{53.33in^3} = 695.57\,psi$$

It's back to the 5x9:

$$F_b = \frac{M}{S} = \frac{37{,}097in \cdot lb}{67.5in^3} = 549.58\,psi$$

Looking at deflection, Figure 7 in *Beam Diagrams & Formulas* (shown above) tells us that, for a single, centered point load

$$\Delta = \frac{PL^3}{48EI}$$

We remember that Modulus of Elasticity (E) for No. 2 EWP is 900,000 psi and that

$$I = \frac{bd^3}{12} = \frac{5in \times (9in)^3}{12} = 303.75in^4$$

Thus

$$\Delta = \frac{PL^3}{48EI} = \frac{750 \times 150^3}{48 \times 900{,}000 \times 303.75} = -0.19in$$

Which amounts to a live load deflection of L /778, a very stiff floor. Accounting for dead load (once again the two are addable at midspan),

$$\Delta_{\Delta L} = \frac{5wL^4}{384EI} = \frac{5 \times -3.19 \times 150^4}{384 \times 900{,}000 \times 303.75} = -0.08in$$

$$\Delta = \Delta_{LL} + \Delta_{DL} == -0.19in + -0.08in = -0.27in$$

Combined live plus dead load deflection computes to L/556, half again as stiff as L/360 standard.

Finally, looking at shear (under combined load),

$$V = \frac{750lb}{2} = 375lb$$

$$A = 5in \times 9in = 45in^2$$

and

$$f_v = \frac{3V}{2A} = \frac{3 \times 375lb}{2 \times 45in^2} = 12.5psi$$

So our 5x9 under concentrated midspan live load plus uniform dead load looks okay in bending, deflection and shear, and we are good to go.

Conclusions Time to take off the green eyeshade, put down the calculator and put up the feet. We've introduced some fundamental principles of beam design and wound our way through a few basic beam sizing calculations. It's valuable to have this exercise under your belt and the experience should prove useful down the road. But, for everyday purposes, there are more efficient and useful methods of getting the answers you need.

One way is to embody the beam formulas in a spreadsheet, where you can enter the essential parameters— beam width, depth and length, timber strength and stiffness, live and dead loads—and let the computer spit out resultant forces, stresses and deflections.

A beam design spreadsheet in use for years in my office reworks the bending and deflection formulas to solve for beam depth for a given beam width, species, grade, length, loading, etc. Since shear stress is almost never the governing factor in beam sizing (and since the shear math is pretty simple), there's no need to rewrite the shear formula.

So, for instance, the calculation for bending stress under uniform load is typically written

$$f_b = \frac{M}{S} = \frac{wL^2}{8} \div \frac{bd^2}{6}$$

Solving this equation for d, you get

$$d = \sqrt{\frac{3wL^2}{4f_b b}}$$

To solve for deflection, it's useful to specify the deflection standard. Thus the uniform load deflection formula

$$\Delta = \frac{5wL^4}{384EI} \quad \text{can be rewritten as} \quad \frac{L}{\delta} = \frac{5wL^4}{384EI}$$

where δ is the denominator of the fraction L/360, L/240, etc. Then, substituting $bd^3 \div 12$ for I and solving for d, you get

$$d = \sqrt[3]{\frac{5w\delta L^3}{32Eb}}$$

Input entries include timber width and length, allowable bending stress, Modulus of Elasticity and line load. As any of the inputs vary (notably b and w), the changes percolate through and d is recalculated. So one can easily ask the question, *To carry this load, how deep would my timber need to be as a 5x (a 6x, an 8x, etc.)?* and then select whichever resultant depth value is greater, that for bending or deflection, depending on which mechanism governs in the particular case. —Ed Levin

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GLOSSARY

Anisotropy The condition of certain materials whose structural properties are not identical in every direction. Wood is anisotropic—stronger and stiffer along than across the grain—unlike steel, concrete, aluminum or plastic, which are isotropic.

Bending Moment A moment is a force acting over a distance. Moments are expressed in units of torque, like foot-pounds (ft-lb). For a simple beam under uniform distributed load, bending moment is greatest at midspan and drops to zero over the supports.

Camber Curvature in a beam. Deflection is inevitable in rafters, joists and beams, but you can take advantage of bowed timbers to introduce upward camber in floor or roof members and perhaps avoid downward curvature under load.

Compression State of stress in which particles of material tend to be pushed together. Compression is the opposite of tension.

Dead Load The weight of the structure itself and all loads permanently on it. For our purposes this generally means the weight of the timber frame (or a given member) plus flooring or roofing and insulation. You need dead load values to design a structure, but you can't determine them until after the structure is designed. So engineers have to start with an educated guess.

Elasticity Attribute of a material that deforms in proportion to applied load, but whose deformation vanishes when the load is removed. Materials that remain deformed after loads are removed are described as plastic. As with any elastic material, there are limits to the loads wood can bear without exhibiting plastic behavior.

Elastic Range and Yield Load Wood's elastic range is the set of stress values that it responds to elastically. The point at which it begins to exhibit plasticity is its yield load. In most cases, persistent plastic behavior immediately precedes the failure of the timber.

Extreme Fiber Stress In bending, maximum compressive and tensile forces occur at the extreme fiber at the upper and lower faces of a loaded timber (see Neutral Axis below). Bending strength is limited by the maximum safe extreme fiber stress. For a given load, bending strength varies directly with the breadth and the square of the depth of the timber, and inversely with the length of the span.

Live Load All loads other than dead loads—usually the weight of people and their furniture as well as wind and snow loads.

Modulus of Elasticity (*E*) The proportion of load to deformation, written as the ratio of stress to strain for a particular species of wood, and the constant used to calculate stiffness in beams of a given species (expressed in units of lbs. per sq. in. or psi).Within its elastic range, wood deformation under load is directly proportional to that load. *E* expresses the linear relation between a given stress (load) and the resulting strain (deformation) in the material.

Moment of Inertia (*I*) In deflection calculations, timber size is usually expressed as Moment of Inertia, in units of in^4. For rectangular beams, $I = bd^3 \div 12$.

Neutral Axis Axis of a beam along which lie fibers neither tensed nor compressed. Bending stresses are greatest at the top and bottom surfaces. They decrease toward the center and at the very middle of the beam theoretically cease (see Stiffness and Deflection below).

Reaction Response of a structural system to an applied force, manifested as a corresponding force equal in magnitude but opposite in direction to the applied force.

Section Modulus Expression of timber size in bending calculations (*S*, in units of in^3). For rectangular beams, $S = bd^2 \div 6$.

Shear, Horizontal and Vertical State of stress in which particles of material tend to slide relative to each other. In a beam, vertical (cross-grain) shear is always accompanied by horizontal (long-grain) shear. Horizontal timbers under load tend to fail in horizontal shear at their supports. To understand this phenomenon, take a half dozen or so pieces of wood about ⅛ in. by 2 in. and at least a foot long and stack them flat. Support the ends and depress the middle. You'll notice that as the center bends downward the individual strips of wood tend to slide along one another. Horizontal shear force operates the same way in a solid timber but adhesion between the wood fibers keeps them from sliding, which induces shear stress in the timber. Shear stress acts to split the timber along the grain, the direction in which wood is weakest.

Stiffness and Deflection Stiffness is a timber's ability to remain rigid in use, as distinct from bending *strength*, its ability to carry a load without breaking. Stiffness is measured by beam deflection, the amount a loaded beam will bend below the horizontal. For a fixed load, stiffness varies directly with the breadth and the cube of the depth of the timber, and inversely with the cube of the length of the span. Roof rafters must be strong enough to take snow loads, and moderate springiness is not a problem unless you plan to finish the underside of the roof. Rafter scantlings thus are often determined by bending strength. But floor joists and the timbers that carry them must be not only strong but stiff as well (if you don't want a springy floor), so joist sizes are limited by deflection.

Strain Lengthwise deformation per unit length of a material under load, expressed in inches of deformation per linear inch of material or in/in.

Stress Force per unit area, typically expressed in psi. A compressive or tensile force acting on an elastic material sets up stress (f). This causes the material to be slightly shortened or stretched.

Tension State of stress in which particles of material tend to be pulled apart. When a simple beam is bent downward under load, its top is in compression and its bottom is in tension. (By contrast, ropes, purely tensile elements, cannot assume compressive stress.) Knots are a significant disadvantage in tension and thus are preferably limited to the top rather than the bottom surface of beams.

Bibliography

Levin, Ed. "Sizing Roughsawn Joists and Beams." *Fine Homebuilding* No. 7, February–March 1982.

Roberts, Rex. *Your Engineered House.* 1964, also published online at www.soilandhealth.org.

Salvadori, Mario and Robert Heller. *Structure in Architecture*. 1975.

Salvadori, Mario and Mathys Levy. *Structural Design in Architecture.* 1981.

Beam Calculator Software Review

TAKING a step further into automation, these days beam-sizing programs run on personal computers and, more recently, on smartphones. To learn which of these packages work well for timber frame design, I canvassed the members of the Timber Frame Engineering Council (TFEC) for their preferences. Brief reviews follow of the favorites that emerged from that survey. I tried out the most popular candidates—some in use for as long as five years—on the load cases described in the accompanying article, with the resulting experience summarized below.

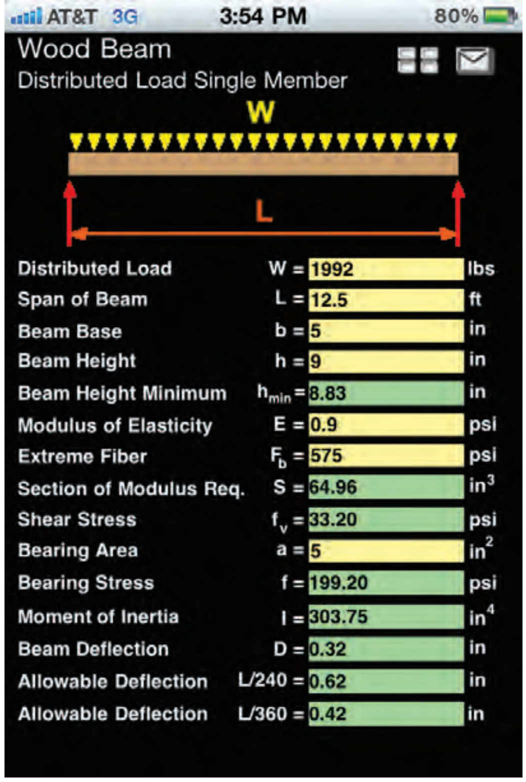
BeamChek (AC Software, Inc., Edmonds, Washington 98020, beamchek.com.; single-user license $149). BeamChek will analyze beams and columns. It accounts for live and dead load and will calculate line loads given sq. ft. unit loads and tributary area width. You can include or neglect the self-weight of the beam, and specify load duration and deflection criteria (and other variables including load adjustments for rafter pitch and repetitive use). In addition to full and partial length uniform loads, you can also apply multiple point loads. Load conditions include single-span, overhang at one end, hips and valleys (with tapered load option) and continuous beams with uniform load over two or three equal spans. BeamChek will also analyze timber and steel columns and calculate shear at end notches.

The data-entry dialog box offers the option of custom wood values, making the program useful to the timber framer who employs a wide range of species and uses nonstandard timber sizes. Once all parameters are entered, you can call up a load diagram to check the data, then hit the Calculate button, which takes you to a second data entry screen to load timber species, grade, design values, breadth and depth. Hit the Test button and you get a report with a table comparing actual and required section moduli, area and deflection under live and total load, so you know how your candidate fared in bending, shear and stiffness. The program does not produce diagrams of resultant moments, stresses and deflection. If any aspect of the test fails, you go back a screen and adjust; otherwise hit the Select Member button for a printout of inputs and results.

At first it was hard to figure out how to get the program to accept oddball timber species and sizes. But once I discovered and selected the custom wood value option in the first dialog box, it was smooth sailing. Take care to check the section readout, however, as you proceed. I found that to test a full-size 5x9, I had to input it as a 5½x9½.

StruCalc (StruCalc, Inc., Corvallis, Oregon 97339, strucalc.com; single-user license $346 full, $134 lite). StruCalc's design modules include footings, beams, collar ties and columns. It will look at floor and roof beams (and combinations) and up to three variable spans with uniform, concentrated and trapezoidal loads separately and in combination.

StruCalc begins with a Design screen to enter data on timber and loads. There is no custom wood values option—the program allows you to use the full range of species and grades and design values out of the *NDS*. Flexible, but no opportunity to define design values for timber not tabulated there. But StruCalc does allow you a free hand in specifying timber sizes; you are not restricted to standard nominal sections, as with BeamChek. The second screen is a Loading Diagram, but where BeamChek does not quantify reactions until after you have run a calculation, StruCalc gives reactions in the loading diagram, broken down into live and dead load components—a handy check on data entry, allowing you to make adjustments before proceeding.



SWDC software running on an iPhone. Screen shows uniform distributed load page with combined load data for joist analyzed earlier.

After checking the loading diagram, it's time to AutoSize. Click on the selected grade, the beam calculator runs and you are offered a number of acceptable timber choices rated by performance. Then it's on to diagrams that map shear, moment and deflection. There is a Stress Values screen to show you the *NDS* design values for the material you have chosen, and finally a Print Preview compiling the whole shooting match.

Where BeamChek proceeds in linear fashion forward or backward from one step to the next, StruCalc allows you to skip directly among any of its screens, which I found useful. But it was a bit easier to interpret results in BeamChek, which presents a simple table contrasting actual vs. critical bending, shear and deflection. The same information is available in the StruCalc print preview, but you have to look around to find it.

One peculiarity of both programs has to do with their accounting for timber self-weight. BeamChek gives you the option of neglecting the dead load of the timber, StruCalc does not (or I couldn't find it). In BeamChek, remember, for a full 5 x 9 I had to enter it as a 5½x9½ to produce the correct section modulus and area and the correct resultant bending and shear, but it apparently calculated reactions and deflection for the entered oversize section. StruCalc, on the other hand, seemingly undervalued beam self-weight, explicitly listing it at 7 lbs. per lineal foot (plf) in the print preview. (Taking pine density as 30 lbs. cu. ft. (pcf), I come up with a self-weight of 9.375 plf.) Still, these are minor quibbles that don't undercut the power and versatility of these beam calculators.

Woodworks Sizer (Canadian Wood Council, Ottawa, Ontario, Canada K1P 6B9, cwc.ca/woodworks+software; single-user license $295). Woodworks Sizer offers column and beam analysis, with optional multiple spans, loads, load types and cantilevers, plus choices for governing code, load duration and a range of other parameters. It has the familiar succession of windows: Input, Loads, a Run button and several choices of tabulated output. The principal results page, the Design Check Calculation Sheet, contains a load diagram with dead and live load reactions, with shear and bending stresses and deflection nicely laid out showing resultants vs. design values. Another click will get you Analysis Diagrams of reactions, shear, bending and deflection. To enable the widest range of timber inputs, selecting Timber-Other as the material choice gives you the full *NDS* palette. On the self-weight question, Woodworks Sizer offers the choice to account for it automatically or manually and posits a value of 7.87 plf for Eastern white pine (which works out to a density of 25.2 pcf). Of these three programs, I found that Woodworks Sizer operated most closely to my usual procedure of generating resultant stresses and deflections and comparing them to allowables. Plus it's broadly configurable, but with options that don't encumber data entry or force the user to run a bureaucratic gantlet.

Enercalc (Enercalc, Inc., Corona del Mar, California 92525, enercalc.com; single-user license $1,495). Enercalc takes a step up in complexity and capability, to a whole lot more project management and broader scope of calculation (beyond design of beams and columns to foundations, concrete and masonry walls, diaphragm, shear wall and 2D frame analysis, etc.). Along with this enhanced range and performance comes a steep learning curve and a significantly higher cost.

So, getting to the dog-ate-the-homework disclosure, I did not put Enercalc through its paces, the issue being one of apples and oranges. Target applications for our beam software assessment are dedicated spreadsheets aimed at easing the calculation load for basic beam analysis. Enercalc—in the price and function range of full-blown frame-analysis programs—is out of the ambit of this review, though certainly of interest to the professional timber engineers among our readers.

Smartphone apps Recently a whole repertoire of beam calculators appeared on the market in the form of smartphone applications. With over 300,000 iPhone apps plus 100,000 Android apps in circulation and more appearing daily, it's pretty tough to keep up with the offerings in any one area. So there can be no pretense here of covering this new waterfront.

Browsing the iPhone app store one day, I tripped across a joist sizer from the Western Wood Products Association (WWPA). As you might expect, it was limited to western wood species and standard joist sections and spacings. But it opened my eyes to timber design app possibilities. Browsing further, I came across a dozen or so for the iPhone that showed some promise.

Here's my favorite to date: **Structural Wood Design Calculator** (Studio.618, Inc., Shaanxi, P.R. China, gonkculator.com, iPhone app, $4.95 full, $0.99 lite). The wood calculator comes adapted for both major unit systems, in feet-and-inch and metric configurations, available alike in full and lite versions. The full calculators will deal with columns as well as beams with full and partial uniform loads, midspan and randomly located point loads, single- and double-tapered loads, uniform loads with beam fixed at one end, cantilever beams with point load at the free end, and double and triple symmetrically placed point loads.

The SWDC lite versions are limited to simple uniformly loaded beams. The opening screen on both full and lite versions has a link to a help-tutorial page (www.gonkculator.com/wbchelpus.htm). Yellow fields are for data entry, green fields give resultant calculations, as shown in the screenshot above, displaying our 5x9. —*E.L.*

Are SIPs Necesssary?

AT least once a year, I have a client in my office who'd like to build a timber frame and enclose it in strawbales. The first time this happened, we agreed and pushed forward, eager to learn about strawbale enclosures. We raised the frame in January of 2007, and the owner-builders and a strawbale subcontractor set to work enclosing it, followed by plastering. The owners survived (kind of) two weird winter rainstorms, daily wind, and an incompetent contractor, and finally got the thing plastered by late fall. They've since replaced the exterior plaster in its entirety twice, and they're still dealing with plaster problems.

I now promote a dialogue with clients that examines alternative enclosure systems for every project we do. While strawbale enclosure may well be practical or even ideal in other circumstances, for our local conditions (we're in Fort Collins, Colorado), local contractors and the budgets of our clients, we have never since concluded that the best enclosure system was strawbales.

What about structural insulated panels (SIPs), then? Since most timber frame contractors encourage the use of SIPs, they must be the best, right? We were certainly happy for some time to promote them as the best choice. It was easy to dismiss the strawbale alternative because of the bad experience of our clients, and building external stud walls and then insulating *them* surely couldn't be energy efficient, could it? Were SIPs the answer?

I accepted as true all the benefits claimed by SIP salesmen and manufacturers, taught them to our clients and tried to sell panels to everyone we worked with. I hadn't done a lot of research and didn't understand alternatives. I wasn't even thinking about return on investment at that point, and in sum I wasn't really able to give my clients enough information. I was a SIP salesman.

The last part of 2008 changed the way we do business. We trotted along until about October 15 when a sale we thought was a done deal canceled with a 60-second phone call. Frustrating, but we still had January through March sold; maybe we could start a little early on that project. We could not, that job officially canceled via a message on my cell phone on Christmas Eve. I became familiar with the phrase "wave of cancellations."

I laid off staff and tried to find anything to do to keep remaining help in groceries. I set about a project of business introspection that I had never done. Since there was no work to manage, I had time to obsess with what-if questions. A free consultation from the Small Business Development Center introduced me to the concept of gross margin. A light bulb went off about how we'd been estimating, what our estimates might look like under an accurate analysis and, of course, the perpetual what-ifs. What if we lowered our prices and did more volume? What if we sold only timber frames and made no money on panels? What if we offered more services? What if we offered fewer services?

At the root of all those questions was one real goal—to sell more work—so I did the most examination of the jobs that had canceled. I naturally wondered whether if we'd somehow been able to charge less, would those jobs not have canceled? It wasn't as if we were getting rich or building exorbitant budgets, so how on earth could we reduce the price? The spreadsheet I was using at the time tracked percentage of overall budget next to each line item, and I had by then noticed that the number next to SIPs was consistently around 30 percent. I had all the motivation I needed (hunger) to take a close look at these panels and try to figure out if they were worth their apparent premium.

The alternative I began to consider was a studded wall of 2x6s, 24 in. on center, filled with sprayed open-cell foam. I went through all of the last ten jobs we hadn't sold and built estimates for replacing panels with the frame and spray system. I got consistent results. In those ten jobs, using the prices of the day, it would have cost about $10 more per sq. ft. of finished house to use SIPs for walls and roofs. In more recent analyses, this number falls between $5 and $10 per sq. ft. Panel prices in our area have gone down substantially, while the price we've been willing to pay to framing carpenters has gone up. Commodity lumber pricing is also fickle. When I ran the original price comparisons, street price for oriented strand board (OSB) sheathing was less than $7 a sheet. Last summer, it got as high as $14 a sheet, and right now in early 2011 it's at $9.57.

I've stopped running panel estimates because of my conclusions, but I did go back to analyze a 2065-sq.-ft. house we built last summer. I know what we actually paid to enclose it, and I know current rates for panels here. To have built the roof and exterior walls with panels would have cost an additional $14,000, or $6.78 per sq. ft. That's less than the premium I've cited above but more than the $5.14 per sq. ft. we'll use below for some payback scenarios. It's probably right to assume that I might get a different figure for every house I analyzed, but I do believe my research identifies a definite trend. When we look at payback numbers later in this article, we'll use the bottom of the identified range of savings.

You may come up with different numbers because of your location, your climate, your local suppliers and your building season. In our area, framing carpentry rates are competitive, there are a lot of local lumberyards competing to sell studs and OSB sheathing, and we apparently get pretty good rates on spray foam. For these reasons, a favorable cost-benefit ratio for SIPs just doesn't appear to pan out here.

IT'S important to consider the entire cost of an installed system. Let's look at panels' advertised benefits to see if they are real for me or my clients.

1. *SIPs reduce waste.* Do they really? Or do they just move it to the manufacturer's or fabricator's warehouse where neither I nor my customers have to look at it? What about the chunks of panel that end up in the dumpster because of imperfect project management? Panel scraps, as far as I know, have almost no alternative use, while the pile of cutoffs from a 2x6 stud wall may have a second life as blocking, backing or, at worst, firewood.

2. *SIPs reduce labor.* Again, do they really, or do they just move it to the fabricator's shop and off my budget line? Even if they do reduce labor because the labor's more efficient in a factory setting, does that offset the additional cost of the system as a whole?

3. *SIPs install quickly, thereby saving time and money.* It's generally accepted that the actual applying of walls to the exterior of a timber frame takes a few days less with panels. As a general contractor, I'm not sure this benefit is more than academic. First, the installation time (and consequent cost of field labor) saved is outweighed by the cost of the panels compared with the cost of the insulated stud walls. And the time saved as a portion of the whole project is insignificant. Custom residential construction simply isn't managed tightly enough that a few days of time gained in wall installation provides a notable financial benefit (loan interest).

To take an actual example, the 2065-sq.-ft. house I mentioned above was studded out in 11 days by a crew of four. That framing time included the insulating walls, interior partitions, second floor framing and stairs. I'm not convinced you could actually install the panels and do all of the interior framing in less time than that. A good study of total project schedules also recognizes that other subcontractors take longer to perform their work later in the project, potentially nullifying any overall elapsed time advantage of SIPs. If you believe that saving a few days in a construction schedule is of benefit, do the math on the cost of the construction loan (if there is one), and see just how many dollars could be saved by shortening a construction schedule.

4. *SIPs save energy and "pay for themselves" in reduced heating and cooling costs.* I am not a scientist. I am a thinking carpenter turned general contractor who expects to be able to understand science as it applies to what I do. I make no exception for "building science," specifically in my search to understand the value of R-value. I've learned a lot about building energy-efficient buildings in the last couple of years, not least that it's very difficult to find consistent information about this particular subject. Sorry, but consistent effort as a researcher reveals a snake-oil environment. Everybody selling one kind of insulation or another (including SIP manufacturers who use urethane foams instead of polystyrene and vice versa) claims everybody else is wrong about initial R-values, R-value creep, air seal, greenhouse emissions, off-gassing, etc. More to the point, though, is this question: "If my clients spend extra money at installation, can I demonstrate that they'll recover that money over a reasonable time via reduced fuel bills and, if so, how long it will take?"

I would expect building scientists to be able to model the following relationship. At R-values of *X* for walls and roof, the calculated heat loss of your house is equal to *Y* Btu/hr (British thermal units per hour). At current (and projected) costs of fuel, the cost to generate those Btu/hr is $*Z*. If this calculation existed, we could then change R-values on the input side, and monitor the result in dollars on the output side. (If we can land a man on the moon, we can make Excel generate these values.) I've found only one person running anything close to this software, one of our local radiant heating contractors. In an attempt to approach this question scientifically, he and I modeled a sample 1500-sq.-ft. house three times, with the only variable the wall system. (At the time we ran this comparison, I was only considering changing wall systems. I was still assuming SIP roofs were the best choice. I don't any longer.)

We analyzed three wall systems: 6-in expanded polystyrene (EPS) SIPs; stud-framed 2x6 walls filled with open-cell foam; and 6-in. stud-framed, similarly foamed walls but this time with woven 2x4 studs to eliminate thermal bridging. Thermal bridging, for the record, is not the demon that SIP manufacturers would have you believe. The effect of thermal bridging on overall R-value of a wall can be calculated in much the same fashion as for windows in a wall. After modeling the three systems, we achieved the following heat loss values in Btu/hr: 6-in. EPS, 25,343; 2x6 wall, 25,757; and 2x6 wall with staggered 2x4 studs, 25,659.

As a point of reference, note that a standing human generates 400–450 Btu/hr, and closer to 800 when dancing. The total difference from worst to first here is 414. Run the numbers and you'll notice that there's 1.6 percent more heat loss in the 2x6 framed wall than there is in the SIP wall. Although I can't prove it, I suspect 1.6 percent is within the range of error in the calculations, and that we've actually proven that there is no significant difference in thermal performance between a foam-insulated stud wall and a SIP.

To put this into a financial perspective, upgrading to panels on the job we were designing at the time, a 2100-sq.-ft. house, would have cost an extra $10,800. That includes labor and materials for a complete installation of wall and roof systems, although it does not include additional costs that I believe SIP manufacturers would prefer you to ignore, such as for window-jamb extensions, extra charges by your electrician for rough-in, the difficulty of hanging cabinets on SIP walls and, in general, the fairly constant slight increases in cost when subcontractors are dealing with unfamiliar building systems.

Although the cost of increasing indoor temperature in the winter is not linear (it takes less energy to increase indoor air temperature from 50 to 60 than it does from 60 to 70), let's look at this question as if it were. To pay back a $10,800 investment in 10 years, we'd need to be able to reduce our heating bills by $1,080 per year or $90 a month. If the supposedly less-efficient 2x6 system in fact requires us to generate 1.6 percent more energy to maintain temperature, $90 needs to be about equal to 1.6 percent of the heating bill. For a ten-year payback on additional SIP expense, then, our heating bills would need to be $67,500 per year or $5,625 a month for this 2100-sq.-ft. house!

Using typical heating numbers from my area, it would probably cost less than half of $5,625 to heat an insulated 2100-sq.-ft. house *for a year* with propane, the most expensive fuel option here. (I live in a 2800-sq.-ft. house built in 1918. The remodeled parts are well insulated with cellulose but the one-third of the house that hasn't been touched still has old windows that air quite literally blows through. The most expensive heating bill I've ever had was $210 in one month; most months are less than $100.) Supposing a conservative total of $2,400 a year to heat the 2100-sq.-ft. house we designed, SIPs would then save $38.40 a year and payback of the $10,800 investment would take a staggering 281 years.

If there's a flaw in my reasoning, I'd like it pointed out by a neutral authority, and I'd also like to make it clear that I'm not suggesting SIPs have no place in the timber frame industry. I am pleading for us to have and to use better research and information when we help our clients make decisions about enclosure. If SIPs are a lot less expensive in your area than in mine, or carpenters are much more expensive or unavailable, or the closest place to find a spray foam installer is a day's drive away, or your building season is really short, panels may be the best choice for you.

For me, there are additional factors to consider about enclosure systems. The largest one is longevity. I'm not sure when structural insulated panels were first put in service, but I do know that we don't have a lot of experience in how they endure. I believe it's worth noting that the entire panel system, from exterior sheathing to interior finish, is dependent on adhesives. How long are those going to last? What will happen to these buildings if the adhesives fail? Are they repairable? We may clad our studded walls in adhesive-dependent OSB, but the connection of cladding to studding is mechanical and thus reversible. If a sheet (or an entire wall) of OSB goes bad on a studded wall, it can be replaced without disturbing the interior. Not possible with SIPs: the whole sandwich has to be removed, including interior finish.

There's a lot more history available for light wood frames. I grant that a lot of it's not good in terms of energy consumption. Light wood frames, though, seem to last fairly well, they're repairable in part if parts are damaged and they can actually be recycled. (Anyone recycling SIPs?) I know roofs shouldn't leak, but I also know that almost all of them do, sooner or later. One of the principles of really long-term buildings is first to accept that they will all eventually fall into disrepair. When that happens, are they fixable? And when they are ultimately deconstructed, are their materials recyclable? As far as I can tell, SIPs fail these tests. For me, that adds two more reasons for us to not use them on our buildings. —Adrian Jones

Adrian Jones (adrian@frameworkstimber.com) operates Frameworks Timber in Fort Collins, Colorado. *A survey of contemporary timber frame enclosure methods appears on page 24.*

Elementary Building Geometry

THE geometry used by the architects of the Middle Ages was adapted to the design of structures more complex and ornate than ordinary houses. For purposes of house design, it's expedient to simplify the method, with two primary goals in mind. The first is ease of use with little training. The method is intended to be largely self-teaching: you learn by observing and doing. The second goal is for the method to be done just as easily on the ground at full scale, providing lines for a scribed layout, as it can be done on paper. To this end, after the initial circle we will rely on lines much more than circles. Circles are difficult to mark out on a large scale, particularly if a building is being laid out on rough ground.

A few basic rules must be followed to guide the geometer:

1. The circle must be drawn first to ensure that the lines relating to it will be accurately made.

2. All points and lines derive from the first circle and its bisector. Ideally this concept should be applied down to the smallest aspects of the design to ensure that everything "fits" harmoniously. The notable exceptions are the location of the center of the first circle itself and its chosen radius.

3. Any line segment can be extended infinitely in either direction beyond the two points used to create it.

The tools are simple and easy to come by. The particular tools used, however, depend on where you are performing the geometry. If on paper (as in this exercise), you will need an accurate compass, a reliable straightedge and at least one pen. Several pens and markers of different colors can be useful. If at full scale, you will replace the compass with a strong rope and an assistant or appliance to hold center, and the straightedge with string or chalk line. If on rough ground, you will also need stakes to mark points.

The photos, in some cases unfortuately distorted, illustrate the design process for a simple building. Many lines must be drawn before any of the building lines are defined, but once the bounds of the structure are defined, all the necessary geometry is available.

This example uses the mode of geometry known as *Ad Quadratum*, which relies on squares to construct designs and yield proportions and relationships. (Another mode, *Ad Triangulum*, relies on equilateral triangles for similar purposes. Its most basic representation is the daisy wheel. See TF 95.) The geometry can be used for more complicated structures than the simple rectangular building shown here, by extending or repeating the basic operations shown. Certain elements such as windows, doors and decorations can also be laid out with their own geometry, using the basic outlines established by the primary geometry but not relying on its lines for anything else.

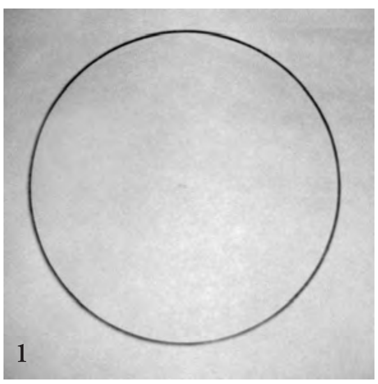
—David Bahler

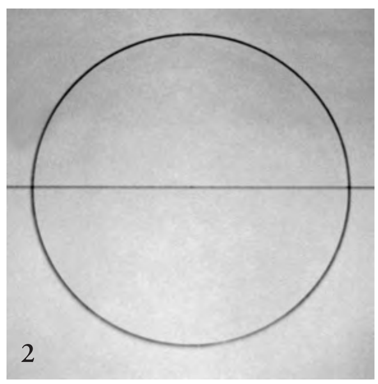
David Bahler (dlbahler@live.com) is a carpenter near Kokomo, Ind.

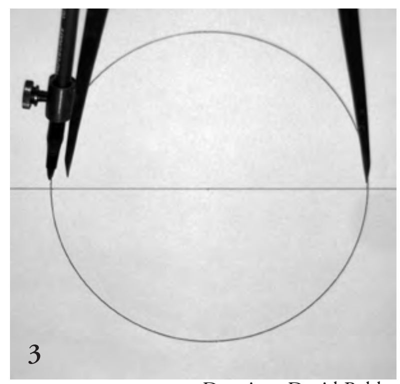
1 First circle. If full scale, size of circle will ultimately determine dimension of building.

2 Bisector line through center establishes longitudinal axis of floor plan or horizontal axis of elevation and provides reference point for other lines and circles.

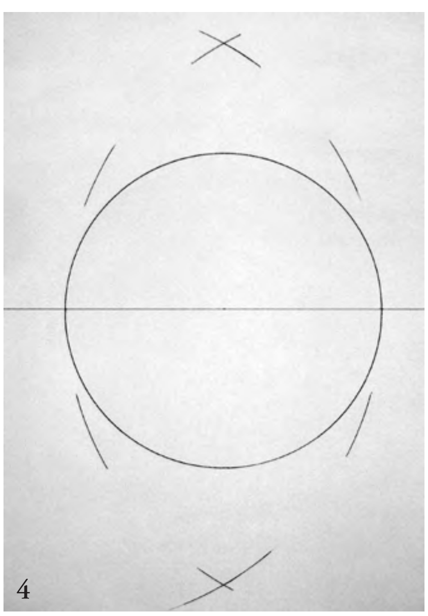
3 Reset compass to diameter of master circle to develop perpendicular line in Figs. 4 and 5.

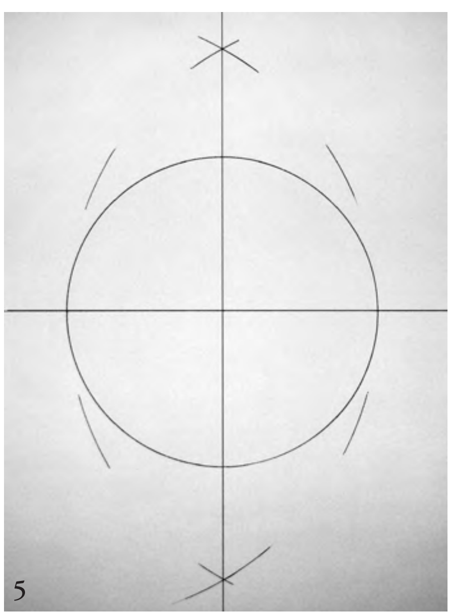




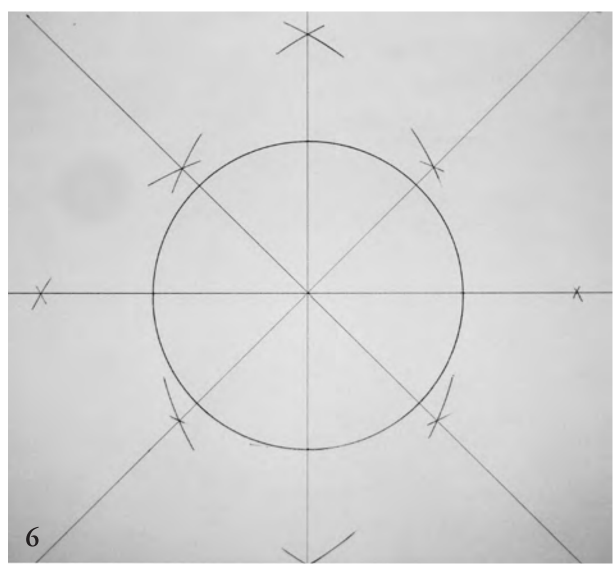


Drawings David Bahler





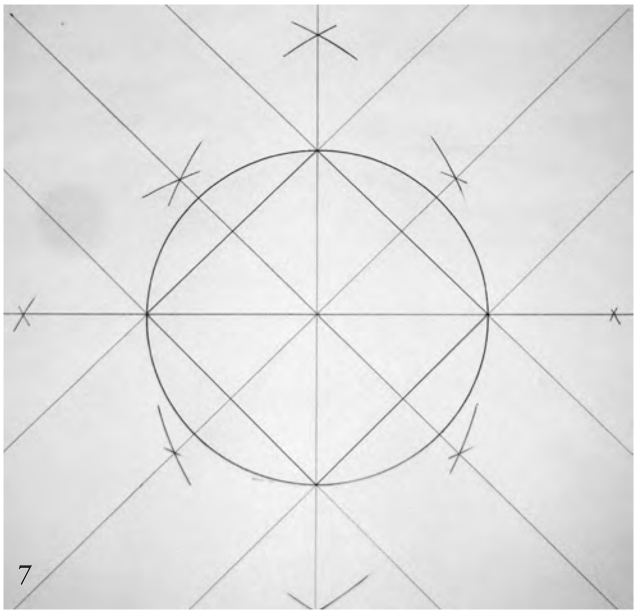
4, 5 Swing arcs from points where axis line intersects circle and erect perpendicular to form transverse axis of floor plan or vertical axis of elevation.

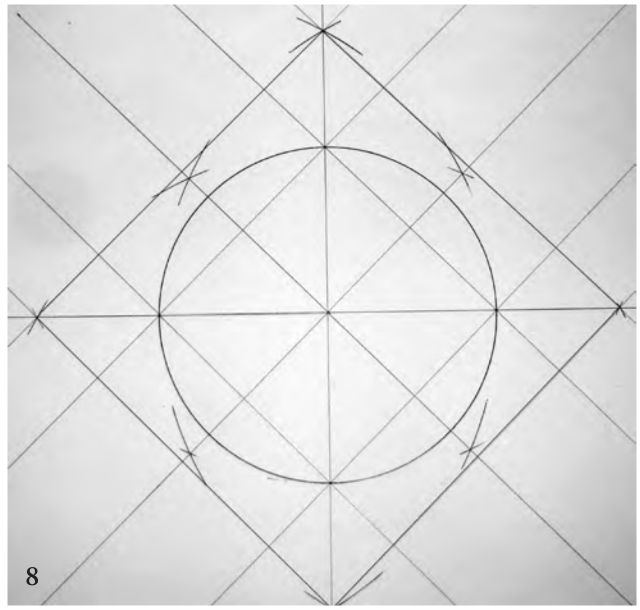


6 Similarly, swing second set of arcs from intersection points of master circle and second axis and erect second set of perpendiculars at 45-degree angles to the first set.

7 Connect intersection of axes and circle to inscribe square. Extend lines beyond circle for future reference.

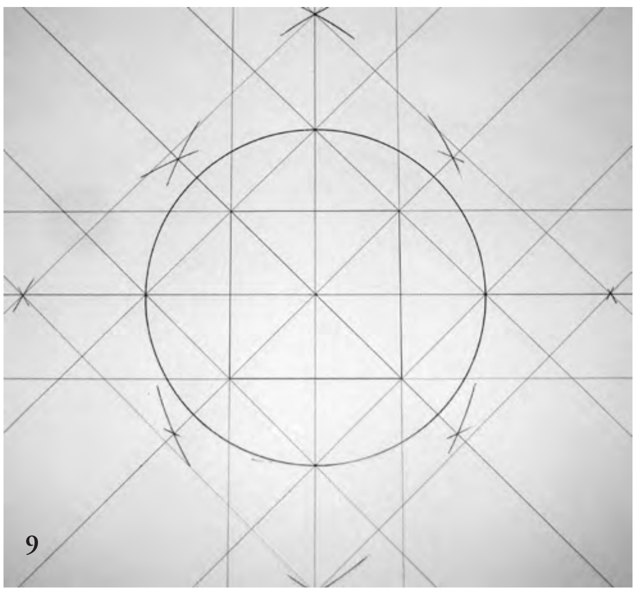
8 Connect outer arc intersections to form outer square.

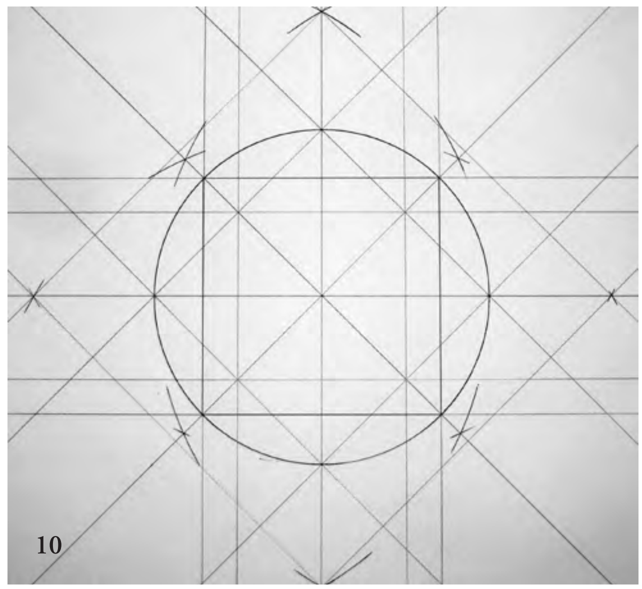




9 Develop second inner square with lines extended.

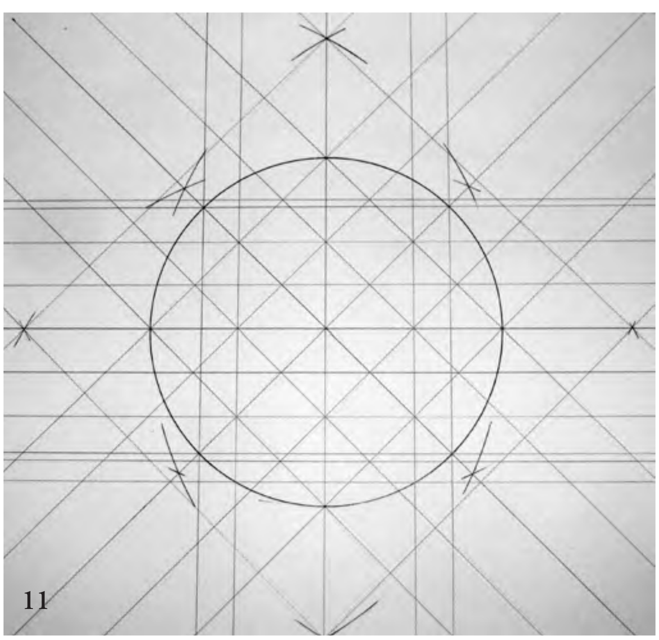
10 Connect four 45-degree perpendicular intersection points with circle to develop third inner square.

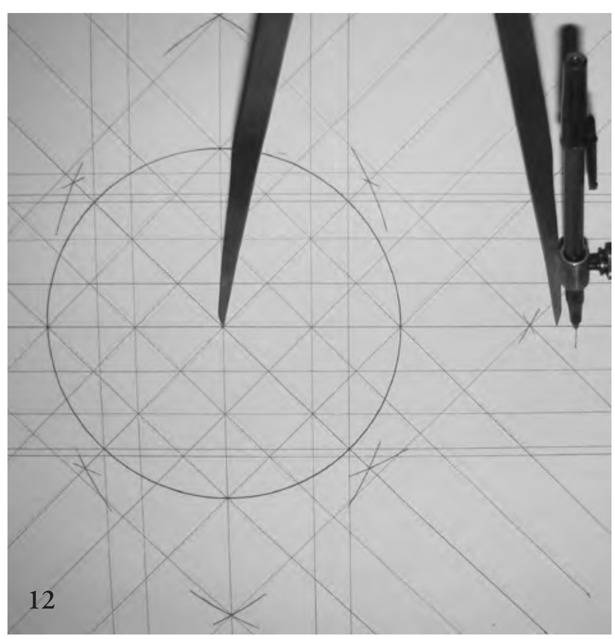


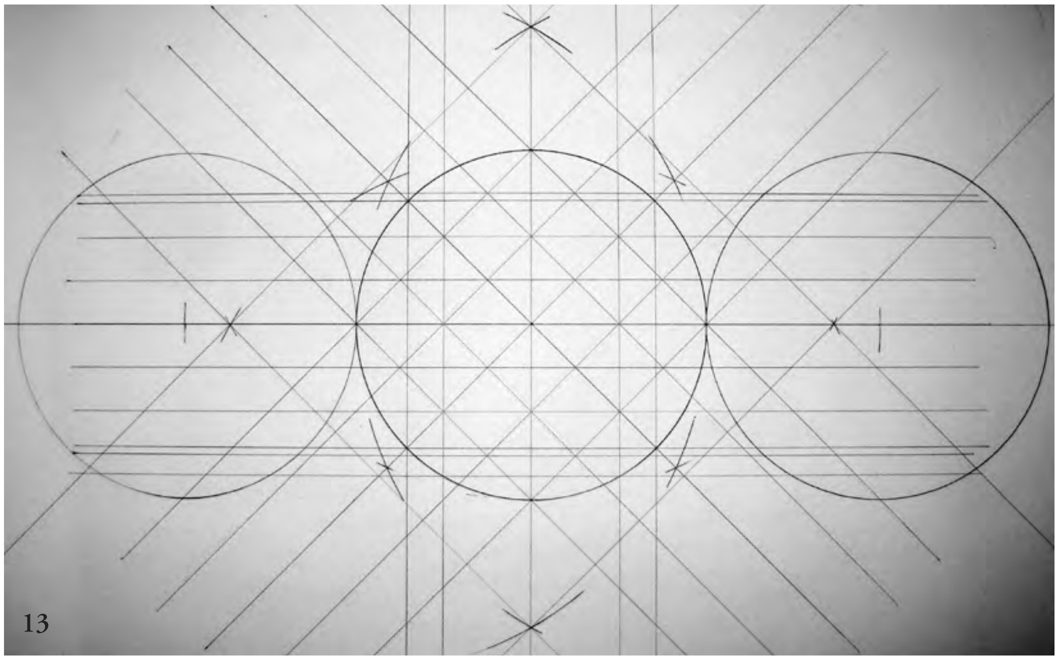


11 Connect numerous intersection points to extend grid, including intersection points of generated grid lines.

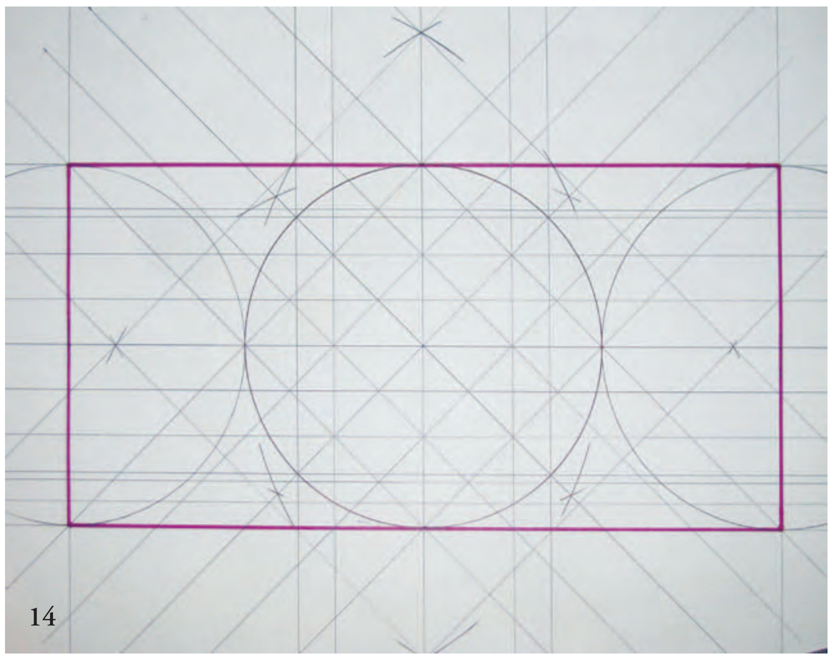
12 Continue with compass set to diameter of master circle, step off for two contiguous circles.



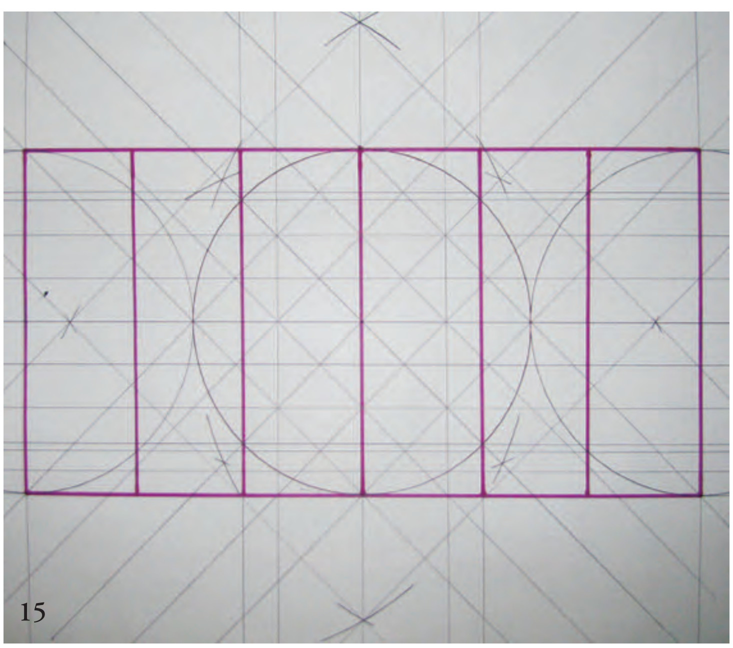




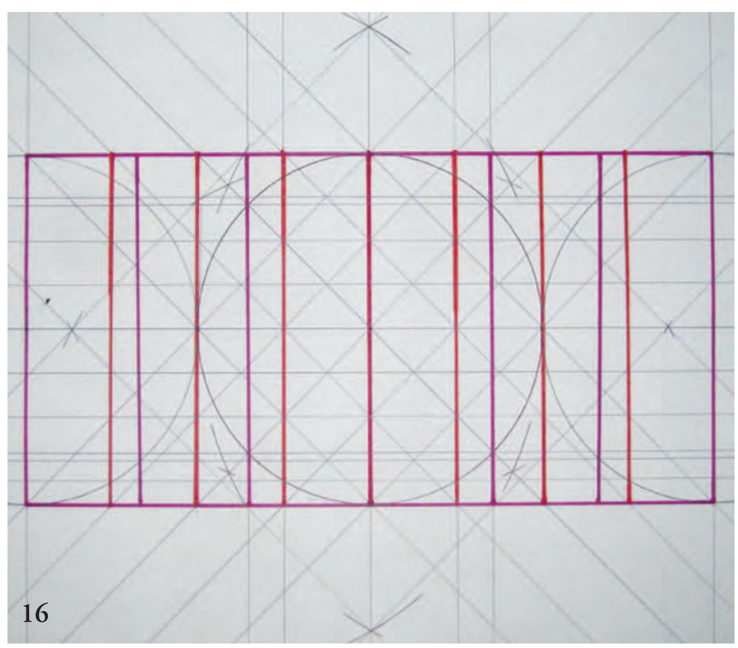
13 Draw equal-diameter circles on new centers (and tangent to master circle) to help establish wall positions of ultimate floor plan.



14 Draw tangents to connect circles and establish width of building, then connect them with vertical lines to establish desired length of building. In this example we connect diagonals that intersect flanking circles at tangent points (simultaneously passing through circles' centerpoints) to produce a length-to-width ratio of 2:1. The geometric lines of interest are over-marked with a felt-tip pen, as are all such lines in the following photos.

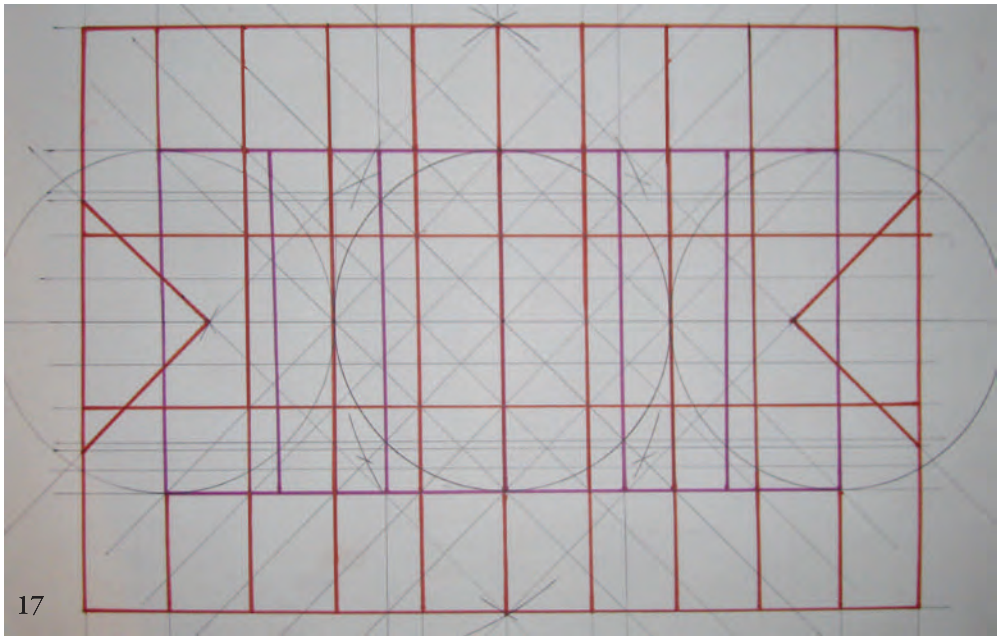


15 Divide the floor plan according to the location of principal posts or crossframes. Cross-frames here shown overmarked purple.

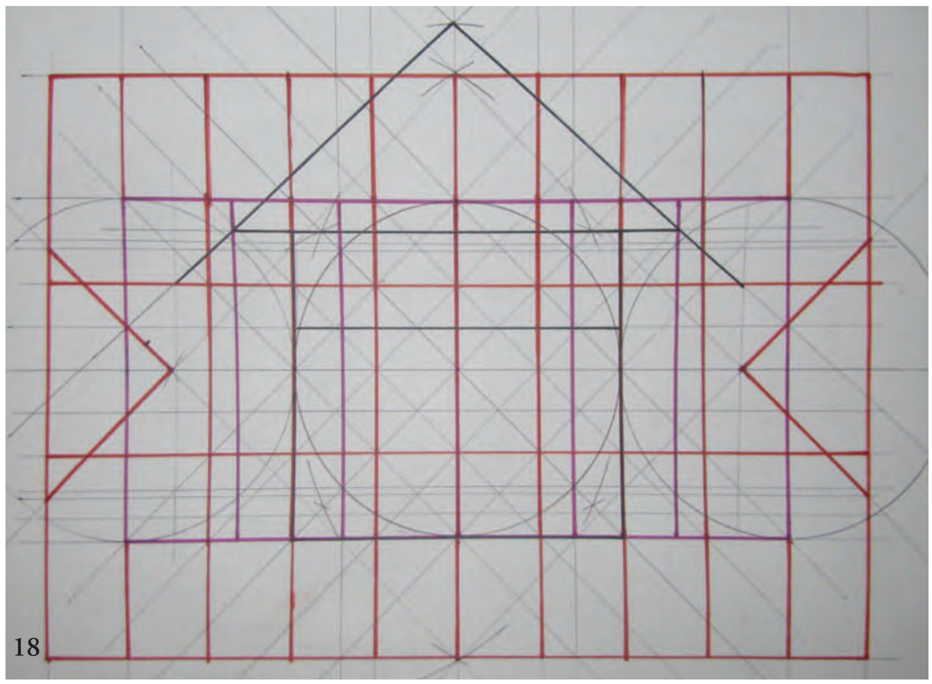


16 Divide the floor plan further according to location of joists or, here, rafters. Rafter divisions overmarked freehand in red.

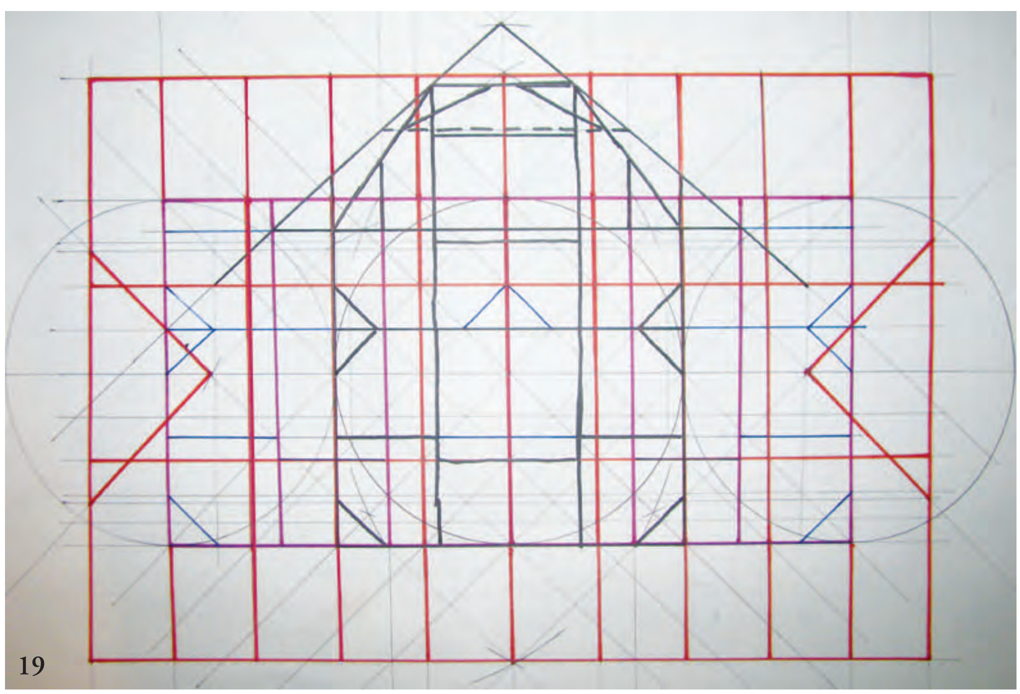
17 Draw roof plan using desired gable and eaves overhangs. In this example, wide overhangs are chosen partly for architectural style and partly to help illustrate the geometry. Note that building lines are defined by lines of geometry yielded previously. Very little new work is needed. This photo, though imperfectly corrected for camera distortion, shows locations of rafters, purlins and lines of hip rafters for half-hip roof ends, overdrawn freehand.



18 Add front elevation (heavier black lines) following mostly established lines of geometry and using existing points to establish certain new lines such as a roof line. Elevation shown is one and a half stories in height. Roof line shown has particularly wide overhang, cantilevered joists and pitch of approximately 10½ in 12. The roof pitch is one example of how geometric relations do not equate to modern mathematical relations.



19 Front elevation completely defined, showing locations of all framing members including braces and roof support structure (black lines) as well as window locations (center of the gable end wall on both levels). These lines are made mostly by extending geometry already present. Blue lines illustrate framing of side walls, including bracing. Thus a completed frame has been laid out according to principles of Ad Quadratum geometry.



New Hope Methodist Church

Photos and drawings Seth Kelley

1 New Hope Methodist Church (1854), Waits River, Vermont, photographed in 2008.

2, 3 Scarfed sill and post repair test-assembled on horses, and fitted in place. Timberwork by Michael Cuba.

4 Facing page, top, double-main-braced queenpost truss with doubled straining beams supporting rear tower posts. Sleepers to support front tower posts run longitudinally between rear faces of front gable posts and front faces of rear tower posts.

5 Facing page, middle, one of two median queenpost trusses supporting purlins and common rafters.

6 Facing page, bottom, partial longitudinal frame section, from rear tower posts to back of church, taken at upper purlin line but also showing edge of interrupted flying plate.

THE New Hope Methodist Church (1854), a Greek Revival building nestled among the houses and barns of Waits River, Vermont, is one of the most photographed churches in the state (Fig. 1). Arriving one fine summer's day to restore a section of rotted sill and post, we unloaded tools, cribbing and screw jacks, donned our headlamps and crawled under the building, pushing one block of cribbing at a time over and under heating ducts and clawing through some serious spider webs. After getting the joists supported, I looked over to my coworker Michael Cuba, and we both grinned. We couldn't wait any longer to go to the attic to see the trusses.

Specializing in timber frame restoration, we get to see some of the most impressive work out there, even if it does come at a cost. Not all of our work is glamorous. We can't be claustrophobic or afraid to get wicked dirty. But we believe it's the best way to learn and understand the craft of framing. If we pay attention and know where to look, we might just find a detail that will change the way we frame. We enjoy working on old buildings and the technology they display.

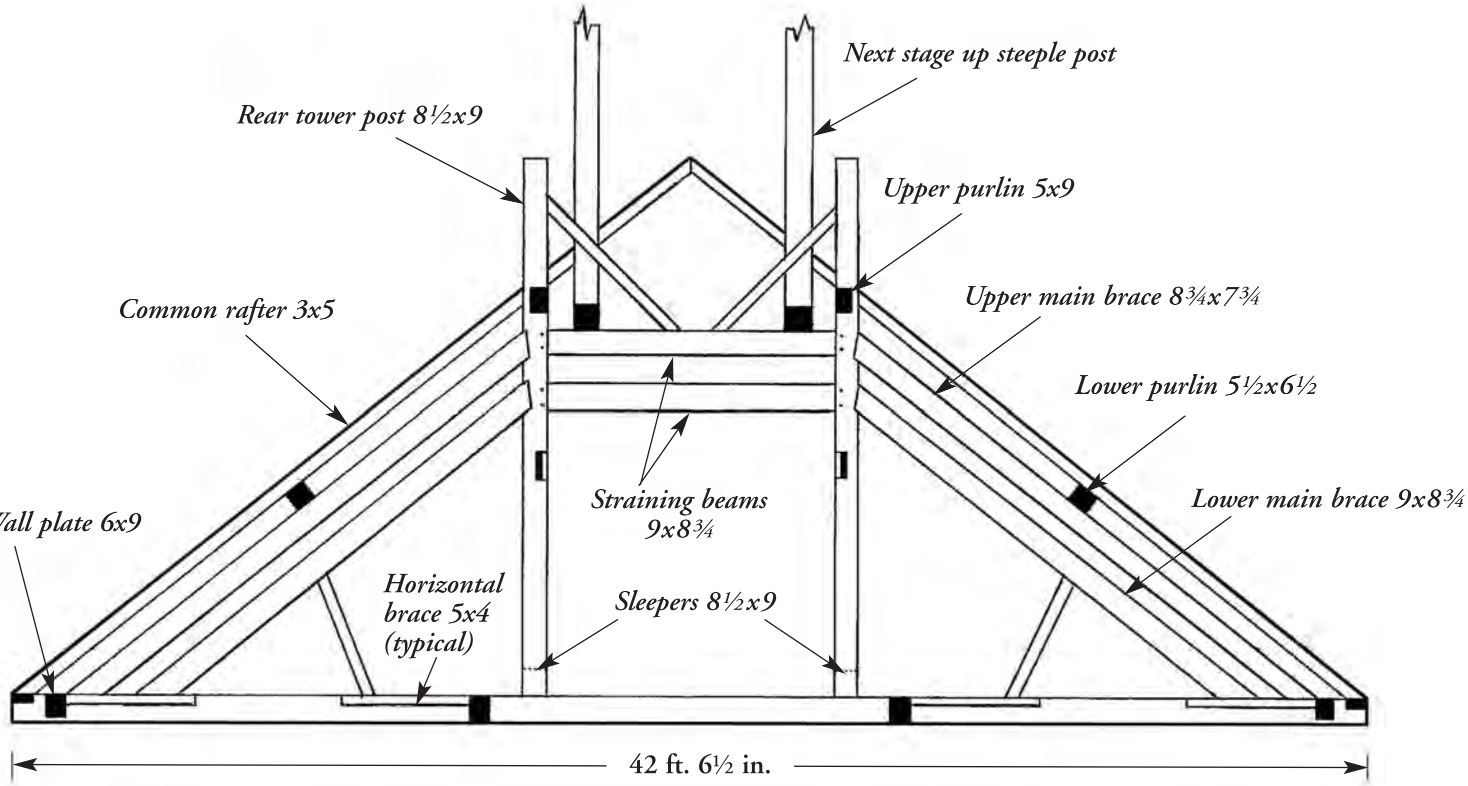
Climbing the stairs up into the attic of churches has always been exciting. What are we going to see? A massive truss, builder's marks, beautifully hewn hardwood timbers? Standing on the lower chord of a roof truss that spans over 40 ft. is certainly humbling. Thus we stood for a few moments in the New Hope attic to take it all in—and we did find something new to us.

After we had completed the sill and post work (Figs. 2, 3), we returned to the attic to observe and measure and to understand how the tower was framed into the truss. The roof frame of the New Hope church has features we're not used to seeing in truss work. First, while the rear tower posts of the telescoping steeple are built into a queenpost truss to help with the weight of the steeple, as is frequently seen, in this case the truss additionally employs doubled main braces and straining beams, with the latter untypically pinned to the queenposts (Fig. 4).

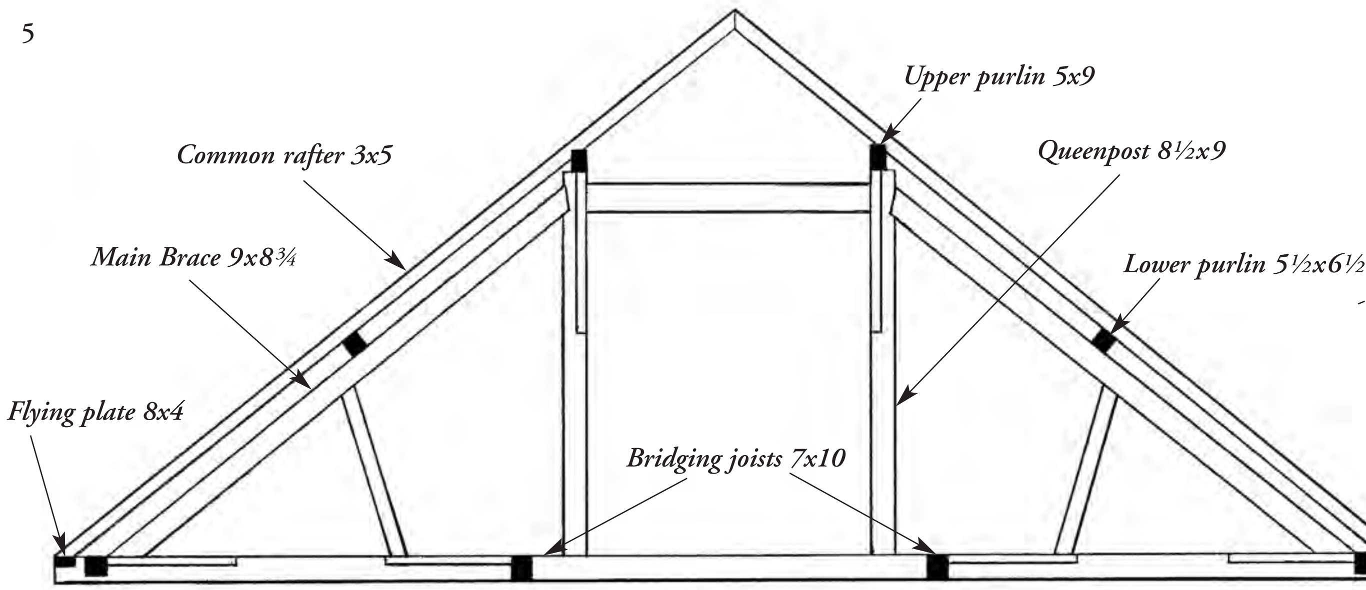
Second, both gable ends of the attic are framed in queenpost trusses similar to the median trusses (Fig. 5), rather than simply studded from wall plate to rafters, especially surprising at the rear of the church (Fig. 6), where there are no nearby concentrated loads. Given the support of posts, studs and braces in the wall below, there is normally no need to truss a gable-end frame. The queenposts in the gable-end framing can perhaps be explained as support for the purlin ends, but the main braces (upper chords), representing extra work, are puzzling. Standardization of parts? In any case, the queenposts at the front gable end and at the rear of the tower (all high-quality sawn spruce) are housed 2 in. deep at their feet to accept the ends of 8½x9 sleepers, which bear on the tie beams and support the front tower posts about 2 ft. back from the front gable end (Figs. 7–11 overleaf).

The trusses support two lines of purlins on each side of the gable roof. The lower purlins (mixed hewn and sash-sawn) run parallel

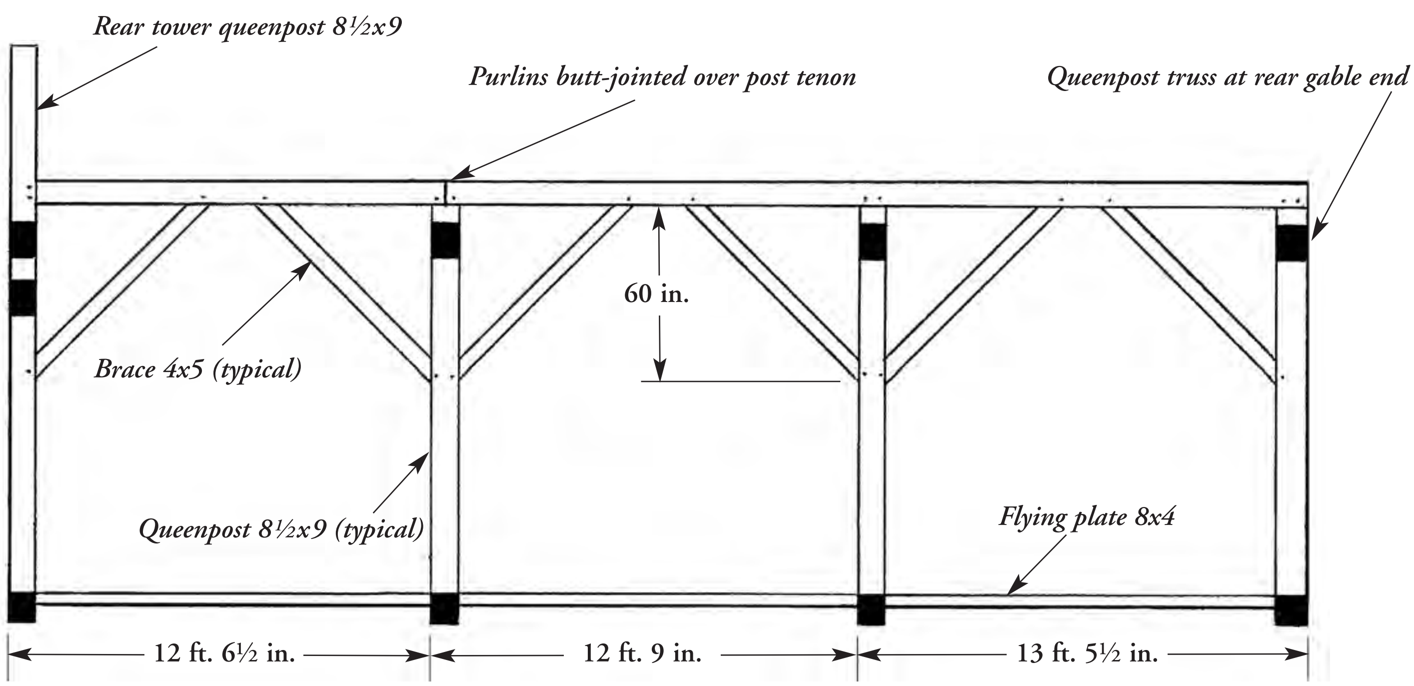
4


Next stage up steeple post
Rear tower post 8½x9
Upper purlin 5x9
Common rafter 3x5
Upper main brace 8¾x7¾
Lower purlin 5½x6½
Straining beams 9x8¾
Lower main brace 9x8¾
Wall plate 6x9
Horizontal brace 5x4 (typical)
Sleepers 8½x9
42 ft. 6½ in.

5


Upper purlin 5x9
Common rafter 3x5
Queenpost 8½x9
Main Brace 9x8¾
Lower purlin 5½x6½
Flying plate 8x4
Bridging joists 7x10

6


Rear tower queenpost 8½x9
Purlins butt-jointed over post tenon
Queenpost truss at rear gable end
60 in.
Brace 4x5 (typical)
Queenpost 8½x9 (typical)
Flying plate 8x4
12 ft. 6½ in.
12 ft. 9 in.
13 ft. 5½ in.

to the roof plane and rest on truss main braces, fastened by single 1¼-in. ash pegs at each crossing. Where they meet in the length, they butt together over a main brace, and just below this joint a strut rises from the tie beam to the main brace to help with the bending force of the roof weight (Fig. 9).

The upper purlins (again mixed hewn and sash-sawn) run from gable end to gable end, mortised over the tops of the queenposts (Figs. 6 and 10). They are interrupted by the tower frame (Fig. 8) and, one truss back in the roof frame, end-butted and mortised over a shared queenpost tenon—a doubtful joint still performing well after 156 years. Unusually long knee braces (more than 7 ft.) rise from the queenposts to these purlins (Fig. 10).

Meanwhile, at the bottom of the queenposts, the framing includes two bridging (longitudinal) joists in each bay connecting the tie beams of the trusses, set 1 ft. outboard of the queenposts and horizontally braced in opposition to the wall plate braces. A set of light ceiling joists (unseen) completes the attic frame.

The New Hope queenpost roof is perhaps unusual in being fitted with full-length common rafters. In other queenpost-trussed roofs we have seen, the queenpost tops tenon into the underside of principal rafters that carry principal purlins in the same plane, with short common rafters filling in between purlin and ridge and between purlin and plate. The 3x5 common rafters here land on an interrupted flying plate, which tenons into the tie beam faces right at the end of the tie beam. The wall plate likewise tenons into the tie beam faces, several inches inboard of the flying plate, the latter connection stiffened with horizontal braces (Fig. 12).

Rafters landing on flying plates are worrisome. At the plate's skimpy connection to the tie beam, there's little relish and only a single peg to resist outward thrust rafters might impose. As long as the trusses support the rafters, there shouldn't be any thrust, but we all know that trusses settle. In this frame, however, all the lines are tight and the trusses seem to be performing well. As I stood in the attic, I appreciated how simple and elegant all the framing really was.

A year later we made an inspection visit to the New Hope church, and my colleague Michael found something we had missed the first time—wooden plugs in the sides of the queenposts a foot up from the tie beams. Michael managed to wiggle one block out to reveal a square nut, wedged against rotation and presumably fitting a hidden bolt clamping the tie beam up to the post (Figs. 13 and 14), an arrangement I have seen drawn in a 19th-century builder's manual by Asher Benjamin. —Seth Kelley

Seth Kelley runs Knobb Hill Joinery (www.knobbhill.com) in Plainfield, Vermont.

7 Upper rear tower queenpost with doubled main braces and doubled, pegged straining beams, and longitudinal brace to upper purlin. High quality evident of spruce timber and joinery.

8 Front gable-end framing with queenpost and main brace. Front tower post rises at right, interrupting upper purlin tenoned to it on front and rear faces.

9 Strut assisting main brace to support lower purlin.

10 Detail of upper median queenpost truss with long brace to upper purlin supporting 3x5 common rafters.

11 Housed sleeper connection at front gable-end queenpost. Sleeper rests on tie beam here and at rear tower queenpost truss. Member at left passing over sleeper is not structural.

12 Tie beam, main brace, wall plates with horizontal braces to tie beam, flying plates, common rafters.

13, 14 Two queenposts, showing a plug *in situ* and another removed to reveal square nut for bolt passing up from tie beam into post.

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10

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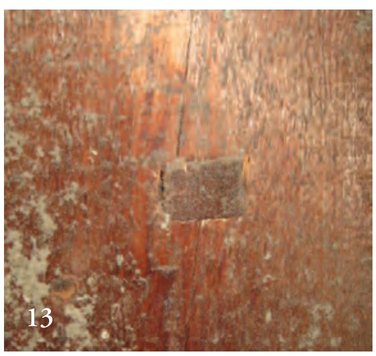


13

14

Enclosing the Timber Frame

IN the modern revival of American timber framing, timber framers have had access to a wealth of source material in developing standards for joinery design, layout methods and even cutting techniques. Centuries-old barns, houses and other buildings still stand to serve as examples. With a little sleuthing, and adaptations to modern engineering standards, traditional timber joinery details can be used in contemporary timber framing.

But what of building enclosures, specifically for timber frames meant to be heated—the exterior walls of a building, including interior finish, insulation and exterior finish? For an unheated outbuilding such as a barn, a contemporary enclosure system would not be unfamiliar to an 18th-century carpenter, for instance vertical siding over horizontal nailers, or horizontal lap siding over interim studding. But the translation from traditional to contemporary is not so straightforward in designing insulated building enclosures. For stylistic as well as thermal reasons, today's enclosure systems are usually installed outside of the timber frame, which exposes three sides of the timbers to the interior and allows for a more nearly seamless "wrap" of the frame, making an airtight shell (or something close to it) feasible. This configuration contrasts markedly with the enclosure of first-period American 17th-century houses, where a thin exterior shell (if any) of weatherboarding was installed over masonry or earthen infill between timbers. It contrasts as well with later enclosures, from the 18th century up to about the middle of the 20th century, where the exterior shell was a double-layer wood membrane, still relatively thin, that aspired to be draftproof and, together with a lath-and-plastered interior membrane, enclosed an air space, completely concealing the frame within (Figs. 1–3).

What of the framer who wants to use traditional enclosure systems? Historical examples abound, but traditional enclosure systems will not be compatible with modern expectations of comfort or energy efficiency. Carl Bridenbaugh, the historian of colonial America, says (in Bill Bryson's *A Short History of Private Life*, 2010) that the average Colonial home required 15 to 20 cords of wood per year. Even including cooking, the figure still reflects a huge consumption of energy, hardly in line with contemporary expectations of energy efficiency or sustainable fuel consumption.

Looking to fulfill expectations of comfort using an enclosure system from a climate that differs markedly from your own may yield undesirable results, both extrinsic (resistance or refusal by code officials) and intrinsic (moisture problems and degradation of building materials). But exposing timbers to the outside, when the climate allows, fits well with certain architectural styles such as the Japanese, frequently adapted for the US West Coast (Fig. 4).

A number of enclosure systems were used in the 1970s revival of American timber framing. Before the wide production in the mid-1980s of foam-core sandwich panels (known today as structural insulated panels or SIPs), timber frames were often enclosed with a light-framed wall system variously insulated. Books of the period describe such systems and show partial or full infill systems as well, again framed with construction lumber. Since the 1980s, SIPs have become the most common enclosure method for timber-framed houses. In recent years, however, a number of alternatives have emerged or reemerged. "Natural" enclosure systems using straw, hemp or wood chips are in limited but widespread use in North America. In addition, some builders have begun to integrate high-performance light-framing systems (such as those conforming to the Passive House standard) into their timber frame enclosure systems. As energy efficiency and sustainability (in its many interpretations) become more and more critical in construction, a new standard for enclosure systems may emerge. What follows is a comparative description of systems in common use in the first decade of the 21st century. Different systems have strengths and weaknesses and may be more or less appealing according to the requirements of the particular building and the priorities of the owners. What works in one context may be inappropriate in another.

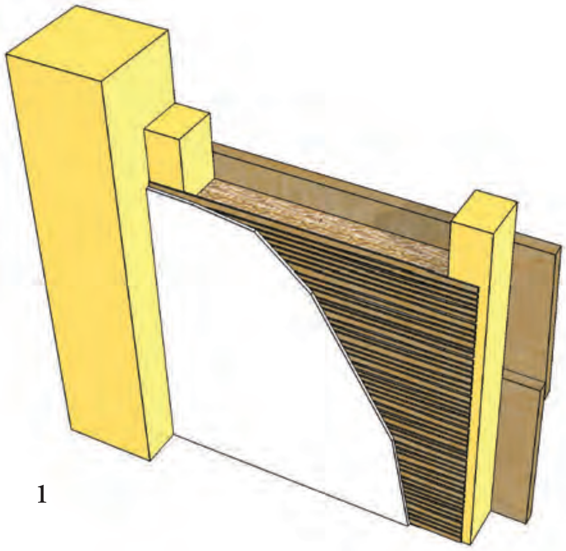
Structural Insulated Panels (SIPs) Sometimes labeled "stress-skin" panels, SIPs are by far the most common enclosure system for heated (or cooled) timber-framed structures. They comprise an outer sheet of oriented strand board (OSB), a plastic foam core and an inner sheet of OSB, gypsum board or tongue-and-groove boards. (The Structural Insulated Panel Association website, www.sips.org, has links to suppliers as well as technical details.) Panel sizes vary, with a minimum of 4x8 ft. and a maximum of 8x24 ft. Foam thickness usually corresponds to nominal lumber dimensions, 3½ in. or 5½ in., to allow trimming of openings with standard framing lumber. R-value varies with the kind of foam used—expanded polystyrene (EPS), extruded polystyrene (XPS) or isocyanurate—and the thickness of the core. Typical claimed wall R-values are R-14.4 for a 3½-in. EPS core and R-21.7 for a 3½-in. isocyanurate core (Fig. 5).

SIPs are manufactured products made in a factory and shipped to the job site whole or with window and door openings precut. Because they are high-value-added product (they embody lots of labor), material cost is relatively high compared to other enclosure systems. Installation cost can be relatively high as well, but the installation goes comparatively quickly.

Pros. As standard manufactured products, the panels' structural performance and insulating value have been measured and documented, which can help make code approval go smoothly, in particular passing the energy audit now required by most US states. In fact, since the insulation is considered to be continuous (uninterrupted by studs or rafters), most codes give a bonus to the R-value of a wall or roof with SIPs. When installed properly, a SIPs enclosure can be close to airtight. If the panels are precut before delivery to the building site, there is very little on-site waste. Installation by a trained crew goes quickly. A modest-sized house can be enclosed in a week's time.

Cons. Since SIPs are manufactured products, money that could be spent locally on material and labor instead leaves the community. The panels must be ordered with sufficient lead time to allow for delivery exactly when needed on the site, and they have a relatively high upfront cost. The plastic foams used contain petroleum byproducts, which to some are unacceptable materials. Waste panel material cannot easily be disassembled into components for disposal or reuse. Because of its near approach to an airtight shell, close attention must be paid to moisture control and the provision of fresh air within the living space of a SIP-enclosed house.

Technical issues. SIPs must be installed and properly sealed to create a tight shell. Installation errors can result in decay of adjacent wood materials such as window jambs, trim and siding. Most manufacturers recommend adding a ventilated ("cold") roof over a SIPs roof system, to avoid most melting and consequent ice-damming at the eaves, and to preserve the life of the roofing material. Likewise, most manufacturers recommend a capillary break between wall panels (strapping or other ventilating wrap) and the exterior finish layer. Because SIPs create a tightly sealed shell, mechanical ventilation is required.



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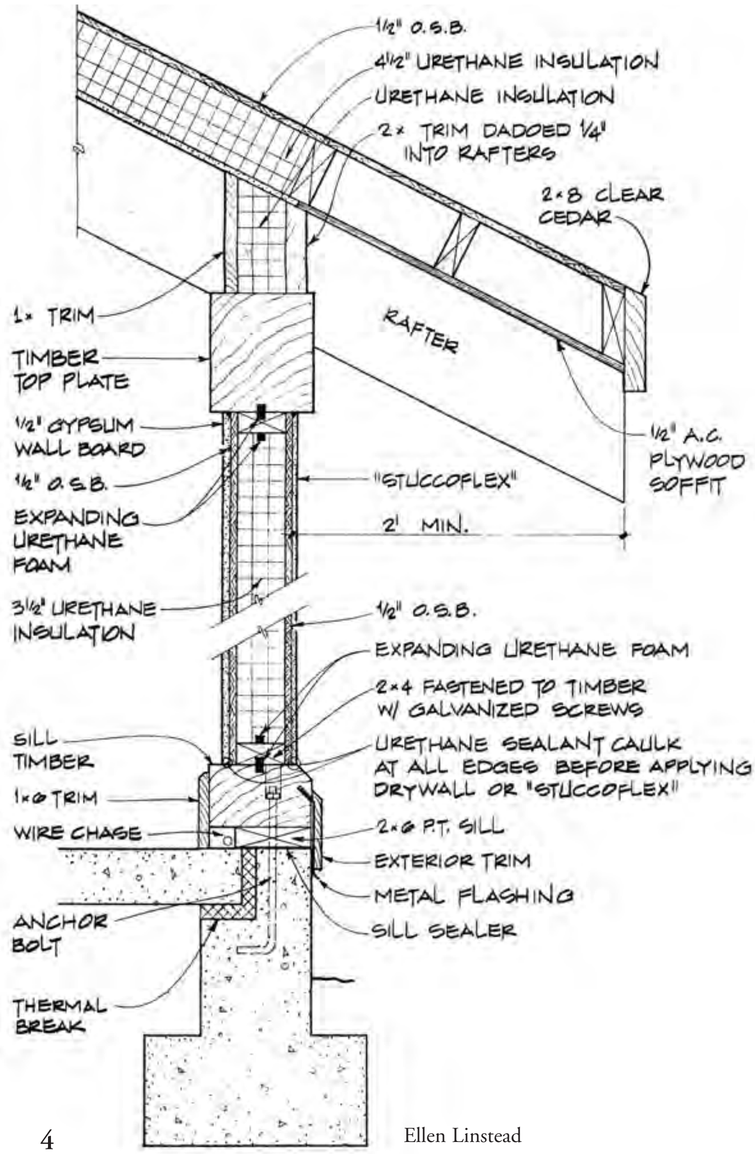

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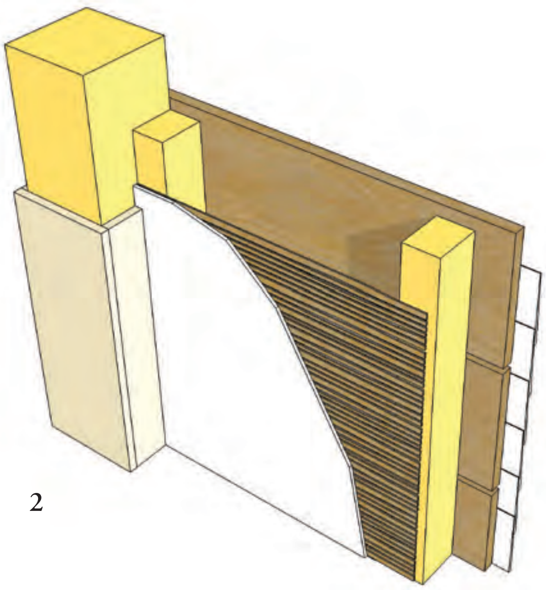
Ellen Linstead



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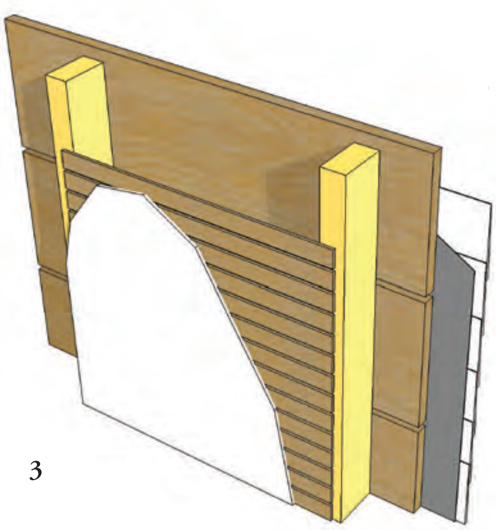

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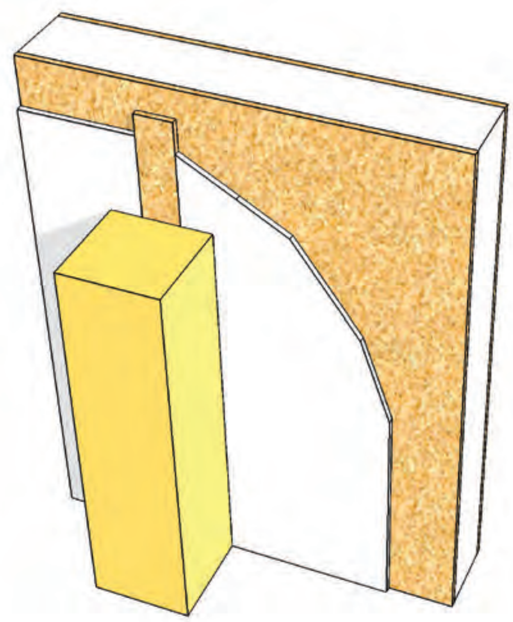


5

Renderings Andrea Warchaizer

1 Plausible 17th-century Colonial enclosure: weatherboarding outside, clay-bearing infill in wall cavity, lath and plaster inside.

2 Typical 18th-century Federal enclosure: clapboards over sheathing outside, empty wall cavity, lath and plaster inside, cased timber.

3 Typical light-frame enclosure through the 20th century: clapboards, building paper and sheathing boards outside, empty wall cavity, sawn lath and plaster inside.

4 Modern enclosure scheme with careful detailing for a Japanese-style house in a mild, wet climate. Timbers exposed inside and out.

5 Representative wall enclosure for modern American timber frame with structural insulated panel outside of timber, and plaster detail.

Light Framing A conventional nailed-together light-framed shell fully or partially surrounds the timber frame, its walls typically of 2x4 or 2x6 material and roof typically of 2x10 or 2x12 rafters or purlins, with wood sheathing to the outside (Fig. 6). Possible insulation materials between studs or rafters include fiberglass, cellulose and polyurethane spray foam. R-value varies with thickness of wall or roof cavity and insulation material. Typical wall R-values range from a minimum of R-11 for a 3½-in. fiberglass cavity to R-37 for a 5½-in. cavity sprayed with urethane foam. Buildingscience.com offers an online resource of information on conventionally framed wall assemblies, in particular high-performance wall assemblies.

Since the raw materials have less added value than SIPs, they can be less expensive overall. The standard conventional framing system, however, with extra material at corners and doubled studs and headers at openings necessary for attachment of finish and adequate support of windows and doors, is more material-intensive than SIPs. Some builders report they can install a stick-framed, insulated enclosure for less than the cost of SIPs (see page 14).

Pros. Any experienced carpenter knows how to execute a light-framed wall or roof system. For builders who have little experience working with SIPs, the enclosure system is simpler to plan and execute. For owner-builders, the 2x framing system can be installed piecemeal and the framing cavities left open and accessible for installation of electrical rough-in. Material is easily sourced at a local lumberyard, and can be locally grown or certified (FSC) lumber. Some assemblies can be dismantled for reuse of materials.

Cons. Wrapping a free-standing timber frame with a complete stick frame yields a redundant structure. Certain timbers at the perimeter can be omitted to mitigate the redundancy, but this tactic may be unacceptable. In another compromise approach, when installing the enclosure as partial infill, junctions between light and heavy timber members can be tricky, and it can be difficult to seal wall and roof assemblies. It's generally believed that the most important thermal disadvantage compared with SIPs is conduction by the numerous stick-framing members through the wall or roof.

Technical issues. A successful wall assembly depends on the interaction of all components, and careful detailing in a stick-framed wall assembly is essential to proper performance. In particular, this type of assembly requires a good understanding of building behavior to avoid unwanted air leakage and condensation problems within wall cavities.

Other Light-Framing Systems This category includes "wrap-and-strap" (overlapping layers of rigid foam and strapping), Larsen truss framing (light, site-built ladderlike assemblies similar in appearance to I-joists), and cross-strapping (vertical and horizontal 2x framing members). The idea is to mitigate some of the structural redundancies and thermal bridging of standard stick-frame enclosure systems while creating a wall cavity deep enough to allow desired levels of insulation (Fig. 7).

Framing systems in this category use small members, 2x2s or 2x3s for the chords of the Larsen trusses, 2x4s and 2x3s in the cross-strapping and wrap-and-strap systems. Material cost thus can be relatively low; assembly of wall components, however, can be extremely labor intensive. Thus Larsen truss or cross-strapped insulation framing might be a good choice for owner-builders or others who have access to a source of inexpensive labor.

Pros. These systems rely on methods familiar to most carpenters. Framing material is typically of small dimension and the design allows for great R-values with less framing material, and with reduced thermal bridging compared with stick-framed enclosures.

Cons. Framing systems can be extremely labor intensive to install. Many smaller framing members means there are many more individual components to assemble. Detailing at windows and other openings can be tricky.

Technical issues. The technical issues are much the same as for stick-framed enclosure systems, although the addition of multiple layers and connections further complicates the design of the wall assembly.

Strawbale Enclosure System Exterior walls are usually built of the smaller 14x18x36-in. strawbales (typically wheat, rice, rye, oat or barley straw) laid in a running bond, on edge for a 14-in. wall or flatwise for an 18-in. wall, with 1½-in. plaster skins on both sides (Fig. 8). Because of a difference in thermal resistance according to orientation of the fibers, both the 14-in. and 18-in. plastered walls are considered to have total R-values of 30.

Bales are fastened to the timber frame, typically exposed on the interior of the bale walls, and fastened also to the window and door framing, which lies on the exterior plane of the bale plane. Because this is a "bale wrap," where the bales wrap the frame like SIPs, an internal pinning system is not necessary as in load-bearing strawbale designs. Strawbale wall gaps are stuffed as necessary with clay-coated straw for fire and thermal performance, and then receive a base coat of clay plaster on both sides. The finish coat is typically lime plaster on the exterior and clay or lime on the interior. Strawbale walls can also be covered with exterior siding, if properly designed. The timber frame is most commonly exposed to the interior, but can also be embedded within the bale walls (much less common), with resulting thermal loss.

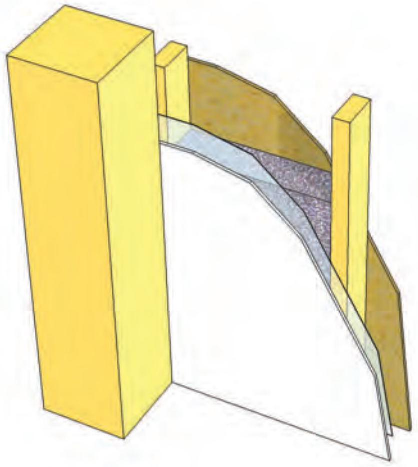
Roofs over timber frames to be enclosed with strawbales are typically insulated using light-framing methods described earlier. They must be framed and dried-in before wall bales are installed.

Depending on where materials are sourced and transportation costs, material costs may be relatively low. Strawbale wall systems are fairly labor intensive to install. If the work is to be done by the homeowner, it can be quite inexpensive; if the work is to be subcontracted out, strawbale wall systems end up being comparable in cost to other super-insulation methods. Whoever does the work, skilled technical advice is always advisable to achieve a successful, long-lasting installation.

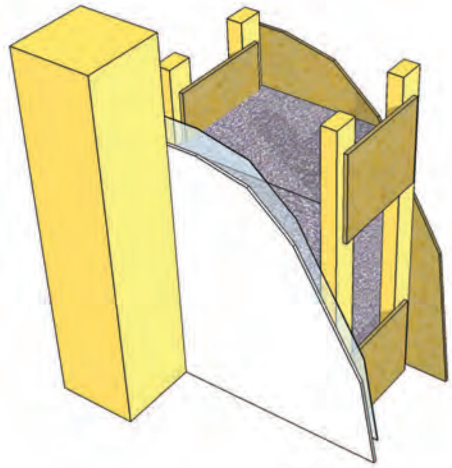
Pros. Strawbale wall systems can achieve a high R-value, with little air leakage, using a material that's ecologically low-impact and nontoxic. Materials often can be sourced locally and are low in embodied energy. If materials are indeed local, money spent on materials (and possibly labor) feeds back into the local community. Strawbales are composed of plant material and contain no petroleum products. The thick walls have slightly irregular surfaces and window openings are typically splayed to allow more light to enter. Texture and detail at deep openings produce a very different look from other enclosure systems and can exert a strong appeal.

Cons. Proposed strawbale enclosure systems may meet with resistance by local code enforcement officials, although this problem grows less common as green and natural wall systems become more mainstream. Deep walls present design challenges, particularly in detailing window and door openings, and require a broader foundation than other enclosure systems. The cost of finishing bale walls can be high according to choice of finish and labor, a case of low cost over the long term obtained for high initial cost—or, as one natural builder puts it, of "the high cost of low-cost things."

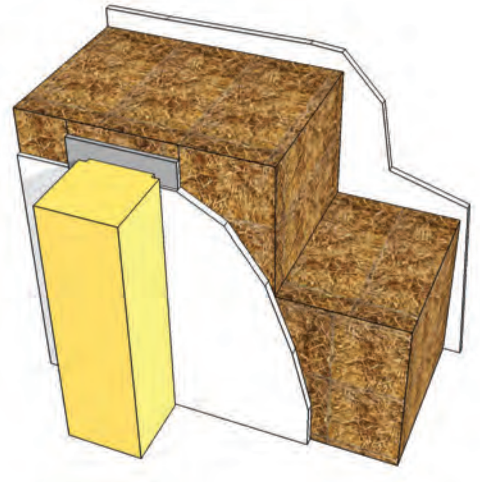
Technical issues. Proper detailing for strawbale enclosures is critical. An uninterrupted *air* (not vapor) barrier should be maintained on the interior. One strawbale builder, Ace McArleton of Frameworks Natural Building in Montpelier, Vermont (newframeworks.com), installs gasketed "air fins" at each exterior timber surface, typically of ½-in. Homasote exposed 2 to 3 in. past the timber edges, which can then be plastered over for an airtight detail (Fig. 8). McArleton also sets back the finish plaster into a rabbet on the back side of the timbers, so the plaster runs slightly behind the



6 Light-frame spray-foam insulated envelope outside timber post. Drywall covered by vapor barrier forms inside of membrane.



7 Larsen truss superinsulated envelope using web construction to minimize thermal conduction of light frame.



8 Strawbale wall applied to timber post with "air fins" and plaster detail. A straw-clay wall would not require the gasket.

frame, making for a clean visual detail. Bale walls must be raised well above grade to avoid moisture problems. Large overhangs are recommended against rain-wetting, and window and door openings must be set to the exterior of the wall, carefully flashed and detailed to avoid moisture penetration into the bales.

Straw-Clay and Woodchip-Clay Enclosure System Historically, clay-fiber was the natural enclosure system for timber frames throughout Europe and Asia, in the form of 4 to 6 in. (or thicker) infill between framing members. The modern innovation in North America is to wrap the timber frame with a 12-in.-thick monolithic clay-fiber envelope.

Walls are formed in place using a mixture of clay and straw, woodchips or other fibers and generally called called "light clay." Only enough clay slurry is added to the straw or wood fibers to completely coat the material and allow the wall to form a cohesive unit. Straw-clay mixtures are generally tamped into removable formwork, while woodchip mixtures are generally poured into lathwork left in place and plastered over. Bamboo, 2x2 sticks or saplings reinforce walls internally. Interior surfaces are finished with lime or clay plaster, exterior surfaces with plaster or wood siding. R-value estimates range from 19 to 25 for a 12-in. wall. Roofs are typically light-framed and insulated conventionally.

If sourced nearby, materials can cost relatively little. Forming of the walls can be labor intensive, and labor costs can be higher than for other enclosure systems. Some practitioners offer workshops to train professional builders and owner-builders in light-clay building techniques.

Pros. As with strawbale construction, materials can often be locally sourced and are low in embodied energy. Straw and clay contain no petroleum products. Thick exterior walls can be finished as smoothly or as "naturally" as desired; window and door openings can be splayed or left as deep recesses. As in strawbale construction, the natural look is part of the aesthetic appeal for many. Clay is a hygroscopic material and will retain and then release water vapor, modulating interior relative humidity and helping to avoid structural moisture damage. Light clay walls create a flow-through or vapor-permeable envelope. (There is no vapor barrier as such.) The clay mass combined with the straw or fiber insulation creates what Europeans refer to as *dynamic* insulation, contributing to a comfortable, healthy and energy-efficient indoor climate. According to Robert Laporte and Paula Baker-Laporte, of Econest in Tesuque, New Mexico (econest.com), that state now offers official guidelines for clay-straw construction. In North America, permitted homes have now been built in 17 states and four Canadian provinces.

Cons. Light clay is not appropriate for all climates. In fairly dry weather, walls take approximately 12 weeks to dry. In wetter climates, the drying process requires an internal heat source and fans. Climates without a dry season to allow for the curing of the clay are not appropriate for light clay enclosure systems. As a wet technique, clay fiber wall building is not practical in freezing conditions. If one of the requirements of the building project is to source materials locally, clayless sites or regions are not appropriate.

Technical issues. Light clay is not appropriate for all climates and users. It's a mass wall system that yields best R-value performance in climates where outside temperatures fluctuate significantly and daily below (or above) desired interior temperature. In climates where exterior temperatures stay steadily well below desired interior temperature for days or weeks at a time, the mass wall system can yield reduced R-value performance. It's important that vapor barriers are not used in the wall construction.

A clear understanding of the thermal performance of mass walls is necessary during design to determine if the system will perform well. (See buildinggreen.com/auth/article.cfm/1998/4/1/Thermal-Mass-and-R-value-Making-Sense-of-a-Confusing-Issue; Passive House information may also be found on this site, which archives the respected journal *Environmental Building News*.) Light-clay walls must be allowed to dry before applying finish plaster. Clay used in the fiber mix must be "sticky" enough to hold the wall together properly. Despite their name, light-clay walls are quite heavy. Clay fiber wall densities range between 20 to 50 lbs. per cu. ft.; structural members such as window and door headers need to be designed accordingly.

Summary Which enclosure system is best? The answer depends on the particulars of an individual building project and the client's and builder's wishes. All elements must be considered: location, climate, access to materials, budget, schedule and the skills of the building crew should all be weighed. Aesthetic, scientific, ecological and even ideological concerns all come into play.

No matter which enclosure system is to be used, a good understanding of building behavior is key. Does the system require an exterior air or interior vapor barrier? How are joints detailed between materials and openings in walls? How does the entire wall or roof assembly work as a system? Actual building behavior is often imperfectly understood. If we intend our timber frames to last indefinitely, we must ensure they are not destroyed by their own enclosures. —Andrea Warchaizer

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