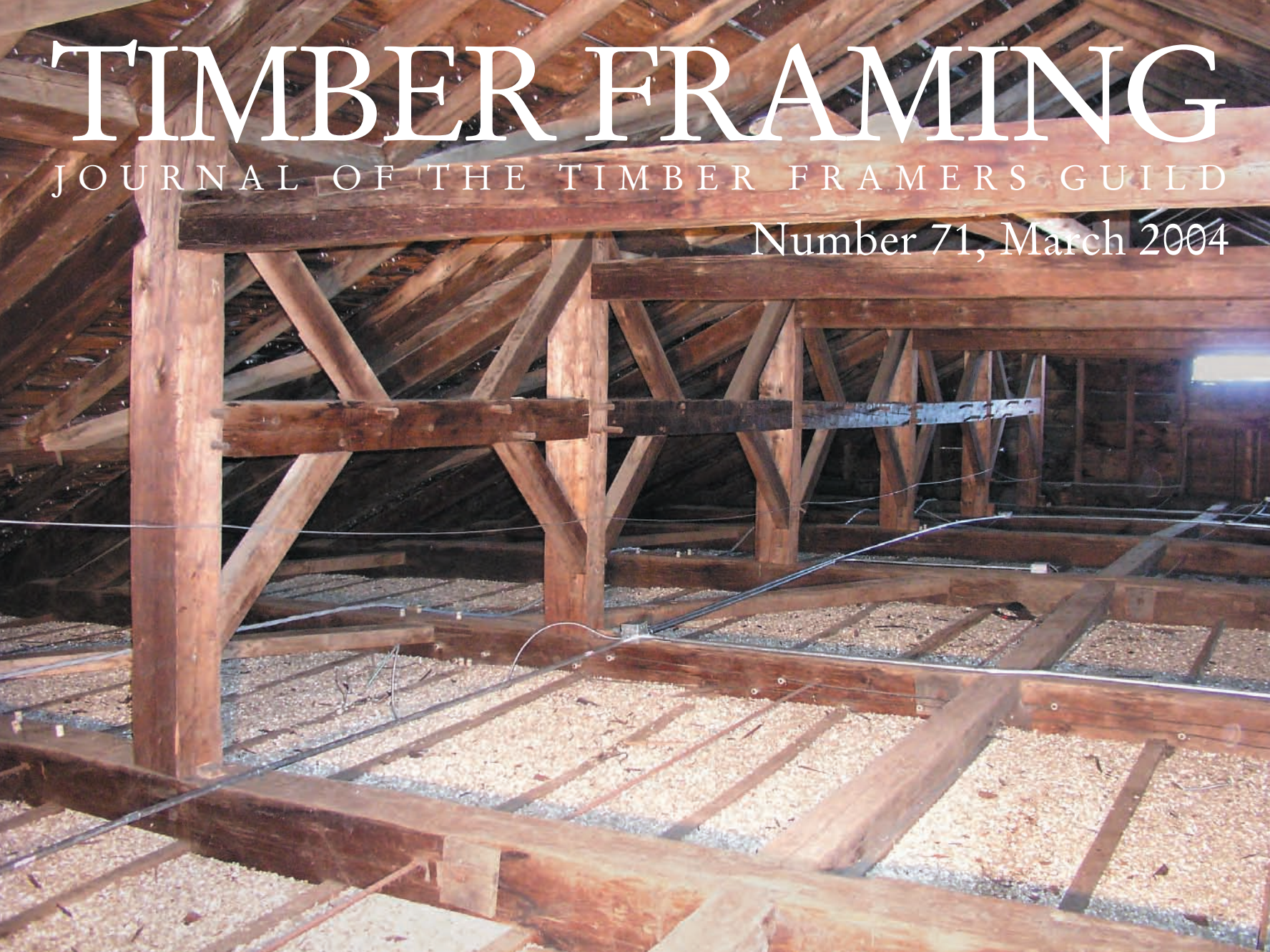


TIMBER FRAMING

JOURNAL OF THE TIMBER FRAMERS GUILD

Number 71, March 2004



Historic Queenpost Trusses

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On the cover, views of the queenpost truss framing in the attic of the Congregational Church, Peacham, Vermont, 1806. Major members are of high-quality spruce and pine; the smaller braces, remarkably profuse, are of mixed hardwoods. Floor is cambered lengthwise as well as transversely to produce a shallow dome in the ceiling below. White spots are frost. Photos by Ken Rower.

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Q & A

A Bracing Exchange

Clark Bremer, Minneapolis. Many of the plans in timber framing books show both braces in the plane of the bent on almost every interior post of a bent section. But the same is not true of braces perpendicular to the bent. They always seem to be there in exterior walls, but on the interior posts they are often omitted. Is there a reason for this? Let's say I'm planning a small house with four bents, each with three posts. That's 12 posts, where only two of them are interior. All of the possible knee braces are in place, except for the ones on these two interior posts. On these two interior posts, there are eight possible knee braces. I'd like to omit two of them, one on each post, perpendicular to the bent plane. Does that seem reasonable?

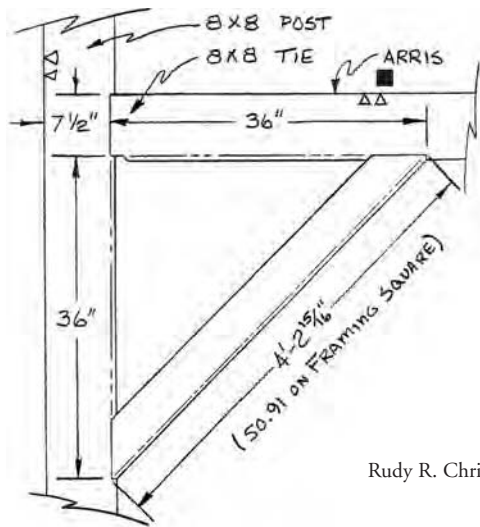
Rudy R. Christian, Burbank, Ohio. It's worth noting that braces are usually placed in pairs. The purpose of bracing is to withstand intermittent transverse loading. The stress of raising is one example. Wind and snow are others. Wind loads are hard to predict, both in strength and direction. By placing braces in opposed pairs, one works to resist the wind from one direction while the other rests. When the wind switches direction, the opposite occurs. In many early barns, braces were not pegged, which indicates the early framer's understanding of the work a brace does: resisting a force in compression. Moving a brace to keep it paired is the best solution. If you have to remove one, it's likely you should remove its mate also. Having an engineer look at the drawings is, of course, the most dependable solution.

Chris Hoppe, Athens, N.Y. Knee braces are not often found perpendicular to the bent at the center post because framing is based largely on tradition and the old hay barns did not have a timber running perpendicular to the bent at the top of the center post. (The center post usually terminated at the bottom of the tie beam.) A second reason is that in new frames the joists often run perpendicular to the bents and, to avoid conflicting joints, it's best to avoid having a joist arrive at the same location as the center post. So there may be no member to brace to, whereas at the eaves there is usually a heavy plate available. An engineer can help make an informed decision on the number and location of braces required.

Grigg Mullen, Lexington, Va. One of the main purposes of the braces is to resist racking of the frame because of wind loads. Loads applied parallel to the bents strike a much larger exterior surface area (usually including the roof) than loads perpendicular to the bents, where there's only the end wall to catch wind, and the braces in several bents to resist that load. The braces are generally not as heavily loaded in this direction, and it's possible that one set can be omitted without compromising the stability of the frame.

John Miller, Floyd, Va.

It's worth noting in any discussion of knee braces that the tension created in the joint braced is usually the limiting factor in their effectiveness. It's often the case that more braces do not make a building stronger. Any structure begins failing at its weakest link. That's where attention should be focused.



Rudy R. Christian

Idealized brace layout in a square rafter frame.

Bart Popenoe, Hood River, Oregon. For purposes of resolving the static forces acting in a timber frame through its braces, an engineer tells me that I should treat a brace as if it were a post, and divide the supported beam above into two separate beams: one beam between the post and brace, and another between the brace and the next post. But if this were true, braces would almost always be carrying huge loads and would have to be some of the largest timbers in a timber frame. Inspection tells me this is not the case. On the other hand, the authors of a book on timber engineering I just read consider braces strictly as members for resisting lateral loads—wind, seismic, etc. When they calculate for beam size, joint shear and the like, they size the beams as if the braces weren't even there. They conclude that considering the braces for weight-bearing is dubious, since beam shrinkage likely increases the distances that braces need to span and braces stay the same length. This makes much more sense to me, although I think that the truth lies somewhere between both methods.

Rudy R. Christian. My understanding of the correct use of braces is that they serve in pairs (in general). The loading condition that requires bracing, wind for example, will likely work from more than one direction. The effect is that braces in pairs actually function as one brace. When a lateral load is applied to the system, one half of the brace pair goes into compression while the other goes on vacation. When this condition occurs, the brace indeed can assume a tremendous load, but in true compression, allowing a small-section member to do a great deal of work. The timber that accepts the load from the brace, however, normally ends up dealing with bending and shear forces. For this reason I have been taught to think of braces more as pry bars than posts.

My experience with barn frames has shown that braces do indeed resist static loading as well as dynamic. Braced purlins are a good example. The braces are typically opposed at the purlin posts, which allows them to work directly against one another (no bending), and the brace legs are appropriately increased in length with the span of the purlin. Of course, this pairing doesn't work at the gable end. For load study, I do consider it practical to reduce the span of a member by one brace leg if the member is braced at both ends. If it's only braced on one end, as in your example, the span cannot be reduced. Conversely, if the brace is pegged, you may want to increase the loading factors for the beam since, from time to time, the brace may end up in tension.

Bart Popenoe. I'm engineering the joinery for a timber-framed garage-loft design, 24x28, with a principal rafter-common purlin 12/12 roof and kingpost truss bents with a 24-in. kneewall. I've

been using the Guild's *Joinery & Design Workbook* as well as other texts to educate myself. Everything was going great until I started trying to resolve the forces being carried by the knee braces.

Specifically, my design uses a strut between the principal rafter and tie beam to reduce the roof load at the end posts. This strut, in worst case scenario on the center bent, transfers 8721 lbs. of roof load to the 8x10 tie beam at 40 in. from its end and offset 8 in. from the knee brace below. I have been told that the entire 8721 lbs. should be transferred directly to the brace, which sounds to me like it may cause some problems with bending stresses on the post, as well as requiring one honkin' big brace. I've seen this strut solution used on many larger frames without abnormally large braces, so I think that this approach must be in error.

Brian Wormington, Great Barrington, Mass. Don't worry. I think the problem of how much vertical load the brace carries is more a function of the tightness of the joinery than of the actual geometry. If the brace is a little undersized, it will carry nothing until there is enough bending in the cross beam to load it and then start to share the load with the end support (vice versa for an oversized brace). Wood is very strong in compression parallel to the grain (from the *NDS [National Design Specification]* tables, the value is 1500-2000 psi for most species). If you assume 1500 psi, you'd only need 5.8 sq. in. of cross-section to support your entire load. A 3x5 brace would have almost three times this cross-section. Because of the load sharing effect, I think even an undersized brace would never fail.

Bart Popenoe. Hmmm . . . My *NDS* tables show design values for compression parallel to the grain generally in the 350-900 psi range. For my species, Ponderosa pine, the allowable compression parallel to the grain is 700 psi for #1 posts, and 325 psi for #2 posts, which will place me closer to allowable limits if I use a 4x6 brace ($8721 \div 24 \text{ sq. in.} = 364 \text{ psi}$). Am I still missing something here?

Brian Wormington. What I see on page 16, *NDS Revised Supplement to the Revised 1991 Edition: Dry service conditions, sawn lumber, 2 in. to 4 in. thick.* Bearing design values parallel to the grain F_g : Ponderosa pine = 1580 psi.

[Editor's Note: Bart Popenoe cites a design value for compression parallel to the grain taken axially (F_c) whereas Brian Wormington cites a design value for compression parallel to the grain taken in bearing (F_g). The first is a measure of the strength of the entire member, and so controls member size, the second a measure of bearing ability that allows calculation of necessary cross-sectional area at the joint.]

Rudy R. Christian. No input on the numbers, but I am a little puzzled by the location of the bracing in your truss. When we design a kingpost truss, the braces join the midspan of the rafters to the base of the kingpost, not to the lower chord. The typical layout is to run the brace parallel to the opposing roof pitch. The trick is to allow sufficient wood on the kingpost beyond the connection to resist shear. This arrangement keeps the force in the brace from pushing down on the bottom chord of the truss and is typical of historic trusses. In my opinion, it's the correct layout and allows the chord to work purely in tension (other than to support its own weight). There are even old kingpost trusses, again using struts between the rafters and the kingpost, that have no connection at all between the kingpost and the bottom chord. These and other historic trusses seem to only get in trouble when a ceiling load is added. Which is why I'm a little concerned about your bracing setup.

This exchange is excerpted from discussions in the Ask the Experts section at the Guild Website (tfguild.org/subbcgibin/ultimatebb.cgi).

TIMBER FRAMING FOR BEGINNERS

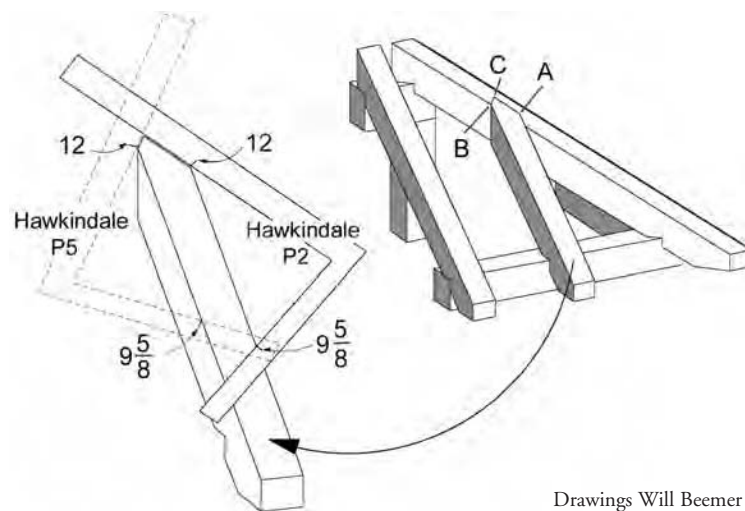
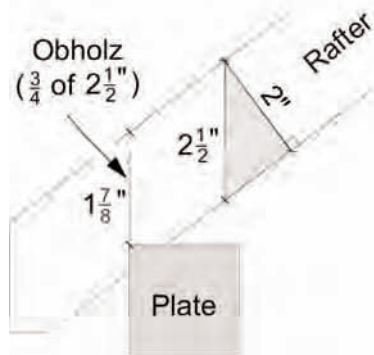
VIII. When Roofs Collide 2

WE closed the first part of this examination of compound roofs with the challenge for the reader to draw the jack rafter and its housing in the hip. It would be helpful for the reader to have the first article from TF 70 at hand as we continue with the exercise.

In Fig. 2 (facing page) we see the solution to the challenge. The jack rafter is identical to a common rafter in slope and at the tail and lower birdsmouth, but its length has been shortened at the top end by its intersection with the hip. Hence, we can use the common rafter elevation triangle to find its true length by projecting lines down, perpendicular to the fold line, from the intersection of the jack and the hip in the plan view. (All housing depths in our examples will be $\frac{1}{2}$ in.). Three plumb lines define the cuts at the top of the jack: one beginning on the shortest (downhill side) at *A*, another beginning where the longer side of the jack enters the hip at *B* and a third beginning at the longest point of the jack where the two end cuts meet *C*. The stock for the jack rafter can be laid on this part of the drawing and the measurements transferred.

We are lacking the roof surface (top view) of the jack but, since we know where to lay out the plumb line at *A* on the side of the jack (plumb in this case being 9:12), we can draw the angle from *A* to *C* using Hawkindale P2 (see Fig. 17 in the previous article), or using the fifth row down in the table on the rafter square, labeled Side Cut of Jacks. The number under the 9 column in this row reads $9\frac{5}{8}$. Holding 12 on the body of the square aligned with point *A* on the top surface of the rafter, and $9\frac{5}{8}$ on the tongue on the same arris, the body will define the correct angle for *A-C* (Fig. 1).

Author's note. In the first article (see TF 70), we learned how to generate true, full-scale views of compound roof timbers using developed drawing techniques, and proposed eventually cutting a model. Before we continue, however, I must correct the depth of the common rafter in the model, given erroneously in Figs. 12 and 14 of the first article. The depth of the common rafter should be 2 in. (not $2\frac{1}{2}$ in.), for a plumb height of $2\frac{1}{2}$ inches, resulting in an obholz of $1\frac{7}{8}$ in. ($\frac{3}{4}$ of $2\frac{1}{2}$ in.). For the model to work out correctly, this measurement, as shown in the figure, should be substituted for the obholz of $2\frac{11}{32}$ in. given throughout the rest of the first article. And, regarding the German term obholz, I said that "in English we don't have a word for this part of the rafter." But reader William Dillon of Chilmark, Mass., kindly points out that in Marshall Gross's book, Roof Framing, the term HAP, for height above plate, is used to mean that very part of the rafter, while union carpenters reportedly use the term "meat cut."



Drawings Will Beemer

Fig. 1. Laying out the cuts at the top of the jack, called "Side Cut of Jacks" on the table of values engraved in a framing square.

The other line *B-C* is the same angle struck from the other side in the case of regular roofs, so you can flip the square over and lay it out the same way. However, this short nose, which helps define the bearing surface of the jack in the housing, is actually made by Hawkindale P5, which in an irregular roof would be different from P2. Nor would you be able to use the framing square tables if the pitches were different. Looking at our Hawkindale table in the last article verifies that P2 and P5 are the same angle in our example.

To lay out the housing on the hip rafter, lines from points defining it in the plan view are projected into the hip elevation view, again staying perpendicular to the fold line. Note that the housing, $\frac{1}{2}$ in. deep in plan, is cut with the load-bearing (uphill) side at 90 degrees to the face of the timber, while the downhill side follows the angle formed by the jack and hip (45 degrees in plan). The 90-degree configuration is usual for housings and avoids difficult undercutting and possible splitting out under load.

Using dividers, we can take the height of the plumb cut on the jack (lower part of Fig. 2) and transfer it to the hip elevation, to establish the bottom of the housing. In this case, the jack is deeper than the depth of the hip where it enters and thus the housing is cut right through. (Remember that the hip side has been reduced in height by the backing angle.) In the upper part of Fig. 2, we can see that transferring the lines from the hip elevation to the bottom view of the hip shows the layout on the underside, with the depth remaining at $\frac{1}{2}$ in. in this view.

Drawing roof members in their entirety at full scale is an accurate method but requires space, time and careful technique. With a little simple math we can eliminate a lot of the space taken up by the stretch of timber where there is no joinery and use a technique to draw only the joinery full scale. Here we will introduce two powerful tools to make complex roof joinery simpler: mathematical multipliers to locate the joinery and the kernel to draw the joinery.

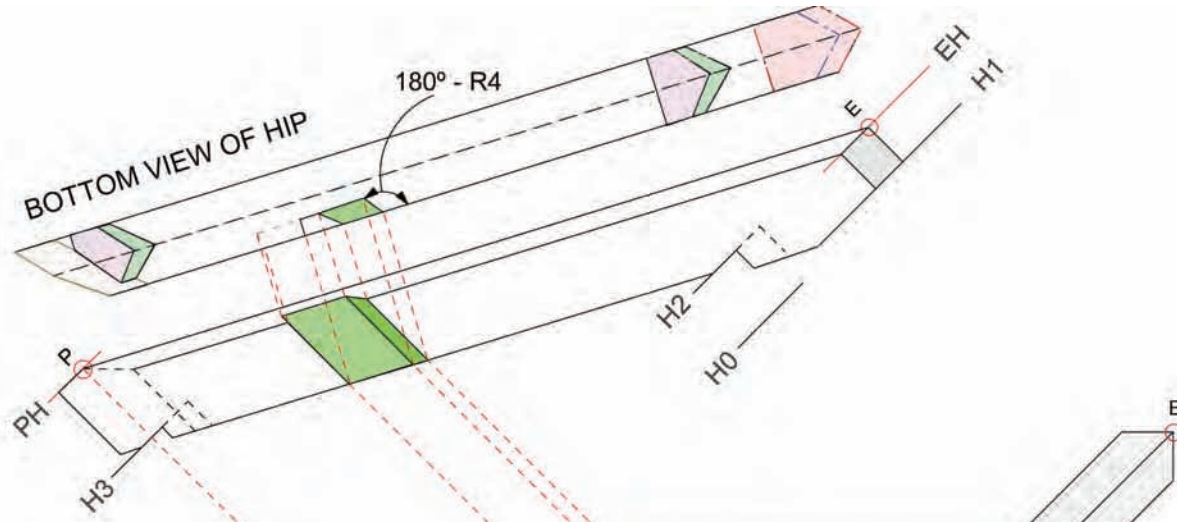
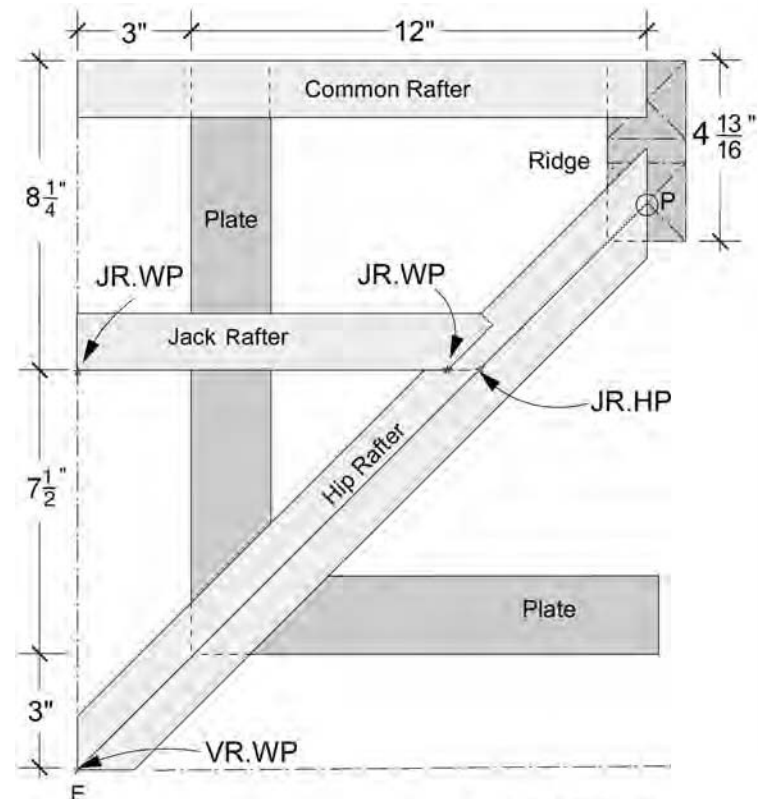


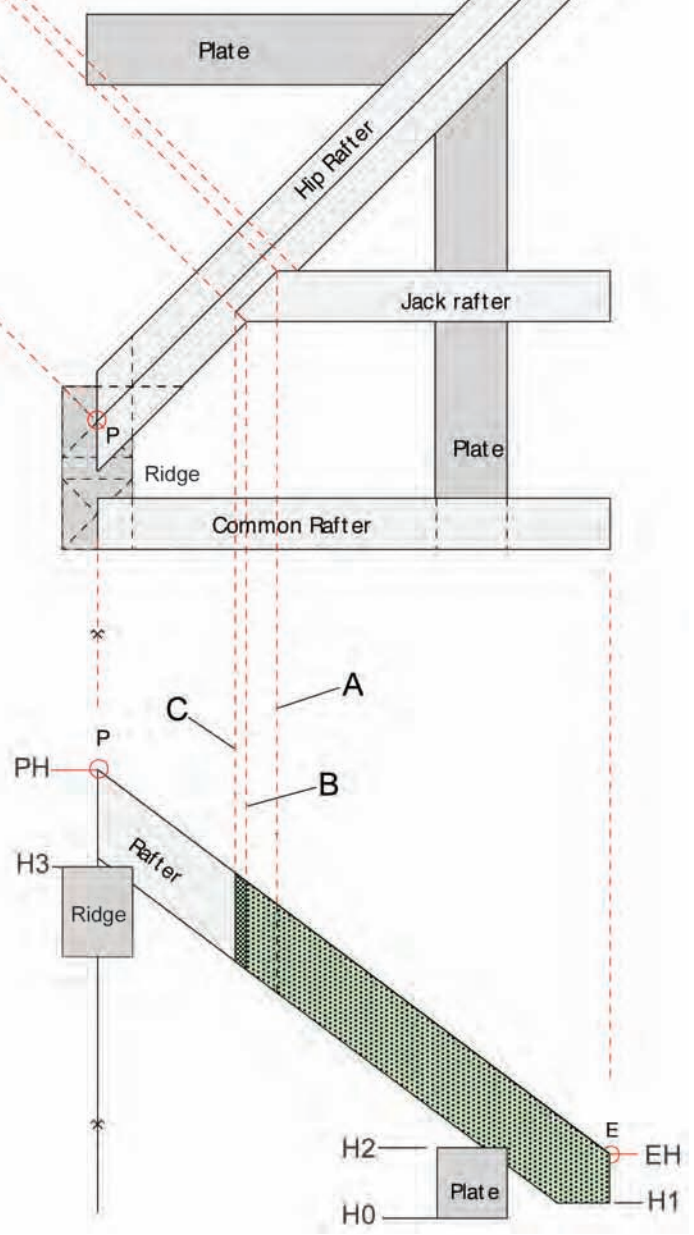
Fig. 2. Development of the jack rafter and its housing in the hip.

WORKING POINTS. If we know precisely where on the uncut timber the joinery occurs, then we can use the framing square, a full-scale drawing of the joint or the Hawkindale angles to lay out the joint. Locating the joinery is a matter of finding working points along the length of the timber. The working point, or WP, of a joint is located on either the cen-



DIMENSIONS OF PIECES		WIDTH	DEPTH
PLATE:		2"	2"
RIDGE:		2"	2 1/2"
POSTS:		2"	2"
RAFTERS:		1 1/2"	2"
HIP RAFTER:		2"	2 1/2"

Fig. 3. Establishing the Working Points for the layout of the joints.



terline (if on a hip, valley or ridge) or the arris (edge) of a reference face for a jack rafter or purlin. Once the working points have been found and marked for each joint along the timber, we can take our bevel gauges and square and lay out the joinery from those points. Working points for various joints in our hip model are shown in Fig. 3.

We find the working points by remembering our mantra, *Stay in plan as long as you can*. On the plan view, we take the run dimension to the intersection we are considering and multiply it by the

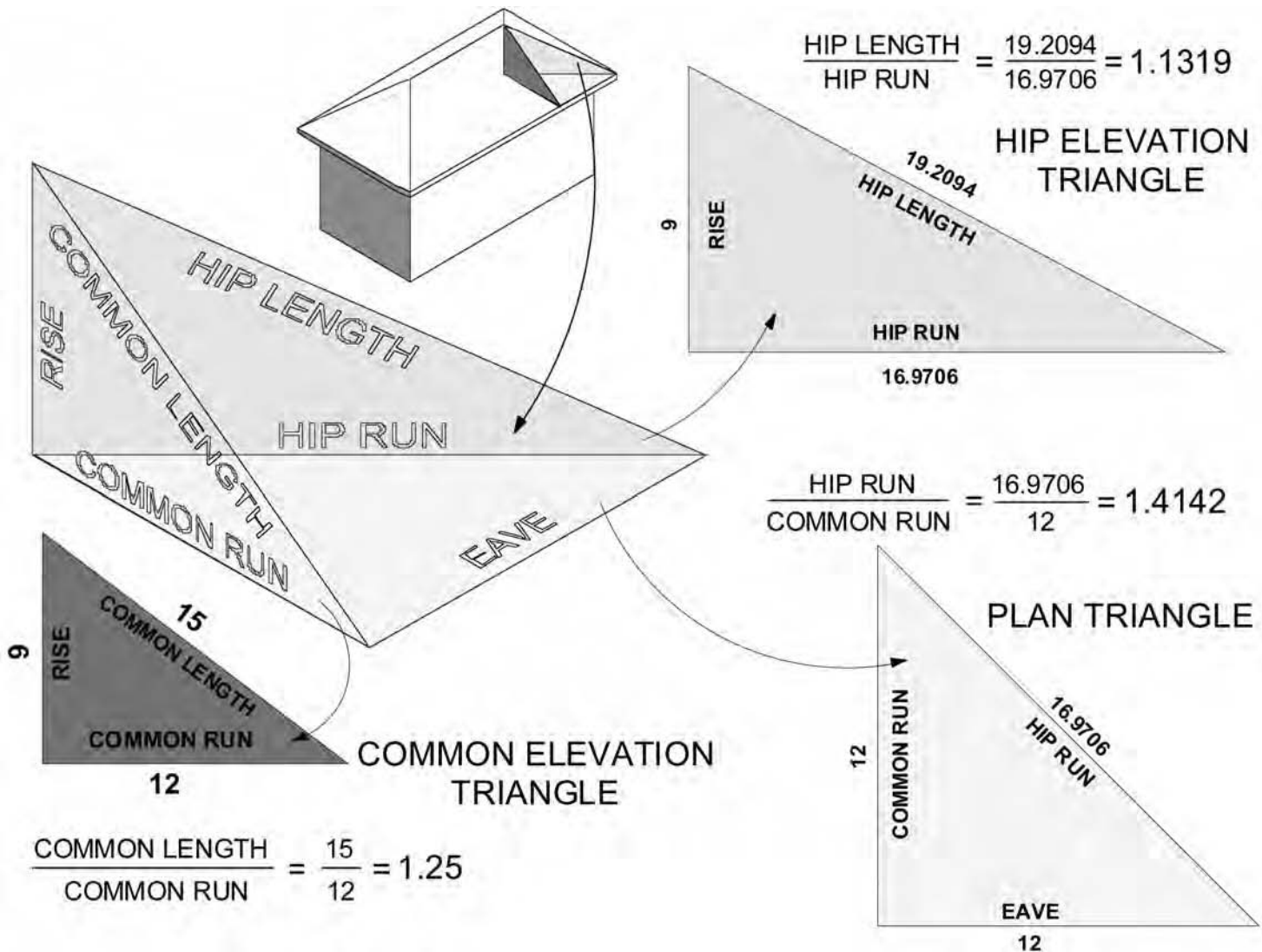


Fig. 4. Calculation of the multipliers, ratios determined by the given roof angles and lengths.

ratio of the unit length to the unit run of the piece to find the WP along its length. (Remember, *run* always refers to a plan dimension and *length* to a dimension along the piece on the roof surface.) This proportional relationship within the similar triangles is termed a *multiplier*—one number we can use to multiply by the common run to find the location of any WP. There are usually just four ratios we need for the entire roof:

1. The common length to common run.
2. The hip run to common run.
3. The hip length to hip run.
4. The hip length to common run (the most useful).

As we'll see, multipliers are powerful tools not only for finding the working points, but also for finding dimensions within the joint layout. To find the ratio of the common length to common run, look at the common rafter elevation triangle in Fig. 4. For every common unit run of 12, the common unit length is 15; this ratio can be expressed as 15 divided by 12, which yields the multiplier 1.25. Now that we have this number, any dimensions along common or jack rafters in plan can be multiplied by 1.25 to find the plumb point along the common length. File that number—1.25—for future use.

Next, continuing with Fig. 4, look at the plan triangle showing the relationship of the hip run to the common run. For every common unit run of 12, the common hip run is 16.9706; the ratio of 16.9706 divided by 12 is 1.4142. This multiplier will also prove very useful. I carry out to four decimal points here for accuracy since these multipliers will be used for lengths, where a slight error could show up on long pieces.

For the hip length to hip run ratio, we need first to find out the hip length. Using the Pythagorean Theorem, $a^2 + b^2 = c^2$:

$$\text{Hip Run}^2 + \text{Hip Rise}^2 = \text{Hip Length}^2$$

$$16.9706^2 + 9^2 = 369.0013.$$

The square root of 369.0013 is 19.2094. Hence the ratio of hip length to hip run is 19.2094 divided by 16.9706, or 1.1319.

The final multiplier we need is the ratio of hip length to common run, and this will prove most valuable. The hip length 19.2094 divided by the common run 12 equals 1.6008 (Fig. 5). If we know the run of the jack rafter, multiplying that distance by 1.6008 will give us its intersection with the hip centerline.

Here's another powerful advantage of the multipliers: once we know them for this roof, we can use them for any 9 pitch regular roof, hip or valley, and never have to go back to Pythagoras again. These four are all we'll need. Let's put them to use.

To figure our common rafter length, we look for the first multiplier, 1.25, the ratio of common length to common run. In Fig. 3 (previous page), since the common run is 15 in. (including the overhang at the eave), the overall length of the common rafter is 15 in. x 1.25, or 18.75 in. Done. How about the hip length? Since we have a ratio for hip length to common run, take 15 in. and multiply it by 1.6008 to get 24.012 in., the hip length from *E* to *P*. Simple.

Now it gets really cool. We want to find next the jack rafter hip point, or JR.HP, the point on the centerline of the hip where the jack rafter's projected short (or downhill) side intersects (Fig. 3). On valleys and hips, we locate any working point on the centerline because it won't change position if the width of the timber varies

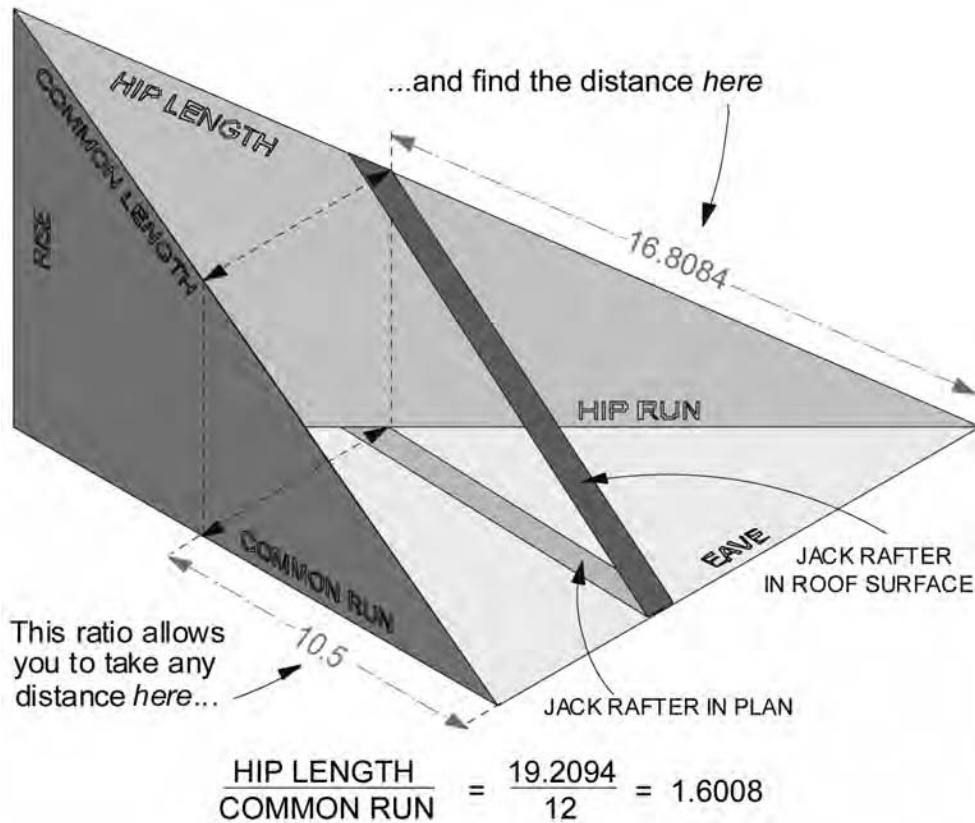


Fig. 5. The ratio of hip length to common run allows quick calculation of distances on the hip.

slightly. Regardless of width, a line projected back out at the proper angle will still exit the side at the right point to accept the jack. (The angle on the unbacked hip would be Hawkendale R4, or the Side Cut of a Hip or Valley from the rafter square table, as shown in the last article.) Once you have a point on the arris of the hip, you can continue with the layout of the jack housing on the side.

To find JR.HP, take the run of the jack from eave to centerline of hip and multiply it by our hip length to common run ratio. (A jack has the same slope and properties as a common rafter; it's just shorter.) As shown in Fig. 3 (page 5), since the hip runs at 45 degrees in plan, the jack's run to the centerline of the hip is the same as the measurement along the eave from the reference side of the jack out to the end of the hip, or 10½ in. Now, 10½ in. times our multiplier of 1.6008 equals the location of the JR.HP on the roof surface: 16.8084 inches measured up the centerline from the VR.WP at the foot of the hip rafter.

Before we get to the jack housing layout, let's also find the working point on the jack rafter, or JR.WP. One JR.WP is at the eave end and the other is at the end of the jack in the housing in the hip, on the short (downhill) side. We use the short side as our reference because the jack may not be of consistent width and we want it to enter the housing at a fixed seat dimension. The uphill side of the jack can then freely vary in width (Fig. 6).

To find the JR.WP at the housing end, take the run of the jack and multiply it times 1.25, our common length to common run ratio. The run of the jack in this case, however, is less than 10½ in. since the jack stops at the back of the housing. Staying in plan as long as we can, we subtract the difference in run from the centerline to the back of the housing (Fig. 6). We can see that since the housing is ½ in. deep and the hip 2 in. wide, that leaves ½ in. from the centerline to the back of the housing, measured square to the face of the hip. The run we want to subtract forms the hypotenuse of an isosceles right triangle with ½-in. sides. Since this triangle has the same proportions as our plan triangle, we can use our ratio of hip run to common run to find that ½ in. times 1.4142 equals .7071 in., or slightly more than 11/16 in. You can see that the mul-

tipliers will be used a number of times during the process of finding working points. Try to do as much adding and subtracting as possible in the plan view. Stay in decimals, too, until after you make the final multiplication to get up into the roof surface dimensions. So, we subtract .7071 in. from 10.5 in. to get 9.7929 as the actual run of the jack along its short side from eave to back of housing. Then we multiply that times 1.25 to get the length of the jack, which is 12.2411 in., or slightly less than 12¼ in.

Now that we have located the working points, we can go ahead and mark them on the timbers, and then lay out the joinery.

THE KERNEL. The developed drawing technique we used in the first article relies less on math than the multipliers, and so may be more appealing for some workers. But it requires lots of room and time because the whole timber is being drawn. The multipliers allow us to locate the joinery without drawing and are thus much quicker to use, but we still need to know what the joint looks like. The roof kernel seen in Figs. 4 and 5 is a great tool to develop the joinery on paper, full-scale,

which can then be transferred to the timber using the established working points as points of departure.

The kernel is representative of the roof system by showing proportional adjacent triangles that contain all of the necessary information about the angles we seek. It can (figuratively) be pulled out from the roof and looked at from all sides. Because each of the triangles shares an edge with another (identical to the fold line we used in our previous exercises), the kernel can be constructed on paper and unfolded to lay flat. Drawn at any convenient scale, it contains no information about the lengths of pieces, but we can

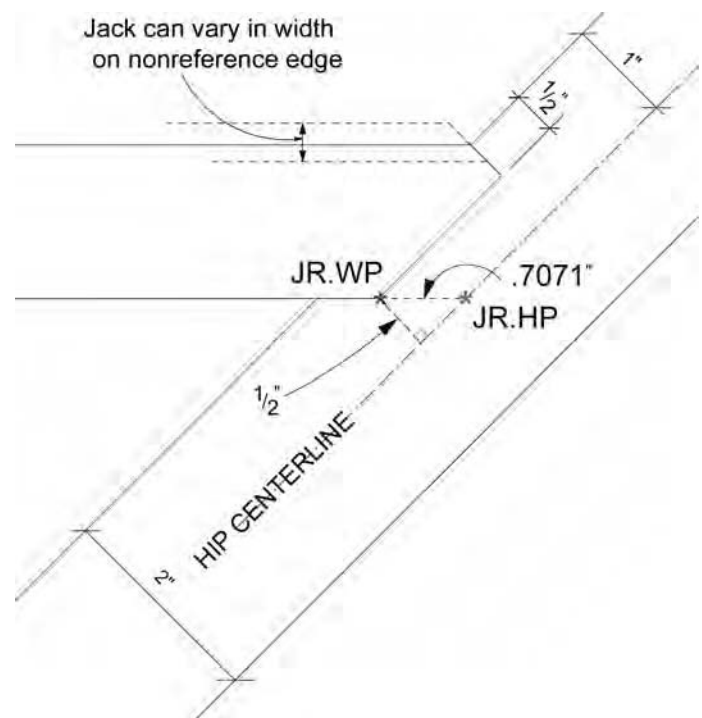


Fig. 6. Working point for jack is at short side to allow for variation in jack width without affecting length of housing seat.

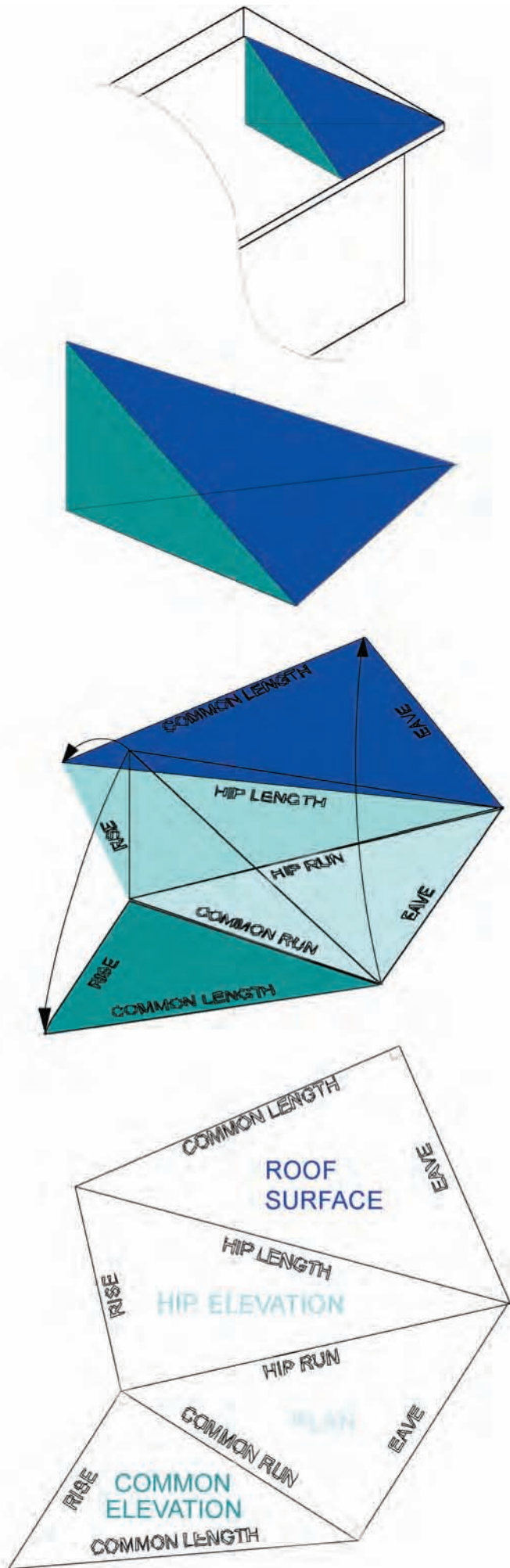


Fig. 7. The roof kernel removed and unfolded onto a flat plane.

obtain that with our multipliers. Just like the multipliers, the kernel for a 9:12 pitch regular roof can be used for any roof of that pitch, regardless of size. (The kernel for a valley roof, however, will be constructed slightly differently from a hip kernel; we'll see that in the next article.) Fig. 7 shows the kernel for our hip exercise removed and unfolded.

It contains the three triangles we used for the multipliers—the plan, common elevation and hip elevation—to which we have added the roof surface triangle. Once the kernel for a 9:12 hip is drawn, we can then draw the end of our jack rafter on it at full scale. (We can also lay out the jacks on both sides and see what the joinery looks like without having to use Hawkindales or other math, but we will save that exercise for next time.) Notice that if we had an irregular roof, with the pitch of the adjacent roof unequal to the pitch of the main roof, we would have to construct two kernels, one for each side of the hip or valley.

DRAWING THE KERNEL (Fig. 8, facing page). Let's construct a 9:12 hip kernel for a regular pitch roof using the script below. If you're doing this at home, a piece of 27x34 flip-chart paper works well at the 1:1 scale we'll be using. You can also work at a smaller scale, but the drawing of the jack may get crowded if the sheet is much smaller than recommended.

1. **PLAN TRIANGLE.** Secure the paper with the short dimension (27 in.) running away from you and start at a point 1 in. up from the bottom edge and 10 in. in from the left edge. From this starting point, run a level line to the right 12 in. and a plumb line 12 in. to form a 90-degree angle. The first line represents the eave line and the second the common run. Connect the two end points to show the hip run. Remember, these lengths have nothing to do with the size of our roof. We are simply constructing a 9:12 model at 1 in. to 1 in. scale, with a rise of 9 and a run of 12.

2. **COMMON RAFTER ELEVATION TRIANGLE.** From the farther end of the plumb line just drawn, extend a level line to the left at 90 degrees to the common run. This line represents the common rise of 9, so make it 9 in. long. Join its endpoint, which represents the peak of the roof, to the eave line to make the common length (it should be 15 in.).

3. **HIP RAFTER ELEVATION TRIANGLE.** Returning to the starting point of step 2 above, extend a line up and to the right at 90 degrees to the hip run. Since the rise of the hip is the same as the common, this line too should be 9 in. long. Connect the far end, which represents the roof peak, to the foot of the hip run to form the hip length, which should be $19\frac{3}{16}$ in.

4. **ROOF SURFACE TRIANGLE.** With dividers set to the eave length and centered on the hip foot, swing an arc. Reset the dividers to the common length and swing a second arc, centered on the hip peak, to intersect the first arc. Now connect the dots to outline the roof surface. Note that the eave and the common meet at right angles.

Label all lines and surfaces. Try to keep the labels outside of the drawing, or make them small, light or of a different color so as not to interfere with the rest of the drawing. If you construct this kernel out of card stock or posterboard, it's revealing to cut it out, score the fold lines and fold it up into a three-dimensional mass model of the roof. Now we're ready to draw the jack end, or any other timber joinery in the roof.

DRAWING THE JACK (Fig. 9, facing page). Joinery is drawn next at full scale, using the actual dimensions of the timbers, so that the results can be directly transferred to the timber. If done at a different scale, extreme care must be taken to adjust every transferred dimension. Use the following script to lay out the jack rafter to hip connection. Colored pencils work well to distinguish fold lines from construction and projection lines.

Fig. 8. Drawing the roof kernel.

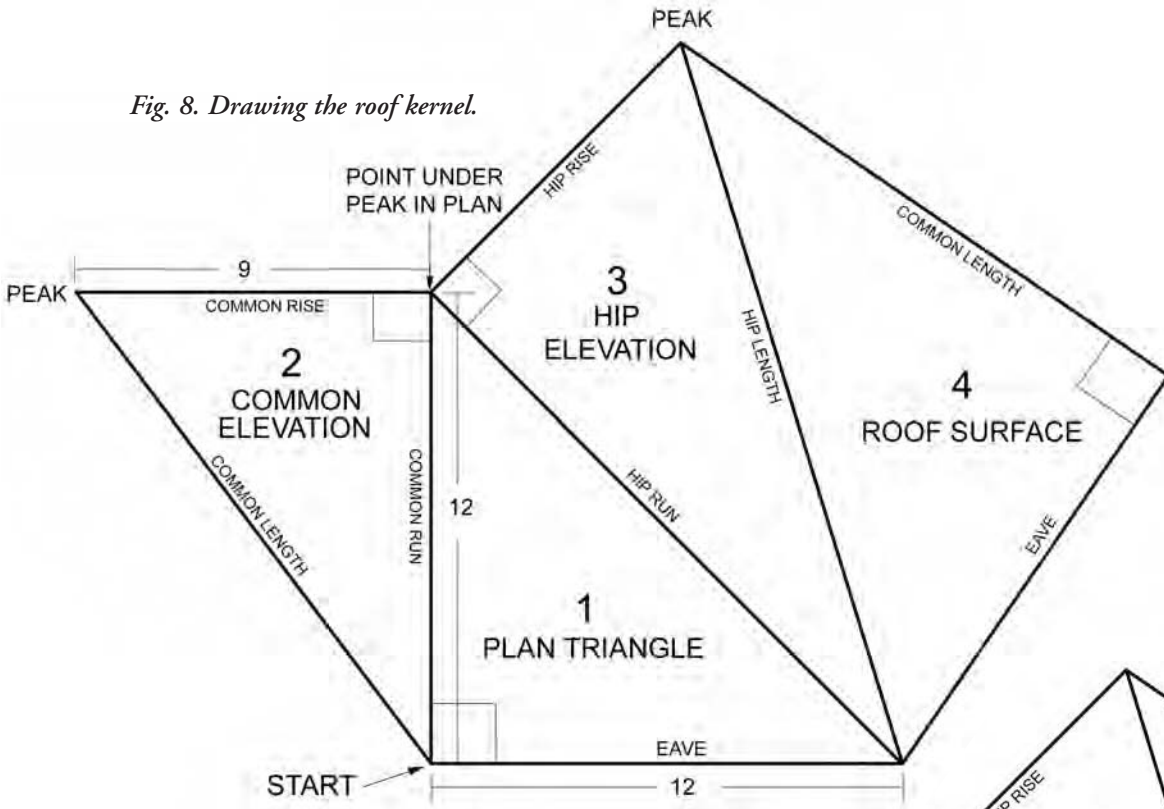
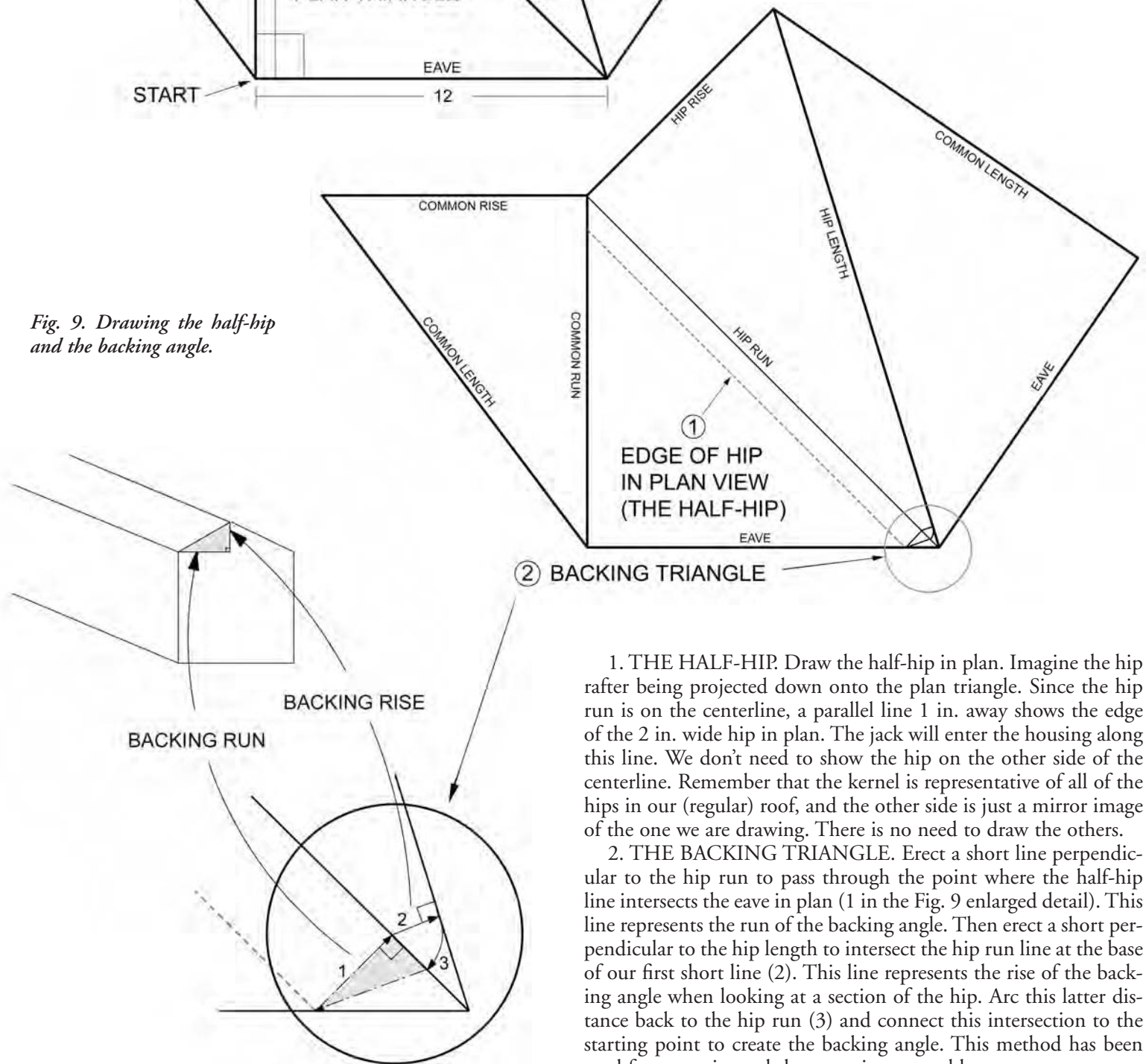


Fig. 9. Drawing the half-hip and the backing angle.



1. THE HALF-HIP. Draw the half-hip in plan. Imagine the hip rafter being projected down onto the plan triangle. Since the hip run is on the centerline, a parallel line 1 in. away shows the edge of the 2 in. wide hip in plan. The jack will enter the housing along this line. We don't need to show the hip on the other side of the centerline. Remember that the kernel is representative of all of the hips in our (regular) roof, and the other side is just a mirror image of the one we are drawing. There is no need to draw the others.

2. THE BACKING TRIANGLE. Erect a short line perpendicular to the hip run to pass through the point where the half-hip line intersects the eave in plan (1 in the Fig. 9 enlarged detail). This line represents the run of the backing angle. Then erect a short perpendicular to the hip length to intersect the hip run line at the base of our first short line (2). This line represents the rise of the backing angle when looking at a section of the hip. Arc this latter distance back to the hip run (3) and connect this intersection to the starting point to create the backing angle. This method has been used for centuries and shows up in many old carpentry texts.

3. HIP SECTION, BACKING SURFACE, JACK ELEVATION (Fig. 10).

We can use the resulting information (backing run, rise and length) to set out, respectively, the backing of the hip in elevation and the backing surface in the roof plane. In the hip elevation triangle, draw a line parallel to the hip length line from the point *X* where the backing rise springs from the hip run line. This line represents the arris of the backing on the side of the hip. Somewhere near the peak in the same triangle, draw a rotated section of our 2-in. hip perpendicular to the hip length line. Connect a center point on the hip length line to flanking points 1 in. away but on the backing arris line. Then drop two lines from these points, perpendicular to the backing arris, to a line offset by the hip depth (2½ in.); these three lines complete the sides and bottom edge of the hip section. Now that we have the hip section drawn, we can take the length of the backing (remember, length refers to the hypotenuse of a rise and run triangle) and offset a line by that distance parallel to the hip length in the roof surface triangle. This shows us what the half-hip looks like when viewed perpendicular to the roof surface, and it's slightly wider than the half-hip in plan.

We can next show the profile of the jack as it enters the hip section. Project a line from the top surface of the hip section to represent the top surface of the jack. Offset a parallel line 2 in. to represent the bottom edge of the jack; because we are looking parallel to the roof surface, the jack appears in its true depth. Last, draw the housing (½ in. deep measured from the side of the hip) in the hip section, and you'll see where the bottom of the jack intersects the back of the housing at the bottom of the hip, indicating this is a through housing from top to bottom.

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4. THE JACK AND ITS HOUSED END IN THE HIP (Fig. 11).

Draw the run of the jack rafter in the plan triangle, arbitrarily starting anywhere near the center of the eave line and bringing up parallel lines 1½ in. apart until they intersect the side of the hip (the half-hip line we drew in Step 1). Since we're only working with angles at the end of the jack, and not with length of the piece, it doesn't matter where along the eave these lines spring from. Offset a parallel line for the back of the housing ½ in. away from the half-hip line, and extend the shorter edge of the jack to the back of the housing. The long edge of the jack takes a 90-degree turn to form the nose for the bearing surface of the housing. Remember that housings and mortises rarely form acutely angled abutments, which would be required if you continued the long edge of the jack straight to the back of the housing. The bearing surface of the housing lies instead square to the face of the hip, easy to lay out and cut, as well as highly resistant to load. Meanwhile, the unloaded edge of the jack, a nonbearing surface, forms an obtuse angle where it enters the hip, again easy to cut.

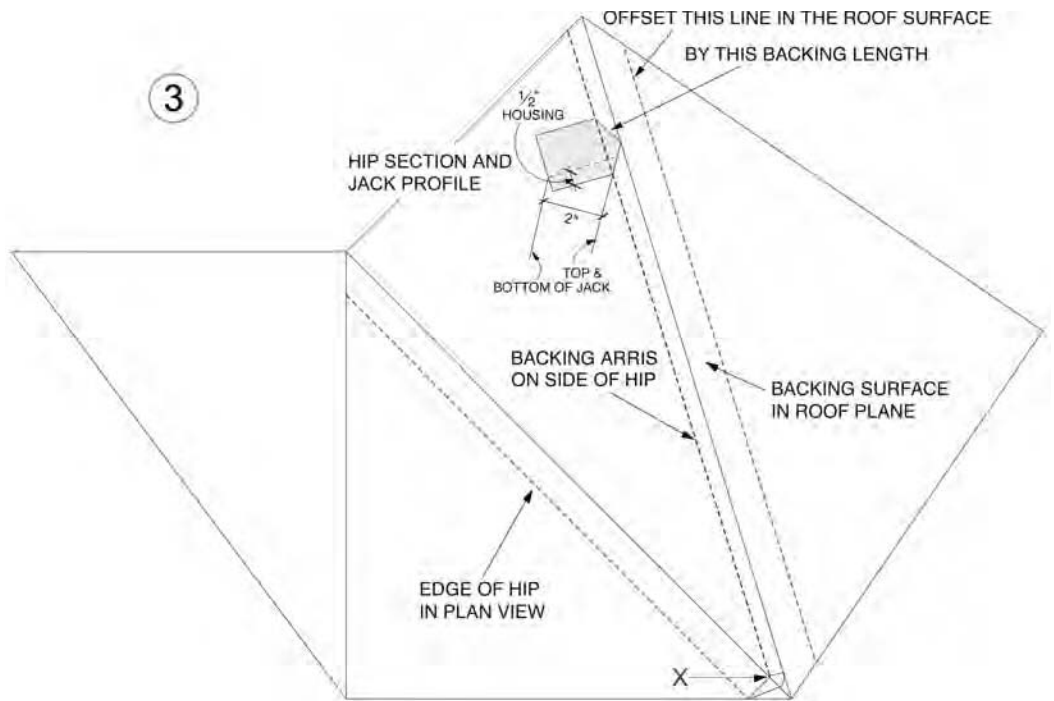


Fig. 10. Drawing the hip section, backing surface and jack elevation.

From the plan triangle, now strike lines from the intersection of the jack and hip perpendicular to and across the common rafter run line into the common rafter elevation triangle and on to intersect the common rafter length line. Offset a parallel line 2 in. from the common length line to represent the bottom edge of the jack in this true view of the side of the jack. The three lines brought over from the plan define the end of the jack; it's helpful to shade such surfaces that lie or slope out of the plane of the paper. These end lines can be transferred directly to the sides of the timber. We will get the top view shortly.

5. THE HOUSING IN THE HIP. Repeat the process on the hip elevation, erecting lines perpendicular to the hip run line to mark the upslope and downslope sides of the jack housing on the face of the hip. Strike a line parallel to the hip length line starting from the meeting point of the hip section and the lower edge of the

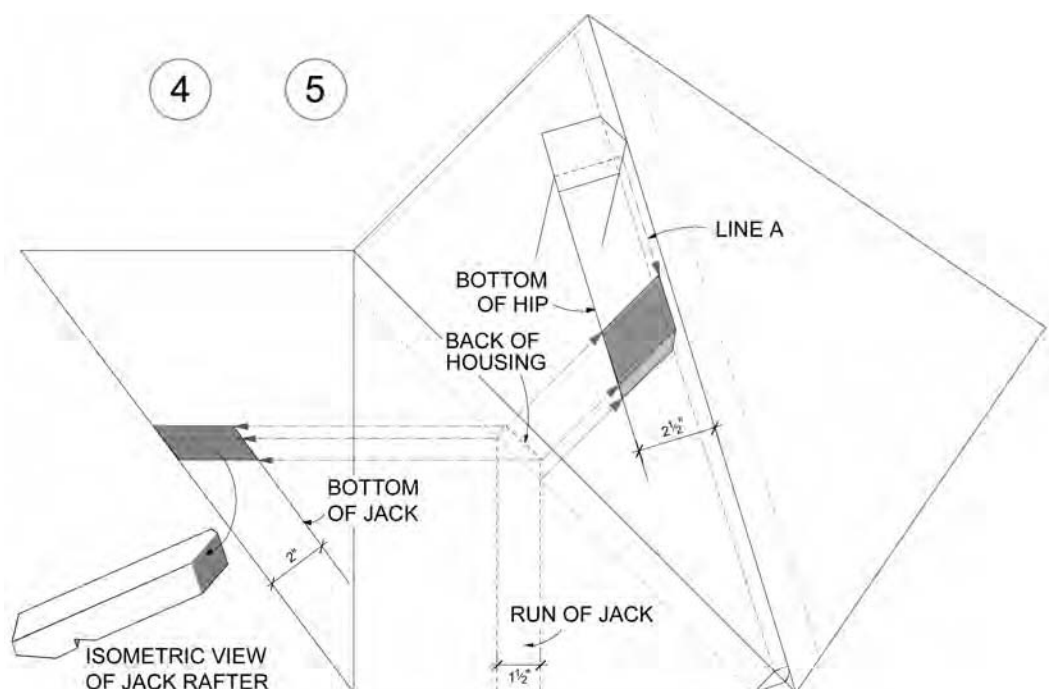


Fig. 11. Drawing the jack and its housed end in the hip.

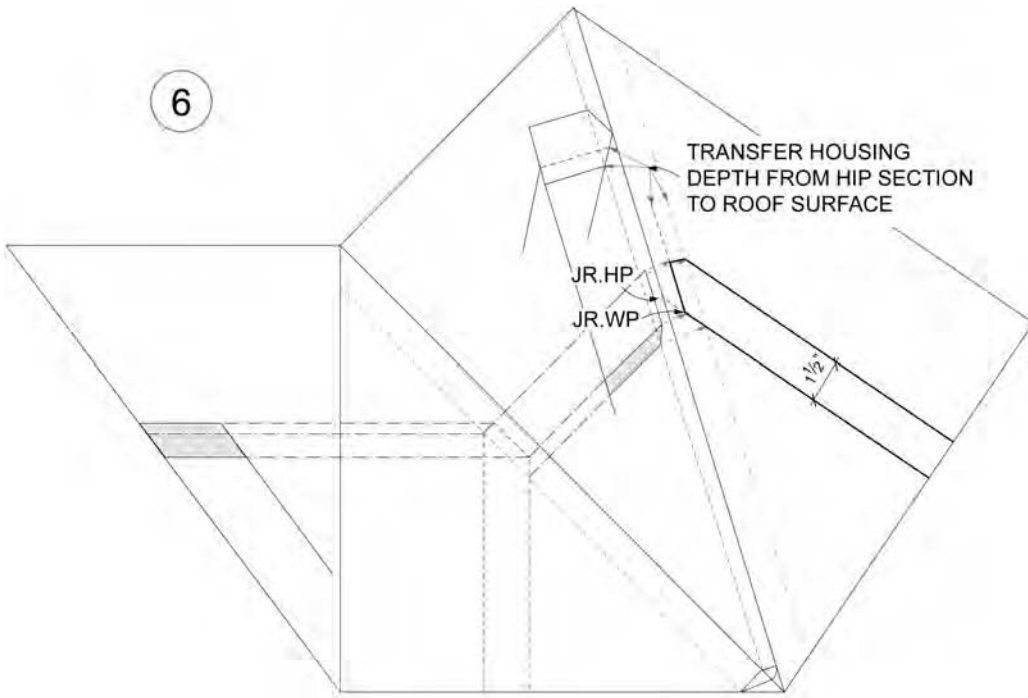


Fig. 12. Drawing the hip housing in the roof surface.

jack. (If the hip were deeper, the crossing points of this construction line with the projected lines from the previous step would delineate the lower edge of the jack housing on the side of the hip. In our case, the hip is shallow enough for the jack housing to be open at the bottom.) Now, in the little superimposed hip section, find the intersection of the backing surface and the back of the housing. Strike a line A from there parallel to the hip length to delineate the back of the housing on the hip elevation.

6. THE HIP HOUSING IN THE ROOF SURFACE (Fig. 12). Square up (perpendicular to the hip length fold line) from the jack housing on the hip elevation to the hip edge in the roof surface triangle and draw the length of the jack intersection in the roof surface. From these points, run lines square to the eave to get the top surface view of the jack. If you've done everything right, these lines should be 1½ in. apart.

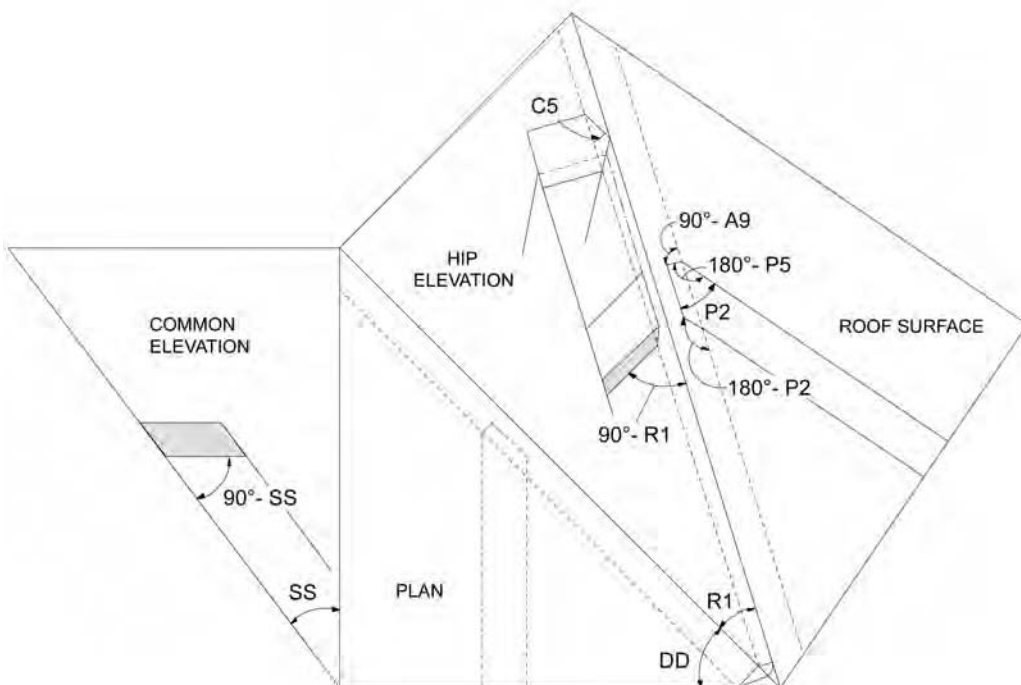


Fig. 13. Hawkindale angles for the joints.

Transfer distances with dividers from the plane of the backing on the hip section to get the depth of the housing on the roof surface. It's not ½ in. as in the plan or elevation view, but slightly larger. Square across from the back and front of the housing in the hip elevation to the roof surface to lay out the upslope and downslope housing lines. Note that the upslope line, which is 90 degrees in plan, is no longer square. This angle relates to Hawkindale A9 (see Fig. 17 in the first article of this series).

ALL of the Hawkindales applicable to these views are shown in Fig. 13. Hawkindale R4 does not appear on these drawings since the underside of the hip is not in the plane of any of our triangles, but you can see in Fig. 2 (p. 5) that it's a simple matter to represent the underside of the hip in one more view, by projecting lines from the housing in the hip elevation triangle to intersect lines with slightly different offsets from those in the roof surface.

It's important to see that the Hawkindales are labeled carefully to show the piece on which they occur. For example, in the roof surface triangle, P2 and 180°-P5 occur on the jack rafter, while 180°-P2 and 90°-A9 are the corresponding angles laid out on the hip. We often show the complement or supplement of a Hawkindale angle, such as 180°-P2 instead of the Hawkindale P2, because the angle must relate to an edge we can measure from. Learning in which direction to draw a Hawkindale and which edge to measure it from are elusive things for a beginner. With experience, you will recognize the ones that show up most often and know how to apply them. You will find that most Hawkindales for pitches 12:12 or under will be less than 45 degrees.

We have now completed the drawings necessary to execute our model. The top and sides of the jack end and the housing on the side and top of the hip can be laid out by transferring the angles and dimensions to the timbers starting at the working points.

To recap, the kernel is a tool to lay out joinery from working points, which are found (along with overall lengths) using the multipliers. Remember that we need to be looking perpendicular to a surface to get a true view of its dimensions. This is the fundamental principle of developed drawing. We can get most of the information for laying out joinery from a top and side view, which the kernel gives us for both the jack and the hip.

In the next article, we will take a very common timber framing problem, the valley dormer, and lay it out using only Hawkindale angles, multipliers and a slightly altered kernel to figure a jack purlin intersection with a valley. We will demonstrate also how to use the same tools to tackle an irregular roof.

—WILL BEEMER
Will Beemer is co-Executive Director of the Guild in charge of education and has taught numerous compound roof courses at the Heartwood School in Washington, Mass.

HISTORIC AMERICAN ROOF TRUSSES

II. Queenpost Trusses

THIS article is second in a series to discuss and illustrate the form, function and joinery of American timber-framed roof trusses of the past, showing typical examples with variations. The series was developed from original research under a grant from the National Park Service and the National Center for Preservation Technology and Training. Its contents are solely the responsibility of the authors and do not represent the official position of the NPS or the NCPTT. Further articles to appear in TIMBER FRAMING will treat Kingpost Trusses and Composite Trusses.

... exhibits the design for a roof whose tie beam is intended to have 40 ft. bearing, but it may be extended to 45 ft. without increasing the size of the timbers. This example is provided with two queenposts instead of one kingpost. (Asher Benjamin, *Elements of Architecture*, 1843)

... a roof supported by two queenposts, instead of a kingpost, to give room for a passage or any other conveniency in the roof. (Peter Nicholson, *The Carpenter's New Guide*, 1837)

A QUEENPOST TRUSS typically comprises two posts spread apart by a straining beam joined near their heads and supporting a tie beam (bottom chord) at their feet, and substantially braced by members rising from the outer ends of the tie beam to the heads of the queen posts (Fig. 1, facing page). These main braces and the straining beam form the top chord of the truss. In service, the queenposts are in axial tension, although they are also compressed transversely between rafter and straining beam at their heads. The straining beam and main braces are in compression. The tie beam is in tension but, because of its length and any loads imposed upon it, it is also subject to bending. It's common for the queenposts to have top tenons carrying principal purlins or principal rafters in the plane of the roof, but these members are part of the load on the truss rather than essential to its operation.

A queenpost truss is to be distinguished from any of a great variety of double-posted roof frames called queen-posted, queen-strutted or post-and-purlin roofs. In a truss, the queenposts hold up the tie beam rather than bearing upon it. Among its other advantages, the queenpost truss can span the same or greater distance than a kingpost truss while using shorter members (or, in the case of the tie beam, smaller sections) since it is supported at two intermediate points. An early example, the 1755 Market Street Meetinghouse in Philadelphia, was roofed with straightforward and well-detailed queenpost trusses of 57-ft. span (Nelson 1996, 16).

Builders' guides of the 17th, 18th and 19th centuries commonly illustrated these truss types, recommending kingpost trusses for shorter spans and queenposts for longer ones. For example, Thomas Treadgold in *Elementary Principles of Carpentry* (1828) conservatively recommends the kingpost truss for spans between 20 and 30 ft. and the queenpost for spans of 30 to 45 ft.

(Treadgold, 88). Edward Shaw in his sixth edition of *Civil Architecture* (1852) stretches the simple queenpost truss to 60 ft., but using queen rods instead of posts (Shaw, 118). Actual practice was complicated by kingpost variants using struts, double or even triple rafters and secondary posts (called queens or princes) for very long spans, as much as 75 ft. in the clear at First Congregational Church in New Haven, Ct., built 1811-14. Ithiel Town designed this church, and the truss, while long, is not novel in form. As usual, it's unclear whether the truss was designed by the architect, the framer or both.

Queenpost trusses were also commonly built with smaller kingpost or kingrod trusses encased within them or above them (providing a peak to the roof), or with large kingpost trusses superimposed upon them or framed among them. The small, subsidiary posts in these trusses have also been called princesses in both English and American practice (Brunskill, 72). St. Helena's Episcopal Church (1842 roof system) in Beaufort, S.C., is a good example of a conventional queenpost truss extended to 61 ft. wide through the use of strutted princess posts. St. Michael's Church (1761) in Charleston, S.C., is an example of a queenpost truss, spanning 54 ft., where principal rafters suspend a kingpost that in turn supports the middle of the queenpost truss straining beam.

Queenpost trusses with smaller, encased kingposts appear in what is probably the earliest illustration of the form, the bridge at Cismone shown in Palladio's *Four Books of Architecture* (1570). This same truss, sometimes with rods instead of posts (even Palladio's version has metal tension connections at the bottoms of all the posts), survives today in many historic wooden bridges and church attics throughout New England, with at least one 1881 bridge example in Elmira, Ont. This category of queen-dominated composite trusses presents clear load paths to the eye and to the experienced framer's intuition and uses no more material than is necessary to do the work (Fig. 2).

The same cannot be said for the trusses where neither king or queen dominates, and the appearance given is of a redundant and sometimes confusing superimposition of forms, statically indeterminate and functioning in parallel or even interfering with each other. Examples are found in the First Moravian Church, Bethlehem, Pa., 1803 (Fig. 3), the large First Congregational Church, Hartford, Ct., 1806 (Kelly, I-203), and Piper's Opera House, Virginia City, Nevada, 1883 (Fig. 4). However, such trusses are recommended for long spans by builders' guides as early as *The British Carpenter* (1733) by Francis Price, whose understanding of truss action is primitive, and forward to William Bell's highly sophisticated (despite its title) *The Art and Science of Carpentry Made Easy* (1857).

Fortunately, the student of historic truss form does not have to depend upon quantitative or graphical analysis or fully understand the operation of a truss to decide whether it is functioning successfully. A long-standing truss can be analyzed qualitatively by directly examining its joints and members for signs of distortion, displacement or failure, and declared good or otherwise.

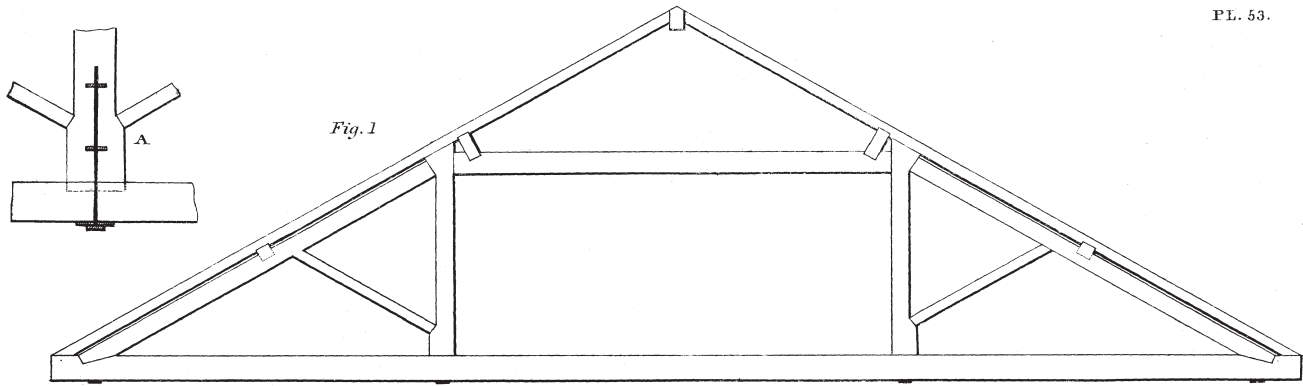


FIG. 1. QUEENPOST TRUSS BY 19TH-CENTURY AMERICAN ARCHITECT ASHER BENJAMIN. INSET SHOWS HIDDEN TENSION BOLT AT POST JOINTS.

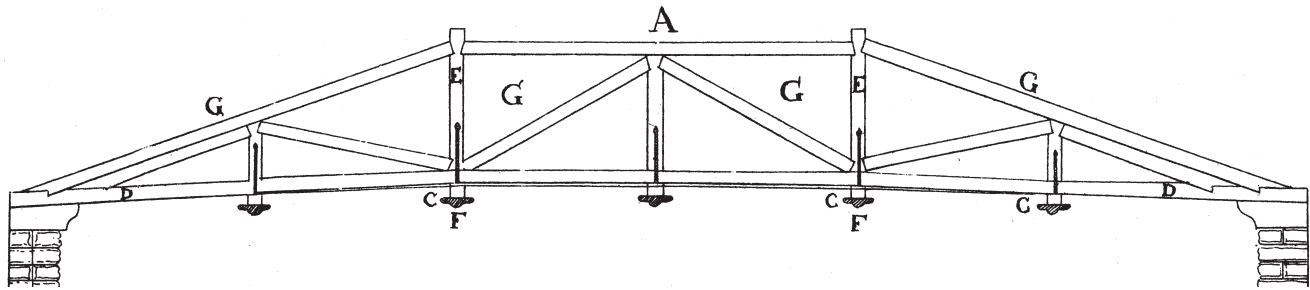


FIG. 2. QUEENPOST TRUSS OF THE BRIDGE AT CISMONE, SHOWN IN PALLADIO'S *FOUR BOOKS OF ARCHITECTURE* (1570).

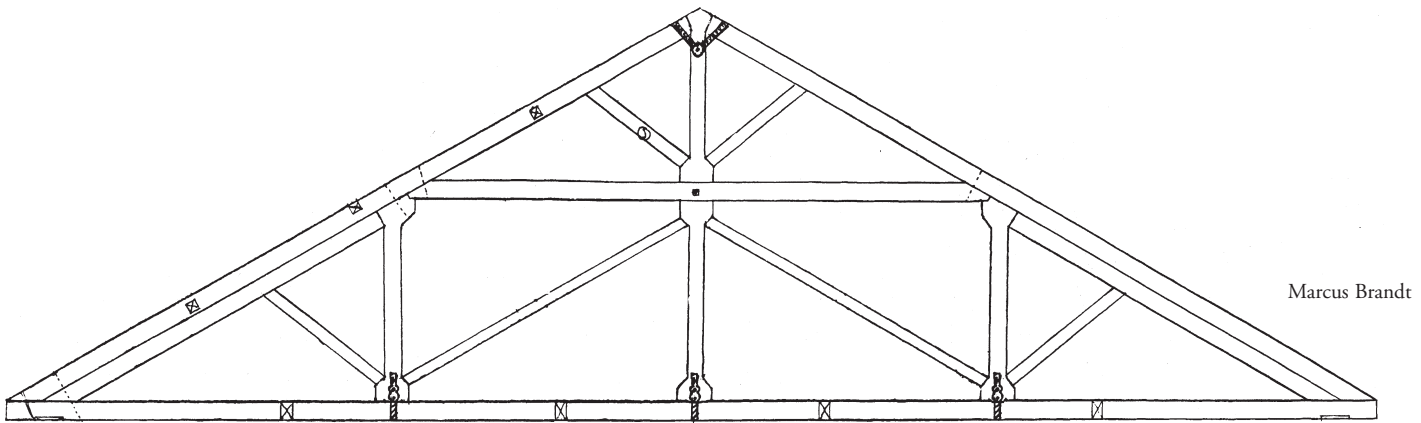


FIG. 3. PRINCIPAL TRUSS, CENTRAL MORAVIAN CHURCH, BETHLEHEM, PA., 1803, SPAN 60 FT.

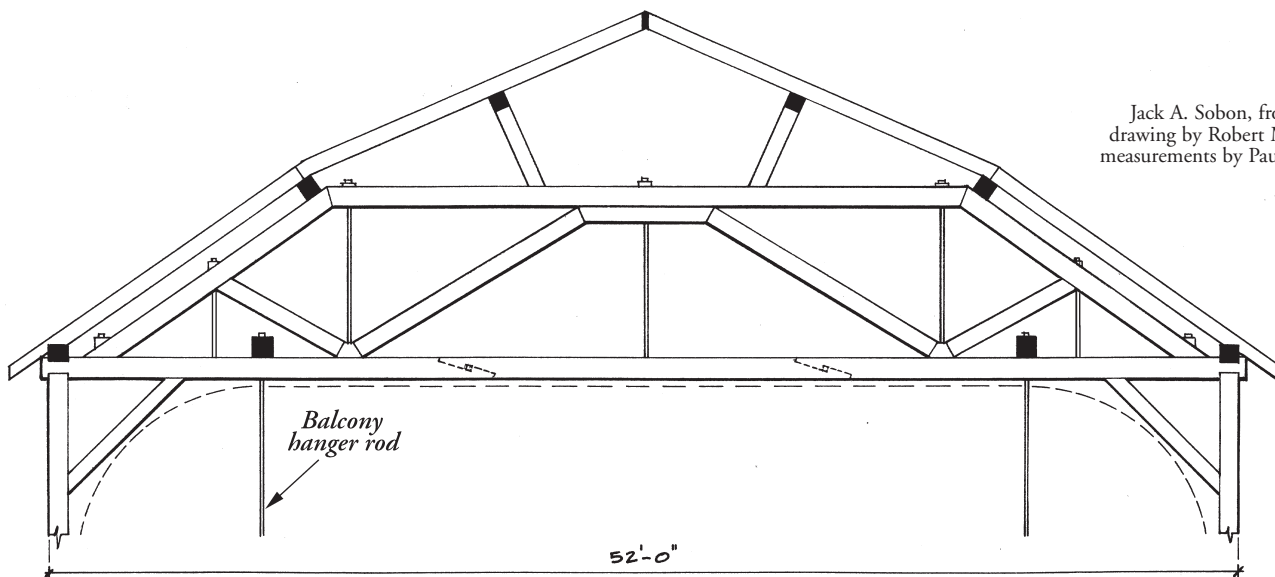


FIG. 4. COMPLEX ADAPTATION OF QUEENROD TRUSS, PIPER'S OPERA HOUSE, VIRGINIA CITY, NEVADA, 1883. NOTE SCARF JOINTS.

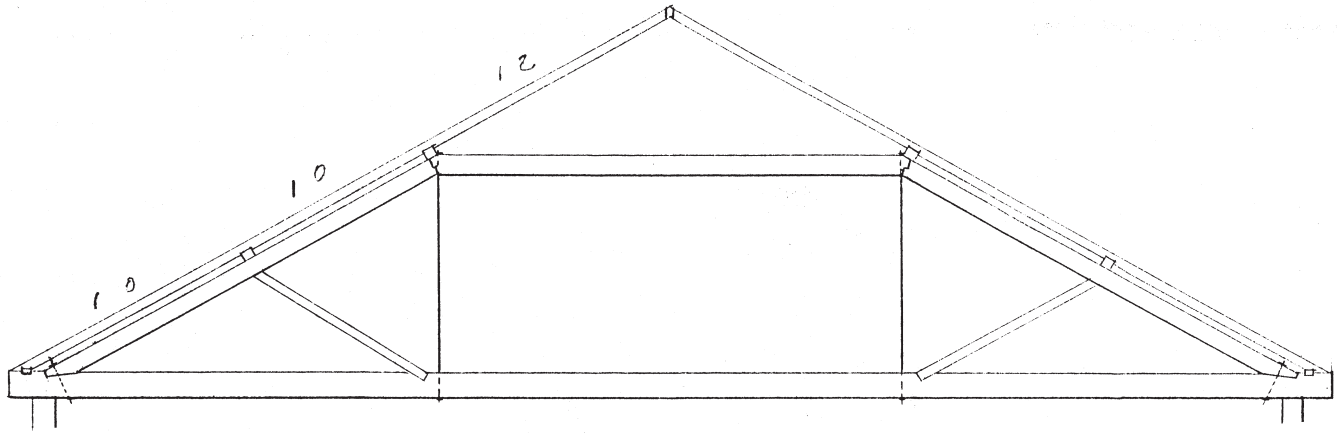


FIG. 5. QUEENROD TRUSS, 54 FT. SPAN, AT THE FIRST CHURCH OF BRIMFIELD, MASS., 1847,
DRAWN BY THE CHURCH'S DESIGNER, CAPT. EDWARD LAMB, ARCHITECT.

Quantitative analysis of the properties of wood and of timber structure is not new. It began in the 1790s in Europe and continued with the work of engineers such as Peter Barlow at the British naval yards in the early 19th century and the American bridge engineers Herman Haupt and Squire Whipple at mid-century. Quantitative analysis remains controversial today. As recently as 1900, the third edition of a widely used text, W.C. Foster's *A Treatise on Wooden Trestle Bridges*, argued at the outset, "A few engineers have advocated the use of mathematics in the designing of trestles, but as wood is an article whose strength and properties vary widely with each piece, no dependence whatsoever can be placed upon the results, and such practice is to be condemned. It is far wiser to merely follow one's judgement and the results of the experience of others as to the proper proportioning of the various parts."

We should avoid the tendency to see inevitable historic progress from the darkness of confused forms to the light of simplified, "cleaner" design over time. The bridge over the Sarine at Fribourg, Switzerland, built in 1653 and still standing, was a very uncomplicated two-span queenpost truss. The next 150 years in the same country saw the construction of fabulously complex, statically indeterminate and increasingly longer-span wooden bridges, often incorporating queenpost elements, culminating in the internationally celebrated Schaffhausen Bridge of 1756-8 (Soane, 130).

Queenpost trusses are repeatedly recommended by builders' guides such as Benjamin's *Elements of Architecture* (Benjamin, 51) for attic spaces, where lodging rooms are conveniently accommodated by the open quadrangular section formed by the truss. Queenpost trusses are widely used also at the rear wall of steeples, where the tower posts can become the queenposts. This arrangement is doubly efficient because the two verticals are already available for use and because the back of the steeple imposes a greater load than the kingpost trusses in the rest of the roof are asked to bear. Examples of the queenpost steeple truss, as in the Universalist Church, South Strafford, Vt., 1833, or the United Church, Craftsbury Common, Vt., 1816, are so common as to be considered standard practice for the period in the northeastern US. (The alternative methods of carrying steeple loads in the late 18th and 19th centuries set the tower posts to bear on large-dimension sleeper timbers crossing three trusses, or used a vestibule wall to bring steeple loads to the ground if the interior aesthetics of the church permitted.) In many other 18th- and early 19th-century churches, notably in Connecticut, apparent queenposts in the attic are actually extensions of gallery posts rising from below, producing what J.F. Kelly in *Early Connecticut Meetinghouses* (1948) calls a post-and-purlin system, not a truss in the modern sense.

By 1839, Asher Benjamin is recommending iron queen rods in place of wooden queenposts. Edward Shaw in *Civil Architecture*

(Shaw, 118) credits Benjamin with the first publication of this idea as well as first using it in trusses as early as 1828. There are at least two advantages to replacing wooden posts with iron rods. First, a major source of settlement in any truss is shrinkage across the king- or queenpost head. The post timbers are frequently 12 to 14 in. across to accommodate perpendicular (square-ended) bearings of the main braces at the head. Shrinkage and compression can easily amount to $\frac{5}{8}$ in. across this joint, accumulating well more than an inch for the two joints, which translates into sag in the truss. The change in shape produces an eccentric bearing of the main brace at the head, causing the sharp end-grain corner of the brace to indent even further into the side grain of the post until some equilibrium is reached. This process takes some time and must be anticipated by increasing the initial camber of the truss by some guessed or calculated amount.

Using rods instead of posts allows the main braces and the straining beam to meet directly, with largely axial, end-grain bearing, effectively forming a polygonal-arch top chord. On the other hand, the loss of a substantial timber volume between main brace and straining beam renders it impossible to create normal (90-degree) bearing surfaces; rather, the angle of intersection between the two members is typically mitred. A second reason to use iron rods instead of posts is the difficulty of developing a satisfactory tension joint in wood within the depth of the tie beam, i.e. without the ends of the queenposts penetrating the ceiling normally found just below the truss. The wedged half dovetail tenon, usually pinned as well, became the timber joint of choice between the queenpost and the tie, but its dependence on side-grain bearing, short double-shear distances on the pins and shrinkage make it subject to creep over time, though it rarely fails completely. Iron straps or inset bolts have been in use at these connections since the Middle Ages (Hewitt, 144, 244; Nelson, 1996, 11-23; Palladio), so the step to a completely iron member was not a large one. Queenrod trusses are found in many large structures, ranging from the First Church of Brimfield, Mass. (1847), with its 54-ft. truss span (Fig. 5) to the railroad depot freight shed in Virginia City, Nevada (1875), whose truss spans 32 ft. (Fig. 6). While it is difficult to say if Benjamin's was the first use of vertical rods as the primary tension members of a truss, it is known that rods were being used diagonally as counterbraces or suspension elements at the Schaffhausen Bridge in Switzerland by 1780 (Maggi, Haupt, Nelson 1990) and in Louis Wernwag's "Colossus of 1812" across the Schuylkill in Philadelphia (Nelson 1990).

Railroad Freight Shed, Virginia City, Nevada, 1875. The queenrod trusses in this shed span 30 ft. 8 in. in the clear and belong to a class of trusses that do not bear on a wall plate but rather tenon

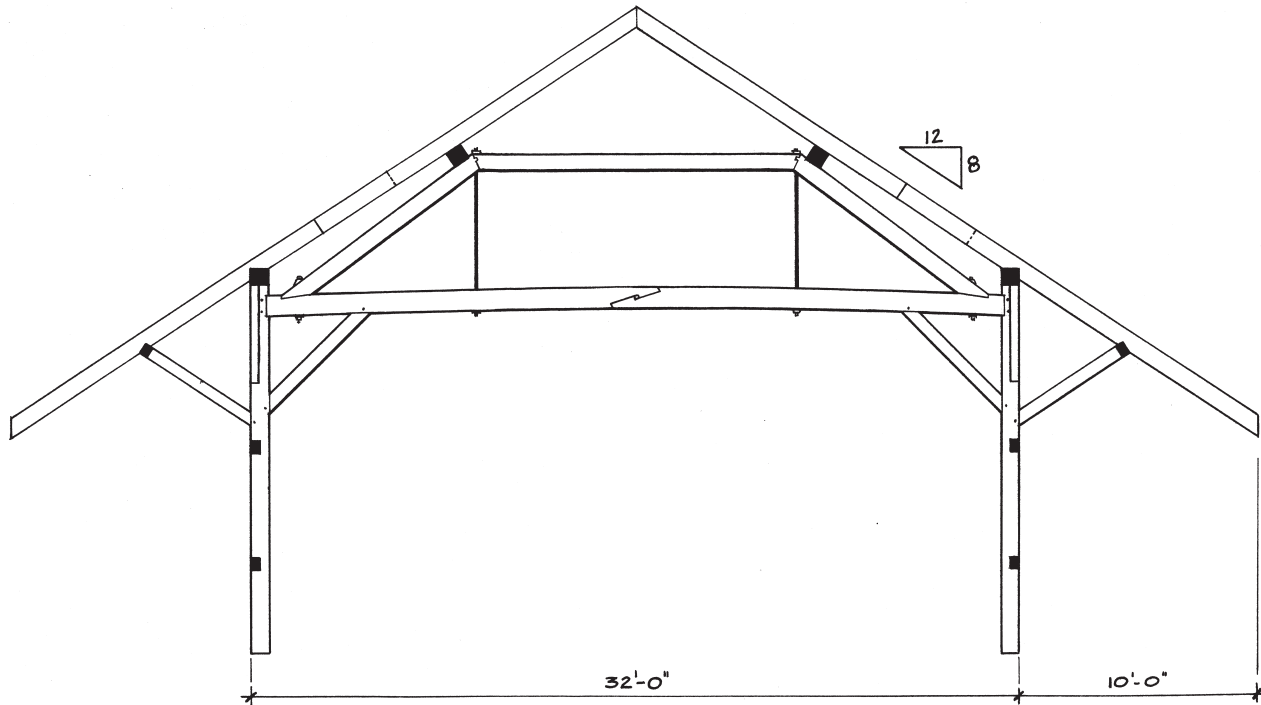


FIG. 6. QUEENROD TRUSS IN A 52X138-FT. RAILROAD FREIGHT SHED, VIRGINIA CITY, NEVADA, 1875. TRUSSES STAND ON 16-FT. CENTERS.

into the side of a wall post (Fig. 6). In this case, the 8x10 scarfed bottom chord bears on a 1-in. shoulder as well. Since neither ceiling nor floor is carried on this bottom chord, the 24-in. stop-splayed tabled and wedged scarf doesn't suffer significant bending. Knee braces rising from the post also add support against shear and reduce the overall span. The 1-in. iron rods nutted with ogee washers drop from the junction of the main brace and straining beam to support the bottom chord about 9 ft. out from the posts on each end. Trusses stand 16 ft. on center in this 138-ft.-long building. No timber is longer than 18 ft.

The angle between main brace and straining beam is mitred with a small integral tenon to keep the members in line. The main braces dap 3 in. into a bearing shoulder in the bottom chord where they are fastened by a 1-in. bolt. The ultimate bearing of the brace is 6 in. from the face of the post. A projected line of the main brace's slope if carried through the tie beam would end up within the post before exiting the tie beam, suggesting that little bending or shear will occur.

The 2x8 freight shed rafters, lapped and spiked to form a 31-ft. length, are all commons supported on a purlin carried at the queenrod head and at the plate. They continue outboard to form a 10-ft. overhang supported by 4x6 bracing rising from the wall posts in approximate opposition to the interior braces that rise to

the tie beam. The extensive cantilever of the rafters and their continuity reduce the loading of the truss and place more of their weight on the wall posts.

Peacham Congregational Church, Peacham, Vt., 1806 (photo p. 20). The roof of this church rests on six queenpost trusses, 9 ft. on center, tightly joined and densely framed, 46 ft. 8 in. in the clear and 50 ft. 5 in. overall. The lower chords of the trusses are cambered progressively from the ends of the church toward the middle, in the pattern 8-14-17-16-14-8 in. of transverse rise, forming curvature in both directions and thus a shallow dome in the ceiling of the audience room below (Fig. 7).

The 7x10 queenposts taper slightly in all dimensions toward the top, and the 7x8 main braces and straining beams engage the queenposts with normal bearing, tenoned but without pins. Principal rafters 7x10 by 30 ft. sit on tenons at the heads of the queenposts, each affixed by a single 1-in. pin. The principal rafters are pinned in mortises at the extremity of the 11x14 bottom chord, bearing over the wall plate and extending to the eaves. The main braces of the truss are shouldered and tenoned (but again unpinned) into the bottom chord 23 in. from the inside of the plate. The queenposts, centered 15 ft. apart, support the bottom chord by means of the through-wedged half-dovetail joint, wedged

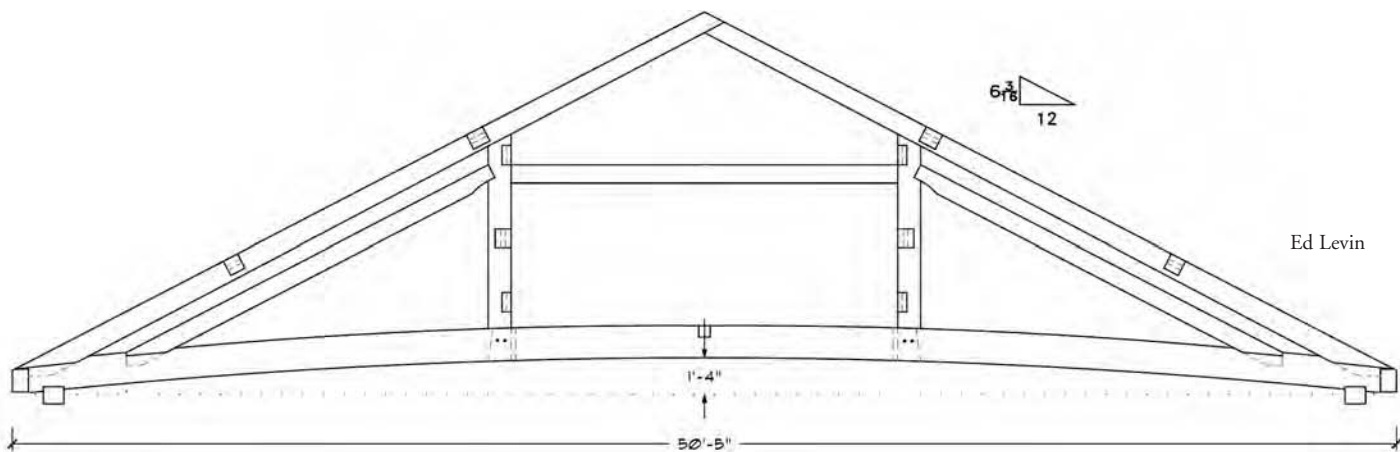


FIG. 7. QUEENPOST TRUSS AT CONGREGATIONAL CHURCH, PEACHAM, VT., 1806. TRUSSES ARE CAMBERED TO PRODUCE DOMED CEILING.

from above and with 1½ in. of taper on the dovetailed tenon. A pair of inline 1-in. pins also transfixes the dovetail 6 in. down from the top of the chord.

The trusses at Peacham are joined longitudinally by 7x8 connecting girts with rising and falling 4x5 hardwood braces at each post. The principal rafters carry 7x8 horizontal purlins mortised in at two positions, dividing the roof plane in three, and 3x4 and 4x4 common rafters soffit tenon into these purlins. There is also extensive 4x6 diagonal bracing in the plane of the ceiling and of the roof, but not arranged in the typical diamond pattern of opposing short 45-degree braces. Instead, the roof braces rise from the plates in long parallel lines crossing several trusses, and then descend again to the plates. The braces lying in the ceiling plane form a giant X seen in plan. All of these braces are not actually passing braces since they comprise mortise-and-tenoned segments between each major framing member, but their collective appearance and effect are those of long continuous braces (see cover photos).

Peacham is framed in hewn spruce and pine for the longer timbers and vertically sawn maple, birch and beech for braces. The frame is fully scribed, with Roman numerals at every joint. The level lines are obvious on the posts, but 2-ft. marks cannot be distinguished at the expected locations. Except for its rich color, the timber appears in like-new condition, densely framed but with no superfluous members. The framer, we know, was one Edward Clark of Peacham.

Rindge Meetinghouse, Rindge, N. H., 1797 (photo p. 20). The large and highly cambered queenpost trusses at Rindge belong to a recognizable subset of trusses (both queenpost and kingpost) that use naturally curved main braces, always working in concert with a straight principal rafter directly above (Fig. 8). These curved main braces occur also, for example, in kingpost trusses at the Old Ship Church (1681) in Hingham, Mass., and the 1714 Lynnfield, Mass., Meetinghouse. The British framing scholar David Yeomans comments that this form does not appear in the English truss work that he is familiar with, and he leaves the question of its origin open (Yeomans 1981). Likely the inspiration comes from late medieval kingpost and crownpost roof framing, where it was common for curved braces to rise along the span of a tie beam and tenon into a kingpost (or the shorter crownpost), aiding the post in supporting the ridge or a collar. Examples are found in the 14th-century Major Barn at Lenham in Kent, the 14th-century Frindsbury Barn, also in Kent, the even earlier Warnavillers Barn in France, and the late 16th-century Bishop's Palace in Fulham, London (Kirk, 1994, 59, 101, 115; Hewitt, 1980, 214). The form of an inner curved brace strutting to an outer and straight rafter is characteristic of cruck framing as well. In all the cases cited, the upward arching of the inner brace serves to stiffen the rafter more

effectively than a straight member with its tendency to bend. In turn, the weight of the roof stiffens the arching brace against buckling out and upward from the load imposed by the kingpost and its dependent areas. These kingpost, crownpost and cruck roofs were usually open to view and thus easily absorbed into the vernacular framer's worldview of good practice.

The roof frame at Rindge is notable for both the size of some of its timbers (Fig. 9) and the tendency of almost all members to curve or taper in some dimension, giving it an especially dramatic pre-modern appearance. The wall posts are 28-ft. oak 10x12s and the tie beams atop them are 55-ft. white pine timbers, tapered from 12x14 to 12x12 and cambered between 14 and 25 in. to produce a shallow dome, as is found later at Peacham. The camber is so great in the ties that the framer had to use a system of 3-ft. marks rather than the usual 2-ft. marks to scribe the queenpost to the tie beam marks.

Rindge is considered a single-braced queenpost truss since the main braces form the primary loadpath, but the principal rafters, while mostly in bearing, are tightly joined to the truss. The queenposts are pine, typically 11x11 at the bottom and tapering to 8x10 at the head, where they tenon into the principal rafters, long pine timbers tapering from 8x12 at the eaves to 8x8 at the peak. The queenpost main braces and straining beam engage the queenposts 15 in. below the rafter and are tenoned and pinned, the straining beam normal to the face of the post and the main brace dapped in at its top end producing a 1½-in. shoulder (Fig. 10). The main braces and the straining beam, all roughly 7x7s, are of mixed oak species and slightly upcurved. A 4x6 hardwood strut rises from a mortise low on the queenpost to support the main brace near its midpoint. Pairs of short 4x6 blocks then strut from the main brace to the principal rafter, though neither of them is directly over the queenpost strut. The stoutest timber in this assemblage is the interrupted flying plate supporting the feet of the common rafters: each segment, 13 ft. long and tenoned into the sides of the tie beam right at the eaves, measures 24x12 in section.

The joint between the queenpost and the tie beam uses a 3-in. through-wedged half-dovetail tenon with a reinforcing ¾x1½-in. U-strap with forelock bolt (Fig. 10). The iron appears original and was likely necessitated by the difficulty of pulling and holding the huge tie beams into the exact curve required by the dome, even if the ties started with some natural sweep and hewn camber. Bending to strike two points correctly (the queenposts) is much more demanding than pulling up a long tie beam to one kingpost at its center.

At the foot of the oak main braces is a 3-in. tenon, end wedged. The wedge facilitates the assembly of these huge curved forms, and driving or changing the size of the wedge allows the framer to fine-tune the camber of the truss and guarantee that the main brace is the major bearing member. The principal rafters are mortised over

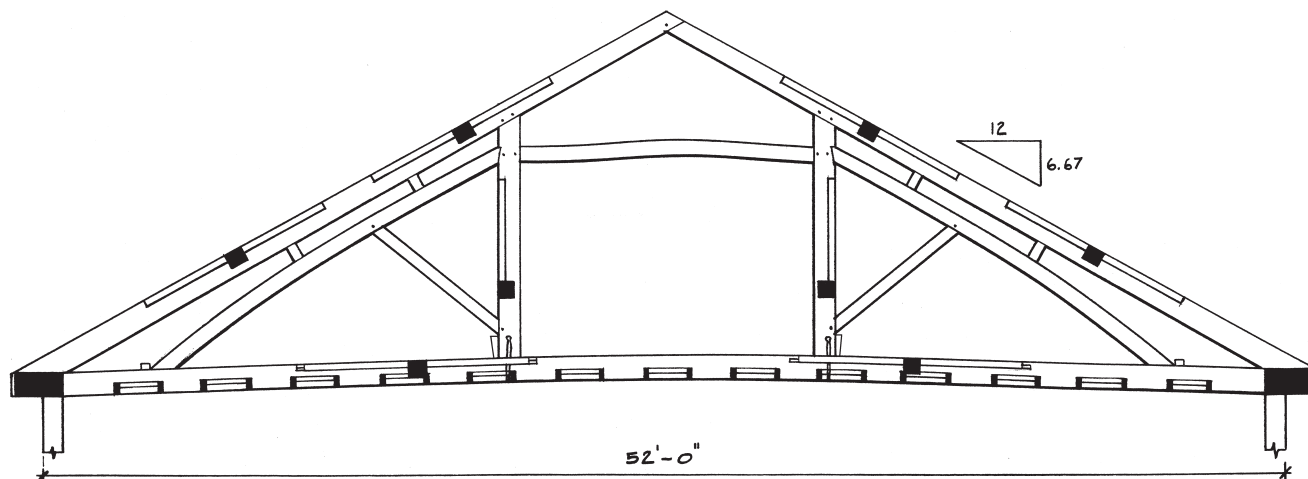


FIG. 8. EARLY-FORM QUEENPOST WITH CURVED OAK MAIN BRACES AND STRAINING BEAM AT MEETINGHOUSE, RINDGE, N.H., 1797.

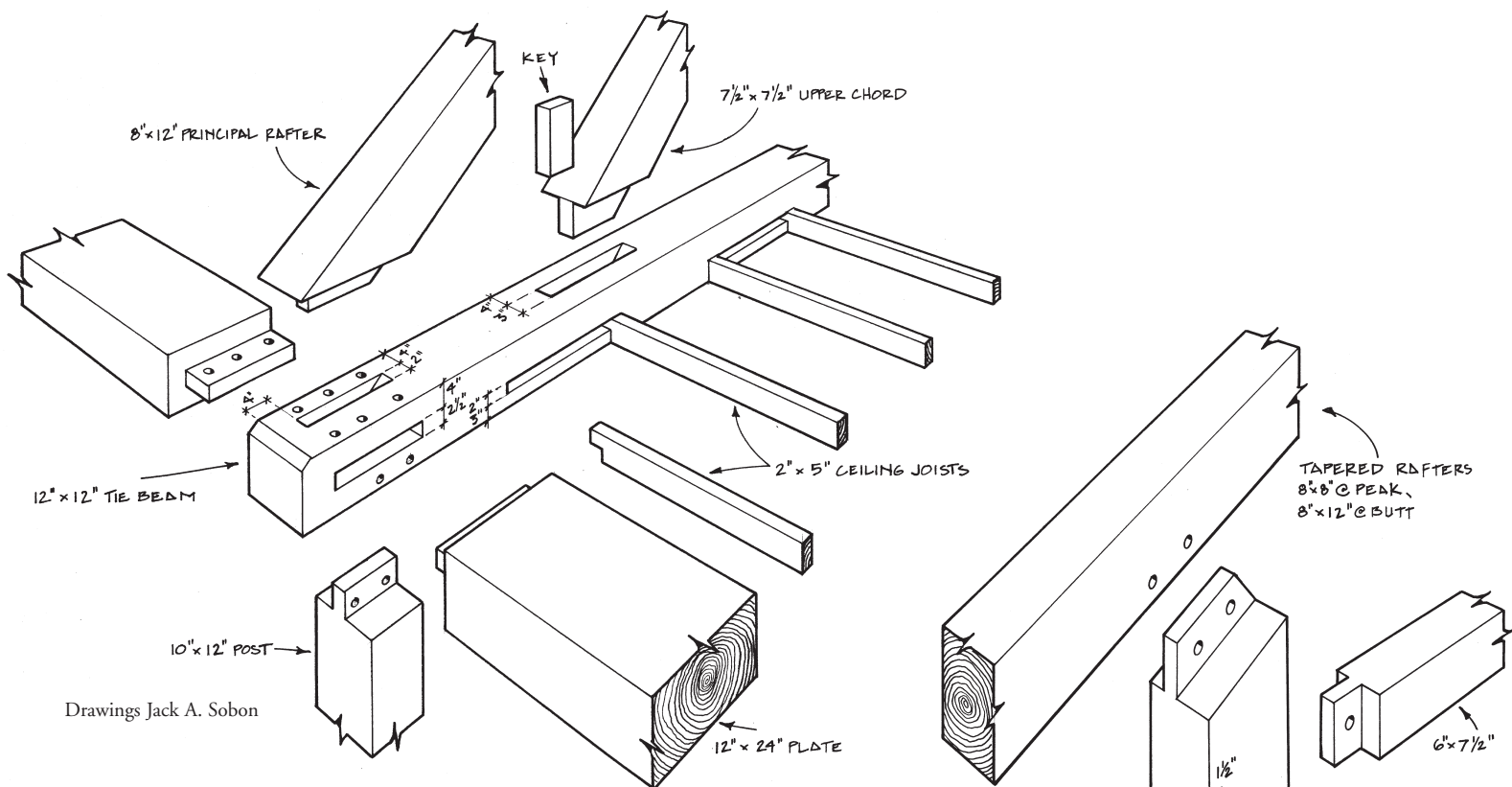


FIG. 9. AT RINDGE, PRINCIPAL RAFTERS AND END-WEDGED MAIN BRACES SEAT IN THE LOWER CHORD OF THE TRUSS.

the queenposts and tenon at their feet into the ends of the tie beam. The queenposts are all connected longitudinally with 7x8 girts, diagonally braced off each post. There are substantial connecting girts with braces joining bottom chords in the plane of the ceiling as well. The bottom chords are fitted with long horizontal chase or pulley mortises on one side, allowing the ceiling joists to be slipped in after erection of the trusses (Fig. 9). This scribe-rule mixed pine and oak frame is all handhewn or vertically sawn. Apart from minor overall sagging of the heavy roof system from shrinkage and two centuries of compression, the queenpost trusses at the Rindge Meetinghouse are performing well.

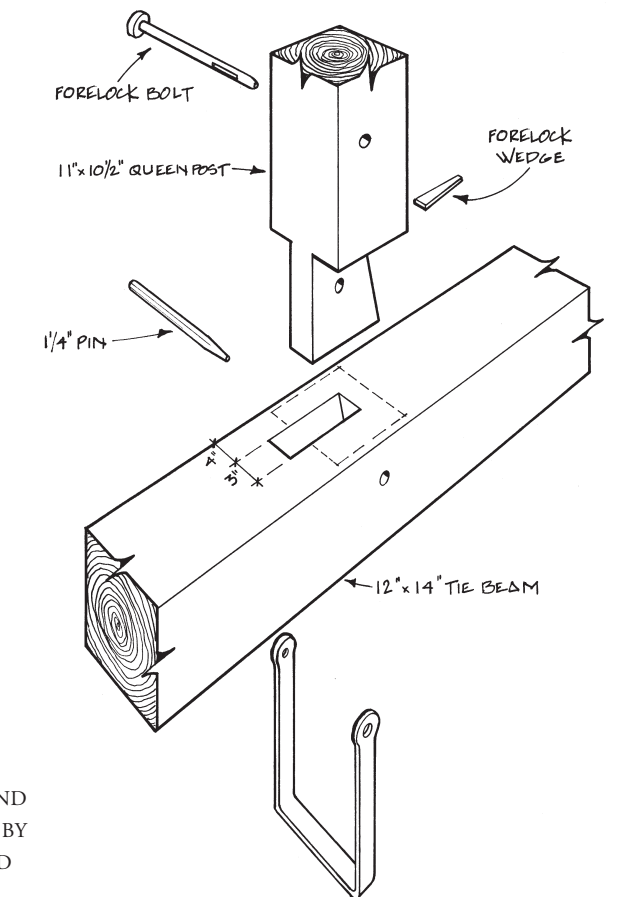
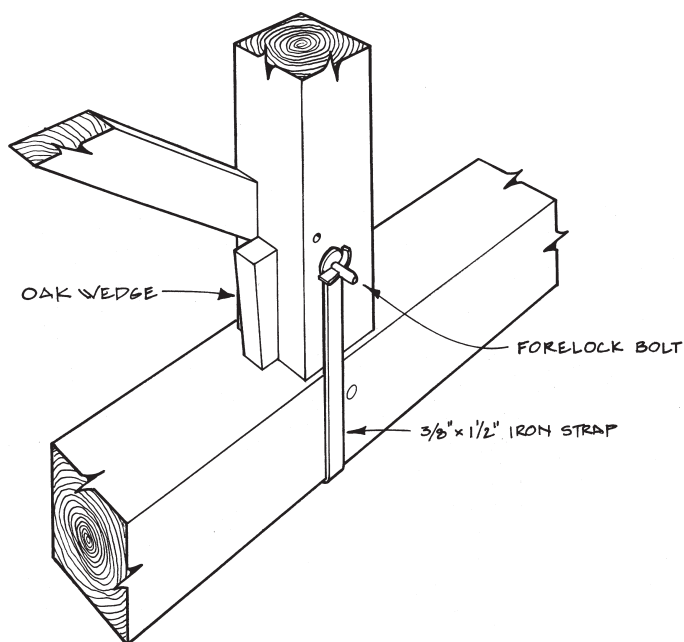


FIG. 10. ABOVE, TO RESTRAIN THE LOADED TENSION JOINTS BETWEEN QUEENPOSTS AND TIE BEAM AT RINDGE, WEDGED AND PINNED HALF-DOVETAIL TENONS ARE REINFORCED BY STOUT IRON STRAPS WITH FORELOCK BOLTS. ABOVE RIGHT AND AT RIGHT, EXPLODED VIEWS OF QUEENPOST TOP AND BOTTOM (TENON WEDGE NOT SHOWN).

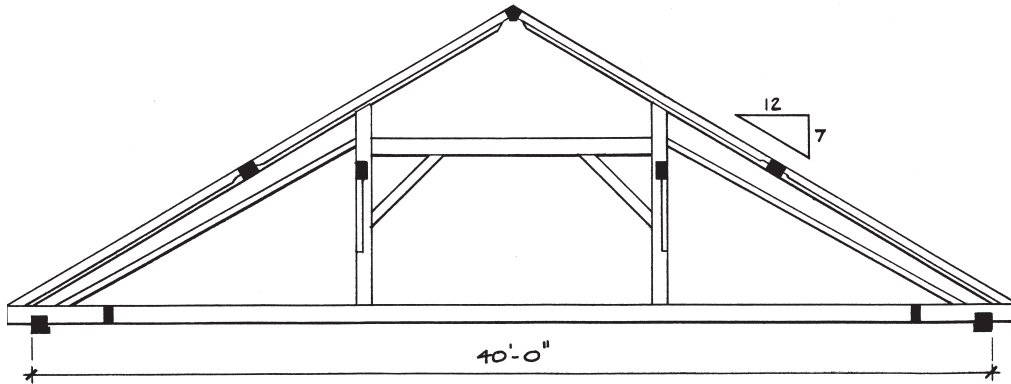


FIG. 11. COMMUNITY CHURCH, WATERBURY CENTER, VT., 1831, WITH NAÏVE KNEE BRACING AND UNDERSIZED MAIN BRACES.



FIG. 12. DETAIL OF TOWER TRUSS, UNITED CHURCH, CRAFTSBURY, VT., 1816.

The Waterbury Center Community Church, Waterbury Center, Vt., 1831. Significantly remodeled in the later 19th century, this 40-ft.-wide brick church (photo p. 20) is spanned by queenpost trusses of good material and joinery, but with disproportionately undersized main braces (Fig. 11). This potentially fatal flaw has already produced local deflections in the trusses and roof of 2 in. to 6 in.

The 9x10 bottom chord is 42 ft. long and laps and bears on an 8x8 plate. The 8x8 queenposts stand 14 ft. apart separated by an 8x9 straining beam, with 4x4 braces rising to it from the queenposts. While bracing of this sort is common in the so-called queenpost purlin systems of houses and barns, it is rare and unnecessary in trusses, where the compressive load of the roof system and the main braces of the truss itself, unsupported across a long span, are constantly forcing the heads of the queenposts inward and down, with little possibility of transverse racking. These unneeded braces may be the first clue that the framer did not understand how different the behavior of a truss was from a frame with intermediate posts. The main braces are the weak point of this truss: often waney, they are variously sized between 3, 4 and 5 in. thick by 7 in. deep. In service, they are all buckling along their 16 ft. length and compressing their inadequate end-grain sections into queenpost and bottom chord—all of this exacerbated by the low 7:12 roof pitch. Queenpost tenons enter the principal rafters above in greatly elongated mortises, removing the possibility of adding stability to the truss.

There is a pattern of patched holes in the original floor of the audience room below that may indicate the former presence of posts supporting a gallery. If gallery posts had continued upward to support the bottom chords of the trusses, even though they would have arrived at the truss several feet outboard of the queenposts, they may have mitigated the great flaws of this design. However, 1831 is late for galleries. Also, there is no evidence along the examinable sides of the bottom chords, in the form of shoulders or pegholes, or traces of old compression, to indicate such posts. Further, the church has no written or oral tradition of a galieried form. The frame is entirely spruce and hemlock, square ruled. Currently the Waterbury Center trusses are supported with cables, stiffening planks and nailed bracing while being monitored for ongoing movement.

The United Church of Craftsbury Common, Vt., 1816. Like many other church roofs framed with kingpost trusses, Craftsbury's also employs one queenpost truss at the rear wall of the steeple, incorporating the tower posts (Fig. 12). These double-duty 10x10 posts, set about 9 ft. apart, are 20 ft. tall, with 22-ft. belfry posts rising from within them in telescoping fashion. An additional 33 ft. of spire and vane sit atop the belfry. Half of this collected load is carried by this queenpost truss. Main braces (8x9)

rise from mortised bearings on the bottom chords almost 2 ft. inboard from the wall posts and tenon into the queenposts, directly opposed by an 8x8 tower girt acting as a straining beam. The 6x6 principal rafters at this truss, rising at a slightly steeper pitch than the main braces, also tenon into the sides of the tower posts and contribute some support (Fig. 12, second rafter from left). There is no straining beam in a direct line with these principal rafters, but 4x5 hardwood braces descend to the straining beam below, where they are opposed by rising braces, and so provide some additional stiffness to the tower post. Unlike Waterbury Center, where the diagonal braces to the straining beam were superfluous, those at Craftsbury expand the effectiveness of the straining beam and, more important, help brace the tall steeple against racking movement.

The truss at Craftsbury Common has hewn spruce major members and vertically sawn and riven maple and yellow birch braces. In spite of its late date of 1816, it is scribe ruled and marked with Roman numerals at every joint. Roof leakage once caused the north end of the queenpost truss to deteriorate badly; it is now assisted by posts rising through the back of the audience room to the bottom of the chord near the posts. Nonetheless, there is no evidence at any of the joints or members of past distress from overloading, such as cracked mortise cheeks or withdrawn joinery.

The Stowe Community Church, Stowe, Vt., 1867. This large, tall and prominent wooden church (photo p. 20) stands on the main street of a busy commercial village. The sophisticated, substantial framing of the stages of the 165-ft. steeple uses paired members, called partners, that eventually clasp a tall spire mast. The steeple work was carried out by a Mr. Edgerton of Charlotte, Vt., a steeple specialist, and appears different in kind from the truss work.

The queenrod trusses supporting the roof and the ceiling of the audience room span 50 ft. (Fig. 13). They are lightly framed, with little mortise-and-tenon joinery, but they stand only 8 ft. apart and are performing well today. Other examples of queenrod framing are common from this period and even earlier, as in the 1847 Brimfield, Mass., Congregational Church (Fig. 5), where queenrod trusses with minor struts rising from the tie beam to the main braces span 54 ft.

The bottom chords of the Stowe trusses do not bear on a plate or post but (like the 1875 Nevada freight shed we saw earlier) instead use shouldered tenons to engage the wall posts about 1 ft. below the plate. The 9:12-pitch main braces sit in a 1½-in.-deep housing on the bottom chord, about 6 in. in from the post (Fig. 14). Instead of a tenon, a ⅝-in. bolt secures the connection. The 8:12 pitch common rafters bear on the top surface of the plate. The 6x7 main braces meet the 6x6 straining beam in a mitred butt joint fitted with a small key to help keep the members aligned (Fig. 15). Iron rods drop through this junction and support the bottom

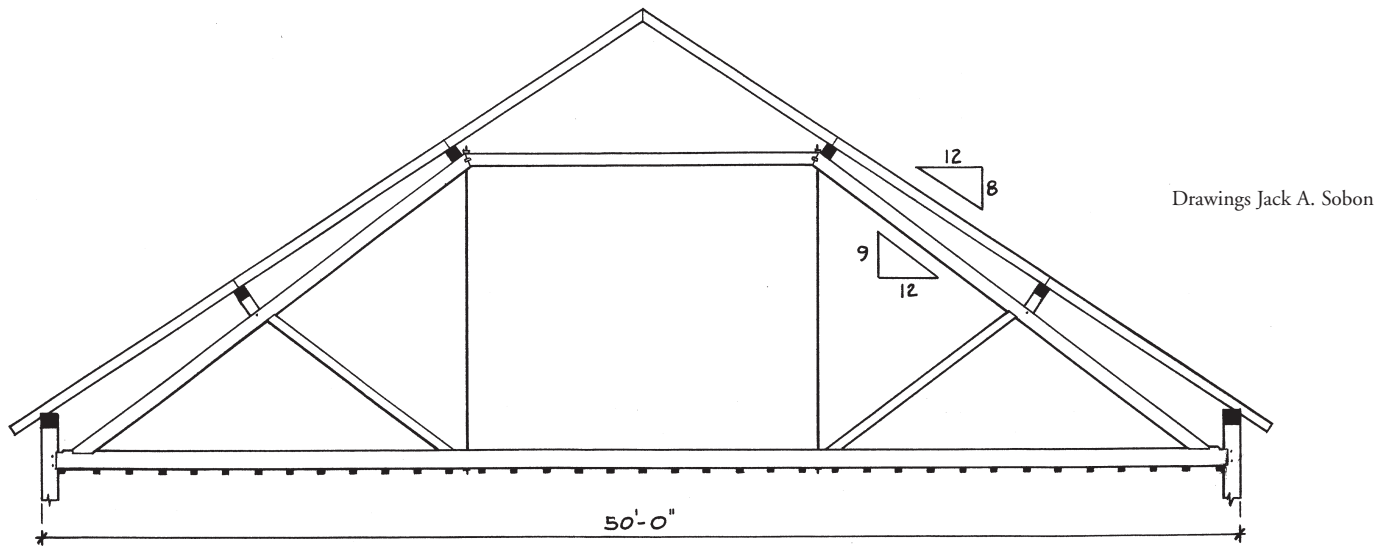


FIG. 13. LIGHT, SOPHISTICATED QUEENROD FRAMING AT THE COMMUNITY CHURCH, STOWE, VT., 1867.

chord at two points. A 4x6 strut rises from the bottom chord next to the rod and supports the midpoint of the main brace. Another short strut, nearly in line with this one, rises from the main brace to support a purlin under the common rafters of the roof plane. Another 6x6 purlin carries out the same function while sitting atop the main brace right next to the junction with the straining beam. In the monumental and architecturally elaborate Stowe Community Church we see the beginning of modern wood framing, where the roof truss is efficient, the joinery minimal, more metal is included and the appearance of the hidden structure inspires little awe.

—JAN LEWANDOSKI

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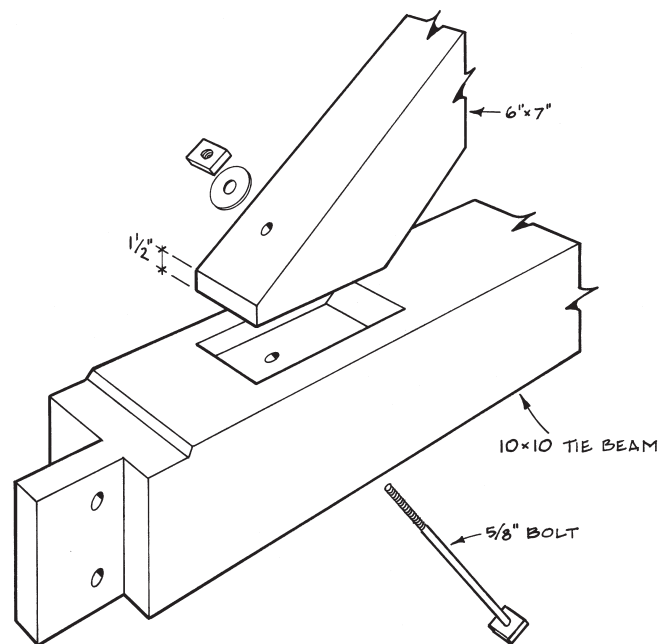


FIG. 14. MAIN BRACE FOOT IS DAPPED AND BOLTED TO THE TIE BEAM.

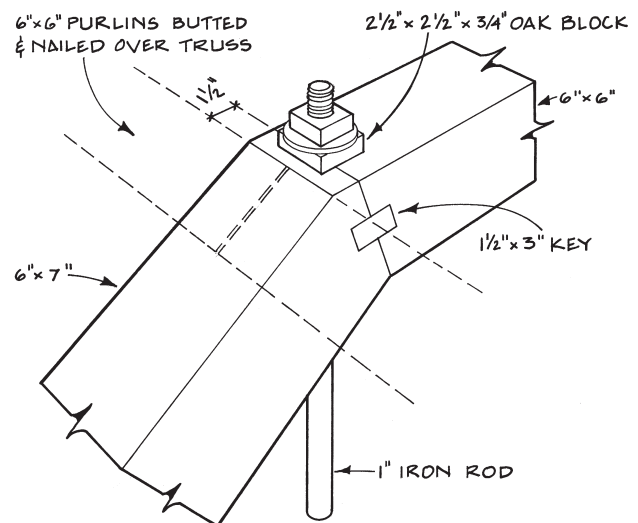


FIG. 15. MAIN BRACE IS BUTTED AND KEYED TO STRAINING BEAM.



Community Church, Waterbury Ctr., Vt., 1856.

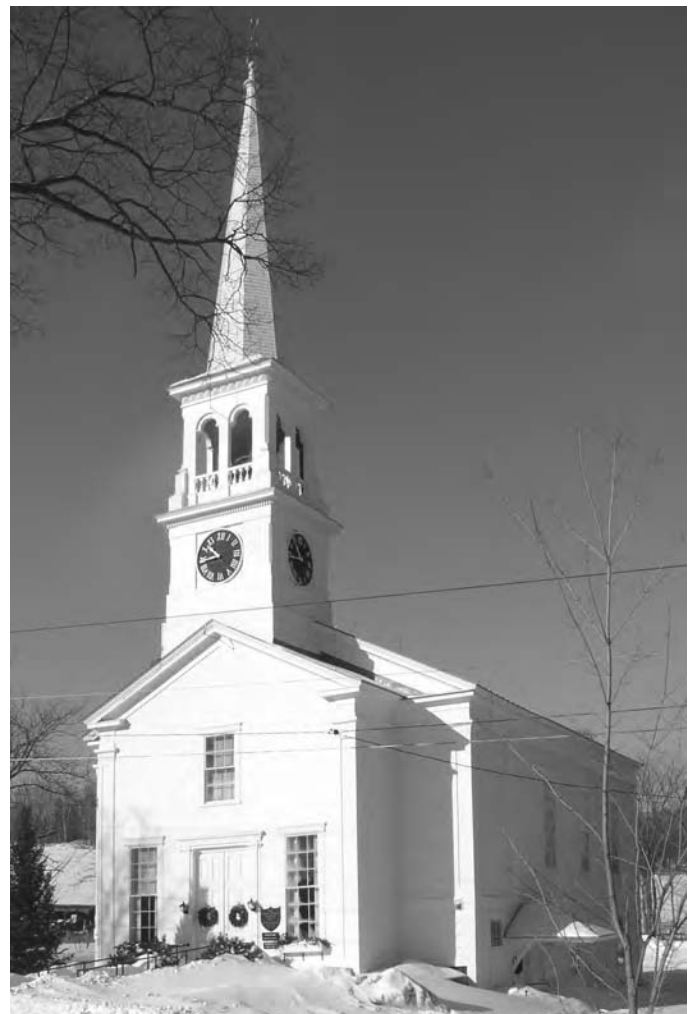


Meetinghouse, Rindge, N.H., 1797.

Photos K. Rower, J. Sobon



Community Church, Stowe, Vt., 1867.



Congregational Church, Peacham, Vt., 1806.

Historic Queenpost Truss Analysis

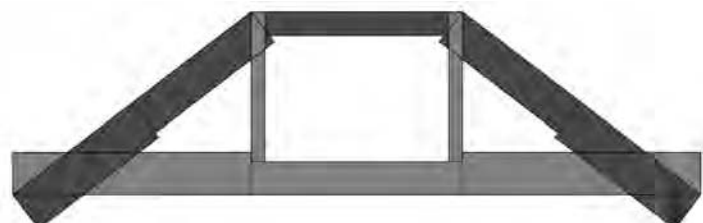
AN ATTIC census of early American public buildings would show that the vast majority of roofs are kingpost trusses and variants thereon, though early American builders were certainly familiar with queenpost trusses via the popular builders' guides of the day—indeed, queenposts were the recommended solution for long span roofs.

Over the course of the 19th century, queenpost trusses evolved considerably: the trusses of the 1797 Rindge Meetinghouse evoke English predecessors with their highly cambered tie beams and naturally curved oak main braces and straining beams, while the 1867 Stowe Community Church queenrod roof trusses use relatively little wood and make essential, efficient use of iron.

TRUSS VITAL STATISTICS						
Name	Date	Span	Truss Spacing	Pitch	Timbers	Iron Rods
Rindge Meetinghouse	1797	52'	13'-0"	6.67:12	14	0
Peacham Congregational Church	1806	48'-2"	9'-0"	6.2:12	8	0
Waterbury Center Community Church	1833	40'	10'-3"	7:12	10	0
Stowe Community Church	1867	50'	8'-0"	8:12	8	2

One post good, two posts better? To quantify the supposed advantage of queen over king, I constructed comparative Finite Element Analysis (FEA) models of kingpost and queenpost trusses, using configurations typical of the church roofs in our study: 50-ft. span, 7:12 pitch, trusses 10 ft. on center. In modeling trusses here, our governing load case was a balanced load combination based on 65 psf ground snow load plus the dead load of frame, ceiling and roof, mitigated and magnified by factors based on load duration, attic temperature, audience room capacity, wind exposure and probability of concurrent combined loading.

It was not surprising to find that axial tension in the lower chords (tie beams) and compression in the upper chords (principal rafters or main braces) was unaffected by the switch from king to queen. But maximum vertical deflection was reduced by 20 percent, and tie beam (lower chord) bending stress dropped by 40 to 50 percent. Compression force in the main braces decreased by 20 percent, probably a result of these sticks being more closely aligned with the direction of the forces they carry. Finally, tension in the posts fell by a third from 18,500 lbs. in the kingpost to 12,500 lbs. in each of the queenposts. This last decrease may be the most significant, since separation of post foot from tie beam is probably the most common joinery failure in traditional timber truss work.



Ed Levin

The essential core of the queenpost truss is a trapezoid founded on a long tie beam or bottom chord. In the force diagram above, darker shading indicates compression and lighter shading tension. Sloping inward and upward from the tie beam spring two main braces that rise to meet paired queenposts held apart by a horizontal straining beam. (In the case of queenrod trusses, the main braces meet the straining beam directly.) Load is applied by the

roof above and the floor or hung ceiling below, putting the composite upper chord (main braces and straining beam) into compression while the tie beam and queenposts are placed in tension. Earlier queenpost roof trusses, such as those at Peacham, Rindge and Waterbury (Figs. 7, 8, 11, pp. 15, 16, 18, respectively), complicate this structure by doubling the main braces with principal rafters above and, in the case of Rindge, by adding struts linking the more or less parallel inner and outer elements. The rafter-doubling strengthens the truss but also makes it more difficult to sort out the load path and quantify forces and stresses.

Taking Rindge as our example, the distribution of axial load between the main braces and principal rafters varies directly with the relative stiffness of the joints connecting rafter and brace feet with the tie beam (Fig. 8, p. 16). What do we mean by joint stiffness? Like the beams that they connect, joints are not infinitely stiff, but rather act as powerful springs linking the timbers. Pull on a beam and the joint securing it will open up a bit. Put the same beam in compression and the joint will tighten up.

Now, if there is a first law of framing, it is that load goes to stiffness. So, given a choice, axial compression in the upper chord of our queen post truss will prefer the path of greatest resistance. If we model the outer joint as significantly stiffer than the inner one, then the principal rafter carries the bulk (70 percent) of the axial force. This high compression load (around 36,000 lbs. in the Rindge model) delivers a strong inward push to the top of the queenpost where it cantilevers above the straining beam, causing a spike in post bending (up to 2000 psi). When the respective joints at the feet of the main brace and the principal rafter are equally stiff, the principal rafter still carries 64 percent of the load (about 33,400 lbs.) and queenpost bending is slightly reduced (to 1800 psi). In all cases, the predicted total load, the sum of the compression in principal rafter and main brace, is about 53,000 lbs.

As the inner joint at the foot of the main brace becomes stiffer than the outer joint at the foot of the principal, the main brace starts to pick up the lion's share of the force and bending stress at the head of the queenpost drops into the allowable range. But the bending problem has relocated rather than disappeared: now the considerable force in the main brace is delivering a jolt where it intersects the tie beam, with tie bending stress climbing rapidly. With the inner joint four times as stiff as the outer, the rafter and brace share load equally—and predicted bending stress in the tie at the unsupported brace joint exceeds 2000 psi. Double the inner joint stiffness advantage and the main brace has 62 percent of the load, treble it and brace load share rises to 72 percent, with tie bending stress climbing, respectively, above 2600 and then 3000 psi. Finally, when the outer joint at the foot of the principal loses all its capacity to retain thrust, the main brace carries the entirety of the axial load, and apparent bending stress in the tie spikes to an attention-getting 4000 psi.

Where along this theoretical load spectrum does the truth lie? The connections in question are blind mortises, those for the principal rafters located (often inaccessibly) at the extreme ends of tie beams, encumbered with secondary framing, sheathing, insulation and debris. So we have no way of knowing, but educated guesses are possible. To the degree that the builders understood the play of forces in queenpost trusses, they had to see the main brace as the preferred load vehicle, trading its secure inboard location (abundant relish opposing outthrust) against the risk of bending in the tie beam. Certainly the location of the principal rafter foot joint so

close to the end of the tie implies high risk of shear failure in the mortise and housing.

The results of our modeling exercise provide a clue regarding likely force disposition: where principal rafters shoulder the load, we would expect to see evidence of bending at the tops of the queenposts; where the main braces take the brunt, one might note a local sag in the tie beam. Our visits to historic trusses have disclosed several of the latter, none of the former. These findings combined with our modeling evidence lead us to conclude that main braces provide the primary upper chord load path for queenpost trusses.

Remember those big tie bending stress numbers in the model? What if this load is excessive? Why don't we see more substantial sag in tie beams? Might they not break altogether? The modulus of rupture of dry, clear Eastern white pine is 8600 psi, so the Rindge tie beams are probably not prone to failure even if the inner joints carry the entire load. We shouldn't expect to see main braces poking down through audience room ceilings. But why don't we find more—and more pronounced—sagging in tie beams?

One answer is that wood as a material, and timber framing as a structural system, have built-in load accommodation mechanisms. As a tie sags under brace compression, the brace in question falls away from its superimposed load, some of which then flows elsewhere in the frame, most likely down the principal rafter. This inherent load sharing maintains equilibrium by shunting force and stress between and among members in a reverse Robin Hood process, taking from the poor and giving to the rich, a classic case of load going to stiffness. A similar process equilibrates load at the joinery level, where connections can be out of alignment because of subsidence, shrinkage or poor workmanship. The resilience and relative softness of timber (compared with, say, steel) allows first elastic and later plastic deformation, bringing joint abutment surfaces back into alignment.

Another probable reason for better than expected truss performance is that our codes and computer models posit loading harsher than the real world conditions experienced by the buildings. In Rindge (Fig. 8, p. 16), we note a pair of short struts joining principal rafter and main brace opposite a single strut carrying on from brace to queenpost, elements that help support point loads delivered by the purlins to the principal rafter. At Peacham (Fig. 7, p. 15), these struts are absent, and the principal rafters must handle substantial point loads from the lower purlins (estimated at 4500 lbs.) on their own, resulting in model deflections of up to 1.8 in. and bending stress up to 2300 psi in the spruce principals. You would expect the magnitude of these results to indicate a kink in the rafter, but apparently the Peacham principals never got this memo, and deflection is undetectable with the naked eye. Could we have overdone the model loading? I don't doubt the 65 psf ground snow, but we're talking about a tall church near the top of a hilltop village in a windy place, and for some time now the church has had a metal roof. The rules mandate conservative loading, but how much snow stays on these roofs?

The unsupported purlin arrangement in Peacham is also found in Waterbury (Fig. 11, p. 18), but here the load effects predicted by the frame model are abundantly clear in the flesh. Here, truly, is the little roof that couldn't, a frame caught in the act of failure. So why does Peacham persevere while Waterbury comes to grief? In his discussion of Waterbury in the article just preceding, Jan Lewandoski pegs the villain by describing the church as "spanned by queenpost trusses of good material and joinery, but with disproportionately undersized main braces. . . a potentially fatal flaw."

Our analyses show the two structures with similar nodes of high bending stress, and accompanying deflection. But trusses are not about bending. By definition, a truss is a framework that bears its burden via axial loading, in pure tension and compression. In this capacity, timber (and indeed all framing material) is many times

stiffer than it is in bending. As a beam bridge, a 12-ft. 6x6 carries a midspan point load of 1000 lbs. with a half-inch of deflection. Stand the same stick up on end, load its top with half a ton and the newborn column will shorten a few thousands of an inch. Loaded axially the timber is a hundred times—two orders of magnitude—stiffer than it is bending.

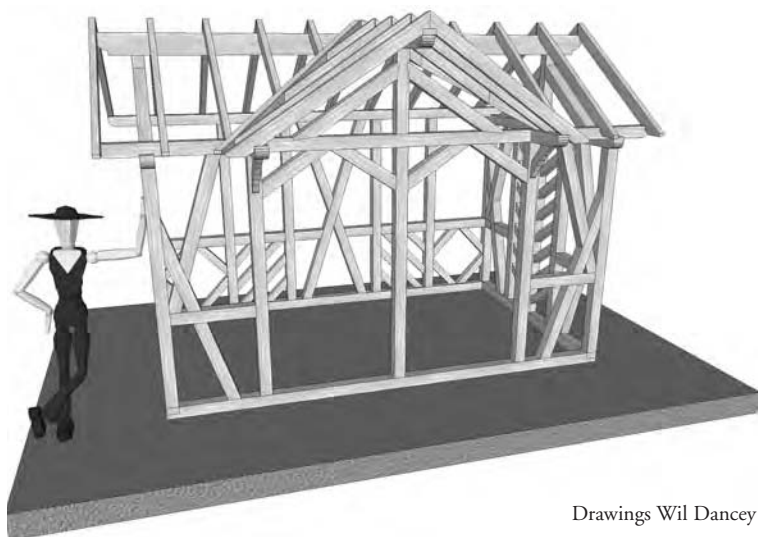
Conversely, consider the effect of an undersized truss member: in bending (beam action) it can cause a local distortion of the frame but, in buckling under axial load, it stands fair to bring down the whole structure. As specified by the *National Design Specification for Wood Construction (NDS)*, the controlling design value used to size a column or other timber loaded in compression is compression parallel to grain (F_c) as modified by the column stability factor (C_p). F_c specifies maximum allowable axial compression stress by species and grade. To avoid the tendency of long thin members to buckle in compression, C_p lowers the working value of F_c via a complex calculation involving stiffness, slenderness ratio (effective length divided by least cross-section dimension), end conditions and other terms (Sect. 3.7.1, p. 22). Additional equations come into play if the timber in question is loaded in bending as well as compression.

Survive running the *NDS* math gantlet, and you find the 7x8 main braces in Peacham rated for a load of 25,500 lbs., which compares fairly well with the predicted load of 27,500 lbs. At Waterbury, on the other hand, the 7x3 main braces are limited by code to carry a mere 2,100 lbs. This against the 29,000-pound load estimate from the FEA model! Compared in terms of stress values, Peacham's main braces are rated to carry 455 psi (down from a tabulated F_c value of 900 psi) and Waterbury's to carry 100 psi (much reduced from the tabulated F_c of 800 psi)—while dealing with a whopping 1385 psi axial stress! So the Peacham main braces weigh in working at 108 percent of capacity, while at Waterbury they struggle to fill shoes 1385 percent too big. Put another way, the Waterbury main braces are undersized by a factor of 14.

In contrast to Waterbury, consider the later, highly evolved queenrod truss at Stowe (Fig. 13, p. 19). Here the 6x6 and 6x7 upper chord members (main braces) seem dangerously light for their job. But both our structural analysis and the excellent condition of the frame belie this judgment. So well did the builders understand their business that load effects are the lowest of any of the frames we have studied. Maximum deflection is less than half an inch. Bending stress is close to zero, except for a jolt in the tie where the main braces land. Maximum tensile stress in the tie is 180 psi against a tabulated value of $F_t = 700$ psi. Maximum compression in the main braces tops out at 499 psi (tabulated $F_c = 800$ psi, calculated allowable $F_c' = 577$ psi, and maximum straining beam compression is 320 psi vs. tabulated F_c of 800 psi and calculated F_c' of 389 psi. In sum, under the biggest load we can throw at it, the Stowe truss operates at 25 percent of capacity in tension and 82 percent to 85 percent capacity in compression, and does so using 55 percent of the volume of material employed at Peacham and 65 percent of the total used in Rindge (material percentages adjusted for truss spacing).

Two posts better, redux. Turning back to the Rindge and Peacham FEA models, it's worth pointing out that, despite the high compression heroics in the upper chord components, tension in the queenposts never exceeds 12,000 lbs. at Rindge and 7000 lbs. at Peacham. If the genius of the scissor truss is its ability to carry significant tension load in its lower members without resort to tension joinery (see TF 69), queenpost trusses approximate this tactic, with joints everywhere in compression save at the post feet. (Of course, queenrods, as at Stowe, are in tension top and bottom.) Thus we discern a primary survival strategy of the queenpost truss: eliminate all tension joints apart from post feet and minimize joint tension there by doubling the tensile element. —ED LEVIN

Framing the Montebello Pavilion



Drawings Wil Dancy

Fig. 1. The Montebello Pavilion frame, with virtual Zimmermann.

THE idea of our workshop at the Guild's 2003 Eastern Conference at Château Montebello in Québec was to build a little *Fachwerk* pavilion using the basic structural elements and joinery of German carpentry in a design that could be executed in three to four days by a handful of students. We got the best students we could think of, and we ended up with over 140 pieces to be cut.

The structural framework. The basic structure is taken from typical historic design of framed houses. The gable walls, including sills, sit over the sills of the lengthwise walls, and the top plates connect by a dovetail lap to the gable walls. To make the assembly of the building easier, we designed the gable walls as independent units. They can be preassembled and hauled into place.

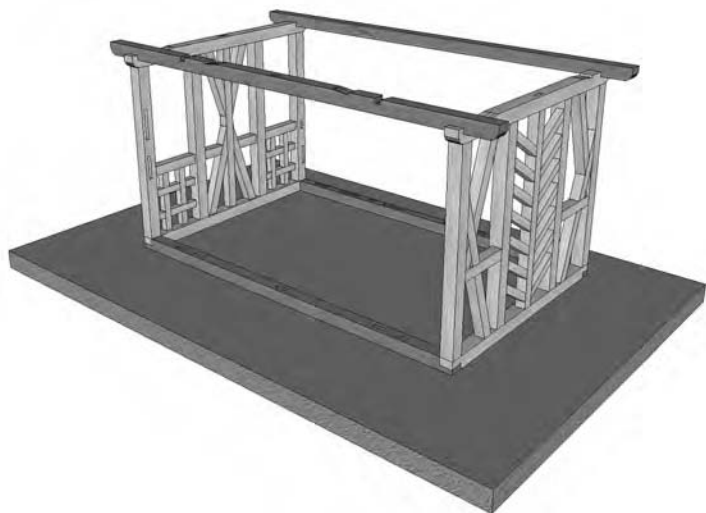


Fig. 2. The gable walls, demonstrations of traditional brace patterns.

The gable walls are stiffened panels. Each has an oversized structural element to stiffen it in plane and to deal with shear loads. One gable wall uses a half-*Wildermann* (wild man) brace scheme at the corners with *Gräten* (fish bone) bracing in the middle; the other uses a *Weible* (woman) brace pattern in the middle with simple square bracing below the rails. The ridge beam, supported by little kingpost trusses, directs much roof loading to the gable walls.

The platform-oriented system makes the assembly of the whole building fast and easy. No assistance is needed to put in the wall posts on the side walls as the rails keep them from falling. The final step is to place the top plates, which lock the walls together. It took us about 30 minutes to raise the entire frame in the lobby of the hotel, including the roof.

Except for the dovetail lap joints on the plates, the traditional *Fachwerk* joints are compression-only connections. The entire building has eight pegged tenons and two wedges in a scarf joint. Everywhere else, self-weight and loads keep the joints tight.

The idea of long bracing in the gable walls. The advantage of long foot-braces instead of the short head-braces familiar to North Americans is that the foot-braces transfer loads directly to ground (the sills). Foot-braces, tenoned at both ends, stand at an elevation of 70 to 75 degrees and connect the main posts of a wall to the sills; they can also extend full height to connect wall plates to sills. Compared to a short head-brace, a long foot-brace reduces leverage on the post and minimizes bending. The arrows indicate where loads are transferred into the sill (Fig. 3). Crossing braces are half lapped, as are the lower brace and short rail (Figs. 3 and 4).

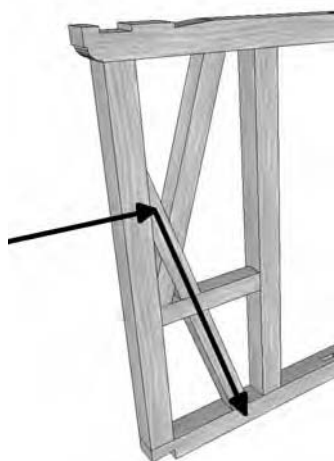


Fig. 3. Representative load path at a foot-braced post.



Fig. 4. K-Runes brace pattern at a corner post.

The head-braces in this building are more ornamental than structural. The set of head- and foot-braces with rails, repeated around a corner (Fig. 4), presents *K-Runes*, a special case of the traditional bracing pattern *Wildermann* (wild man), the latter usually found in one plane and without rails at a median wall post.

The Weible. The *Weible* pattern is structural as well as ornamental. As distinct from the *Wildermann*, the head-braces and foot-braces don't cross and they connect directly to the post. (This pattern, too, can also be used around a corner.) The joinery of a *Weible* can vary. In our case (Fig. 5 overleaf), the braces are tenoned into the plates and half lapped to the center post. The half laps are held in place with pegs. On older buildings, you might see an ornament, perhaps a bull's head, carved into the post to show where the braces meet.

The long walls. The long walls stiffen the building lengthwise. On the front wall, the two foot-braces at the corners transfer shear force to the foundation (Fig. 1). The short braces over the openings work as door heads; structurally they are redundant. The ridge of the dormer is supported by a kingpost truss (Fig. 1) and dove-



Fig. 5. The Weible (woman) bracing pattern.

tailed into the main ridge (Fig. 7). At the eaves, the dormer rafters are supported by two short plates dovetailed into the top plate of the wall (Fig. 7) and held up with braces (Fig. 1). On the rear wall, the main structural element is the Andreaskreutz (St. Andrew's Cross) in the center of the wall (Fig. 6). The infill pieces are traditional ornaments and don't have much effect on the structure.

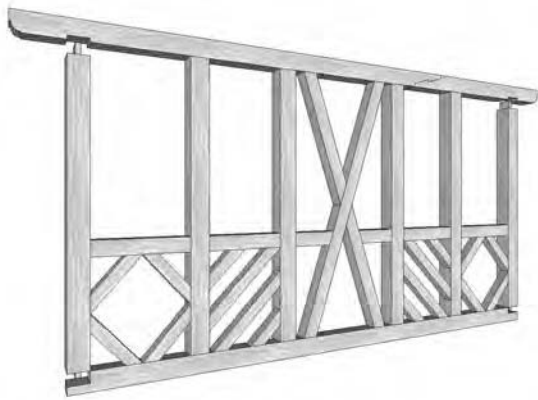


Fig. 6. The long wall (rear) stiffened by central St. Andrew's Cross.

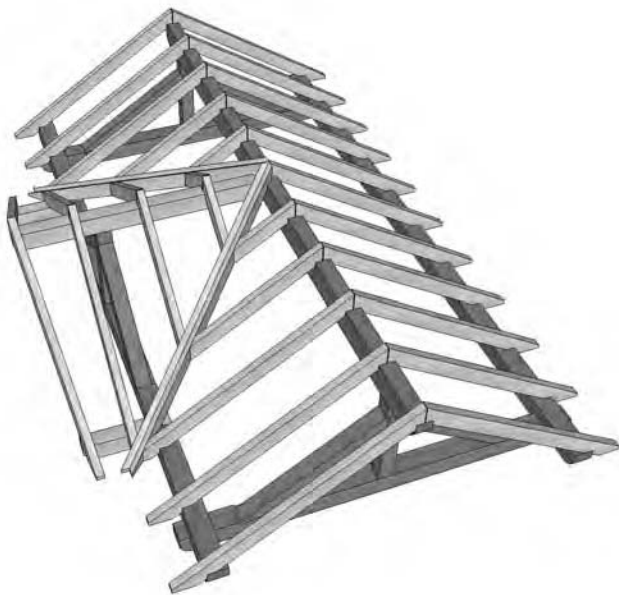


Fig. 7. Roof framing. Common rafters, birdsmouthed at both ends, do not abut at ridge. Dormer ridge is dovetailed to main ridge.

The roof system. The roof is a standard design still very common in Germany. The rafters are birdsmouthed to the ridge and plates and so transfer gravity and snow loads directly to the plates and supporting posts. To avoid any outward thrust whatsoever, the ends of the rafters do not touch at the ridge; each rafter behaves like a shed rafter. The deliberate air gap is only close at the outermost rafters, for appearance. The rafters are fixed with long screws through the birdmouths into the plates.

Shear in the roof is taken by the two valleys. The valley rafters are fixed to the plates and ridge in the same way as the common rafters; the jack rafters are screwed to the valley rafter as well as to the ridge.

Even today, most roofs in Germany are cut in this fashion. Nailed or plated trusses are, by far, less popular for residential buildings than in North America. The German preference for traditional roof construction has led to the development of fairly straightforward roof joinery.

Most of the rafters for our pavilion were gang cut in strapped-together groups—including their birdmouths, swiftly made with a dedicated shaper. The valley rafters and the barge rafters (the visible end rafters) were single pieces cut individually.

Insight into rules of building. Most of the joinery in traditional German carpentry depends on ratios, not absolute dimensions. The proportions derive from experience and rules of thumb for basic engineering. For the mortise and tenon joint, a division into three equal parts, adjusted for convenience, is common. For example, on a 10cm x 10cm (4x4) post, the tenon thickness would be one-third of the timber thickness, or 3.33cm. This dimension is then rounded to the next even value, 4cm, to simplify the work and to coordinate with popular layout tools graduated in whole centimeters.

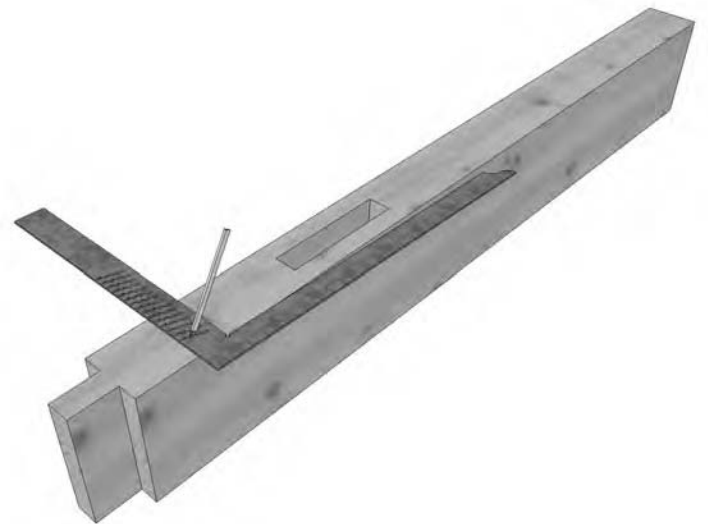


Fig. 8. German layout square perforated for marking at standard joint dimensions.

Timbers come with variations in thickness and width. To accommodate these variations, the following rules are applied:

1. All layout is done from the reference face of the piece, which, according to the case, might be the top, the outside or the visible face.
2. For most posts, rails, and braces, all tenons are cut 40mm (about 1⁷/₁₆ in.) from the face side, 40mm wide and 40mm long.
3. On wall posts except corner posts, tenons are 80mm (about 3¹/₈ in.) long instead of 40mm.

Generally, the joints are not housed for appearance since, in traditional *Fachwerk*, timbers are exposed to the outdoors and stay at a rather stable moisture content, therefore moving much less than timbers exposed entirely to the drying conditions indoors.

Following our rules, it was quite easy to lay out all the pieces in full scale. For training purposes, we laid down white-painted sheets of plywood on which the wall elevations were drawn at full scale. The lengths and cutting angles were transferred from the layout to the appropriate piece of timber, and the joinery was laid out using the protractor as necessary.

We agreed on two conventions so that it wouldn't be necessary for the same person to do the layout and the cutting—a single X to mark the part intended to be cut off, and a double X on a line (XX) to indicate the cutting line at the end of a piece. In a traditional German timber framing shop, you would find one or two highly qualified masters doing the layout and some apprentices or regular framers doing the cutting. The lofting method was used in the dark old times when nobody worked from proper drawings. Today almost every German timber framer uses a CAD system to produce scale drawings for the guys in the shop, with fewer mistakes and more efficiency. The lofting method is still used for educational purposes, to understand 3D contexts, especially of roofs. Today, we take advantage of a simple-to-use software module to do all kinds of tricky roofs.

The marking system. The marking system we use in German carpentry is based on Roman numerals. Some are altered to make them easier to chisel out or hew with an axe and to prevent misreadings. Numbering starts at the southwest corner of the building looking north. The corner post gets a I, the next post is II, the next III and so on. To identify the wall and floor for the piece, runes are added to the number.

The *Rute* (slash) indicates walls running lengthwise. You start counting at the outside left wall of the building as you face one corner. The first wall is marked by one slash (/), the next parallel wall to the left is marked by two slashes (//), the next by three (///) and so on.

The *Ausstich* (flag) indicates walls running crosswise. Start counting at the gable wall connected to the first long wall. The gable wall gets one flag, the next parallel to this two flags and so on.

The floor or story level for a given piece is indicated by the *Stockzeichen* (solid Delta). There is no story sign for the ground floor. The second story (in US parlance) gets one Delta, the third story two Deltas and so on. Germans count stories beginning above the ground floor. Here are some examples.



First post, second long wall, first story.



Second post, second cross wall, ground floor.



Second post, second cross wall, second story



First post, first long wall, ground floor.

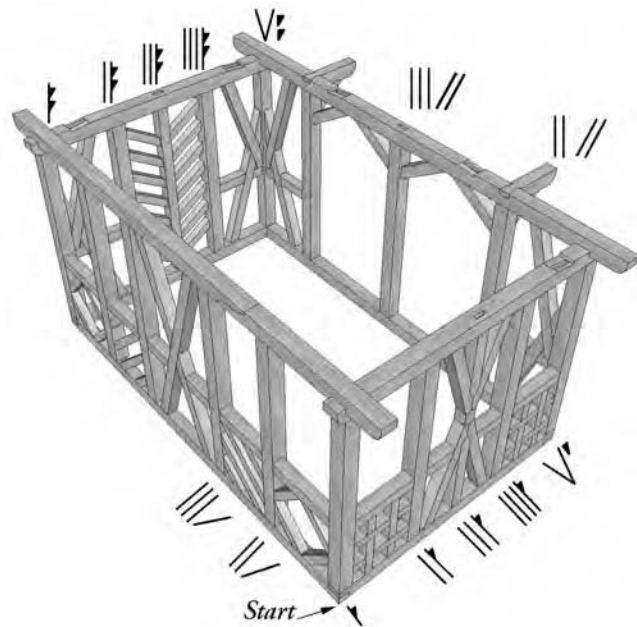


Fig. 9. The completed wall framing with post locations flagged.

Carpenter's marks are always located on the face side of beams (as previously defined), near the bottom ends of posts and at the "beginning" of horizontal members. This means that long wall members are marked at the bottom left end with respect to the observer at the first corner. Cross wall members are marked at the bottom right end.

The wall plates are numbered like the posts. So the sill in long wall I will be marked I/. The top plate would be marked II/. It's up to the framer on site to recognize whether he's standing in front of a plate or a post.

Rails are numbered the same way. The rail that belongs to the first post I/ is numbered I/ as well. The second rail is marked II/ and so on. Again, it's up to the framer on site to identify the piece as the rail and not the post or plate.

Foot-braces are numbered like posts. The matching head-brace gets the same number as the down-brace but with an additional mark (e.g., a carat, >). In some regions those braces are numbered like rails and up-braces have no additional mark. Whichever way you go, as each series of marks (post-rail-plate) is unique to one building, there is only one way to put the pieces together. There is no need for a detailed drawing or explanations on site, an important advantage in the old days when most people could neither read nor write. All they had to do was to sort and assemble the pieces by matching the marks and knowing the difference between a post and a brace, or a brace and a rail. Some corner pieces might end up with two sets of marks (although this practice is avoided when possible). As the marks are also used to sort your timbers on site, it makes sense to place the wall number first, following the right sequence of the assembly of the building, but again there is no law to follow. Fig. 9 shows in part how we numbered the workshop frame. It might be helpful for its new owner, Paul Cooper, who won the frame at auction and later ended up with all the individual pieces on his truck.

—OLIVER AMANDI AND MARKUS BRUNN

Oliver Amandi and Markus Brunn of Dietrich's NA (software), together with the Gewerbe Akademie of Rottweil, Germany, ran a workshop at Montebello '03 on traditional German layout and joinery. The instructors were master framer Filippo Campagna of the Academie and Wil Dancey of Dietrich's. The design, by Markus Brunn of Dietrich's, took about five hours to develop. A set of drawings for the building, including the timber list, is available in PDF format from checkout@dietrich's.com.



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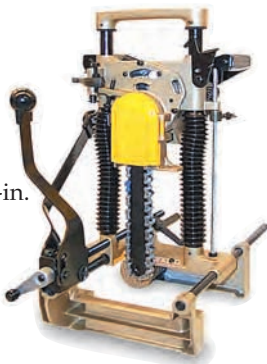
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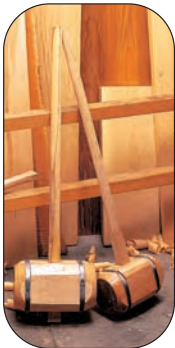
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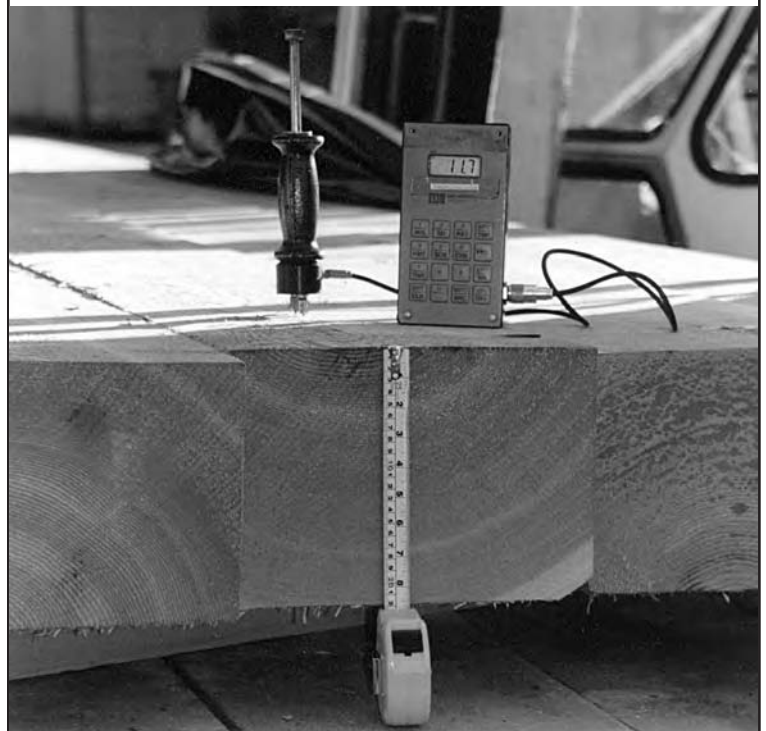
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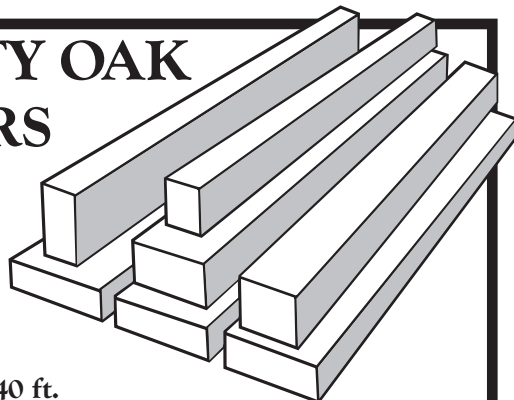
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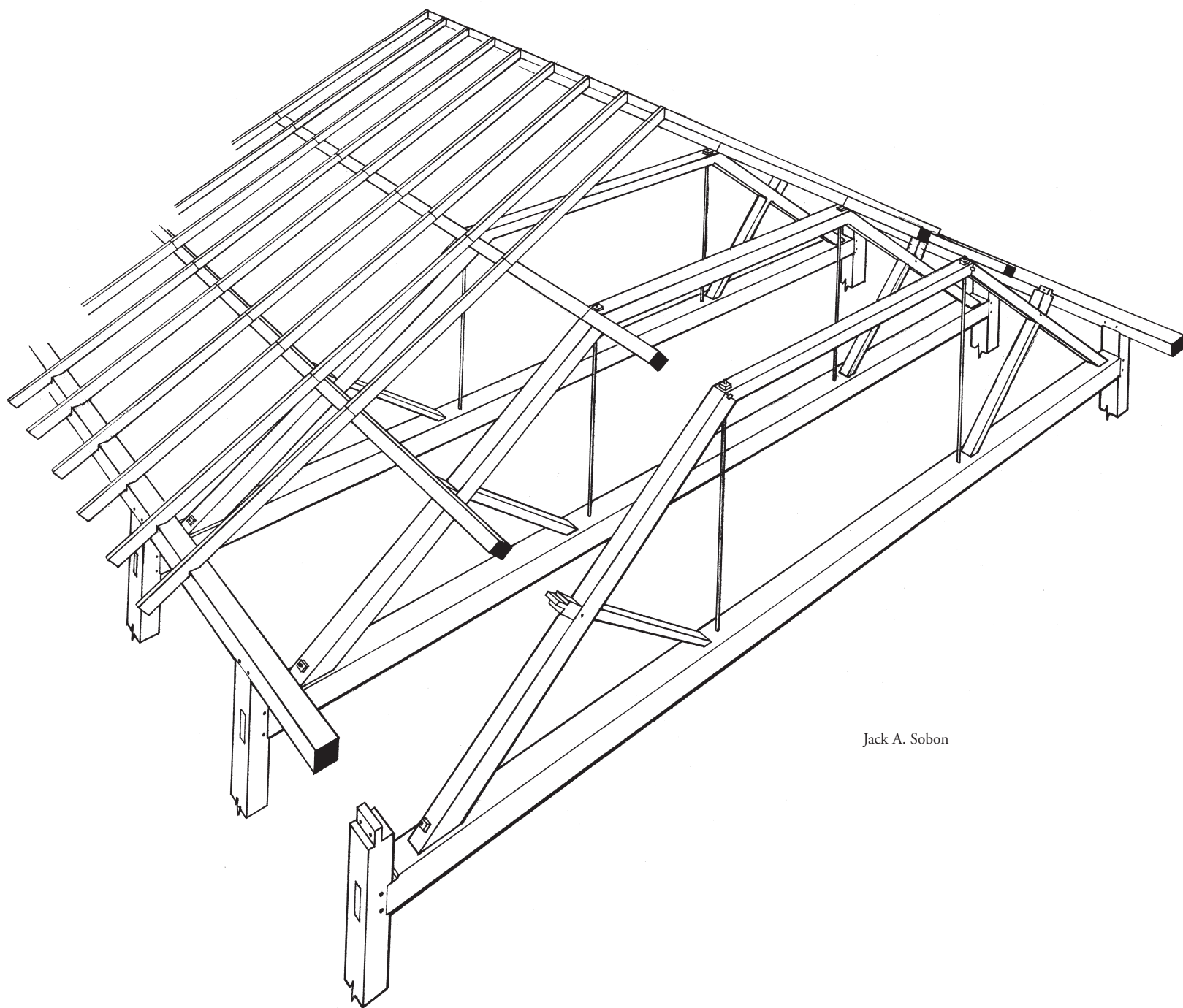
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Jack A. Sobon

Perspective view of 50-ft. queenrod roof trusses at the Community Church, Stowe, Vt., 1867. Framing is as light as possible, with simplified joinery and effective use of iron in tension. See page 12.