TIMBER FRAMING

JOURNAL OF THE TIMBER FRAMERS GUILD

Number 87, March 2008

Reproducing a Burned Steeple

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CONTENTS

On the front cover, winding trail of debris at the grounds of the Weathersfield, Vermont, Meetinghouse, built 1826. In 1984 a consuming fire in the meetinghouse left the brick walls standing and charred timber remains. On the back cover, steeple frame, reproduced in 1986 from analysis of the remains and documentary evidence, is lifted to the meetinghouse. Story page 16. Photos by Jan Lewandoski.

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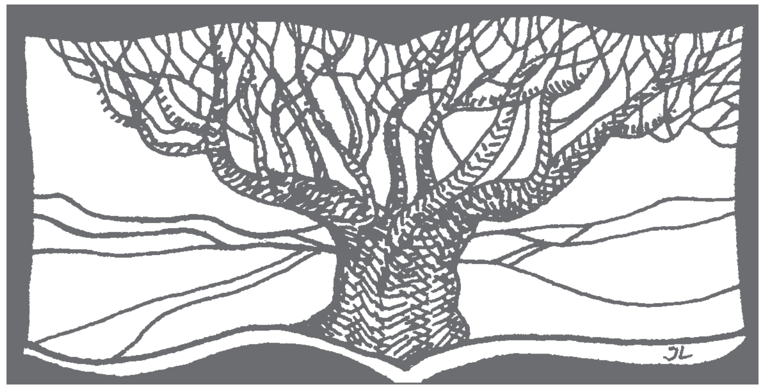


BOOKS:
Intermountain Travel Guides

Historic Barns of Northern Utah: A Self-Guided Tour, by Lisa Duskin-Goede and Elaine Thatcher, and *Historic Barns of Southeastern Idaho: A Self-Guided Tour*, by Lisa Duskin-Goede. Logan, Utah, Bear River Association of Governments, 2004, 2007 (*Utah*), 2007 (*Idaho*). 8½ x 11 in., spiral-bound. *Utah*, 64 pp. *Idaho*, 60 pp. Copiously illustrated. $15.00 each plus $2.13 postage ($2.47 for both). Available with check or money order from Bear River Heritage Area, 170 North Main, Logan, UT 84321.

THESE well-written guidebooks give a concise introduction to the history of farm building design and construction, which applies not just to Idaho or Utah but also to all of North America. They take you on a photographic and written tour of farm buildings of the intermountain region of northern Utah and southeastern Idaho, and they take you back in time to when the barns were full of life. They also provide maps for a driving tour, showing the building locations, but with instructions to drive carefully and respect private property. Only selected barns were included in the books: there are many more barns in the intermountain West than these.

The stories with each building give firsthand accounts of farm life and history, building construction and reuse, renovations and preservation. Interesting tidbits are found throughout the stories, for example this one from the Bend in the Road barn (*Utah*, p. 46): “Mr. Anhder believes the barn was originally set up as a horse barn because of the horizontal placement of siding up to the level of the loft floor, and the tight tongue-and-groove construction of the loft floor, ensuring a cleaner lower loft for the housing of valued animals. Anhder says ‘My grandfather always told me that you can walk into a barn—if it is tongue-and-groove on top, you know they’ve got horses.’ ”

The Amos R. Wright barn (*Idaho*, p. 52), possibly built “as early as 1870,” is half log and half framed with vertical board sheathing (although the book refers to the board sheathing as “vertical planking”). This barn comes with an interesting story of settlement, living, struggle against the elements and creative use of one’s resources.

The books discuss styles and types of farm buildings such as sheds, granaries, stables, English barns, the “northern European two-story barn,” intermountain barns and “20th-century specialized” barns such as dairy barns. They also discuss or show photographs of building methods and materials including timber frame (“post and beam”), transitional framing, balloon framing and “panelized” framing (post-WWII military surplus wooden crates). There are also a mail-order barn from 1914, bank barns, stacked 2x4s, hewn logs, round logs, stone, brick, dugouts and corrugated metal! The lean-to in the Andrew Stewart Heggie barn (*Utah*, page 58) even has a rare cobblestone floor.

At least one granary in the books was used by the Women's Relief Society of the Church of Jesus Christ of Latter-day Saints to store grain for times of hunger. It's little known that there are tithe barns in the United States and, although they aren't specifically identified in these books, every Mormon community had a tithing office which may have included barns and granaries. Extremely few tithing barns survive since the Mormons went to a cash tithing system around 1908, putting tithe barns into private hands.

Watkins barn, Mendon, Cache County, Utah, ca. 1922. Horizontal sheathing is atypical.

A few examples of unusual buildings photographed and described include an externally framed round wooden granary with a hexagonal pyramid roof, a brick barn with an arched roof (not a gambrel roof), an octagonal stacked 2x4 silo and a hewn-log full-dovetail granary—even a railroad-tie barn with a sod roof. Of course, there are gable, gambrel and monitor roof barns, too, and a photograph of a Mormon hay derrick. The books can be viewed online at bearriverheritage.com. They are well worth buying even if you never expect to travel in the Bear River Heritage Area. Though presented as travel guides, they are uncharacteristically rich for their genre and excellent examples for other barn enthusiasts and historians to emulate. Every region in the USA should produce books like these. Happy trails. —Jim Derby

Jim Derby (jim_derby@verizon.net) is a restoration carpenter in Waldoboro, Maine.

Hans Sorenson's stacked 2x4 and 2x6 10,000-bushel hexagonal grain bins, Soda Springs, Caribou County, Idaho, ca. 1925.

Marion McBride's squared log dovetail chinked granary, Cache County, Utah, early 1900s.

Long Truss Bridge Framing

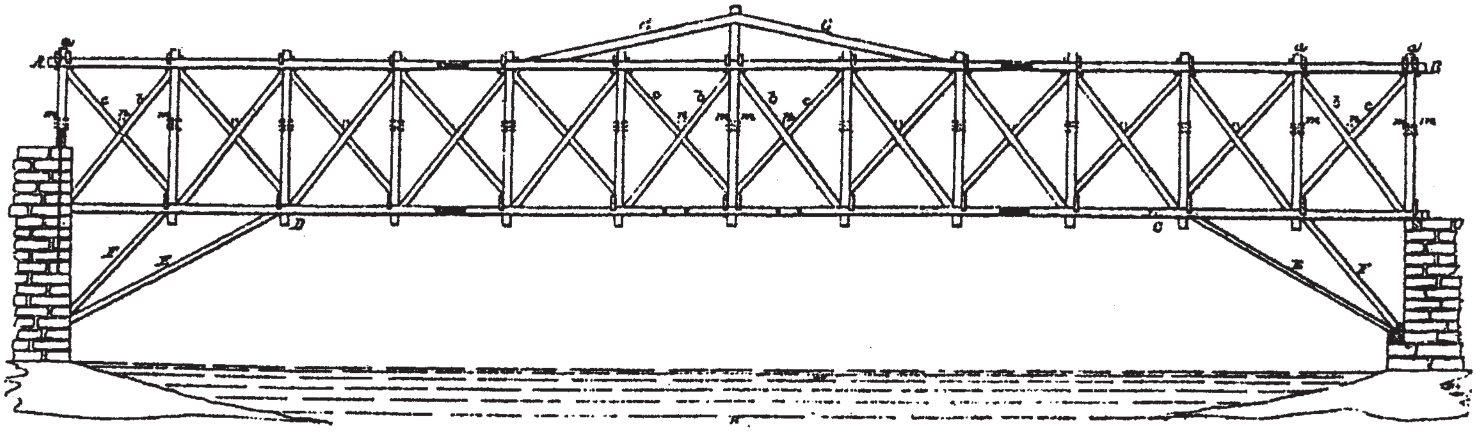


Fig. 1. Col. Stephen Harriman Long's patent, 1830, main drawing.

TIMBER-FRAMED bridges faced new demands with the development of railroads. Increasing loads required more rigidity than ever before. An innovative response to these demands was the Long truss, patented in 1830 by Lieutenant Colonel Stephen Harriman Long (1784–1864), an Army engineer. His truss used traditional joinery in new ways to produce a bridge that would not deflect under load.

The Army was much involved with large-scale transportation issues in the early United States. Before working on his bridge truss, Long commanded an exploratory and scientific expedition as far west as the Front Range in Colorado. Longs Peak, the highest in Rocky Mountain National Park, is named for him. Few examples of his truss survive today, but in his lifetime the colonel was well known and well connected among American bridge builders. Stephen Daniels was a Long patent agent in Marietta, Ohio; his son J. J. Daniels went on to become one of the Midwest's best-known timber bridge builders, although he chose to work with the Burr truss instead. Long himself later patented several other bridge designs, but his original patent of March 6, 1830, was the only one that was influential, and this is the one we generally mean when we say "the Long truss" (Fig. 1).

Before considering the details, note the little kingpost frame above the top chord and the reinforcing struts at the ends under the bottom chord. The colonel was aware that compressive stresses in the top chord and tensile stresses in the bottom chord increase toward the center of a bridge. The special features were intended to reduce the concentrated stresses, by setting up forces in the opposite direction, thus in effect spreading the load more evenly throughout the span. The thought is sophisticated, but in practice it complicated the framing. Except on a few early bridges, these features were usually left out. Long acknowledged this fact in later editions of his writings.

The Long truss consisted of panels each with paired posts, paired braces, a single counterbrace running in the center plane between the braces and three-part top and bottom chords. Timber was of modest dimensions, often either 6x6 or 7x7, except that the center component of the chords was a little wider to make up for section loss due to notching. Bridges of longer span used larger timber. Framing details were traditional throughout. Chord splices were handled with extra pieces using a stop-splayed scarf with multiple tables. The three members of the chord were spaced slightly apart using shear blocks. Joints between posts and chord segments were notched into each other on both faces.

The brace-post joint originally used a double table, the ends of which were tucked between the chord segments in Long's conception, although later builders often modified this joint. The colonel personally preferred white pine for most of his truss timber, and he wanted it quartersawn. He mentioned some modern details such as the possibility of using iron instead of timber for splice plates, and he recommended placing sheet metal in timber joints, but such modifications were probably little used. He specifically stated that bolts in his design were not intended to be load bearing, but meant only to clamp timber parts together firmly.

In the description so far, we do not see anything that seems original enough to warrant a patent, a judgment actually made by engineering historian J. G. James in a 1982 article on wooden bridge trusses.[1] But modern commentators often miss one of the main points of the design: it was prestressed to contain the equivalent of a full load even when the bridge was not loaded. To understand this concept, which Long himself admitted appears "paradoxical," we turn to a small drawing that he published in 1830 (Fig. 2).

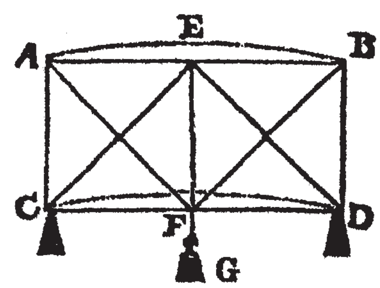


Fig. 2. Col. Long's 1830 diagram explaining prestressing.

In the drawing, the upcurved lines at AB and CD represent the original position of the chords before the truss is loaded, and before the counterbraces AF and BF are inserted. The load G is added so that the bridge deflects into a level position represented by the straight lines at AB and CD. Then the counterbraces are inserted and the load is removed. The counterbraces prevent the truss from springing back to its original position. Any load the bridge later carries (up to the original weight of G) will only shift stress from the counterbraces back to the braces EC and ED. Of course this explanation is somewhat simplified; allowance must be made for compression of the counterbraces themselves when they are loaded. The prestressing feature made Long's patent original.

In actual building, the prestressing was achieved by using wedges permanently installed over the heads of the counterbraces. The wedges were driven in far enough to produce the desired effect upon completion of the bridge. There is some question how long the effect lasted, given aging and creep of the timber. Regular maintenance would have been required, and surely this detail was forgotten long before the 20th century and its engineering commentators came along. But there is no doubt about the ingenuity of the concept.

The framing of the counterbrace can be inferred from Figs. 3 and 4. The counter is wider than the space between the posts and reduced at the ends to form shoulders that bear against the posts, leaving a tenon to fit in between. At the top, the prestress wedge fits between this tenon and the chord. At the bottom, the counterbrace tenon sits on a small spacer block inserted between the posts, at the same level as the floor beam. When the wedge is driven, either at top or bottom, there is some outward thrust on the posts, but since the panel shape is taller than it is wide, the majority of the thrust is transmitted in the direction of the chords.

The Long truss was used for highway bridges as well as for railroads; all of the existing examples are highway bridges. Later builders often modified the design. The colonel called for the prestress wedges to be at the top of the counterbraces, but adjustment would obviously be easier if they were at the bottom, and some builders made this change (Fig. 4). Occasionally, bridges did not have prestress wedges at all, and had the counterbraces firmly fixed at top and bottom. Their builders apparently did not understand the basic concept even if they followed the general profile of the truss otherwise. Nichols M. Powers's celebrated Blenheim Bridge in Schoharie County, New York, is a modified version of the Long truss; its single span originally measured 210 ft. in the clear, although it has been shortened by recent repairs. The prestress wedges are placed transversely to the chords, an unusual practice. Blenheim is a double-barrel bridge, that is, it has two lanes for travel with a center truss down the middle. This truss reaches to the roof ridge and has an arch incorporated into its central plane. It would be hard to say how the arch interacts with the prestress principle, but it is a very successful bridge.

The nation still has about a dozen Long trusses, although the exact number depends on how much deviation from the patent type is allowed while still calling it a Long truss. Many guidebooks refer to any truss with X-panels and timber posts as a Long truss, and this is misleading. The term is loosely used even in some 19th-century sources but, as J. G. James observed, a generic truss with X-panels could not be patentable. Col. Long's significance is in the originality of his prestress concept, using traditional timber framing to address the special engineering challenge of producing bridges that would not deflect under load. —JOSEPH D. CONWILL

Joseph D. Conwill, of Sandy River Plantation, Maine, is a photographer and editor of Covered Bridge Topics*, as well as author of several books about covered bridges. He has visited every covered bridge in North America. His previous articles in this journal have treated the Paddleford truss (TF 75), the Burr truss (TF 78), and the late versions of the Howe truss (TF 85).*

[1]*Journal* of the Institute of Wood Science, Vol. 9, No. 4, p. 176.

Joseph. D. Conwill

Fig. 3. Framing details of Long truss. Thin ends of counterbrace prestress wedges poke out between braces. Behind the near post an additional wedge can be seen intended to snug up the housed joint, a detail often omitted. Low's Bridge shown, between Guilford and Sangerville, Maine, built in 1857 by Leonard Knowlton and destroyed by flood in 1987, when it was the extant Long truss most closely resembling the patent (though the colonel would have recommended more post-top relish). Jan Lewandoski built a new Long truss bridge here in 1990.

Fig. 4. Counterbrace wedge at the bottom instead of the top. Blair Bridge, Campton, New Hampshire, built 1869.

Basic Design Issues in Timber Frame Engineering II

IN the first part of this article (TF 86:16), we discussed the engineering method generally, common methods for supporting floor loads and specific strategies for handling simple gable roof loads. Before we leave our discussion of roof framing, we should talk briefly about hips and valleys. In a regular square hip roof, where there is no ridge (Fig. 1), it's not hard to see that the opposing pairs of hip rafters function much like simple rafter pairs. In this case the necessary tension tie is provided by the joined and restrained plates, which function as a tension ring.

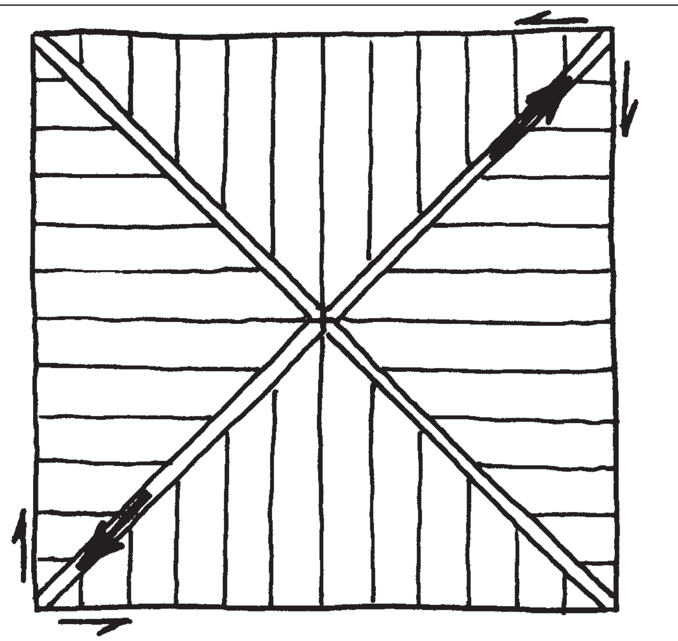
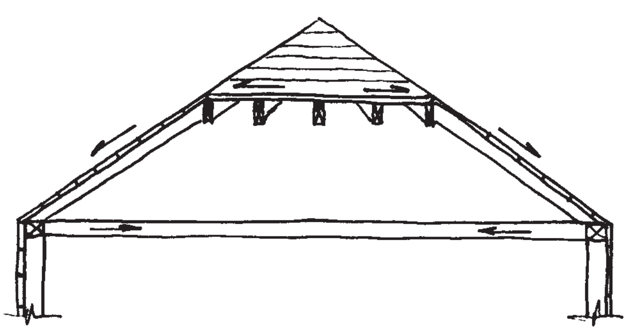


Fig. 1. Ridgeless hip roof frame, its rafter action resolved in the plates.

Hip rafters do not necessarily need to be sized to handle the full gravity load of the jack rafters they appear to support. We know that in many old houses the hip rafters are not much bigger (if at all) than the jack rafters that frame into them, despite the seemingly much larger bending and shear loads they have to support. Yet most of the time they perform pretty well. How come? The roof sheathing and jack rafters must be working together with the ceiling framing to form a kind of arch or truss (Fig. 2).



All drawings Tom Nehil

Fig. 2. Sheathing works with framing to form a kind of arch.

When the roof framing is open, as we often see in timber frame buildings where no ceiling joists tie the feet of the rafters, the jack rafters and hips in the system function more like stiffeners to brace the roof sheathing, which becomes a three-dimensional shell or folded plate.

Hip roofs work on rectangular-shaped buildings as well as on square plans; the arch action is still there. The opposing rafter pairs that frame into the ridge along the main roof of the building, however, still need to be designed using one of the strategies previously discussed for gable roof framing.

Valleys are in some respects just upside-down hips; rather than throwing the roof sheathing into compression as do hips, valley rafters pull on the sheathing as they sag under load. Nevertheless, to simplify design and to be conservative, we usually design both valley and hip rafters to support their full tributary area roof loads, especially in open timber-framed roofs.

We left the discussion of hips and valleys for last in the roof-framing section because it leads us to think about our buildings as three-dimensional assemblies, where the sheathing or skin functions as a part of the structural system. We are no longer looking at our buildings as simple two-dimensional assemblies. Such three-dimensional thinking is exactly what we need when it comes to the issue of dealing with lateral loads.

Strategies for Resisting Lateral Loads. Lateral loads are imposed on our buildings by wind or, in some cases, by seismic activity, but they can also be caused by unbalanced snow loads. Asymmetric frames also have a tendency to drift sideways under gravity loads.

Wind loads are defined by the building codes, as are the forces resulting from ground accelerations. Of all the code requirements, lateral loads are perhaps the most difficult to understand—and to believe. We have looked at many timber frame barns, relatively simple and easily-understood structures, and found they cannot be shown to be capable of resisting full code-required wind loads. Thus a considerable amount of retrofitting is necessary when a barn is to be converted to residential or commercial use. Yet such barns have stood for over 100 years without collapsing or lifting off their foundations despite a lack of anchor bolts. Some old barns will even sit stably for years with no hay stored inside to serve as ballast and with the barn doors open, a so-called "partially enclosed structure" that acts something like a parachute.

It's often difficult for us structural engineers to justify code lateral load requirements in light of such performance. Even so, code wind loads are based on meteorological records and physical measurements of pressures on buildings, and are therefore more than just extravagant guesses. We all have to remember that code requirements are intended to make our houses or commercial buildings safe shelters even in fairly extreme weather conditions. Because of code limits on the stresses we can apply to our framing members and limits on deflection or sway of the frame, the building needs to come through these extreme weather events without much swaying or damage to interior finishes. If you design and build to meet the code requirements for lateral loads, you can feel pretty safe in your building during a storm.

The lateral loads applied to buildings from wind are a function of the wind speed. Maximum design wind speeds are defined in the building codes for various areas of the United States. For design purposes, most of the interior of the country is classified for a 90-mile-per-hour, 3-second-gust maximum wind speed. (Note that we do not try to design for tornados since these are considered too unlikely an event for any individual building, and economically impractical to design for.) On the other hand, design wind speeds along the Gulf Coast are upward of 120 mph. That may not sound like a big increase from 90 until you realize that the pressure the wind exerts on an obstacle in its path, such as a timber-framed building, is proportional to the square of the wind speed. A 120-mph wind thus exerts almost twice as much lateral load on a building as does a 90-mph wind.

So how big are the code wind loads? Let's say you are building a two-story Colonial 30 ft. by 40 ft. with a 12:12 pitch roof, and your building will be in a 90-mph wind speed region in fairly open terrain. The pressure a 90-mph wind applies to your building will be on the order of 15 to 20 lbs. per square foot of vertical sail area. This can quickly add up to a lot of lateral load—you could be looking at 7 to 8 tons. Clearly you need to design to resist these racking forces.

We have two basic strategies for resisting lateral loads in timber frame buildings: frame action, where the racking loads are resisted by the frame using knee braces, full-height diagonal wind braces or even posts cantilevered up from the foundations; and shearwalls. Let's look at the specifics of these strategies.

Frame Action. In a pure timber frame structure such as the typical 19th-century American barn, we usually see numerous relatively small diagonal members connecting posts and beams, termed knee braces (even though they are not actually made from natural-grown knees) to distinguish them from long, wall-height braces typical of other framing traditions. In American timber framing, these knee braces evolved to a standard size, often 4 in. wide by 3 in. deep in the Midwest (vs. 3x4 in New England), with vertical and horizontal runs both at 36 in. Often these braces were not pegged in place but simply held in position by their housings and confinement by the timber frame around them. This configuration has been described as "compression-only" joinery.

What happens when we try to rack a knee-braced frame? The corners formed by the posts and beams change from 90-degree angles to something less on the leeward side and something greater on the windward side. As the angle tries to close on leeward side, the knee brace is put into compression and resists closing of the angle. Remember from our earlier discussion that a knee brace pushes not only down but sideways as well, thus putting the joint between the post and beam into tension; we maintain there is really no such thing as compression-only joinery. As the knee brace is very stiff in compression, the angle is maintained pretty close to its original 90 degrees. The post and beam, however, bend around the knee brace as shown in Fig. 3.

The bending of posts and beams is significant in a large frame despite their hefty cross-sections. If we increase the size of our knee braces so that we have room for decent tenons and good-sized pegs, we can start to develop tension joinery on the windward knee brace and thereby get both the windward and leeward sides of the frame working to resist the racking. On the windward side, however, we have not only the flexibility of the post and beam to consider but also the flexibility of the pegged joinery.

All these effects taken together, a simple knee-braced frame is very flexible. You have probably noticed this on small frames, where it's not hard for one person to get the frame rocking back and forth. Big frames with heavy members and large-diameter pegs such as 1½-in. are still flexible. Though not much of a concern in agricultural buildings or perhaps open pavilion structures, this flexibility is certainly not acceptable for residential or commercial buildings incorporating rigid finishes and often large window walls.

In our structural analysis, we have to take into account the flexibility of tension joinery to properly predict the magnitude of compressive forces in the knee brace on the leeward side of the frame and the resultant bending forces in the associated posts and beams. Analyzing wood tension joints as if they are similar to monolithic concrete or structural steel framing is inappropriate. (See Erikson and Schmidt 2001 for additional information on the stiffness of pegged tension joinery.) Notice in Fig. 3 how the windward post shows less bending than the leeward one: this is due to the flexibility of the pegs in the tension joinery that limit the capacity of the tension brace to "pull" on the post. The compression brace will have a larger load in it than the tension brace, but not as large as if there were no tension joinery at work. (See our companion article, TF 79:18, for further discussion of the interaction between tension and compression joinery.)

If instead of knee braces at the top of our frame, we incorporate so-called down braces at the bottom of the frame, as shown in Fig. 4, we can get better resistance to racking. That's because the foundation that the compression down brace (now on the windward side) pushes against is rigid, unlike the beam at the top of the frame. Note that as the frame pivots around this compression brace, the windward post tends to be pried up out of its joint to the sill, so the forces at that joint have to be considered. Fortunately, we have the weight of the building on the post working in our favor to resist this uplift. If we plan to use tension joinery in down-braces on the leeward side of the frame, then we need to have good anchorage of the sill to the foundation at those points as well as good tension joinery.

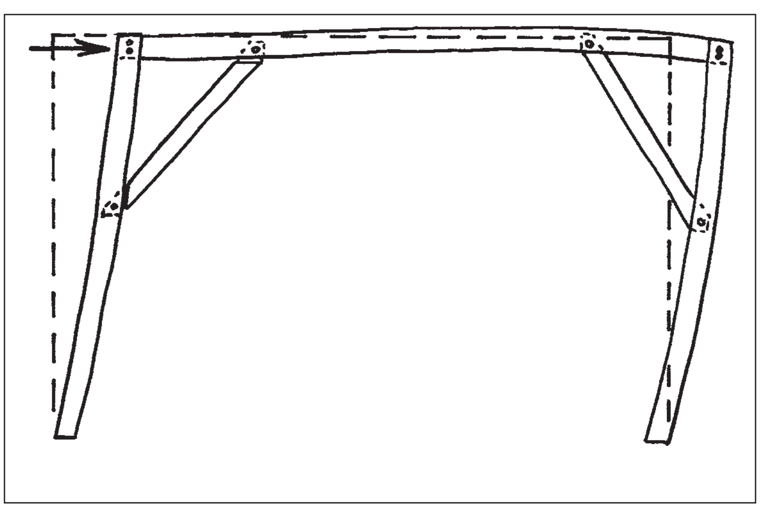


Fig. 3. Racking a simple knee-braced frame. Posts and beams bend to accommodate stresses applied by braces.

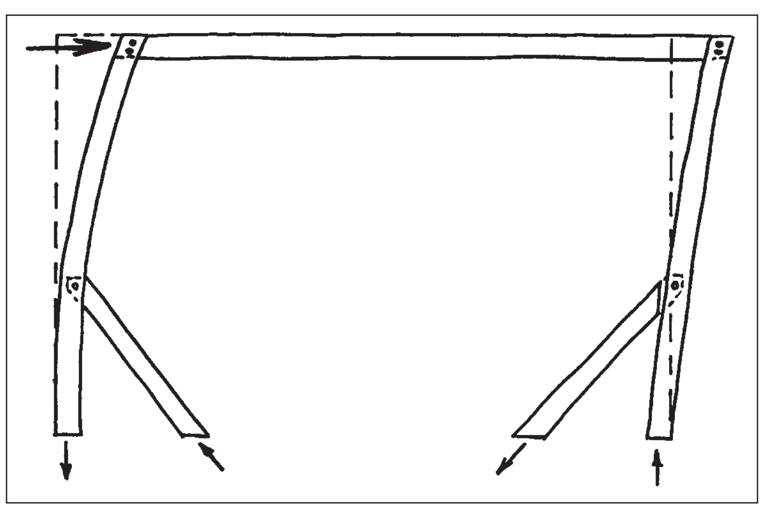


Fig. 4. Locating knee braces at sill rather than plate obtains greater racking resistance for frame since the fully supported sill cannot bend.

There is still flexibility in a down braced frame because the posts can bend. Other bracing options are more effective. Instead of using just 36-in. horizontal and vertical runs for the brace, we can take the brace from corner to corner of a frame, as in X-bracing. We will get a much stiffer building with lower joinery forces. This configuration starts to look like what we might call Old-World bracing, as seen in the half-timbered structures of Germany and England. Sill-to-plate bracing is stiffer because it turns our frame into something more closely resembling a truss. The nearer we bring the ends of the diagonal brace to the intersections of the posts and beams, the less bending there will be in those members. Because the diagonal brace is long, it has a better lever arm to resist the racking forces, and thus the forces in the brace are lower.

When we study those half-timbered structures from Europe, we notice they are not pure timber frames in the same sense as a typical 19th-century American barn. With the kind of infill typically in place in a European timber frame, we start wondering how much work the braces really have to do, which leads us to our other main strategy.

Shearwalls. What is a shearwall? It's a wall or portion of a wall that's essentially rigid in its plane. It will not rack, it will not slide and it will not tip over when design lateral loads are applied. In wood construction the resistance to racking can be provided by a number of arrangements.

¶ Horizontal or vertical sheathing. This forms a relatively soft shearwall, since all the resistance to racking is provided by the nailing of the boards to the framing members.

¶ Diagonal board sheathing. A much better and stiffer method. We still depend on the nails to fasten the boards to the framing members, but now the sheathing boards function as diagonal braces.

¶ Plywood sheathing. Even better since we get a much "smoother" flow of the forces in the panel and we can put many more fasteners through the plywood into the framing members without risk of splitting the sheathing. The more fasteners, the stronger and stiffer the shearwall action.

¶ Structural insulated panels (SIPs). Similar in behavior to plywood sheathing but particularly applicable to timber frames since the panels can span larger distances between framing members and provide an insulated skin at the same time.

Research presented in this journal (Erikson and Schmidt 2002) has shown that shearwall-braced timber frames can be much stiffer than timber frames with knee braces alone, even those incorporating tension joinery. The loads in a structure go to the stiffest elements. With any form of shearwall in place, the timber frame will likely not have much opportunity to resist racking since the shearwalls will take up the load first.

There can be some interaction of braced timber frames and shearwalls in a structure, where loads are shared between the two systems. This is the case when frames are designed with relatively rigid bracing and the diaphragms are relatively flexible (read on for the discussion of diaphragms). We find that in most residential and commercial buildings it's usually more practical to deal with lateral loads simply by the use of shearwalls. Shearwall systems have the advantage that they can be designed using code-accepted rules that define racking resistance as a function of the thickness of the sheathing and the size and spacing of the nails used to fasten it. Research sponsored by the Guild, the Business Council, the USDA and the University of Wyoming has made great strides toward developing accepted standard practice for use of tension joinery in braced timber frames (Schmidt and MacKay 1997, Schmidt and Daniels 1999, Schmidt and Scholl 2000, and Miller and Schmidt 2004), but it's still more straightforward to get a building permit, especially in seismically active regions, using shearwall systems.

For engineered design of SIPs as shearwalls, at this time we need to use manufacturer-specific shearwall resistance values. The International Code Council Evaluation Service has testing procedures in place and evaluates the suitability of a particular SIP manufacturer's products for use as shearwalls in wind and low seismic demand applications. The Structural Insulated Panel Association is working with the American Plywood Association to add evaluation procedures for SIPs used in more seismically active regions of the country. SIPs will be included in the next edition of the International Residential Code for use in prescriptive design (that is, cookbook or pre-engineered design provided in the code) for wall applications, including use as shearwalls.

Diaphragms. Whether you are using braced frames or shearwall systems, keep in mind the function of the floors and roofs as part of the lateral load-resistance system. The sheathing on floors and roofs essentially creates horizontal shearwalls that we call diaphragms. The diaphragms act as horizontal beams that provide lateral support to the walls of our building and transfer the wind loads on the walls to the braced frames or shearwalls, elements of the building that resist racking.

Figs. 5 and 6 show the flow of wind forces through a simple building. Wind causes pressure against the windward face of the building and suction on the leeward face. The wall sheathing and framing direct the wind load to the floor and roof diaphragms, which in turn direct it to the shearwalls or braced frames. These are anchored to the foundation. Diaphragms can be constructed of board sheathing laid perpendicular to the joists but, just as with shearwalls, a stronger and stiffer structure results when the boards are laid diagonally to the joists. Plywood-sheathed diaphragms are even better. Design of diaphragms follows code-accepted rules that define strength as a function of the size and spacing of nails and the thickness of the sheathing.

Diaphragm action allows us to position shearwalls and braced frames in a building in asymmetric arrangements, and it opens the door to creativity in building configuration. We are not confined to a rectangular box with solid walls on four sides. The diaphragms

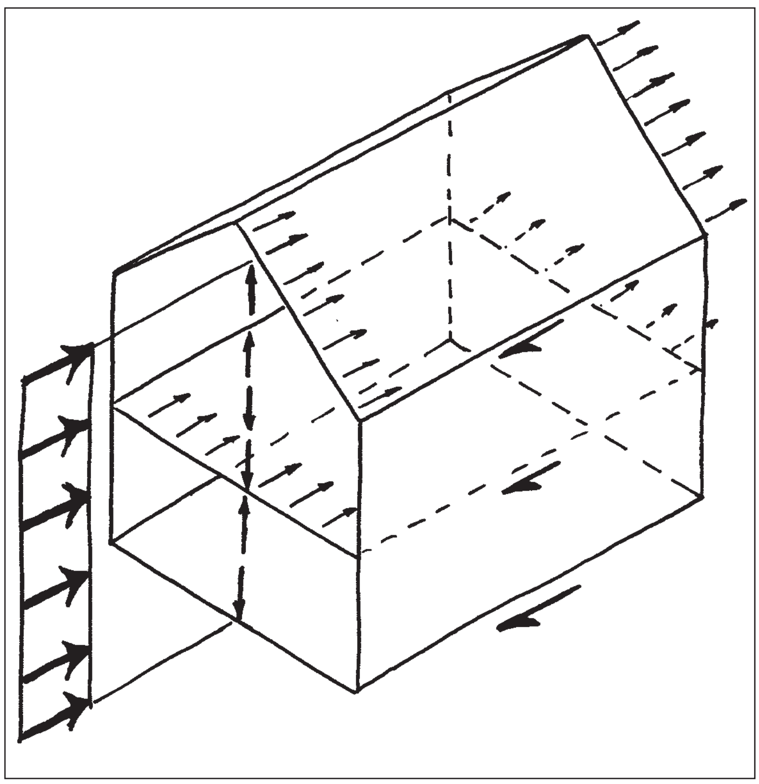


Fig. 5. Diaphragms and shearwalls at work. As windward and leeward walls try to push and pull roof and floor with the wind, side walls hold back roof and floor to stabilize building.

have to be specifically engineered for the forces they must resist, and understanding the three-dimensional behavior and flow of loads through a building is required.

As a system for resisting lateral loads, shearwalls and diaphragms reduce the strength and stiffness required of the timber framing. Posts can be sized to accommodate the joinery at beam intersections without having to worry about the effects of racking that would be at work in an unsheathed braced frame. It was a combination of engineering and trial and error that led barn designers and builders in the early part of the 20th century to appreciate and take advantage of sheathing working as diaphragms and shearwalls to greatly reduce the amount of framing in barn construction. The classic 19th-century gable-roofed timber frame barn of the eastern states evolved into the laminated curved-rafter clear-span dairy barn of the 1920s.

Project Development and Management. When should you get a structural engineer involved in the design of your timber frame project? We encourage you to get architectural and engineering advice as soon as you have developed those first freehand sketches showing rough plans and elevations for the building. Don't try to take your design too far and make it pretty before you discuss the basic issues with an architect and a structural engineer. Definitely do not wait until you have already signed contracts and ordered timbers before contacting an engineer, hoping to get your drawings approved and stamped for a building permit. At that point your options for modifying the building are all going to be expensive and could lead to some very soured relations with the client, or with your bank if the project is for yourself.

Remember that design of a building starts from the top and works down as you figure out the loads and framing for the roof and the gradual accumulation of roof loads and floor loads down to the foundation. If you keep that in mind you will see why it makes no sense to build the foundation until you have a clear plan for resisting both gravity and lateral loads applied to the building.

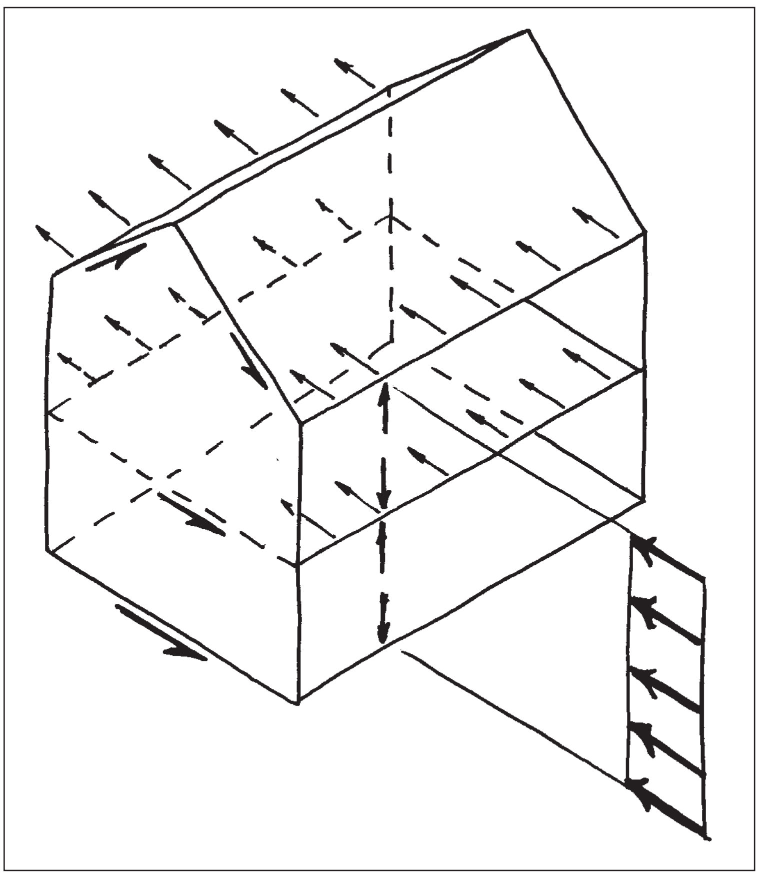


Fig. 6. ***Wind against eaves side of gable roof building puts diaphragms and shearwalls to work but produces more complicated forces because of roof slope. For roofs steeper than 6:12, wind produces positive pressures on windward side but suction on leeward side, while for roofs shallower than 6:12 both sides experience suction (not shown).***

There are often misunderstandings by architect, owner, builder, and even sometimes the structural engineer, of the role of the timber frame components in the completed structure. The team sometimes assumes that the timber frame structure will perform something like the Rock of Gibraltar, capable of resisting all gravity and lateral loads. Because of such misunderstandings, often not enough attention is paid to designing the structure specifically for resistance to lateral loads. The timber framer needs to verify with the architect or engineer how the building is to be braced for wind or seismic loads and whether the design for lateral loads has been provided in the drawings. If SIPs are to act as shearwalls, make sure the SIP suppliers are aware of this fact. Before prices are established and contracts written, clarify whether the suppliers are responsible for design for lateral stability or whether it will be provided by others. To repeat, design for lateral loads needs to be addressed before the foundation is designed because the foundation and the attachment of the lateral-load-resisting system to it are critical components of the lateral stability system for the building. The foundation must have adequate mass and appropriate reinforcing at critical locations.

For architects, engineers and timber frame shop owners, we believe it is negligent not to clearly spell out responsibilities for gravity load and lateral load design on any contract documents and on shop drawings. It is unacceptable and unethical to stamp shop drawings without a thorough review of all critical joinery and a clear statement on the drawings whether or not the timber frame has been designed to resist lateral loads.

Engineering for timber frames is a craft like the craft of timber framing itself. To be proficient requires both training and practice. The basics of structural engineering are well within the grasp of most timber framers, and you can learn to do some of the preliminary design for yourself. The more complex aspects of structural design take specialized training and time to master, so we urge you to get structural engineering review. There is no one right answer to the design of any timber frame structure. Structural engineering can help you achieve creative ends while still having confidence that the building will perform as expected under loads.

—Tom Nehil and Amy Warren

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D-I-Y Down Under III

All photos Rob Hadden and Toni Lumsden

Fig. 1. View of new work at Rob Hadden's ever-growing empire in Castlemaine, Victoria, Australia. Center right, barn and guest house completed in 2004. Far right, masonry-walled house with interior timber framing, completed in 2000. Note drooping jetty on gable end of new work.

HAVING completed the timber-framed barn and guest house in 2004 (Fig. 1, center right), and having enjoyed a very short recovery period, I launched into the main house frame as a man on a mission. A project of this scale was always going to present serious logistical challenges, not the least starting with nowhere near enough timber or regular income to pay for building as well as living expenses. But my firm belief that what I need will somehow manifest itself has been vindicated. A network of people are on the lookout for timber on my behalf, including a number of arborists recruited for the cause and unanimous in their backing of my use of salvaged timber. Over the last two years I have accepted for use a large variety of species, including cypress pine (*Cupressus leylandii*), English oak (*Quercus robur*), elm (*Ulmus minor,* var. vulgaris), Lombardy poplar *(Populus nigra*), silver poplar (*P. alba*), golden poplar (*P. canadensis,* var. serotina aura), Himalayan cedar (*Cedrus deodara*) and Monterey pine (*Pinus radiata*); also, grey box (*Eucalyptus microcarpa*), red box (*E. polyanthemos*), hybrid box (*E.* variety unknown), yellow gum (*E. leucoxylon*) and English ash (*Fraxinus excelsior*). All this timber I transported using my trusty Toyota pickup and a trailer borrowed from my father-in-law.

I towed home large logs up to 20 ft. long from near and far in some heart-stopping journeys, at times along major highways. These logs were then stored on our land and the ends painted to reduce rapid drying and checking. The logs slowly accumulated until I felt I had enough to start.

Using a cutting list and a spare set of plans, I determined where it was all to go and spent much time sorting for length, diameter and position. It was very nearly a nightmare making sure that I didn't misuse one stick of timber or waste a long log when a short one would suffice. Once I reached consensus among the voices in my head, then chainsaw milling, which collectively took many months, could commence. For some strange reason, the heavy work of milling nearly always fell in the hot months of summer.

The most important logs to attack first were cypress pines, for the large jowled two-story posts, tie beams and connecting floor girts. Quality was not an option at times with this pine. I had to work around large knots, hollows and folds in the timber, constantly making adjustments. Sometimes the defects could not be avoided. This was not textbook carpentry but a good example of making do and finding solutions without too much compromising.

The rubble stone foundation I built two years ago in a frenzied two weeks using sandstone from our own property. The 12-in.-thick walls follow the natural slope of the land. The foundation extends from about 12 in. high at the south end to over 4 ft. at the north. This is very much standard-issue domestic random rubble masonry bonded with lime mortar (Fig. 2).

Fig. 2. Author built lime-mortared stone foundation quickly with stone gathered from property.

Fig. 3. Not a fisheye view. Swept corner post posed interesting problems.

Fig. 4. Crossframe in English Midlands style. Studs are 10x5 in.

The frame design, originally put to paper in 1998, turned out to be similar to that of the Merchant's House at the Avonscroft buildings museum in England, which I had never seen. When I later visited the museum I was astonished to see how much there was in common between the two designs, although there are plenty of details that set my building apart. Since then I have completely redrawn the plans based on new ideas, research, philosophy and, quite possibly, my deranged state of mind. These plans now differ from what the local authority has on file and there might be the odd raised eyebrow come inspection time. The "collapsed" jetty on the northwest end (Fig. 1) could be a case in point, and another the very bent southeast post that has given the east side a rather decidedly pregnant bulge, leading motorists to veer off the road when they see it (Fig. 3).

These deliberate deformations are based on actual examples I have seen in the UK. I have no reservations in building like this from scratch. It's sometimes said that there's not much art to modern framing and that contemporary practices have put the muse to flight. I agree and find a lot of contemporary framing bland in design. My inspiration by older English and European domestic buildings takes me down a different aesthetic track and expands my vision of what can be done.

The building is T-shaped, with two separate frames, the first started nearly two years ago with a layout marked full scale on the floor of the barn. Careful work allowed the crossframe lines to be extended and used for the wall layout, saving a whole new marking out. Using different colors, I was able to keep them separate and avoid mistakes. I varnished them to prevent their loss and to saturate the colors. With all systems go, I then scribed, cut, drilled and chiseled all the timbers for three crossframes (one shown in Fig. 4) and the jettied frame with its upper and lower parts (Fig. 1). There were no real dramas here except for the continual moving of the timbers in and out of the barn, which took up a lot of time .

Fig. 5. West wall frame completed on floor of barn, little room to spare.

Lest I get carried away with success on the crossframes, however, the wall frames presented me with two salutary lessons. First, the curved northeast post, more than 5 in. out at the middle, would not sit still even with wedges. Compounding the problem was the fact that all the timbers laid on its curve were then not level and had to be scribed in this position. The 2-ft. marks both top and bottom and the plumb marks helped finally to position the post correctly.

Second, and this was more serious, the long plate scarfs that I had done many months earlier had moved alarmingly during an absence while I did some paid work. One plate had bowed out of level by 3 in. at the upper jetty post end. Drastic problems needed drastic solutions. I assembled the scarf, struck a new straight datum line from the scarf to the end, then resawed the timber from 3 in. to nothing at the scarf. It looked odd, but at least the face side of the plate was now in correct alignment, and there was still enough meat left for the rear tenon of the post. After this little hiccup, it was smooth sailing for the rest of the wall frames. They did look impressive laid out in the barn with only about 18 in. between the barn walls and the frame on both sides (Fig. 5)—rather a tight fit!

Having patted myself on the back for getting this far without a nervous breakdown, I was suddenly summoned to work full time, thus leaving the timbers outdoors covered with tin for the first three months of summer, at the mercy of the intense summer sun and hot drying wind. I used to lie awake at night knowing what was happening to all this green timber sitting under hot metal in the sun. Shrinkage, movement and winding stresses were being unleashed and creating a whole new set of challenges for me. I could hardly wait.

The Raising. I began with crossframe two, raised as a whole bent with the aid of a friend and his old WWII Blitz wagon fitted with a small hydraulic crane on the back. Crossframe two had to be first up so that when the lower part of crossframe one, the jettied end,

Fig. 6. Author pegs off first wall girt, which passes corner post to form end support for jetty. Pipe tripod with block and chain raised the beam.

was in place, the lengthwise 7x5 floor joists could be located in their respective mortises in the transverse floor girt (mortises still empty in Fig. 6). My joy at the success of raising crossframe two was short-lived. After placing the sill and corner posts for the lower frame of the jettied end, a brace on the left side wouldn't fit and all the studs refused to cooperate with the top plate. The cause appeared almost accidentally with a casual glance along the sill plate's datum line. The timber had bowed over an inch, taking all with it, moving in the heat over a few months. Pulling at it with ratchet straps did not improve the situation and mild panic set in as I envisioned the whole frame-raising stopped dead in its tracks. In the end, the only remedy was to saw the sill halfway through in two places and winch the middle section down slowly until I could engage the pegs in the stud tenons. Metal plates housed in place out of sight and secured with galvanized bolts bridge the cuts.

Raising the upper jetty assembly looked a simple procedure on paper, but in the event it upped the heart rate considerably. We had to erect the oak posts and oak bressummer with two elm down-braces up onto the projecting tenons at the ends of the wall girts. The assembly could easily have tipped over, so I tied ropes back to crossframe two and screwed large stops onto the ends of the girts to prevent the frame sliding off.

With my wife Toni on one side and myself on the other, we manually tilted it up in three stages until it was nearly vertical. As the weight of the posts, bressummer and elm braces was considerable, we raised it one side at a time (there being enough flex over 14 ft. to do this) and supported it at each stage with blocks of wood and, finally, sawhorses. At about 70 degrees, it was easy to take on to the vertical position. Checking to make sure the tenons on the ends of the wall girts would engage their mortises in the posts, we lifted one last time and the assembly engaged with a satisfying hiss of air out of the mortises and a solid thud as the posts hit the girts. The rabbeted bressummer also sat nicely on the overhanging floor joists that support it (Fig. 7).

With that in place we could add the remaining studs and window sill, then climb down and have a stiff drink and rest the aching muscles.

The piece-by-piece assembly moved along smoothly with interrupted sills, wall studs and girts raised by hand and the crane lifting crossframes three and four into place. With the aid of my trusty shear legs, Toni and I hauled up the top plates and assembled their respective 3-ft.-long stop-splayed scarfs. A temporary floor of thick plywood reduced any danger to ourselves during these operations. It was, altogether, now starting to resemble a house frame. We hauled up the large tie beams in two stages as the hoist chain could not reach two stories. We slid the ties along the plate to their respective locations and then lifted them onto the teazle tenons of the posts (Fig. 8).

Fig. 8. Shear legs tied off to wall, last tie beam raised over top plates.

Fig. 7. Jetty bressummer rabbet shelters ends of 7x5 pine floor joists. Note distinct drip groove in underside. Tenons of wall girts support end posts.

We did have one diversion. The east post of crossframe three, measuring 8 in. by nearly 14 in. deep at the jowl, had moved inward by three quarters of an inch in the top half of the post only. The tie beam would engage the west post but not the east post. A plan was hatched after much agonizing about bending a post of this size without harming other joinery.

We locked the posts together at floor girt level with heavy chain (just below blue strap in Fig. 9) and turnbuckle, so they couldn't spread and burst the joints when we straightened the top half of the post, and we rigged the top of the west post with a ratchet strap to keep it from yielding outward (red strap Fig. 9). Then we cut a major taper in the teazle tenon of the east post and made minor adjustments to the top tenons of intermediate studs, all to ease the descent of the tie beam.

The tie beam set over all the modified tenons, we rigged two large ratchet straps down to the heavy floor girt directly below and slowly increased tension to pull the tie beam down over the tapered post tenon. Once the tie beam was seated on the post top, we put in all the pegs and left the restraining straps for two months. Sighting down the datum line confirmed that the post was now plumb.

With all the tie beams in place, we hauled up the principal rafters with the shear legs, lifting them at the angle at which they were to rest. Collars and their respective studs went in next and held each principal rafter in place while the next matching rafter was lifted up and lowered over the collar tenon and into its tie beam mortise. Though I had previously used splines through the rafters to connect purlin ends, this time I used drop-in housings in the rafter faces, with half-dovetails on the purlin ends; in some cases these locked to one another (Fig. 9). The lap-jointed wind braces were scribed in situ. I tapered their tenons sharply in thickness to make the corresponding notch in the purlin a simple matter of saw cuts and a small amount of chiseling. Nails hold the tenon in place. This way, little wood is removed from a critical point in the purlin and there is still a good bearing surface. This technique has historical precedence in the UK and was demonstrated to me by an English timber framer.

THERE are many differences between the ell-frame and that of the main section. The height has been reduced to a story and a half and the upper floor is supported by two axial beams, both 12x14 in., resting on the fireplace to the east and on crossframe three to the west. A combination of different frames upstairs gives a potted history of English roof framing, with truss one an interrupted tie beam setup, truss two a two-tier cruck and truss three part base-cruck with a large triangulated truss on top for good measure (Fig. 10).

I framed parts of these trusses from quite a range of timbers, including a large forked branch of yellow gum for one interrupted tie. This timber was incredibly difficult to work, with the plane skidding across it as if on glass, rather than taking a shaving. Sawing and chiseling presented their own dramas and anything less than a Robert Sorby chisel did not pass muster. The crucks for the

Fig. 9. Main frame complete, with straps and chain for troublesome crossframe three post left in place. Purlins lock to each other or to a rafter, wind braces are taper-lapped to rafters and purlins. House in background with twisted chimney dates from 2000.

main two-tier truss had interesting histories. One came from a bifurcated ash tree that had lost one half to the ground (the chickens' loss of shade, and my gain). The other cruck came from the local botanical garden after a golden poplar fell over one weekend. Monday morning saw me at the garden organizing for the log to be cut and delivered by lunchtime. By mid-afternoon it was milled and taken to the barn to be shaped, and two days later it was framed up. Now *that* is green timber framing!

While it's one thing to plan two-tier trusses, it's certainly another to build them when experience is not on your side and one of the crucks is bent compoundly. Double curved and even simply curved blades do not have nice convenient straight edges to work from, nor do they level up with datum lines. Meanwhile, the small spur ties between cruck blades and walls have to sit level during scribe layup even if the cruck plane is far from true. Rupert Newman, in his book *Oak-Framed Buildings* (reviewed in TF 81), gave me the clue I was after. Origin points, where critical timbers intersect, are the only surfaces to consider. So, by using a long straight edge placed on two blocks at these points, I was able to level and shim the layup correctly. With the collar placed on top and leveled, the remaining small upper cruck could be laid up as well, keeping all the members in level planes.

Top plates for the ell also came in for modification compared with the main building, with the usual long stop-splayed scarf usurped by a simpler and shorter bridle joint with a 6-in. tenon that I found illustrated in F. B. Charles's book on timber conservation. The beams for the plates were simply not long enough to make the stop-splayed scarfs. Three edge-pegs stop the plates from wandering. This latest frame was raised much more promptly after cutting compared with the first, so there were fewer problems. Looking ahead, my only real concern is the expected substantial shrinkage of the large poplar crucks that were so green when framed. Poplar shrinks in bizarre ways, including along the grain if there are large knots or shakes at or near a joint. This has already begun to happen. From his review of *Oak-Framed Buildings*, Bill Keir's comforting phrase "the subtle undulation of line and level" springs to mind.

So, here I am at a halfway point, with the last cob-and-timber section to be completed next year. Jointed crucks and more "primitive" framing are in prospect, as well as the use of my only large piece of English oak, a 12x12 girding beam for the floor converted from the trunk of a tree felled illegally (the owner did not secure permission to cut it) not far from home. In a subsequent article I will deal with the final section's framing and cob-walling construction, and cladding, insulation, flooring, roofing and lime plastering of the entire structure, much of it relevant to contemporary timber framing as we all come to terms with the problems of sustainable methods and how to incorporate natural materials.

—Rob Hadden

Previous articles by Rob Hadden (marmalade@mmnet.com.au) appeared in TF 58 and TF 74. Rob will speak about his work at the 2008 Western Conference at Coeur d'Alene, Idaho, in April.

Fig. 10. Framing of the ell continues. Note two tiered-cruck, spur acting as truncated tie, short bridled scarf in plate (three pegs) and lap-jointed wind braces. Additional crucks will rise to the left. The whole will be cob-walled and lime-plastered.

HISTORIC AMERICAN TIMBER-FRAMED STEEPLES

IV. Reproducing Burned or Destroyed Steeples

This article is fourth in a series to discuss the form, function and joinery of selected historic American timber-framed steeples. 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.

> On February 29, 1852, the church was destroyed by a hurricane which struck the spire, threw it directly upon the ridgepole, crushed down the whole of the roof, burst out the side and end walls, and in one movement demolished the entire building. —H. Saddington, *A Backward Glance: History of the Syracuse, N.Y., Unitarian Church,* 1938

CHURCH steeples can fail slowly or dramatically, as in the epigraph above, in different ways and from different causes: structural inadequacy, decay, fire, lightning and wind.

Structural inadequacy. Steeples can be structurally inadequate to bear their own dead load. This problem usually shows up low in the steeple where sleepers or other bearing timbers deflect excessively from the accumulated load. In the 1869 steeple of the First Congregational Church of Brattleboro, Vermont (1853), the tower girts bearing the sleepers that carry the accumulated load (in descending order) of the spire, lantern, belfry and clock stages are too slight for their span and have broken without the presence of rot. More often the problem is located at a non-steeple element. Typically the first interior roof truss clear-spanning a choir loft or nave below is unequal to the dead and live loads imposed upon it by the rear of the steeple framing. The result is the backward lean of the steeple, sometimes alarming, and the locally depressed roof ridge seen on hundreds of wooden churches in the eastern US and Canada.

Decay. Water infiltration can rot steeple framing. Again, this often occurs low in the steeple or at other points where the slope of a stage changes or one stage transitions into another, occasioning flattish skirting roofs, flashings and snow retention, as well as applied ornament such as urns or volutes that pierce the coverings, not to mention the roosting of pigeons with their corrosive droppings and scratchings. Water itself is not the entire problem, but a high moisture content invites wood-destroying organisms and insects to begin their work.

Tall slender spires atop a steeple can usually shed water well and are often successfully covered in only flat boarding. If the spire itself has a water problem, it might be at the flared base for the same reasons cited above, though it's more likely to be at the entry of the weathervane, where leakage, condensation and the extreme difficulty of examination or maintenance allow rot to develop. Water entering midway up a steeple runs down the posts, enters brace mortises on the way and pools at the bottom in mortises for the tower posts in the bearing sleepers laid across the lower chords of the roof trusses.

Jan Lewandoski

Fig 1. Perfect original spire frame on right, totally replaced middle stages on left, awaiting lift onto Salem, N. J., Presbyterian Church, 1854, out of sight at right.

At the Salem, New Jersey, Presbyterian Church (1854), the middle stages rotted, requiring complete reframing, while the healthy spire required only recladding (Figs. 1–3). At Christ Church, Philadelphia, built 1753, the wood sills laid upon the lower brick portion of the tower, as well as the feet of some of the great octagon lantern posts, rotted alarmingly by 1771, and the church's builder and designer Robert Smith was called back to repair it. Given the immense size and good condition of the steeple above, it was left in place (somehow supported on jacks, probably in successive segments) while the rotted timber was removed and replaced by built-up plank. Smith scarfed on new bottoms to the belfry posts as needed, and replaced with plank several of what he

Fig. 2. Salem Presbyterian Church. Italianate tower is 185 ft. high.

Photos this page Ken Rower

Fig. 4. Christ Church, Philadelphia, 1753, designed by Robert Smith.

Fig. 3. Restored middle-stage framing (octagonal stage, Fig. 2) of steeple at Salem. Tall spire above is tied down via long central bolt terminating at laminated crossing that bears up against partners in middle stage.

called hammer beams, cantilevered members designed to carry the upper posts within the perimeter of a lower square tower. The spire, the ultimate stage of this rescued steeple, escaped water and decay but was destroyed by fire and rebuilt in 1908 (Figs. 4–5).

Fig. 5. Laminated beams replaced rotted material at Christ Church in 1771. Iron straps appear to be original, rods 20th-century.

Jan Lewandoski

Fig. 6. Decay in posts and supporting beams led to significant distortion of steeple at the South Woodstock, Vt., Community Church, 1836.

At the South Woodstock, Vermont, Community Church (1836), the combination of rotted sleepers and tower posts and a rotted and deflected first truss chord caused the modest steeple to tilt out of plumb 7 in. on one axis and 5 in. on the other (Fig. 6). The steeple was restored by dismantling its conveniently telescoped and lodged stages (see TF 85).

Fire, lightning and wind damage. Lightning and wind are likely to affect the upper levels of a steeple most severely, tearing off a stage or blasting apart or burning a spire or belfry. Even fires within the body of a church tend to follow a chimney effect into the upper portions of a steeple, collapsing it while charring members below.

Church steeples were designed to be the tallest objects in their towns and thus the most exposed to wind and lightning damage. Perhaps a quarter of the hundreds of steeples I've examined have had their upper portions, usually the lantern, cupola or spire above the belfry, replaced because of wind damage or fire. The reasons for this particular vulnerability may seem obvious—it happens to tall trees in the forest as well as towering figures in history—but there are other reasons peculiar to 17th- through 19th-century stylistic trends in steeple design.

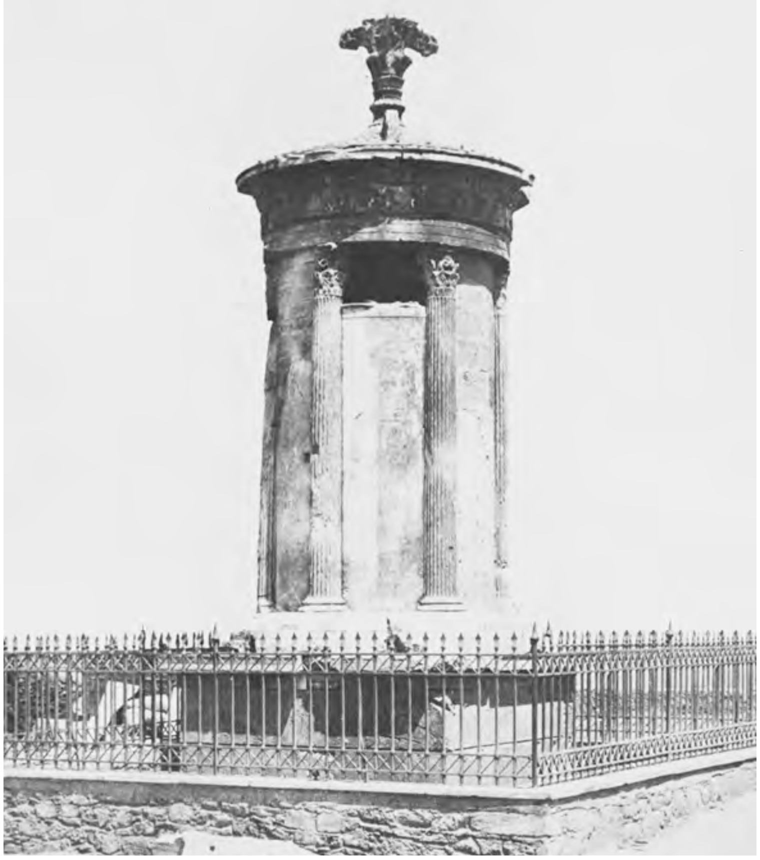
The deep telescoping discussed in previous articles in this series, while offering no lightning protection, is a method of avoiding the tearing off of successive stages by high wind. Telescoping is very different from platform framing. At Ithiel Town's great Center Church (1811) on New Haven Green in Connecticut, for example, the framing of one stage may penetrate as much as 38 ft. into the stage below (Figs. 7–8).

To remove any stage from the one below it, winds have to actually break eight posts and tear apart their surrounding casings, cornice and ornamental work. A key factor in the resistance of these telescoping posts to breakage is that they usually don't contain any weakening joinery where they emerge from a lower stage. Thus they are at full section where a skirting roof might appear to mimic an end condition.

Ken Rower

Figs. 7–8. Center Church, New Haven, Conn., 1811. Deep penetration of stages to those below successfully resists effects of wind. Below, lowest wood-framed stage is deeply lodged in brick tower.

The open bell deck, a stylistic choice of form that threatens many spires, often provides no concealed space for stages above to drop long posts into stages below. The insertion of an open colonnade of four, six or eight slender posts, sometimes surrounding a bell, is a post-Gothic feature based on certain classical structures such as the Choragic Monument of Lysicrates (Fig. 9). Wren and



Bryn Mawr College

Fig. 9. The Choragic Monument of Lysicrates, Athens, ca. 350 B.C. Drum surmounting Corinthian columns was adapted to other forms in 19th-century America.

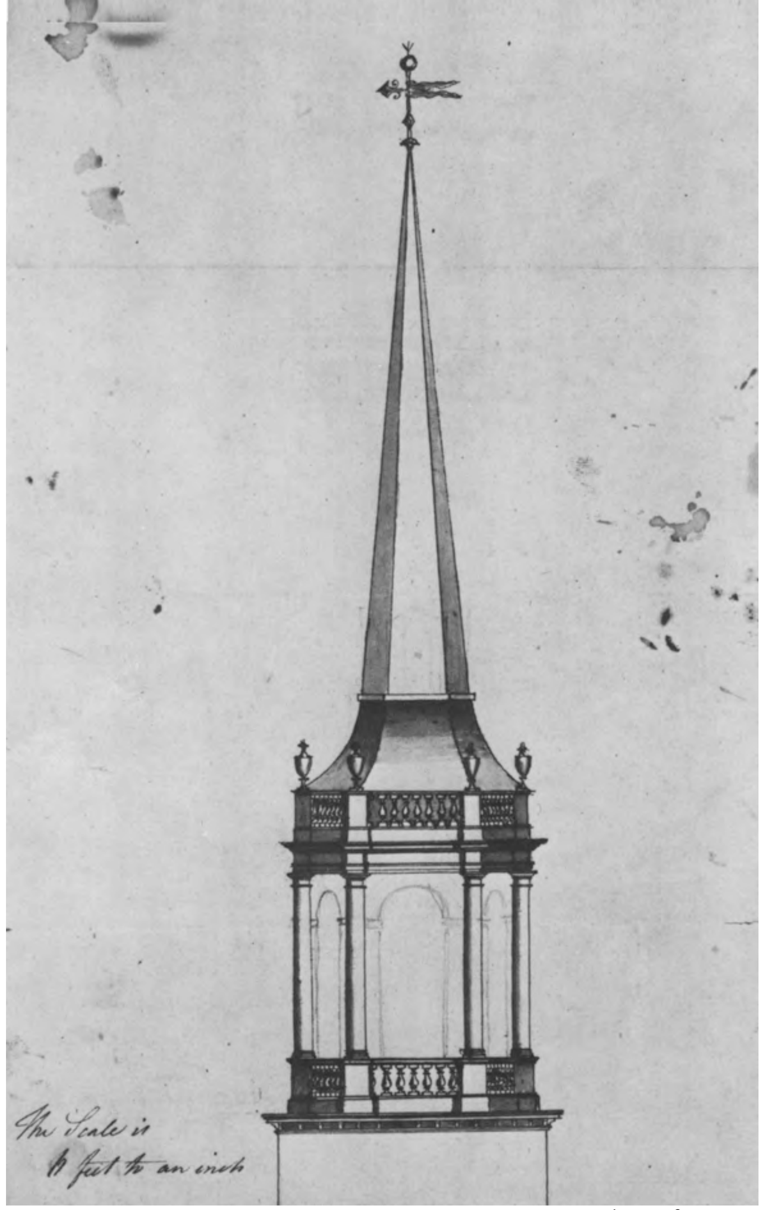


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Fig. 10. Bulfinch's drawing of the steeple at the Congregational Church, Taunton, Mass., 1790–1792. Legend reads: "The Scale is 6 feet to an inch."

other English architects and framers seem to have avoided the feature in their numerous and influential steeple designs, partly because of its unsuitability to execution high up in stone, and also perhaps to a dislike for the lightness and insubstantiality it suggests. (Alternatively, it's possible that English wooden versions simply have not survived.)

In New England, where tall steeples were generally of wood, an open colonnade with substantial height and weight perched above it appeared by 1712 on Boston's Christ Church and by 1723 on Boston's "Old Brick." Charles Bulfinch, designing churches in the same year of 1790 for both Pittsfield and Taunton, Massachusetts, laid the basis for widespread popularity of this form (Fig. 10), the model being disseminated almost immediately in Asher Benjamin's books such as *The Country Builder's Assistant* (1797).

The desire to display a bell on an open platform and to transmit its sound better, and to pierce the body of a steeple with daylight, produced an inherent weakness in the structure. Sometimes the weakness was simply accepted as the price of beauty, or mitigated by attaching the stages above the opening to a very heavy crab or belfry plate (heavier than needed to support the dead load above). Often, iron straps made a tension connection between spire rafters and plate at this point.

Another mitigating technique was the construction of thickened corners for open or partly open belfries. At the large and tall Middlebury, Vermont, Congregational Church (see TF 83), Lavius Fillmore designed an additional square tower, two lantern stages and a spire with vane atop the belfry, amounting to almost 70 vertical ft. of superimposed framing. He anticipated this load by thickening the belfry corner casings and trim so that they could conceal not only the framing surrounding the bell but also the telescoping posts (four pairs of 12x12x28-ft. timbers) that framed the tower above the belfry.

In 1832 at nearby Castleton, Thomas Dake chose the same design to allow telescoping framing of the upper stages to surround the bell invisibly and begin below it. Dake's 9x9x39-ft. white pine posts concealed 28 ft. of their length and emerged for 11 ft. within an irregular octagon above the belfry (Fig. 11).

Ken Rower

Fig. 11. Steeple at Castleton, Vt., Federated Church, 1832, showing thickened belfry corners to conceal telescoped posts from above.

As much as wind, however, lightning is responsible for frequent destruction, striking the tall object usually crowned with a metal weathervane or ornament, or even striking the bell (as it did in June 2007 at Williston, Vermont, destroying the cupola of "Old Brick" church). Lightning-induced fire, taking advantage of the chimney effect within the steeple, is likewise a cause of frequent destruction.

THE historically accurate restoration of such damaged or destroyed structures, including the historic engineering of the steeple—the timber frame, its joinery, wood species and historic metal connectors—is equally as important as recreating its visible form and external detailing. After a disaster, where to start? If lightning, wind or fire destroys or seriously damages a steeple, how do you know how to reconstruct it with historic accuracy?

First, *keep the burned remains.* The artifact itself remains your primary source of information. Timber is rarely consumed entirely by a fire and, in spite of the mess, much can be learned by examining the charred timbers: wood species, actual length of members and type of joinery are all typically readable from the burned remains. True molding profiles for the exterior are also usually recoverable.

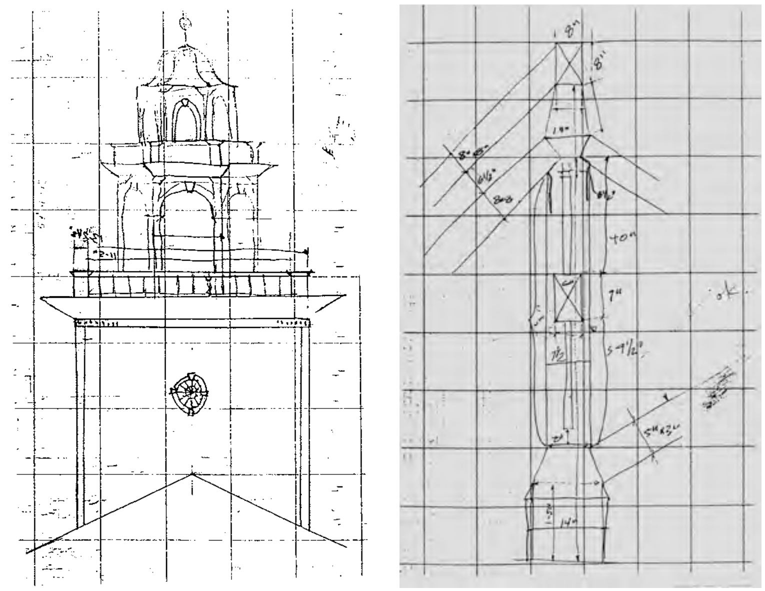
Second, *photographs* of the exterior taken before the event can provide a picture of the desired appearance and a general form for the framing you intend to reproduce. It's rare for photographs to reveal any framing details, but occasionally photos taken during previous repair campaigns show the steeple unsheathed and are of use. If the building has ever been photographed professionally using a large-format camera (perhaps for publication in a book), prints may be available at large size or enlargeable to provide valuable detail on ornament, coverings and sizing. For example, large versions of the photos in H. W. Congdon's *Old Vermont Houses* (1945), archived at the Fleming Museum at the University of Vermont, allowed me to accurately specify a reproduction weathervane for the 1833 Castleton, Vermont, Federated Church, particularly since I had the salvaged directional arrow to scale the other elements against.

Third, *documentary evidence* may exist in the form of Historic American Buildings Surveys done by the National Park Service and available online (memory.loc.gov/ammem/collections/habs_haer). These are invaluable documents, but the drawings vary in accuracy and should be compared with other sources of information. Steeple heights, for instance, are established by transit and may not be accurate to within more than a couple of feet. The architects completing the surveys are usually strong on proportions and historic architectural detail, less so on timber frame structure, particularly when so hard of access and so interpenetrated as steeple framing. Nevertheless, at the Weathersfield, Vermont, Meetinghouse, in addition to sketching the exterior of the steeple, a HABS delineator had penetrated the attic space and produced a very useful measured sketch of a kingpost and its entering members (Figs. 12–13).

Occasionally, if rarely, construction drawings, lumber lists or contracts exist for a destroyed structure. You can look for these in church records, local historical societies or manuscript collections at universities. St. Paul's Episcopal Church, Windsor, Vermont (1822), still possesses the original elevations of the church façade and tower, drawn by its architect Alexander Parris (Fig. 14).

The University of Vermont has extensive archives of original drawings and lumber lists for existing and departed buildings in northern Vermont for the period 1790–1830. A number of important and still-standing early houses, church and public building frames could be reconstructed according to these lumber lists, supposing a knowledgeable eye to examine the lists.

It's not uncommon for church building contracts from the 18th and the first half of the 19th centuries to specify that the steeple or another part of the structure be constructed like another nearby church, which perhaps you can still look at. In their 1836 indenture between the church and the builders, the Woodstock, Vermont, Methodist Episcopal Church repeatedly asked for work to be accomplished "as in the Universalist Chapel." The agreement also specifies "kingposts and principal rafters to be well covered, the roof to be boarded with hemlock boards and shingled with good spruce first quality shingles and to be laid four and one half inches to the weather, the shingles to be fifteen inches," and continues to many other particulars.



Historic American Buildings Survey

Figs. 12–13. HABS fieldworker's notebook sketches of Weathersfield, Vt., Meetinghouse steeple and measured detail of kingpost roof truss.

The original artifact, however, remains primary. At St. John's Church in Portsmouth, New Hampshire, the 1807 lumber list drawn up by the architect Matthew Marsh is framed and hung in the vestibule. It might construct a very similar truss and tower, albeit the cross-sections of the timbers are somewhat different from what the framer eventually used, and none of the original ironwork, such as stirrup-straps at the kingpost-to-chord junctions, was specified in this list. But if St. John's were badly damaged or destroyed, a copy of the lumber list would provide you with the form, even if some of the sizing would be off (Fig. 15).

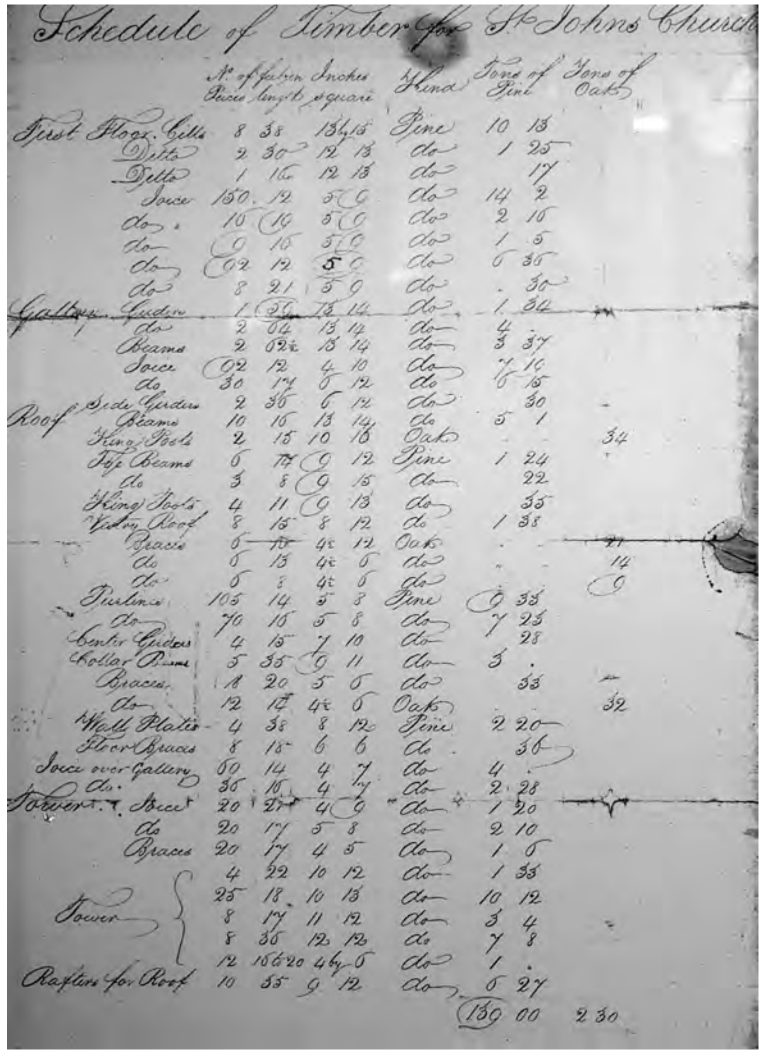
Fourth, *similar examples* built locally at the same time period, or possibly by the same hand, should be visited. They will not give you the frame of the destroyed steeple but they may give you a context and an illustrated vocabulary to help make sense of the burned timbers or the ruinous remains in the attic that you are studying. For example, when asked to rebuild the burned Weathersfield, Vermont, Meetinghouse (1826), I examined the steeple framing of several other 1805–1830 Federal-style churches with similar steeples, all within 40 miles.

While no one steeple frame seemed to provide all the answers, I was able to examine strategies for concealing bracing in the lower parts of an octagon, the typical specialized horizontal frames called crabs, techniques of telescoping and lodged framing and the cambering of bell girts and where they delivered their load. Some of these discoveries were answers to questions I didn't know to ask and allowed me to derive much more information when I returned to Weathersfield's burned remains. Examining wood species in other similar structures allows you to come up with patterns of species choice typical of the time period in your region.

Builder's guides from the period may provide examples of fashionable steeple exterior elevations but rarely contain any steeple framing. Truss forms and their joinery and scarf joints are commonly illustrated in 18th- and 19th-century guides, but the first

St. Paul's Church

Fig. 14. Alexander Parris's front elevation for St. Paul's Episcopal Church, Windsor, Vt., 1822 (drawing cleaned for reproduction).



Schedule of Timber for St Johns Church

	No. of Pieces	feet in length	Inches square	Kind	Tons of Pine	Tons of Oak
First Floor. Cills	8	38	13 by 15	Pine	10 13	
Ditto	2	30	12 13	do	1 25	
Ditto	1	16	12 13	do	17	
Joice	150	12	5 9	do	14 2	
do	16	19	5 9	do	2 16	
do	9	16	5 9	do	1 5	
do	92	12	5 9	do	6 36	
do	8	21	5 9	do	30	
Gallery. Girders	1	56	13 14	do	1 34	
do	2	64	13 14	do	4	
Beams	2	62½	13 14	do	3 37	
Joice	92	12	4 10	do	7 16	
do	30	17	6 12	do	6 15	
Roof. Side Girders	2	36	6 12	do	30	
Beams	10	16	13 14	do	5 1	
King Posts	2	15	10 16	Oak		34
Top Beams	6	14	9 12	Pine	1 24	
do	3	8	9 15	do	22	
King Posts	4	11	9 13	do	35	
Vestry Roof	8	15	8 12	do	1 38	
Braces	6	10	4½ 12	Oak		21
do	6	13	4½ 6	do		14
do	6	8	4½ 6	do		9
Purlins	105	14	5 8	Pine	9 33	
do	70	16	5 8	do	7 23	
Center Girders	4	15	7 10	do	28	
Collar Beams	5	35	9 11	do	3	
Braces	18	20	5 6	do	33	
do	12	14	4½ 6	Oak		32
Wall Plates	4	38	8 12	Pine	2 20	
Floor Braces	8	18	6 6	do	36	
Joice over Gallery	60	14	4 7	do	4	
do	36	16	4 7	do	2 28	
Tower. Joice	20	27	4 9	do	1 20	
do	20	17	5 8	do	2 10	
Braces	20	17	4 5	do	1 6	
Tower	4	22	10 12	do	1 35	
	25	18	10 13	do	10 12	
	8	17	11 12	do	3 4	
	8	36	12 12	do	7 8	
	12	16 & 20	4 by 6	do	1	
Rafters for Roof	10	35	9 12	do	6 27	
					139 00	2 30

St. John's Church

Fig. 15. Timber list for St. John's Church, Portsmouth, N.H., 1807.

English discussion of steeple framing and the mysteries of their erection I have found is in William Bell's *Carpentry Made Easy* (1857). Drawings of steeple framing show up earlier, as in William Pain's *Carpenter's Pocket Directory* (1797), plate XIX, "framing spires for country churches," or in John Clayton's 1848 elevations of the structure of Wren's parochial churches, but there is no discussion of how to get them up there.

J. Frederick Kelly, author of the outstanding and valuable *Early Connecticut Meetinghouses* (1948), which provides measured drawings of the roof trusses of more than 80 churches, religiously stayed out of the steeples. Kelly wrote in the foreword:

> The most difficult aspect of this part of the work has been the measurement of the roof frames, access to which has been far from easy. . . . In many cases the roof space is in total darkness . . . always laden with the dust and cobwebs of centuries, it is often the habitation of bats and wasps. With few exceptions there has been no footing upon which to walk or stand other than the trusses or framing timbers themselves. To climb over and through such a framework and at the same time handle a flashlight, a notebook, a pencil and a six-foot rule, with the peril ever imminent of a serious and possibly fatal fall . . . has been an arduous undertaking. Because of cramped working conditions where truss feet meet the main plates, great difficulty was encountered in obtaining the framing measurements there. The hazard of such work must be actually experienced to be fully understood.

The same can be said of the interior of steeples, only more so.

THE WEATHERSFIELD MEETINGHOUSE. In 1985 a fire destroyed all the timber portions of the 1826 Weathersfield, Vermont, Meetinghouse, leaving the brick walls standing (Fig. 16). The church's insurance company was quick to offer a check for the insured value, but the policy also promised to replace the building in kind. A determined church committee waved away the first offer and decided to discover what "in kind" actually was, from the timber frame down to the Windsor chairs, and felt it needed a year to do so. The burned remains of the church were not disposed of but instead strewn in a long line within the grassy park around the meetinghouse (cover photo).

Jan Lewandoski

Fig. 16. Weathersfield standing timber remains included three tower posts, inset front plate, sleepers, first truss tie beam and vestibule posts.

Fig. 17. One-inch-scale Weathersfield steeple model in spruce and with correct joinery, built by Ted Ingraham. Model stands 70 in. high.

Photos Jan Lewandoski

Figs. 18–19. Weathersfield truss fly-in was complicated by making up the closed mortise and tenon joinery of many connecting members.

The brick walls were shored up and a group of artisans, architects and church members set about examining the remains, old photographs, the HABS survey and nearby churches similar in date and style, all in an attempt to come up with a confident restoration plan. The church committee decided to retain the interior of the church in its mid-19th-century form of two floor levels (originally it had one room with galleries), and it rejected possible floor-system reproduction opportunities, such as the installation of 10x10x50-ft. yellow birch and beech carrying beams, as too difficult. At the same time the committee decided to reproduce exactly the roof frame (six double-rafter kingpost trusses spanning 50 ft. in the clear, with central longitudinal bracing) and the nearly 100 ft. of steeple timber frame with its exterior finish.

Framing the trusses. The HABS field notebook sketch of a kingpost (Fig. 13), reconciled with surviving charred timbers including the timber plate and some bottom chord ends still in place atop the brick walls, gave dimensions and the truss form. There was plenty of evidence among the charred timbers to indicate the joinery, and the original stirrup-straps and forelock bolts that joined kingpost to chord were reusable.

Samples of several salvaged members went to the Forest Products Laboratory in Madison, Wisconsin, for determination of wood species. The 9x14x16-ft. kingposts and the cambered bell girts were found to be white ash, the longer truss members and steeple posts, commonly 10x10s 25–50 ft. long, Eastern spruce. Smaller members such as braces and ceiling joists were mixed spruce, beech, yellow birch and maple.

The next step was to build 1:12 scale models of truss and steeple as aids to framing and, more important, guides to order of assembly and erection. The models, built with correct joinery, would help me explain to the architects, owners and other interested parties how the steeple was designed to be erected in discrete, lodged, telescoping stages, and how we would accomplish this task (Fig. 17).

Acquiring timber from various helpful landowners, loggers and sawmills included considerable walking of forestland to find the large and long enough spruce and ash timber. We ordered Douglas fir for the six 10x10x50-ft. tie beams, which at the time I did not believe I could find in Eastern spruce. Today I believe that I could have found them.

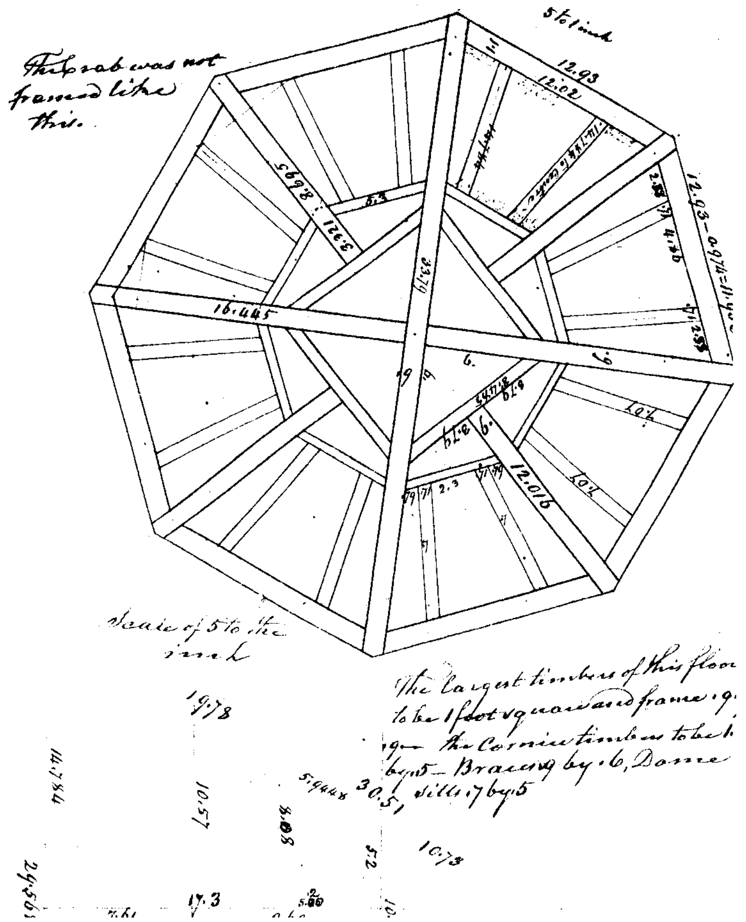
The trusses were fully framed on the ground and cambered according to directions found in Peter Nicholson's *The Carpenter's New Guide* (1837). To produce camber, Nicholson advocates that principal rafters be lengthened and "forced in framing," rather than shortening the kingpost. Lengthening the rafters forces the kingpost upward, dragging the middle of the bottom chord with it. The longer rafters also compensate for shrinkage and compression that normally occur across the flared kingpost head, allowing the truss to sag. Despite Nicholson's advice, builders often added camber to trusses by shortening kingposts or using tie beams naturally curved or hewn to a camber.

The assembled trusses were then flown into position atop the plates and their closed mortises engaged, laboriously, with the 32 tenons awaiting each side of each truss. These tenons belonged to longitudinal girts and bracing entering the kingposts, purlins entering the upper rafters and many ceiling joists entering the tie beams (Figs. 18–19). At this point in my heavy-timber-framing career, I had not discovered that practical framers of the past had invented at least four methods of insertion (TF 76:23) to avoid having to engage all the ceiling joists simultaneously.

Framing the steeple. The original Weathersfield steeple was lodged at the bottom upon two 12x12x28-ft. spruce sleepers, one fortunately still in position after the fire. The sleepers began at the

Fig. 20. Charred remains of inset plate, front tower post and long sleeper to carry steeple. New material is part of front-wall shoring.

Fig. 21. Weathersfield tower frame in place, well braced and studded for exterior finish. Posts descend to long sleepers in attic and carry short sleepers halfway up to foot belfry posts. See also Fig. 17.



University of Vermont Special Collections

Fig. 22. John Johnson's drawing of crab 34 ft. in diameter, for Centre College (later the University of Vermont), 1829, freely adapted for crab atop the belfry at Weathersfield. Johnson worked in decimals.

inset plate on the interior of the front brick wall and ran back to span two kingpost roof trusses (Fig. 20). Two 10x10 vestibule posts rose from the ground to the underside of the first interior truss to help carry the rear steeple load. The plate, sleepers and vestibule posts were all replaced in dimension, species and joinery.

We framed the three stages of the steeple independently on the ground. By using this method, we never needed to scaffold the steeple to its full height of 70 ft. Rather, scaffolding never more than 24 ft. tall was simply moved about in the yard.

The first stage, the square tower, 14 ft. on a side and 25 ft. tall, had abundant crossing bracing and a square of short sleepers lodged halfway up to accept the bottoms of the eight telescoping belfry posts. The sleepers lodged on girts tenoned into the 10x10 tower posts, and the two girts that carried the four extended ends of the 12-ft. belfry crib were reinforced by mortised 4x6 studs that dropped to the long sleepers below. The tower frame made itself further useful and expressive by providing rigid centers for the cased newel posts in the bell deck balustrade (Fig. 21).

The octagonal belfry frame would have been relatively easy to reconstruct correctly, since one entire belfry post had fallen to land in front of the church intact and very little charred. Our pattern was thus a 10x10x28-ft. spruce timber, square for the first 10 ft. and five sided for the upper 18 ft. to provide correct octagon facets. Ironically, we made changes in reproducing these posts. While we copied their size and shape exactly, we added eight new sets of mortises for girts and crossing braces between them where their lower portions would be concealed within the tower. We made this modification in response to church members' accounts of instability and vibration in the original unbraced posts under high wind or other loads. The parishioners had even ceased ringing the bell in recent years because of vibration in the posts. Throughout the 20th century (and perhaps earlier), plank had been spiked and bolted to the octagon posts in an attempt to stiffen them. We used instead a contemporaneous reinforcing system seen in similar Vermont churches such as the Strafford Meetinghouse (1799) and the Norwich Congregational Church (1817).

The design for the major crab, the eight-legged horizontal frame that supports the lantern, was suggested by some burned remains and a complete period design provided by examples from the John Johnson papers at the University of Vermont (Figs. 22 and 24).

The desired framing of the lantern, the top stage, was less obvious. We had the remains of a mast that anchored the weathervane and rose the full height of the lantern and its cupola roof. How it could pass through a crab was unclear, so for the lantern crab that would carry its cupola roof we altered the typical design of the period by turning its main axis into partners, parallel timbers spaced apart, to accommodate the mast we knew to be original. The ogee rafters of the lantern's cupola roof were cut from solid 2½x18 white pine plank, and careful adzing continued their curve onto the supporting crab legs (Fig. 23 overleaf).

Photos Jan Lewandoski

Fig. 23. Mike Cotroneo fairing legs of lantern crab to cupola roof's ogee rafters. An adze in good hands remains unsurpassed for such work. Lantern posts will tenon to major crab atop belfry in Fig. 24.

Fig. 24. Belfry frame with major crab installed on top to carry lantern. Workers rig heavy lifting timbers that will bear under the crab to transport the assembly to the meetinghouse.

Erection. I believe that the Weathersfield steeple was designed with the possibility of framing and finishing the belfry and cupola within the vestibule and framed tower of the meetinghouse, and then bringing it up from within using the tower posts themselves as gin poles (TF 36:6). I couldn't generate much enthusiasm for this method among the owners and architects and the other tradespeople, so we assembled the stages in the yard and allowed the roofers, finish carpenters and painters to do their work (Fig. 25). Once the lantern roof was sheathed I inserted and wedged the weathervane shaft 18 in. deep into the top of the lantern mast.

A 75-ton hydraulic crane, our modern powered version of an external gin pole, first placed the framed tower onto the sleepers crossing the truss chords. Next the crane placed the cupola with its eight post-bottom tenons, eight descending brace tenons and the mast tenon, onto the crab atop the belfry, still on the ground. The belfry sleepers were then flown into place on the steeple tower at the mid-tower girts. The crane lifted the belfry-cupola combination above the tower plate level and then lowered it to the waiting sleeper mortises inside. The cambered bell girts were next; they would have run afoul the belfry octagon bracing had they been in place already. The bell itself was then flown in alongside the belfry, transferred to comealongs within the belfry and hung from the crab, allowing carpenters and roofers to complete the bell deck below them. The steeple assembly took two days. On one day the tower was inserted. On the next all the other stages and heavy girts and the bell were put in place. Flashing, skirting roofs and the remaining exterior woodwork then followed.

Finish work. The wood and metal finish work to cover and decorate steeples is generally performed at a high level of skill suiting a monumental public building. Though finish work isn't our main subject, some applicable principles are worth stating.

Fig. 25. Lantern covered by soldered copper cupola roof, topped by mast with orb (weathervane unseen above), awaits lift to top of belfry. Timbers just under cornice will be strapped to lift the stage.

Fig. 26. Belfry frame with lantern (unseen) lifts off. Scale of work, hardly apparent once steeple is in place, is plain enough near the ground. Braces and girts were added to frame design to counter vibration.

To look right or to be seen at all, everything high above must be larger than it appears from the ground, sometimes by a factor of 100 percent or more depending upon the height and scale of surrounding elements. Consequently, in historic steeple design, ornament and trim tended to be robust and projecting. When estimating repairs to a steeple, often done without full access to the exterior, assume that moldings, dormers and ornament are much larger than they look. If you can, go up in a crane basket and actually measure them.

The higher the work on the steeple, the better (more expensive) should be the materials and methods. The reason is the difficulty of maintenance at heights. No one may examine the vane and its flashing, the spire covering and ornament or anything else above the bell for 50 or 100 years. Consequently you should build to last. No caulking can last that long, so it shouldn't be the main line of defense against water at any point. Rather, detail the metal and wood to shed water. Asphalt shingles or roll roofing, with their short lifespan, are bad choices for high work, particularly on the low pitches of the skirting roofs that surround the stages, or on the bell deck, where folded or soldered metal is preferable.

Pressure-treated wood inside is of no special virtue. Occasionally it might be advantageous outside where runoff or splashback tends to continually soak trim lumber. If water is getting in, it has already rotted sheathing or ornament and eventually will saturate the attic insulation (an occasional source of catastrophic ceiling collapse) and drip through a plaster or wood ceiling. There is no substitute for initial self-preserving design and construction, particularly when regular inspection and maintenance are normally deferred.

Don't substitute synthetic materials for wood. The wood you replace may have lasted 100 to 200 years, and you should not assume that you will get the same performance from fiberglass, vinyl or aluminum, or composite wood products. Even freedom from painting lasts only a few years as synthetic material starts to discolor. Only at the greatest expense of custom fabrication can you get synthetics to mimic the shapes of historic molding and ornament, and the experienced eye can identify them as wrong, by the way they reflect light and weather, even miles away.

Fig. 27. Completed steeple frame installed on meetinghouse, awaiting bell girts and bell and considerable finish work. Crab with partners to allow passage of central mast visible in ceiling of lantern.

—Jan Lewandoski

Jan Lewandoski (jlrt@sover.net) operates Restoration and Traditional Building in Stannard, Vermont. This article is fourth in a series on historic American timber-framed steeples. Ken Rower, Jack Sobon and Ed Levin assisted in steeple research.

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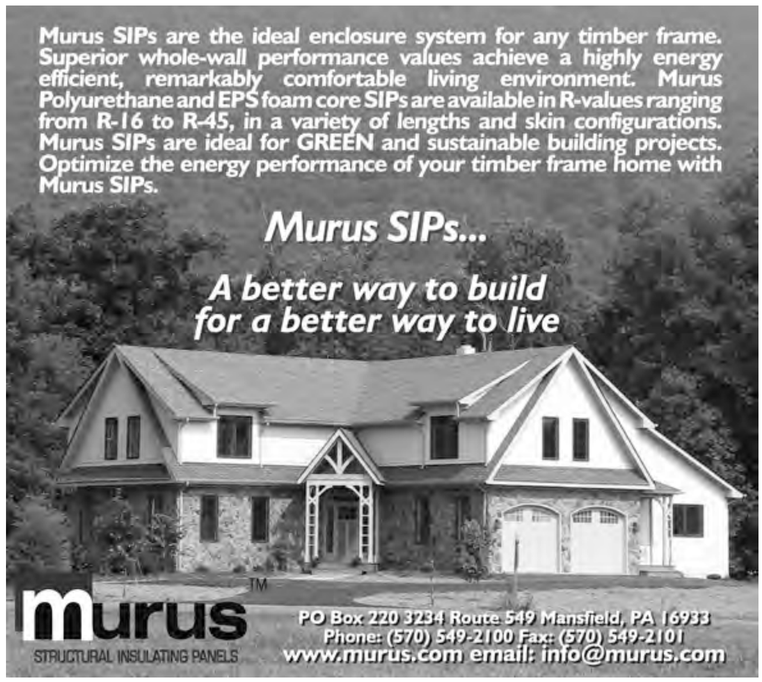
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