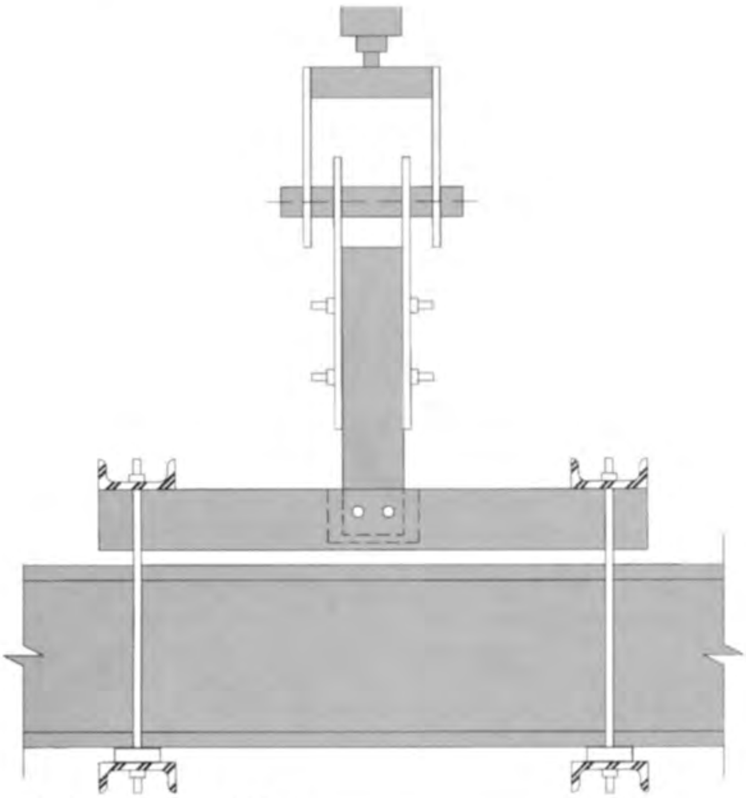
TIMBER FRAMING

JOURNAL OF THE TIMBER FRAMERS GUILD

Number 38, December 1995



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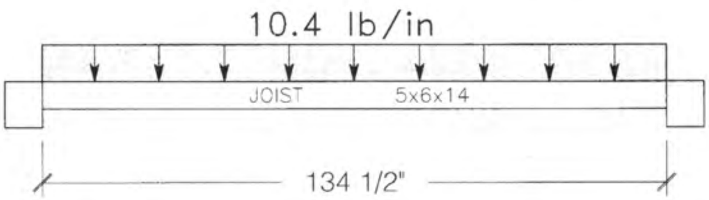

Joint Engineering

TIMBER FRAMING
INCORPORATING TIMBER FRAMERS NEWS

Number 38 December 1995

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Published Quarterly

Timber Framers Guild of North America
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Subscription $20.00 Annually
Or by Membership in the Guild

ISSN 1061-9860

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TOPICS

Ontogeny or Philogeny?

THOSE of you who attended the Guild's foundation meeting in Hancock Shaker Village back in 1985 will have little trouble recalling the passion and contentiousness that were our birth pains. We worried the bone of joinery standards until we (and it) were exhausted, and then we turned to our identity crisis.

I continue to believe that our choice of *Guild* as our totem is emblematic of the deeper themes that brought us, and continue to hold us together. Little in my travels and conversations over the last several years has shaken this conviction.

Except.

Except that life has become (or revealed itself as) more complicated than we first suspected. As we spread our message of quality and passion around the continent through hard work, example and luck, many of our founding companies began to feel compelled by circumstance to grow. Any other industry would consider this a sign of success. Many member companies took this as a larger issue: what do we want to be when we grow up?

Some companies consciously took the Small is Beautiful option, but many others took the plunge.

Are there any among us who came to the craft as business men and women first, timber framers second? Precious few, likely none. Perhaps it's residual 60s guilt or some inherited anti-establishment gene; the common theme in the past few years of conversations has been that growing a business (in Paul Hawken's phrase) is harder work, less fun and much more complex than building a timber frame. Many's the member company that thought it had pioneered the mistakes (occasionally fatal) of sloppy contracts, casual accounting practices, insurance shortfall, tax ignorance, incomplete specifications and the host of other ills our corporate flesh is heir to, only to discover in a tap-room conversation or conference workshop what a large and growing club it belonged to.

In the background of all this activity is the low-frequency hum of the real world refusing to recognize us for what we thought we were: Counsel (the aptly named Mr. Marlarkey) to, and by extension the IRS itself, consider the Guild a trade association, and only incidentally (and inconsequentially) a doer of good deeds, a community of historians and standard-bearers for a permanent, appropriate building technology. Owner-builders have been charmed by our ways, but not so much so as to overlook our lapses as professional builders (and even to take advantage of this in court). Professional builders and bankers and real-estate appraisers like our enthusiasm, admire our skills and work habits and remain unconvinced that we add any real value (by their accounting) to a project. Big-time general contractors may hide their cynical smiles behind their hands, praise our craft and methods, and wait to harvest us from the bush of naïve, ripe, sub-contractors anxious to work on a public prestige job. Many professional architectural firms have difficulty seeing us as providing more than expensive trim ('trimber-framing') and remain unconvinced of the aesthetic and pragmatic power of an integrated structural and insulation system.

Add to this kettle the increasingly plaintive and (sometimes) strident voices of member business owners, many of whom feel ill-served, and occasionally scorned, by traditional Guild events, activities and attitudes, and their attendant expense. Consider the on-going conflicts of whole-house versus frame-against-the-sky philosophies, and the increasingly frequent lamentations of uneven quality (in craft and profession) raised by member companies. Recognize that there are perhaps more non-member than member companies advertising themselves as professional timber framers, and acknowledge that many of these non-member companies quit (the Guild) in disgust or never joined at all for reasons similar to the ones outlined above.

Hence, the Timber Frame Business Council. Born March '94 in Texas. Explored at Skamania (November '94). Born again February 1995 in Colorado. Debated at length in Williamsburg (June), topic of much e-mail and fax traffic between times. Codified in Louisville (October). Refined and finally released into the air at Semi-ah-moo (November). The Guild has caused to come into being a separate corporation that will allow business members to focus undivided attention and energy on professional issues. And free the traditional Guild to rededicate itself to the educational, historical and spiritual pursuits that attracted us to timber framing in the first place.

The mechanics are simple enough: individuals join the Guild, organizations join the Business Council. Business Council members will be Guild members. The Guild will eventually reincorporate under the tax code, making it, among other things, more attractive as a target for charitable contributions and grants. A simple majority of the Business Council Board of Directors will be appointed by the Board of the Timber Framers Guild.

In a conscious and hard-won decision, the Guild is making this happen from within the organization. The formation of the Business Council has been initiated with Guild funds and administrative services. It is the stated goal of the Business Council to become free-standing as soon as possible, and to provide substantial support for the activities of the traditional Guild.

The traditional Guild will become a simpler organization, regaining the clarity of purpose that was so evident in Hancock, resolving the schizophrenia of our recent history. The new Timber Frame Business Council, comprised of a single delegate from each member company, has a similar clarity, but a different purpose. We think we're on the right track here, and hope you'll join us. —JOEL C. MCCARTY

TIMBER FRAMING, Journal of the Timber Framers Guild of North America, reports on the work of the Guild and its members, and appears quarterly, in March, June, September and December. ♻

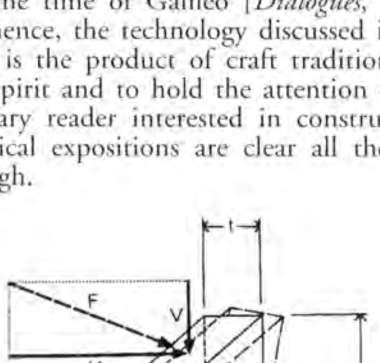
BOOKS

Architectural Technology

Architectural Technology up to the Scientific Revolution, the Art and Structure of Large-Scale Buildings, edited by Robert Mark. The MIT Press, Cambridge, 1993. 8 x 10 in., 252 pp., 206 illustrations. Hardbound, $39.95.

NINE architectural historians and two research engineers have here produced a well-organized work that treats, in separate but parallel-structured chapters, Soils and Foundations, Walls and Other Vertical Elements, Vaults and Domes and, of major interest to the timber framer but by no means the only such chapter, Timber Roofs and Spires. The organizing principle of each chapter is historical: following an introduction to the subject at hand, which can as necessary include schematic drawings and short equations, the principal author or authors take the reader through Greece, Rome, Byzantium, then Early Medieval, Romanesque, Gothic and Renaissance Europe, including England.

These subjects are bracketed by an introduction and a conclusion. The former provides useful insights into the limitations of purely geometric design and the material effects of scale, information on materials and mensuration and the key observation, "The kind of mathematically based predictive engineering we know today was unavailable before the time of Galileo [*Dialogues*, 1638] and hence, the technology discussed in our book is the product of craft tradition." In that spirit and to hold the attention of the ordinary reader interested in construction, technical expositions are clear all the way through.



"Overturning occurs after the base section is cracked and the applied bending force* (H × y) *exceeds the 'righting moment'* (W + V) × (t/2) *set into play by the downward forces tending to rotate the wall oppositely about its outside edge. Hence raising the wall (to increase its weight, W) or splaying out the wall base (to increase its thickness, t) helps to stabilize it."

As for the concluding chapter, the authors review "the factors leading to design success" in the monuments reexamined and propose that they are "predicated on an intimate relationship between the process of design and of building," a relationship that "came to a fitful end across most of Europe during the Renaissance," when "master builders were displaced as principal designers of monumental buildings by artist-architects." Of course this shift proved irreversible and persists to this day in the arrangement whereby the architect inhabits the cultural milieu of the patron and the builder stands a bit to one side. Buildings are credited to those who design and pay for them, not to those who build them. While noting that "until the 19th century, the level of structural experimentation in European architecture never again approached its Gothic zenith," the authors take the broad view that culture is the ultimate beneficiary of the professionalization of the architect, that in the long run the divorce has been a stimulus to experiment.

THE 45-page chapter on timber roofs was written principally by Lynn T. Courtenay, whose studies, both alone and with Robert Mark, of Westminster Hall established that Hugh Herland's masterful 68-ft.-span hammer-beam roof frame (1397) does not behave as a trussed-rafter system taking most of its support at the tops of the walls, but that the main support is well down on the walls, at the level of the masonry corbels, from which spring the great arches, the hammer beam braces and the wall posts. The assertion is supported by evidence that the original design specified an air space behind the upper half of any wall post, "ensuring that it would not bear against the wall behind it."

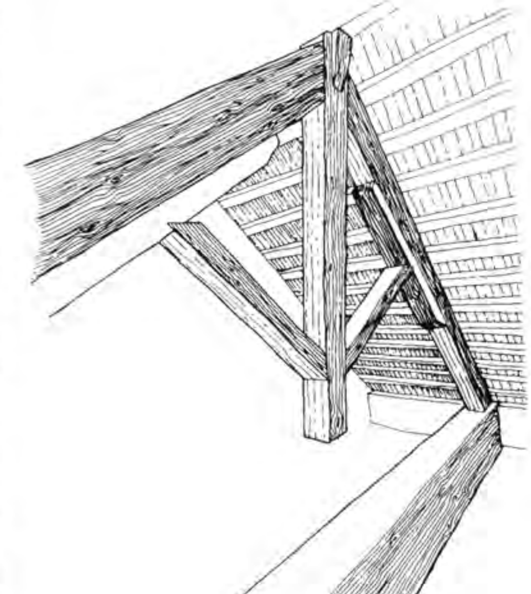
Courtenay is assured and informative in dealing with framing from the Parthenon (432 B.C.), whose west purlins measured, on the evidence of extant masonry pockets, nearly 3 ft. square in section, to the tied-arch iron-bound roof framing with its bolt-of-lightning scarfed tie in Wren's Sheldonian Theatre at Oxford (1666).

The Romans (replacing the Greek system of prop and lintel), developed the stable, base-tied timber truss and used it successfully over spans of 50 ft. and greater, occasionally 100 ft., and devised an ingenious system of hanging scarfed lower chords. Later, in northern Europe, after the early medieval period, trusses and purlins cease to appear in large structures; instead we find what Courtenay calls common-rafter or "equal-scantling" roof frames, with rafter pairs tied at the base and perhaps collared, all relatively small-sectioned and on close centers, 3 ft. or less. This pattern changed somewhat as carpenters had to deal with "the greatly increased use of stone vaulting. . .that intruded into the region between the clerestory walls that was normally occupied by closely spaced tie beams." Notre Dame, Paris (ca. 1200), thus shows a tie-beam only at every fifth rafter-pair, with all pairs on nearly 32-in. centers and of uniform scantling, but with additional members in the tied pairs. The Renaissance south of the Alps renewed essentially Roman framing systems (which had never any reason to change from considerations of efficiency) and the design books of the period merely reflect the regular need for scarf joints to obtain long members.

It is reassuring to be guided by an architectural historian who knows not only the succession of styles, but understands very well what purlins, rafters and ties do, what allows a lap joint to resist tension, why the tabled scarf evolved.

Of spires we are shown only two (Notre Dame, rebuilt 1860 and Amiens, originally ca. 1240, rebuilt 1532) and the framing for the base of a third (Ely Cathedral, 1335), perhaps because of a paucity of extant examples or records of lost ones—they do regularly burn or blow down from natural causes. But this part of the discussion could happily have been longer.

The chapters on foundations, walls, domes and vaults naturally treat stonework above all, and while it has always been remarkable what can be done with stone, and how long it stays put, wood does offer its assistance permanently or temporarily in the form of pilings and grillage for foundations and centering, sometimes intricately built, for arches and vaults. So the woodworker will often feel at home in these chapters. But the chief sensation is awe at the daring and accomplishment of the early builders and the curious purity of stone itself. Not all work is successful or everlasting—domes crack and walls tilt—and late work in the Renaissance grows structurally complex, with admixtures of materials such as the timber, lead, brick, stone and iron chain in the dome of Wren's St. Paul's (1675-1710). By then we have entered the era of separation of powers between architect and builder. —Ken Rower



St. Catherine at Mount Sinai, sixth century roof truss, no ceiling load.

A Dining Rotunda in Syracuse

PIONEER Development, of Syracuse, New York, was well acquainted with the work of our firm. We had recently completed for them a series of pavilions, kiosks, bridges, benches and docks for an award-winning industrial park in nearby Rochester, using salvaged and re-claimed longleaf southern yellow pine and traditional joinery to replace the specified pressure-treated material and heavy metal connections. So when Ross Wooley (of Wooley-Morris, Architects, Syracuse) gave them a plan for Liberty Commons, a retirement residence that included a cylindrical dining room, 30 ft. in diameter and 34 ft. high, with heavy timber posts and a 16-ft. kingpost overhead, they turned back to us.

The developer was interested in minimizing initial construction costs while holding as closely as possible to the original design. We were asked to review the plans with an eye toward simplification and lowering costs. What we found was a graceful and dynamic two-and-a-half story circular space enclosing an array of steel tension rods suspending a kingpost on which the inner ends of all the roof rafters could rest. In this configuration, the roof load not taken directly by the wall posts was transferred down the king post to the tension rods and thence to the compression spokes joining the wall posts and the kingpost.

This approach was unfamiliar to those of us so used to working with wood in compression. Yet as we got used to the idea of using the steel tension rods, we came to realize that this was indeed an elegant solution, an excellent design and financially efficient. We loved it. Our only suggestions to the bean counters were to segment, rather than loft, the roof, and make the central kingpost untapered. Our first suggestion was accepted. We were relieved, though, when the architects successfully argued to keep the bi-directional taper in the kingpost, for it made this large mass of wood overhead a graceful and proportioned statement of loading and space.

Because of the importance of dimensional stability in the members, we again chose salvaged and re-sawn timbers as our medium, specifically Douglas fir for its high strength-to-weight ratio and available large sections free of heart center. We wanted to minimize heart checking in this instance for two reasons. In the 16 tall posts 26 ft. high that ringed the dining area, a well-travelled public space, we felt that deep heart checks might be structurally misinterpreted or visually unacceptable. We also needed very stable kingpost stock for we had decided to glue up the eight tapered sides, leaving the center hollow, rather than try to get the kingpost from a solid piece. This decision added labor costs while lessening material costs, increasing our predicted stability and minimizing the weight of the piece. The 48 connecting girt beams, 6 ft. long, were bandsawn inside and out to follow the cylinder. Diagonal bracing was omitted, with engineering taking into consideration the contribution of plywood sheathing and the buttressing of the large connecting wings (apartments and offices) to calculate loading capacities. No attempt was made to hide signs of previous life in the timbers.

Architect Ross Wooley was frank in reminding me that many of our best inspirations come from the work of others, and this was no exception. Charles Rennie Mackintosh (1868-1928) is recognized as one of the most important Scottish architects since his countryman Robert Adam. His generally acknowledged masterwork, The Glasgow School of Art, uses a series of similar kingposts with tension rods in one of its long (if rectilinear) gathering halls. The elevation of one of those trusses rotated 360° produced the design for the Liberty Commons dining room. This remarkable construction serves its function well while offering a powerful and unique focal point for the building.

The developer certainly earns credit for allowing this part of the project to be completed with little compromise. Yet I smile broadly when thinking back to one of our meetings when we were told that the whole room should be changed, since the residents would never feel comfortable is such a space. In a recent visit to take photos, it was made quite clear to us by everyone there that the Central Rotunda is the residents' very favorite space. —JONATHAN ORPIN

Jonathan Orpin directs New Energy Works Timberframers in Shortsville, New York.

Charles Rennie MacIntosh influence is plain in the exterior design as seen in the photo lower left, facing page. Assembly shown below left gives new meaning to the term "bent." Below right, the completed dining room.

Photos Jonathan Orpin

Load Behavior of Connections with Oak Pegs

IN CASES of renovation, reconstruction or new construction of framed buildings, there often exists the problem that proof of stability and load capacity cannot be shown by structural calculations when traditional carpenter-style connections are used exclusively. The grounds for this are:

- The loads used in design today, in particular wind load.
- The increased live load produced by reuse of a building for a museum, meeting place, restaurant, etc.
- The structural changes produced by reuse involving the removal of joists, supports, etc.
- The low allowable-stress specified in German Industrial Standard (DIN) 1052 for oak timber, since the specification for wood quality corresponds to ungraded timber.
- The absence of design specifications for the type of connection.

A particular problem in trying to establish load capacity of the connections arises when dealing with wind loads that very often require tension connections. It shall be shown in the following tests how such connections can be designed in the form of traditional carpenter-style connections with wooden pegs. In reference [1], allowable stresses for strength-graded, high-strength, oak timber are reported. According to the author, these allowable stresses might be up to 50% higher than those given in DIN 1052.

2. Construction Considerations. When combined with long-term loads, under certain circumstances, wind loads cause tension forces in some connections of framed buildings. On the other hand, these long-term loads cause, as a rule, substantially greater compressive forces in these connections. For example, this can be the case for the post-sill connection as depicted in Fig. 1. The large compressive force D from long-term load can usually be distributed over the area of contact without difficulty. For the distribution of the tensile force Z as the resultant of the loading case of long-term loads and wind, however, no such easy solution can be found. So in reference [2] only the dovetail connection is used in tension.

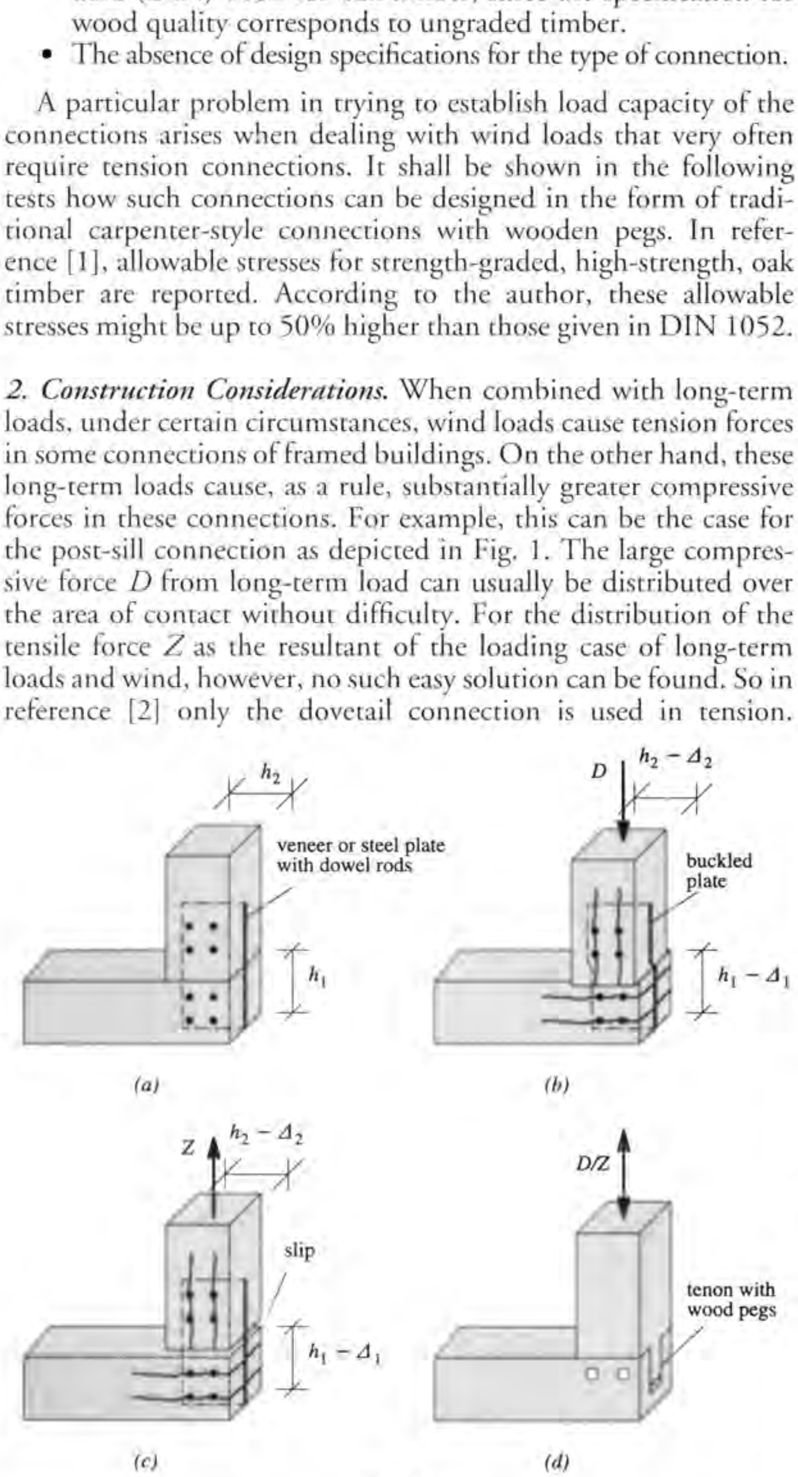


FIG. 1.
POST-SILL CONNECTION VARIATIONS.
(a) Unloaded, M.C. >30%, (b) Loaded in compression, M.C. <15%, (c) Loaded in tension, M.C. <15%, (d) Loaded, M.C. independent.

However, the dovetail connection possesses only small load-bearing capacity when unseasoned wood is used, and was previously used only for the connection of foot and head braces and crossbeams. Therefore, in practice, traditional building methods are not followed and the so-called engineering-style connection, which uses a fastening plate and [often metal] dowels (Fig. 1a), is chosen as the only solution.

The use of fastening plates of steel or veneer in framed-building connections can lead to considerable deficiencies in construction as well as aesthetic quality if the shrink-and-swell behavior of the wood is not considered. For cost reasons, material with a moisture content of 40-90% is used, for the most part, in framed oak work. In later years, the oak dries to an equilibrium moisture content of approximately 10%. This can lead to substantial cross-grain tensile stress, primarily at the fastening points.

With connections using sheet steel or veneer plates and dowels, the fastening plates cannot conform to the shrinkage deformation. Inevitably cross-grain tension splitting occurs in the region of the dowels (Figs. 1b and 1c).

As a result of contraction of the sill, the fastening plate will buckle or kink because it was not designed for the compressive force D, but rather for just the tensile force Z. With additional wind loads, the plate must straighten to transfer Z, which causes slippage in the connection, and results in cracking in the building under certain circumstances.

Therefore it is structurally better to produce the connections in a carpenter's style with tenon and wood pegs as shown in Fig. 1d. Naturally, contraction cracking can not be completely eliminated with this connection. However, the cracking is not caused by confinement. Above and beyond that, the unfavorable influence of compressive force on the tensile load bearing capacity is avoided.

An additional important disadvantage of the metal plate lies in its large coefficient of thermal conduction. In order to meet suitable fire resistance requirements, the metal plate must be insulated, at high cost. In contrast, wood pegs require no additional protective measures.

Unfortunately up to this point, this carpenter-style connection in new framed construction could not be considered because no allowable loads have been available for wood pegs. An exception is wood pins up to 20 mm (about ¾ in.) in diameter, for which permissible values are given in reference [3]. However, for the additional forces in framed buildings, these allowable loads (up to a maximum of 1.4 kN, or 315 pounds) are insignificant due to the small diameter. It has been shown that, in framed construction, the wood peg diameter should be a minimum of 28 mm, or about 1⅛ in.

A report on tests of compression connections with 16-mm (⅝-in.) diameter dowels, made of synthetic resin pressed solid wood, is found in reference [4]. Tests of tension and compression connections with pegs of synthetic resin pressed wood up to 20-mm diameter are reported in reference [5].

3. Production of Wood Pegs. In the past, wood pegs [*holznäglen*, literally "wood nails"] were produced exclusively by hand through splitting of choice blocks or boards. In this way, straight-grain fiber orientation in the wood peg was guaranteed and inclined grain was excluded to the greatest extent. Partially as a consequence of these production methods, the wood pegs were quite irregular in cross section and hardly suited for use in a structural investigation involving larger forces.

The production of the oak pegs that were used for these tests (Fig. 2) is described briefly as follows. When choosing the wood, the fiber orientation and the number of knots and other wood

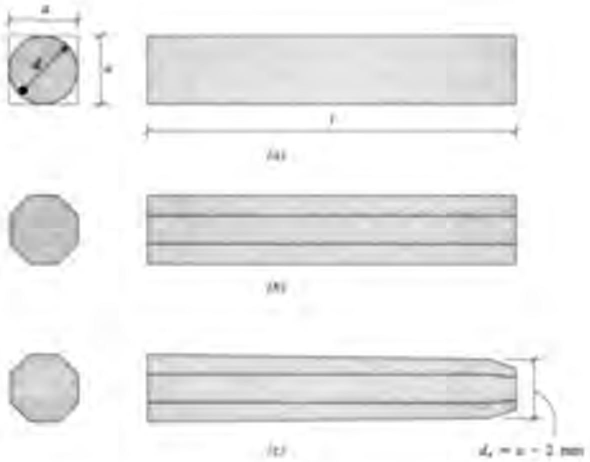


FIG. 2. PEG FABRICATION STAGES FROM SQUARE TO TAPERED OCTAGON.

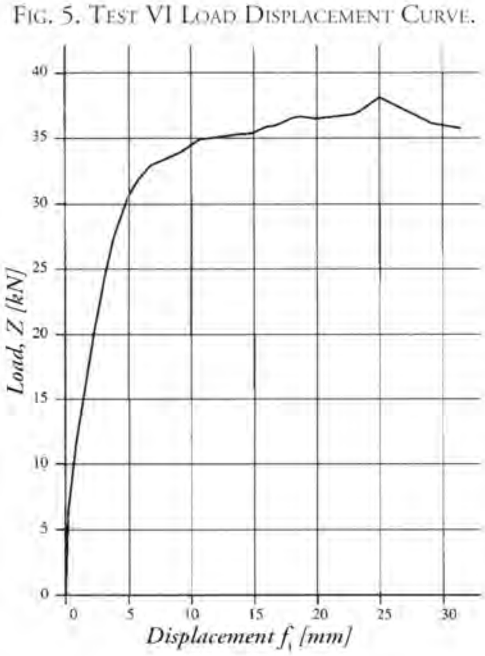
defects were the most important criteria in the attempt to keep the inclined grain as minimal as possible. From the selected wood, square strips were cut with a width and thickness *a* corresponding to the diameter *d* of the drill hole (Fig. 2a). The pieces cut to the length of the future wood pegs were later planed into an octagonal cross section with a hand plane (Fig. 2b). After this the wood pegs were conically planed along their entire length so that their end diameters were approximately 2 mm less than that of the drill hole. Additionally the wood pegs received a bevel on the smaller end so that they could be driven easily (Fig. 2c).

*4. **Preliminary Tests with Wood Peg Connections.*** In the process of reconstruction of the Knochenhauer office building in Hildesheim, Germany, preliminary destructive tests of a total of twelve post-sill connections with wood pegs were conducted in the Laboratory for Wood Technology at the Technical University, Hildesheim. This work was supported by the German Society for Wood Research (DGfH) and was under contract from the Marktplatz Hildesheim, GmbH. The objective of the tests was to determine just how well oak pegs could transfer tensile forces at the fastening points. Simultaneously, the cross-sectional dimensions of the pegs and their placement in the connection region were to be optimized, so that all the parameters that influence the load-bearing capacity of the connection (tenon thickness, end-distance, edge-distances, etc.) were balanced as equally as possible.

Simple mortise and tenon connections, secured by wood pegs against tensile forces (Fig. 3), were studied. The length of the mortise in the sill was always greater than the breadth of the tenon, in order to exclude the favorable influence of a tight fit between mortise and tenon on the load-bearing capacity of the connection. The dimensions and results of tests V-XII are summarized in Table 1 overleaf.

The first four tests were carried out with wood pegs of 20-mm (¾-in.) and 24-mm (15⁄16-in.) diameter. It was quickly shown that the stiffness of such pegs was not sufficient to achieve an equivalent carrying capacity of steel dowels of diameter 10-12 mm. Therefore these tests will not be described here in any greater detail. The wooden peg diameters in tests V and VI were then enlarged to 32 mm (1¼ in.). With a cross-section of 10x14 cm (about 4x5½ in.) for the post and sill, the end-distance in the tenon and the edge-distances of the pegs from the top edge of the sill came to only 1.3 times the peg diameter (Fig. 4, page 9). These conditions can be observed as a general rule with historical construction of wood peg connections.

FIG. 5. TEST VI LOAD DISPLACEMENT CURVE.



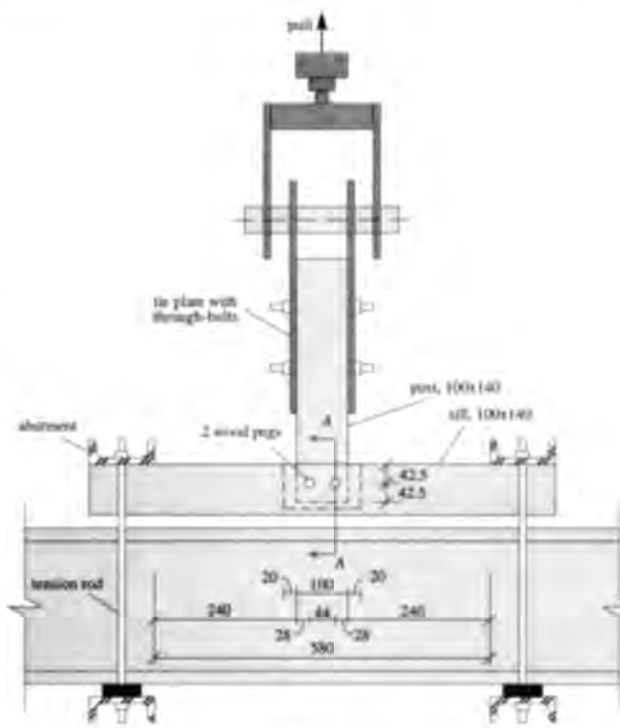
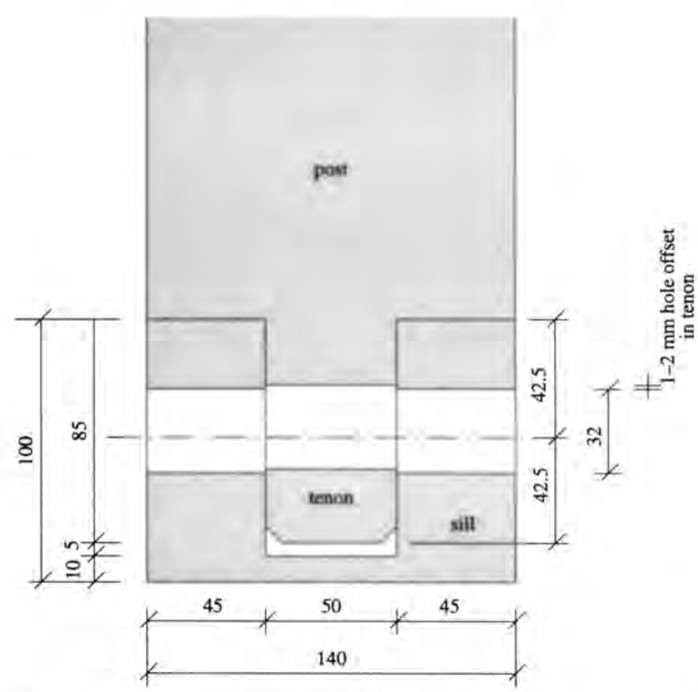


FIG. 3. SET-UP FOR PRELIMINARY TESTS V-XII.
Edge-distances for tests V and VI, dimensions in millimeters.

BELOW: SECTION A-A, DIMENSIONS IN MILLIMETERS.



In Fig. 5 the measured load-displacement curve for connection VI is reproduced. At the breaking load Z_u = 36 kN (about 8,100 pounds), cross-grain tension strength of the sill and the bending strength of the wood pegs was reached. At a deformation in the connection of f_1 = 1.5 mm (less than 1⁄16-in.), the tensile force amounted to more than a third of the breaking load Z_u.

These promising results gave expectations that, with larger post and sill cross-sections and four wooden pegs as a means of connection, still greater forces could be transferred in the connection. In tests VII and VIII, the sill height was increased to 16 cm (about 6¼ in.) in order to allow 4 pegs with identical edge distances as before (Fig. 4). However, these tests did not lead to the desired results, as shown in Table 1. Only the substantially greater wood dimensions of test IX yielded a noteworthy increase in the breaking load. However, it was shown through the bending failure of the wooden pegs that the slenderness λ of the pegs, where a_m is the peg length embedded in the tenon,

$$\lambda = a_m \div d = 80 \div 32 = 2.5,$$

was obviously too great. After an increase of the peg diameter d to 40 mm (1⁹⁄₁₆-in.), the connections XI and XII finally produced breaking loads of Z_u = 25 kN (about 5,625 pounds) per peg. These loads are comparable with those in reference [6] for steel dowels with diameter d = 16 mm. It should be noted further that for test XII, the connection of test IX was reused after the peg holes were redrilled to 40 mm. In addition, the tenon had already cracked along its entire length.

5. Application Examples for the Load Bearing Wood Peg Connections. Based on the described test results, 7,500 oak pegs were used in the reconstruction of the Knochenhauer building in Hildesheim. Among other things, through the use of wood pegs, the wood connections of the 26-m (85-ft.) tall framed building with its eight floors could be carried out in traditional carpenter style, as was intended by architecture Professor Klose. The wood pegs' large fire resistance capacity also spoke well in comparison to metal dowel connection methods.

Due to the electromagnetic properties of metal connection hardware, a new design was specified by the firm Siemens AG for construction of a radio wave interference measurement platform with a 6 x 8 m (about 20 x 26 ft.) base using nonmetallic connections. Therefore, for the transfer of wind loads, mortise and tenon connections were used with 30-mm (1³⁄₁₆-in.) diameter oak pegs. All members were fabricated from laminated wood planks in order to reduce the danger of shrinkage cracks in the connections. The structural safety of the wood peg connections was verified through consideration of the test results listed in Table 1.

6. Conclusions. The described preliminary tests and both of the completed framed construction projects show that oak pegs have been unjustly forgotten as a means of connection in modern wood construction. From a 28-mm (1⅛-in.) and above diameter, the wood peg can be used as a load-bearing connection method, in particular for the transfer of wind loads. Under certain circumstances, the pegs also have definite advantages in fire resistance and freedom from corrosiveness as compared with steel connecting methods. The determination of minimum distances and permissible shearing forces, in the framework of DIN 1052, is an important task for the future. —MARTIN H. KESSEL AND RALF AUGUSTIN

Prof. Dr.-Ing. M. H. Kessel is Director and Professional Engineer, and R. Augustin is Co-worker, at the Laboratory for Wood Technology, Technical University Hildesheim-Holzminden, Germany. *This article is the manuscript of a lecture given by Dr. Kessel on the occasion of the European Campaign for the Rural Environment, March 2, 1988, at the quarters of the firm Haacke & Haacke, Celle, Germany. The article first appeared in German in the journal* bauen mit holz *(Building with Wood), pp. 246-250, April, 1990, published by Bruderverlag, Karlsruhe.*

The article was translated and its illustrations redrawn by Matthew D. Peavy, Undergraduate Research Assistant, Department of Civil and Architectural Engineering, University of Wyoming at Laramie (WY 82071) and Richard J. Schmidt, Associate Professor in that department (schmidt@uwo.edu). Several photographs were included in the original paper to illustrate the test apparatus, failure modes of the joinery and the two built structures mentioned in the text. The translators regret that these photographs could not be reproduced here. A further article describing Dr. Kessel's research into oak-pegged mortise and tenon joints will appear in the next issue of TIMBER FRAMING.

TABLE 1. RESULTS OF PRELIMINARY TESTS OF WOOD PEG CONNECTIONS.

Test		V	VI	VII	VIII	IX	XI	XII
Peg: no. x dia.	mm	2x32		4x32		4x32	4x40	
Post	cm	10x14		10x14		18x22.5		
Sill	cm	10x14		14x16		20.5x27		
Wood moisture content	%	70		15		70		
Tenon thickness	mm	50		50		80	80	80
Peg spacing	mm			48		64	90	72
Tenon end distance	mm	42.5		48		85	90	81
Sill edge distance	mm	42.5		48		85	120	81
Failure locations *Modes* Pegs: bending Sill: cross-grain tension Tenon: shear or notch effect		Sill, Pegs		Tenon	Sill	Pegs	None*	Sill, Tenon
Max. total load, Z_u	kN	32.9	36.0	42.0	59.0	86.0	107.0	98.0
Max. load per peg	kN	16.4	18.0	10.6	14.7	21.5	26.8	24.5
Load at 1.5 mm disp.	kN	12.8	15.2	22.4	17.2	36.0	81.2	52.0
Load per peg at 1.5 mm disp.	kN	6.4	7.6	5.6	4.3	9.0	20.3	13.0
Shear stress in tenon at Z_u	N/mm^2	3.9	4.1	2.2	3.1	1.8	1.9	2.0
Tension stress in tenon at Z_u	N/mm^2	18.2	19.4	23.3	32.8	9.3	13.4	12.2
Bearing stress in tenon at Z_u	N/mm^2	10.2	10.9	6.6	9.2	8.4	8.4	7.6

*"Maximum test load, no break, undamaged"

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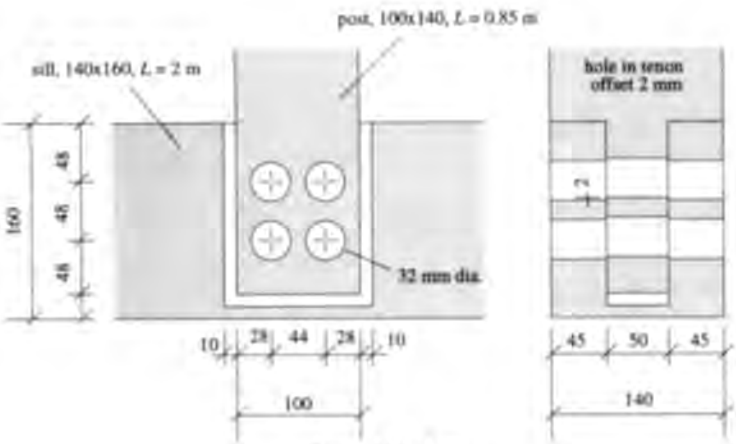
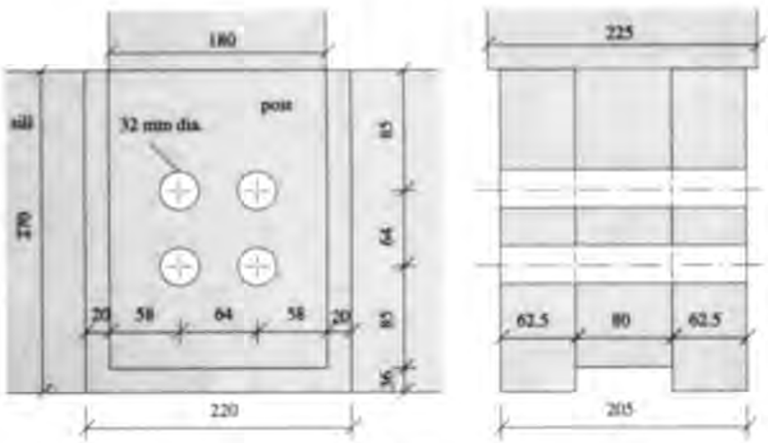
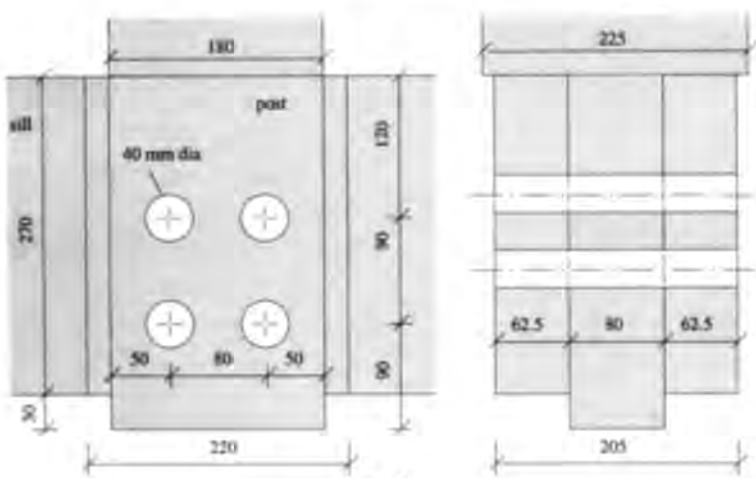


Tests VII, VIII

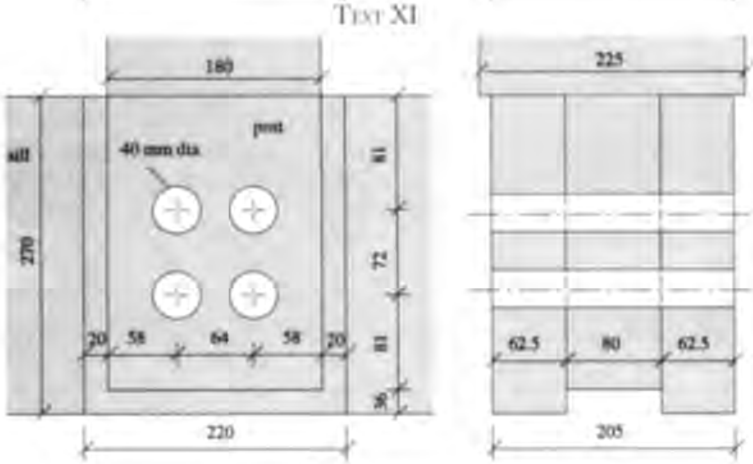


Test IX

Fig. 4. Dimensions and Peg Spacing for the Tests, Post-Sill Connection, Dimensions in Millimeters.



Test XI



Test XII

Commentary

CLEARLY, the research conducted by Dr. Kessel was stimulated by conditions that also affect timber framers in North America today: the lack of definitive technical data, and the attendant building code provisions, to guide the design of traditional timber framing joinery. In particular, Dr. Kessel focused his attention on the tension load capacity of a standard mortise and tenon joint. The objectives of this postscript to his article, which is assumed to represent standard or recommended practice in Germany, are to highlight the more surprising features of that practice and to speculate on their possible impact here.

Dr. Kessel emphasizes that his work deals with the design of mortise and tenon connections to resist short-term tension loading, primarily wind. He does not discuss connection design to resist long-duration tension loading, such as that developed by snow and dead loads in a roof truss. One could infer that German designers in such cases avoid the use of the mortise and tenon joint in favor of alternatives (for alternatives, see Ben Brungraber's article "Engineered Tension Joinery" in TF 23, March 1992).

The "engineering-style" connection described in the second section of the article and shown in Figs. 1a-1c is likely unknown to North American designers and equally likely to have been developed by an engineer, definitely not a timber framer! Kessel clearly presents the deficiencies in this connection when made with unseasoned timber. Although this is unstated in the article, the dimensions h_1 -Δ_1, and h_2 -Δ_2 in Fig. 1 are the final sizes of the sill and post, respectively, after they have seasoned.

Proof testing of the kind conducted by Dr. Kessel is not a practical consideration in most cases. For him, preliminary testing of sample joints was the only way to demonstrate their suitability for the eight-story, 85-ft.-tall Knochenhauer office building (Section 5), an enormous building by timber framing standards. For a project of this magnitude, the costs of preliminary joint testing might be proportionately modest. The same could not be said for a 5,000-bf house frame. However, the drawback to proof testing is that the results are typically limited in scope. In this case, only 12 specimens were tested, with no more than two using the same peg size, post and sill sizes, edge- and end-distances, etc. With a healthy factor of safety, the test results could be applied with some confidence to a specific project for which the tests were designed. But no researcher would draw definitive conclusions about joint strength from such a small sample. Hence, the load capacities in Table 1 should not be viewed as design values, and certainly Dr. Kessel does not present them as such. Still, it is reasonable to observe general trends in behavior in these tests, so long as the differences in results from test to test are distinct. So, although the results of the study might not be immediately applicable to current practice in North America, they do help timber researchers identify the directions that we should take in our own investigations.

Perhaps the most surprising *direction* suggested in the article is towards even-larger diameter pegs and more of them in the joint. Dr. Kessel rejected pegs of 1-in. diameter and smaller as too slender. He was unsatisfied until he obtained a tension capacity of about 100 kN (22,500 lbs.) using four 40-mm (1⁹⁄₁₆-in.) pegs in a joint with members no larger than those used regularly here. In tests IX, XI and XII, his posts were 7x9 and his sills 8x10½. The large-diameter pegs were used in tenons just over 3 in. thick. A ratio of tenon thickness to peg diameter of near 2 appears to be preferred. (This ratio is familiar to the North American framer.) End- and edge-distances of only 1.3 peg diameters also appear to be standard practice. This end-distance is particularly short by our standards.

Dr. Kessel's next article studies over 100 mortise and tenon joint specimens, including 2-peg joints of oak and spruce with oak pegs from 1-in. to 1½-in. diameter, and offers specific recommendations for design strength and joint stiffness. —Dick Schmidt

Joint Engineering I

KNOWING the forces experienced and exerted by individual members is the necessary prerequisite to joint engineering. This information is a useful byproduct of the kind of analysis presented in "Frame Engineering" (TF 30, December 1993), an engineered approach to timber frame design where frame layout and timbers sizes were determined through the use of finite element analysis (FEA). To get to a timber list for a particular structure, we drew up a preliminary frame design, built an FEA model of this frame, loaded the model with live and dead load and ran it on the computer to determine individual member deflections, forces and stresses. The proposed frame design and timber sizes were then adjusted as necessary to meet code, the model run again and the process repeated until all resultant stresses and deflections fell within acceptable limits.

1. Design Values. Most of the information needed for engineered design of timber joinery can be found in the *National Design Specification for Wood Construction* (American Forest & Paper Association, 1991). The notable exception to this rule is tension joinery. By themselves, the provisions of *NDS* are not sufficient for design of mortise and tenon (and spline) tension connections secured with wooden pegs, keys or wedges. However, there is material in the contemporary timber engineering literature as well as ongoing research that will make it possible to take up the tension joint question later.

When sizing timbers in "Frame Engineering," our governing criteria were member deflections and axial, bending and shear stress, all of which had to fall within acceptable limits. Similarly, when specifying joinery we look to keep bearing and shear stress at connections in the allowable range. For material in the examples below we use No. 1 Southern Pine timber (under dry service conditions)[1] with the following design values, taken from the 1991 *NDS* supplement:

E	modulus of elasticity (stiffness)	1,500,000 psi
F_b	extreme fiber in bending (bending strength)	1,350 psi
F_v	shear parallel to the grain	110 psi
F_c	compression parallel to the grain	850 psi
$F_{c\perp}$	compression perpendicular to the grain	560 psi

As with the design of members, the process is circular. First the designer specifies a joint, then calculates shear and bearing stress imposed by the known forces on the connection. If these predicted stresses do not exceed design values the job is done, otherwise the joint must be redesigned and the calcs run again until the resultants fall within tolerances.

2. Floor Joist. Shear. The joint between a floor joist and its carrying timber is a basic shear connection. The joist, loaded from above, wants to move downward but is restrained by its housing into the girder. Take, for example, a floor frame of 5x6 joists tenoned into an 8x10 chimney girt (Fig. 1). Covering a 12-ft. bay, our floor has joists on 28-in. centers with an unsupported span of 134½ in. and a combined live plus dead load, including joist weight, of 53.5 pounds per square foot (psf). This translates into a line load per joist of 10.4 lb/in. So the shear force at either end of the joist[2] is

$$V = 10.4 \text{ lb/in} \times 134.5 \text{ in} \div 2 \approx 700 \text{ lb}$$

In rectangular beams shear stress is determined using the following formula:

$$f_v = 3V \div 2bd$$

where V is the shear force and b and d are the breadth and depth of the timber respectively.

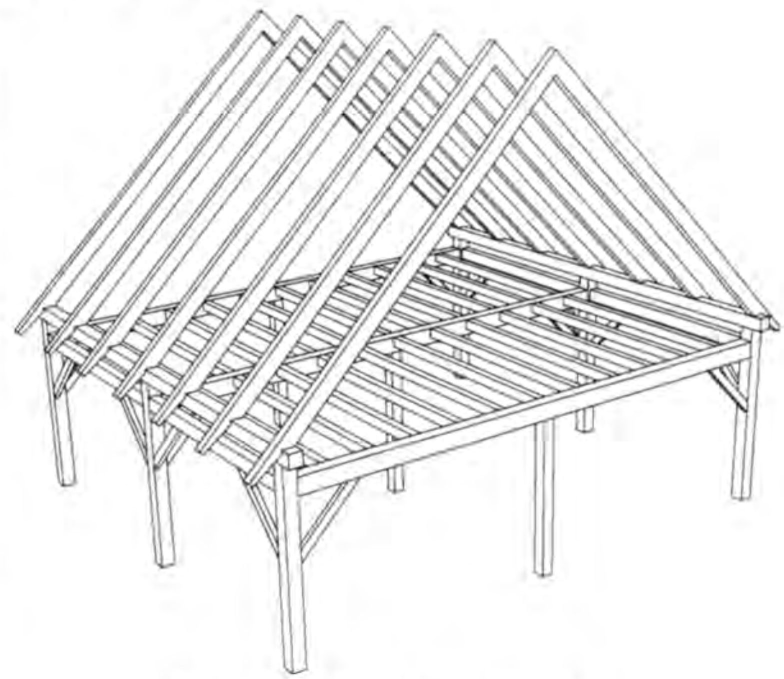
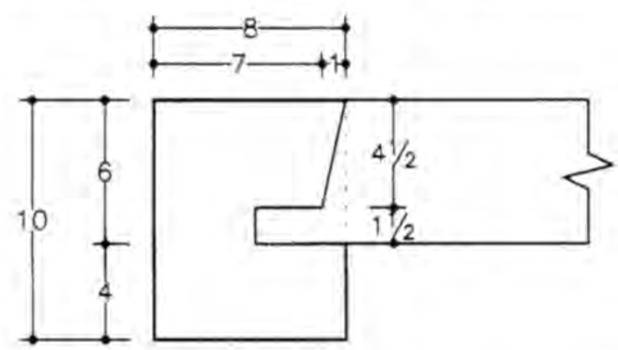


Fig. 1. Model House Frame, 12-ft. Bays, Girders Span 14 ft. Post to Post.

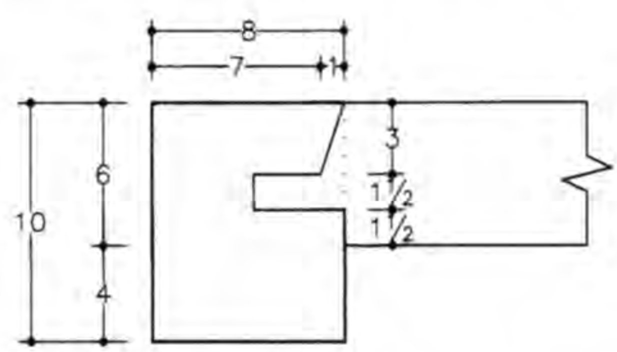
Assuming that the joist is tenoned into its girder using a soffit tenon with entrant shoulder (below), the full 5x6 cross-section acts



in shear and the 700-lb. shear force will induce a shear stress of

$$f_v = 3 \times 700 \text{ lbs} \div (2 \times 5 \text{ in} \times 6 \text{ in}) = 35 \text{ psi},$$

well below the allowable shear stress F_v of 110 psi. Should it prove necessary to use a tusk tenon (below) rather than a soffit tenon (*i.e.*, to notch the lower or tension edge of the joist at the joint, say because of a shallow carrying timber), then shear stress will be



concentrated at the inside corner of the notch. *NDS* makes allowance for notching on the tension side of members at connections with the modified shear stress formula

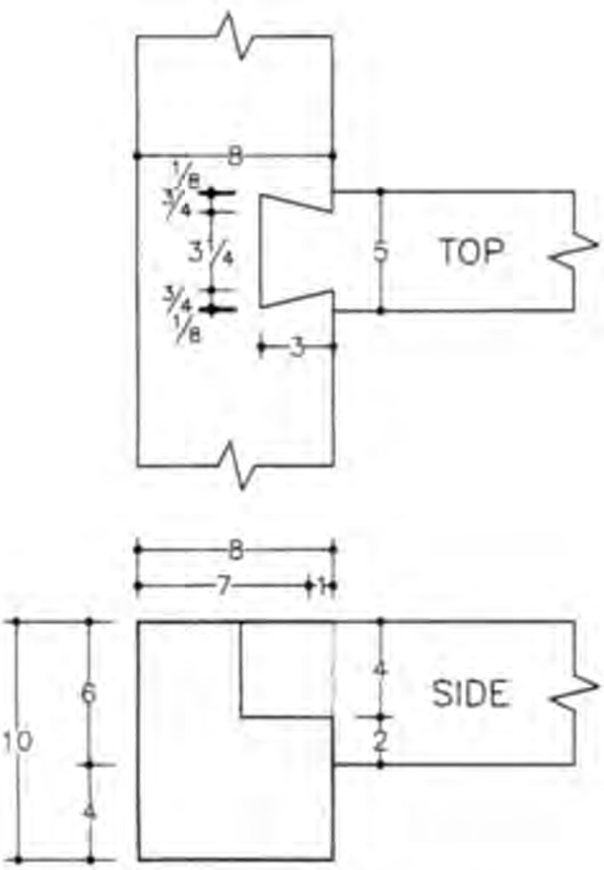
$f_v = [3V \div 2bd_n][d \div d_n]$,

where d is the full depth of the tenoned member and d_n the depth remaining above the notch. As you can see, the formula accounts for notching first by using net area in the cross-section calculation (bd_n), and secondly by increasing the resultant stress by the ratio of the full to notched depths (d/d_n). So if the 5x6 joist is joined to its girder with a tusk tenon having a 3-in. upper shoulder, 1½-in. tenon and 1½-in. unsupported lower shoulder (facing page), then d = 6 in., d_n = 4½ in. and the shear stress calculation looks like this:

$f_v = [3 \times 700 \text{ lbs} \div (2 \times 5 \text{ in} \times 4.5 \text{ in})][6 \text{ in} \div 4.5 \text{ in}] = 62.2$ psi,

showing a dramatic increase in shear stress over the original soffit tenon, but still well within the allowable range. Suppose we then go to a male dovetail drop-in on the joist end with a 4 in., 2 in. layout and 1:4 flare over 3 in. (below). Net timber depth is reduced to 4 in., net width to 3¼ in. and

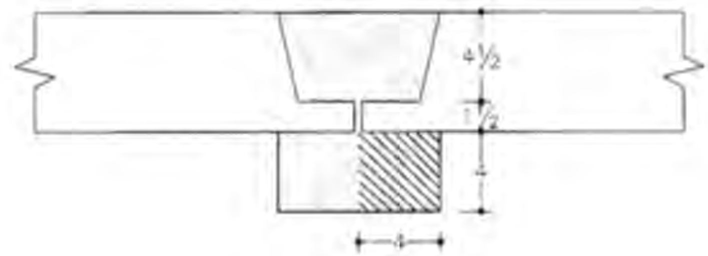
$f_v = [3 \times 700 \text{ lbs} \div (2 \times 3.25 \text{ in} \times 4 \text{ in})][6 \text{ in} \div 4 \text{ in}] = 121.2$ psi.



Here the predicted shear stress exceeds the design value, so this joint is not acceptable[3]. It's clear from this exposition that, for the joist end, the best joint is a full-width soffit tenon and, failing that, when notching it's best to support lower shoulders.

What about the other half of the joint? The code does not deal specifically with the strength in shear of mortised carrying timbers. Timber engineers who on other questions follow definitive and rigorous procedures say, "Oh, I keep a minimum of 3 in. below loaded mortises in girders." Or they might stipulate 2½ in., or 2. Ask why and you'll likely be rewarded with a moment of thoughtful silence. Indeed it is difficult even to define the issue here. It seems obvious that as the mortise for a soffit-tenoned joist approaches the lower surface of its carrying timber, the carrying timber will eventually fail and the joist fall to the floor below. So how much is enough? Three in. seem like plenty in ordinary situations, 2 in. make me uncomfortable under all but the lightest loads, 1 in. is clearly insufficient. And what is the mechanism of this failure? Shear parallel to the grain? Tension perpendicular to the grain?

As a precaution, I generally look at the net timber cross-section supporting a joist and assess its shear capacity as if it were a stand alone beam carrying a superimposed load. In the case of the 5x6 joist with soffit tenon, 4 in. of timber remain below the mortise in the 8x10 girt. In an interior bent, with opposing joists mortised in



from either side (above), each joist would be carried on a 4-in.-sq. cross-section of the girt with a hypothetical stand-alone shear stress of

$f_v = 3 \times 700 \text{ lbs} \div (2 \times 4 \text{ in} \times 4 \text{ in}) = 65.6$ psi.

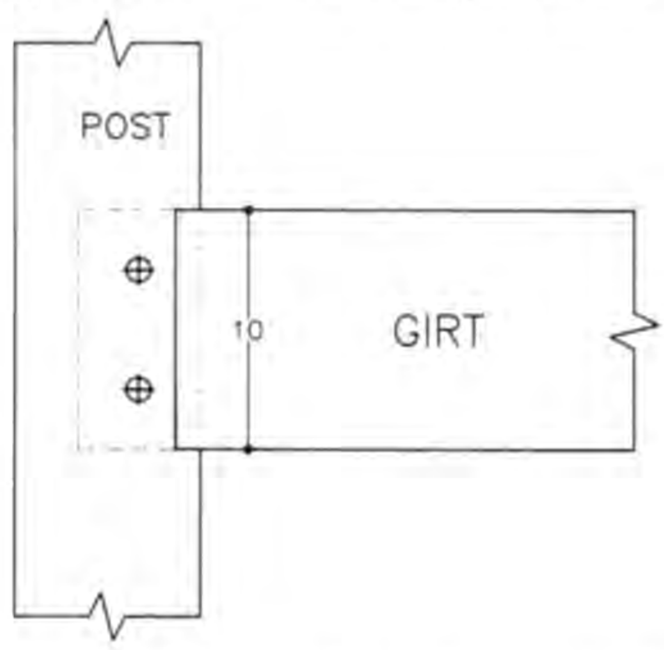
Should the shelf height below the mortise drop to 3 in., this shear calculation yields a value of 87.5 psi. If further reduced to 2 in., the theoretical resultant climbs into the danger zone at 131.3 psi. While this exercise is merely an indicator, rather than a true calculation of actual shear stress, I find it useful for detailing connections in carrying timbers.

Bearing. Turning to joint bearing capacity, we must first determine the size of the bearing surface across which load is transferred. In the case of a horizontal tenon, I typically calculate bearing area as the width of the joist tenon by the depth of its housing into the girder.[4] Thus our 5x6 soffit tenon with 1 in.-deep entrant shoulder would bear on an area of 5 in. x 1 in. or 5 sq. in. In both joist and girt this load is taken perpendicular to the grain. Given the shear force of 700 lbs, the cross-grain bearing stress at the joint would be

$f_{c\perp} = 700 \text{ lbs} \div 5 \text{ in}^2 = 140$ psi

well below the allowable $F_{c\perp}$ of 560 psi.

3. Carrying Timber to Post. Shear. Our 8x10 chimney girt accumulates the load from multiple joists and delivers it to an 8x8 chimney post via a housed mortise and tenon joint (below). Carrying a 12-ft.-wide tributary area, the beam bears a combined live



plus dead line load of 55.2 lb/in. over a 14-ft. span (156½ in. clear between posts). Apportioning one-half to our connection yields a shear load at the post of

$$V = 55 \text{ lb/in} \times 156.5 \text{ in} \div 2 \approx 4{,}320 \text{ lbs}.$$

If the girt is fully housed entire into the post, then resultant shear stress equals

$$f_v = 3V \div 2bd = 3 \times 4{,}320 \text{ lbs} \div (2 \times 8 \text{ in} \times 10 \text{ in}) = 81 \text{ psi}.$$

No problem. But suppose for a moment that the girt is not housed, and the load is borne on a single 2-in. tenon connecting beam to post. The formula for shear stress in the tenon looks like this:

$$f_v = 3V \div (2bd) = 3 \times 4{,}320 \text{ lbs} \div (2 \times 2 \text{ in} \times 10 \text{ in}) = 324 \text{ psi}.$$

Not a good idea.

Shear is not an issue in the post, since it is loaded only with the grain, and feels no shear load unless the girt carries axial tension or compression load—that is, unless the girt is pushing or pulling on the post.

Bearing. In order to check bearing stress, we must first settle the thorny issue of bearing area. A conservative approach might stipulate that all the load is borne on the housing of beam into post, while a liberal one might distribute the load over the combined cross-sections of housing and tenon. Or you could take the middle of the road position so popular these days in public life and assume that the girt is carried on the housing and a portion of the tenon.

Knowing that mortises especially are sometimes undercut, it seems prudent to eschew the extreme left wing position and opt for the housing only or perhaps the housing plus partial tenon options. Taking the most conservative stance, we find that a ¾-in.-deep housing of beam into post yields a bearing area of 8 in. x ¾ in. = 6 sq. in. and bearing stress of

$$f_c = f_{c\perp} = 4{,}320 \text{ lbs} \div 6 \text{ in}^2 = 720 \text{ psi}.$$

This number looks okay on the post, where the load is carried with the grain since allowable compression parallel to the grain equals 850 psi. But the girt must bear the load cross-grain, and allowable compression perpendicular to the grain in No. 1 Southern Pine under dry service conditions is only 560 psi.

If we augment the original ¾-in. housing with 1 in. of tenon length, bearing area increases to 8 in. x ¾ in. + 2 in. x 1 in. = 7 sq. in., cutting bearing stress to

$$f_c = f_{c\perp} = 4{,}320 \text{ lbs} \div 7 \text{ in}^2 = 617.1 \text{ psi}.$$

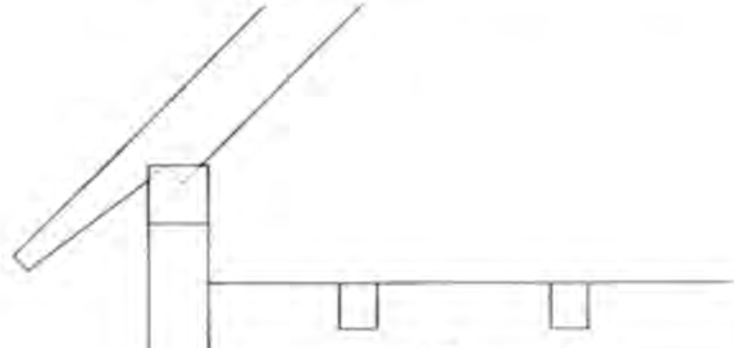
Still too high. Let's run the calculation again for a 1-in.-deep housing (no tenon contribution). Load bearing area is now 8 in. x 1 in. = 8 sq. in. and bearing stress equals

$$f_c = f_{c\perp} = 4{,}320 \text{ lbs} \div 8 \text{ in}^2 = 540 \text{ psi}.$$

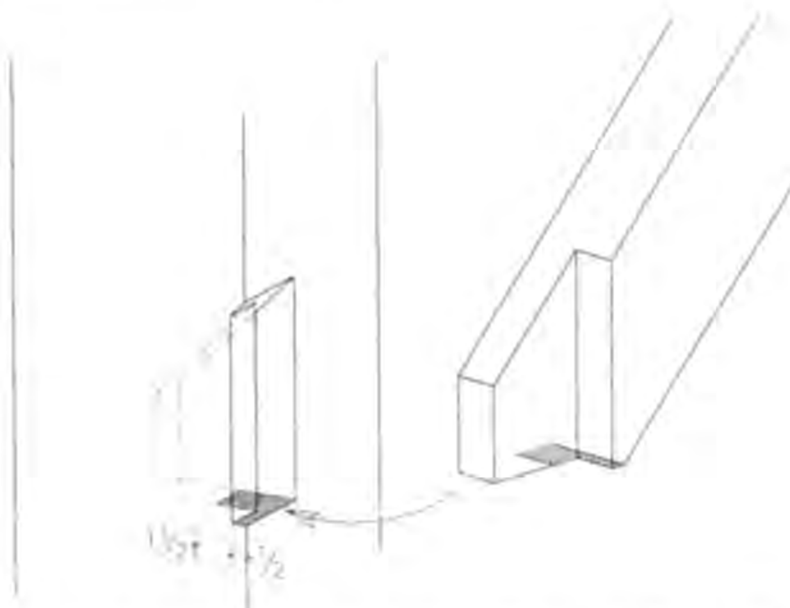
Safe country at last. We could have achieved the same result using the ¾-in. housing by putting 2 in. of tenon length into bearing service, but engineering, with its explicit safety concerns, is one place where all of us should feel comfortable taking a conservative stance.

4. Brace. We move on to examine the connection between knee brace and post or beam. It's a windy day and our diagonal member is doing the job for which it was made, taking up compression load as the frame begins to rack, and stiffening the building against sideways deflection. We stipulate compressive load here since it's difficult to make a decent mortise and tenon tension connection with a short-tenoned small member approaching its bearing at a 45° angle. Indeed some engineers typically discount the tension capacity of knee braces, treating them only as compression elements.

Bearing. We look at a downwind 3x5 brace (below and at bottom) with 1½-in. mortises and ½-in.-deep housings into 8x10 girt and 8x8 post. Our FEA frame model predicts 3,500 lbs.



compressive force in the brace and 1,075 psi bending stress in the post at the brace joint. Looking first at the brace, the 3,500 lb. axial force is distributed over a 3 in. x 5 in. or 15 sq. in. cross-section,



resulting in compressive axial stress f_c = 233.3 psi, well below the allowable F_c = 850 psi[5].

To assess bearing at the joints, we first resolve the 3,500 lb. axial vector acting at 45° to the post into horizontal and vertical components of 2,475 lbs each. If a force F_β acts at angle β to the level, then its horizontal component is

$$F_x = F_\beta \times \cos\beta$$

and its vertical component equals

$$F_y = F_\beta \times \sin\beta.$$

So in our brace, 3,500 lbs along the grain translates into level and plumb components of

$$F_x = 3{,}500 \text{ lbs} \times \cos 45° = 2{,}475 \text{ lbs}$$

and

$$F_y = 3{,}500 \text{ lbs} \times \sin 45° = 2{,}475 \text{ lbs}.$$

The horizontal thrust of the brace is transferred from the shoulders of the tenon to the face of the mortise, the vertical component across the housing and the end of the tenon to corresponding end grain on the post. In order to check bearing stress, we must resort to the *Hankinson Formula* for bearing at an angle to the grain. This useful device reads as follows:

$$F_\Theta = F_c \times F_{c\perp} \div [F_c \times \sin^2\Theta + F_{c\perp} \times \cos^2\Theta]$$

where

- Θ = angle between long grain direction and direction of load
- F_Θ = allowable bearing design value at angle Θ to the grain
- F_c = allowable compression parallel to the grain
- $F_{c\perp}$ = allowable compression perpendicular to the grain

Using the tabulated values of F_c = 850 psi, and $F_{c\perp}$ = 560 psi for No. 1 Southern Pine under dry service conditions (see above) and the standard knee brace angle (Θ = 45°), the Hankinson formula yields an allowable bearing design value at 45° to the grain of 675 psi.

As bearing for the Y-component of the load we take the ½-in.-deep housing plus a square patch of 1½-in.-thick tenon (facing page, bottom) for a total bearing area of

$$3 \text{ in} \times 0.5 \text{ in} + 1.5 \text{ in} \times 1.5 \text{ in} = 3.75 \text{ in}^2.$$

The post takes this vertical load along the grain and so is capable of carrying a vertical force of

$$3.75 \text{ in}^2 \times 850 \text{ psi} = 3{,}187.5 \text{ lbs},$$

while the brace, transmitting the load at 45° to the grain, is limited to

$$3.75 \text{ in}^2 \times 675 \text{ psi} = 2{,}531.3 \text{ lbs},$$

slightly in excess of the predicted 2,475-lb. Y-component. Looking at the horizontal portion of the load, the bearing area on the post housing is

$$6.56 \text{ in} \times 1.5 \text{ in} = 9.84 \text{ in}^2.$$

The post takes this load at right angles to the grain so carrying capacity equals

$$9.84 \text{ in}^2 \times 560 \text{ psi} = 5{,}510 \text{ lbs}.$$

And on the brace, the load is once again felt at a 45° angle to the grain, so the allowable is limited to

$$9.84 \text{ in}^2 \times 675 \text{ psi} = 6{,}642 \text{ lbs}.$$

So the bearing is sufficient to the load, by a country mile for the horizontal vector, by a close shave for the vertical component. It seems fair to conclude that the limiting factor on brace compressive strength is likely to be bearing capacity on the housing and tenon end grain.

Shear. Shear stress is not an issue for the brace, since it carries only axial load. The post takes the horizontal component of brace load in shear (V = 2,475) over a net cross-section of

$$8 \text{ in} \times 8 \text{ in} - 0.5 \text{ in} \times 3 \text{ in} - 1.5 \text{ in} \times 4 \text{ in} = 56.5 \text{ in}^2,$$

imparting shear stress of

$$f_v = 3V \div 2bd = 3 \times 2{,}475 \text{ lbs} \div (2 \times 56.5 \text{ in}^2) = 65.7 \text{ psi}$$

to the post. Note that the shear with notching formula is not used here, since the "notch" in the post (the brace housing and mortise) is located on its compression rather than its tension side.

Before signing off on this joint, some thought must be given to bending stress in the post at the brace mortise (facing page). Just as net post cross-section at the joint is reduced by mortise and housing from 64 sq. in. to 56½ sq. in., so the post section modulus[6] also drops (from 85.3 cu. in. to 69.4 cu. in.). The unaltered bending moment is now resisted by this smaller net section, so bending stress in the post will rise proportionately. If predicted bending stress at the brace joint in the unmortised post was 1,075 psi, in the mortised post bending stress will increase by the ratio of the two section moduli:

$$f_b = 1{,}075 \text{ psi} \times 85.3 \text{ in}^3 \div 69.4 \text{ in}^3 = 1{,}321 \text{ psi}.$$

Less than the allowable (F_b = 1,350 psi), but not by much. Clearly the question of increased bending or shear stress due to reduced section at connections is something to watch for. —Ed Levin

The second part of this article will analyze the rafter-to-plate joint.

Notes

[1] *NDS* tabulates values for F_c and $F_{c\perp}$ of 775 psi and 375 psi respectively for No. 1 Southern Pine under wet service conditions (see Supplement, Table 4D, p. 35). Backing out wet service factors of C_m = 0.91 for F_c and C_m = 0.67 for $F_{c\perp}$ yields dry service values of 850 and 560 psi for F_c and $F_{c\perp}$.

[2] According to *NDS* 3.4.3.1.a (p. 10), "All loads within a distance from supports equal to the depth of the bending member shall be permitted to be ignored for beams supported by full bearing on one surface and loads applied to the opposite surface." We disregard this provision here for simplicity's sake.

[3] *NDS* 3.2.3.2 (p. 9) specifies that "where members are notched at the ends for bearing over a support, the notch depth shall not exceed one-quarter the beam depth," so this arrangement strikes out on two counts. On the other hand, the code does make provision for shear stress factors which increase the allowable shear value in timbers with minimal splits and shakes. For instance, a member which is free of splits and shakes would have a shear stress factor (C_H) of 2.0, to be applied to a nominal value of F_v = 90 psi for all grades of Southern Pine, yielding an allowable shear stress of 180 psi. See *NDS* Supplement, Table 4D, p. 31 for details.

[4] There is some discretion here since one could apportion some or all of the lower tenon cheek as bearing area. By restricting ourselves solely to the cross-section at the housing, we make the most conservative (the most safety-conscious) assumption.

[5] Long or slender section compression members should also be checked for *buckling.* See *NDS* sections 3.7.1 and 3.9.2 and Appendices D and G.

[6] A timber's section modulus (S) is a measurement of its strength in bending. For solid rectangular timbers, $S = bd^2 \div 6$.

Photos Will Beemer

Above, preparing to scribe the upper assembly to the arched tie. Below right, setting and linking the assemblies on the braced wall posts.

Montana (Continued)

AFTER layout of centerlines and datum lines on the arched log ties (see TF 37, "Guild Notes and Comment," for the first part of this article), the upper assembly of square timbers was then leveled and plumbed in the proper orientation above the ties. These pieces also had centerlines which would align with those on the log ties after assembly. For now the ends to be scribed had been left long and the scribers were set to the distance between the centerlines, usually around 10-14 in. Log scribers are dividers fitted with one or more levels at the hinge point and pens at one or both tips. With one tip following the surface of one timber its curvature can be transferred to the other at exactly the location where the timbers will meet, provided the timbers are level and the operator keeps the bubbles centered. The scribers themselves must first be calibrated against a known plumb surface, with the legs opened to the requisite distance between centerlines.

Rather then coping the shoulders of square timbers to meet the curved surfaces of the logs, we cut bearing tables inside housings on the log. This eliminated much gouge work and thin shoulder edges that could crush under load; it was much easier to get a full bearing surface in the housing. The scribe work, then, was to establish the location of the housing on the log and to find the corners on the square timber where it entered the log. The housing depth was established by the lowest corner of the square timber set flush to the surface of the log and the other three corners penetrating accordingly. Some housings were 3 in. deep in places; anything over that resulted in a stepped housing. There were quite a few log builders in the workshop who had no reservations about attacking these joints with chain saws—with excellent results; the dangers of plunge cutting were stressed, however. Most students relied on routers and jigs, drills and a beautiful 3-in. T-auger from Ohio. (It was revealed that all the good timber framing hand tools can be found in Ohio.) This T-auger proved the best tool for roughing out the 3x24 through-mortise for the king post tenon, with the *bisaigue* used for cleanup. This tenon was left 6 in. long after it came through the bottom of the log tie; one student carved a rosette on his and each truss team then picked up on the idea. Other motifs included a brown trout with a fly in its mouth (the ranch is a fly fishing center) and a bull with a ring in its nose.

Round fir posts (14-in. diameter) had been substituted for the square timbers in the original plan, and peeling commenced upon their arrival. With the main bent brace scribed into it, the assembly was then laid on the log tie for scribing into its lower surface. Some teams took the initiative of scribing the post to the tie first, then after fitting it laying the brace across both and scribing it to the post and tie. This latter method proved easier.

Evening classes during the first week included a design discussion and a presentation of structural basics. There was also an impromptu demonstration of carpenter's math for compound joinery; even though there was none in the frame those who had come off recent jobs involving complex roofs wanted to tell what they learned. As the first week drew to a close, the sun came out for good and began to dry out the site. We all felt we were on schedule and looked forward to a day off. Sunday found everyone heading for the substantial hills to go fishing and touring; a group of 17 went to Yellowstone National Park. Geyser- and bison-watching and having too much fun sitting in a hot spring left little time to visit the Old Faithful Inn. After all, no one wanted to miss dinner by Gerard.

During the second week material came off the Wood-Mizer as fast as it could be cut and put on sawhorses. With most of the scribe work done there was still much fine-tuning during fit-up of the bents. After the final

checks were completed the longitudinal work began, with the log ties up on edge for layout of the crossing plates. The main posts and large crossing braces were scribed to each other, with everyone getting faster, more accurate and efficient. The purlins, plates, ridge and their accompanying braces were all laid out by square rule, with this work going quicker still after people understood the layout system. Besides appreciating the break from leveling, scribing and referencing to another piece, everyone could visualize the building much better by this time. There was a marked improvement in skill level the second week. One crew went up to Big Timberworks in Gallatin Gateway to produce the 56 knee braces on the tenon cutter, with a tour of the shop and the straw-bale-enclosed design studio as a bonus.

We had originally planned a two-day hand-raising, but since the 18-in.-diameter concrete piers would have less than 72 hours to cure, the decision was made to make this a one-day crane raising to avoid using the piers as pivot points. The hand-raising would have proved impractical anyway; although the site had dried out enough to dig and pour concrete it was still messy and unstable on the surface.

Saturday morning dawned bright and beautiful. The crane arrived and set up on large, thick plywood pads to keep from sinking in. Many pieces had been brought up to the site the day before (it was about a quarter-mile from our cutting area). Starting at one end, posts were placed over their metal tie-down fins embedded in the concrete, and braced. Each of the seven log ties with kings, queens and braces was pre-assembled in the cutting yard a quarter mile away and brought to the site standing up on a trailer. The crane then hooked on and lifted these assemblies successively onto the main posts and braces. Connecting girts were added, then rafters as we moved down the line. In the waning hours of daylight, with the main frame finished and about half the rafters in place, the crew clambered up for a topping off ceremony and a group photo. Many of us would be leaving in the morning while a skeleton crew finished setting the last pieces.

By now the pavilion is finished. A metal roof was planned; but perhaps they've opted for wood (like the other buildings there) after seeing the finished frame. Then again, remembering the Yellowstone fires, maybe not. Most of the ranch decision-makers and staff (some of whom had to have their arms twisted to go along with this investment of time and energy) declared they had no idea the pavilion would be so beautiful and fit so well, which underscores the need to have good client communication ahead of time for projects such as this. Not everyone at the ranch had seen the Guild videos (luckily the people controlling the purse strings had), excellent tools for promoting and explaining timber framing.

The pavilion at Lone Mountain Ranch is unique among Guild projects in that it was built for a private client instead of a public or institutional group. While that may raise a question, it was also an opportunity we could not afford to pass up. The affordability of this workshop made it possible for more people to attend, and the educational agenda got high marks as the two-week format gave everyone more time to exchange and absorb information.

The ranch owners had contacts in the various media in western Montana, and the pavilion raising showed up on television and newspapers throughout the region. A couple of color magazine spreads are due out soon. As the Guild decides which projects to do in the future, an important consideration should be the potential exposure of the craft to the public. A lot of time and energy go into getting us together to show our stuff; we ought to get appropriate recognition for timber framing as a result. Our workshops benefit the Guild as a whole and not just those lucky enough to attend.

—Will Beemer

Maine Rendezvous

THE Guild Rendezvous '95 in August featured a most spectacular site for building as well as interesting building techniques. The project, to build a boathouse on Green Island (also called Green's Island or simply "Greens") offered a chance to see the beauty and natural ruggedness of the coast "Down East," as the Maine shoreline is called by Boston sailors. Everyone was encouraged to slow down from the usually fast pace of construction to see how satisfying timber framing with hand tools on a remote site can be.

Few places are as beautiful as Green Island and few so lacking in the conveniences we all take for granted. Karen Jackson, a year-round inhabitant of the island, writes: "Simple to make a list of the conveniences that are not available on Green Island. There is no running water, no electricity, no phone, no road, no store. In preparing yourself and your families for a week or more trip to the island, you might consider the zen-ness of what those Nos mean. All water, drinking, cooking, limited washing, is carried either to the island or across the island from a stone well. Any energy generated is human energy."

Perhaps it is no coincidence that beauty appears so readily where luxury is lacking. The pristine quality of the islands in Penobscot Bay is easily marred by human hands. To build a boathouse (or any other structure) there respectfully requires thoughtful design and careful technique. To the credit of Guild members and their Atlantic Challenge colleagues, this environmentally-sensitive project was successfully conceived and completed.

The Atlantic Challenge Foundation has enjoyed a warm relationship with the Timber Framers Guild. Lance Lee, founder of Atlantic Challenge, previously worked with the Guild on the Russian exchange project in Moscow and the Penetang project in Georgian Bay, Ontario, so when Lee envisioned a traditional Scandinavian boathouse, or *naust,* nestled in a secluded cove on Green Island, he again called on the Guild. Lee needed the boathouse to shelter rowing boats which the Apprenticeshop of Rockland had built for the Atlantic Challenge race, a biennial affair that brings together young sail-

Lance Lee leads the beachcombers gathering (sometimes improbable) timber.

ors from many countries.

The traditional Scandinavian design was perfectly suited to the landscape of the island. Such structures were commonly constructed of round timber, much of it driftwood scavenged from rocky beaches and fjords. Not only would the design complement the beauty of Green Island, but it also offered important lessons for constructing a building from the round without the use of power tools.

This rendezvous drew 15 Guild members, as well as members of the Apprenticeshop, apprentices from the Heartwood School in Massachusetts and about 20 students from Hurricane Island Outward Bound who helped with the raising. The organizers and instructors for this affair were Will Beemer, Mike Goldberg and Curtis Milton. The incredible meals—international dishes and local delicacies (yes, that means lobster)—were prepared by Michelle Beemer and Debbie and Micah Goldberg.

The first days on the island involved pitching tents on the abandoned stone pier once used to load quarried stone onto barges, then transporting heavy tool boxes by boat to the site, and simply getting used to the idyllic island life. The first structure built by the framing crew was a shelter for the composting toilet to accommodate the influx of new inhabitants. This structure may not have had great architectural significance, but it did show a reverence for the delicate balance of island life.

In a small cove around the next point of land from the old stone pier was The Powder Hole, a rough camp equipped with a gas stove, where meetings were held and meals served, and witness to repeated happy gatherings set off with music and great food. Deeper in this cove often shrouded by the early morning fog was the site for the *naust*-to-be.

THE first task at hand was to gather the material. Most proved to be spruce with some cedar and pine. An inventory was taken and the pieces divided into piles to be used respectively for posts, plates, rafters and tie-beams.

"Harvesting the logs for the *naust* was a wonderful process," recalls timber frame designer Andrea Warchaizer. "Lance Lee energetically led the group along the shore, walking up to all sorts of washed-up pieces of wood to examine them for usefulness. We used pry bars and poles found on the beaches to break loose the driftwood timbers from the rocks and inch them down to the water.

"Some of these were left at points along the beach so the high tide would float them and the prevailing current carry them into the cove. Others we rolled directly into the water and tied into rafts. Lance, in the

Sam Kirby

Will Beemer

Log scribers (two types shown), indispensible for working with round timber. The double vials shown fixed to the tool at left allow cross-leveling, the essential condition for the technique.

Apprenticeshop's small boat *Perseverance,* and Will Beemer and others in kayaks and small rowing boats pushed and pulled these timber rafts into the cove.

"What impressed me about this," Andrea continues, "is how intimately involved we had become with the forces of Nature. The tide and the currents which had brought the driftwood to the shores of the island were also the forces which would help move the material to the worksite."

Will Beemer explains the process that followed after the gathering of material. "The lines of the building were laid out on the slope of the shore—taking advantage of the position of large stones that would make decent-sized footings." A few of these boulders had to be arduously coaxed into alignment with the column grid.

"An elevation line was then established at 2 ft. above the bottom of the longest posts. We were restricted in terms of height because our found posts were only of a certain length." Calculations were made to assure that the lowest posts at the bottom of the sloping site near the water would be long enough to reach from their respective stone footings to the plate that would rest atop the posts.

Two timbers were scarfed together to make each of the 50-ft. plates and were positioned next to the rock footings. The slow task began of hand drilling each stone footing to accept a re-bar pin at the intersection of the center line of the plate above and the line of that particular bent. This point was also established on the plate. Once the height of the plate from a pre-established elevation line was decided on, the height of the top shoulder of each post could be determined. At each spot where a post would come up to meet the plate, the plate was adzed and planed to a flat level surface 5½ in. down from its center line.

While half the crew set to work cutting 4-in. mortises into these flats along the bottom of the plate, the other half scribed each post to its respective rock footing. The process, here involving an irregularly shaped stone and an equally irregularly round timber, required extreme dexterity with the log scribers, as demonstrated by Will Beemer.

Will Beemer

Mike Goldberg and friends with arsenal of squares used to conquer round timber.

Once the profile of the stone footing was drawn on the post, the post was cut and chiselled to the scribed line. Then the inside of the timber was hollowed to fit or clear the contours of the stone. The posts, properly fitted to the stone footings, would stand plumb without any additional support.

With a post standing in place, an elevation line was marked on it using a water level (a length of ¼-in. plastic tubing filled with water) matched against a reference point on a nearby tree. (After patient adjustment to the heights of the ends of the tube, the meniscus at the tree would align with the reference elevation, at which moment the meniscus at the post would find the same level

Will Beemer

Above, the workshop crew pose on their handiwork. Below, the Rockland apprentices continued on to roof the building and make the remaining provisions for the boats.

Lance Lee

and be marked off.) From there the elevation of the shoulder and top of the tenon could be found. Each post was taken down to be cut for the tenon.

Meanwhile, the plates were turned right-side-up so the tie beams could be set on top of them and scribed for the concave seat that would straddle the plate. After these seats were cut on the bottoms of the tie beams, the beams were turned on their sides so the rafter feet could be scribed to them.

Part way through the second week, some of the frame was ready to raise. Under the direction of Mike Goldberg and Curtis Milton, with the helping hands of the Apprenticeshop and the students from the nearby Outward Bound school, and not least, pairs of shear legs positioned over each end, the plates made their way onto the top of the posts.

New teams then formed to drive pegs, fit each bent with braces, and raise and spike the tie beams into place and last, to raise the rafter pairs. At the end of two weeks, most work on the frame had been completed. Mike and Curtis gave instructions on installing nailers and purlins and advised the Rockland apprentices on an appropriate sheathing and roofing system.

Each visitor left Green Island that weekend having found something special there to remember. Will Beemer recalls, "the most important lesson of the rendezvous was adaptability, working with the materials at hand. People learned to lay out round posts and beams by scribing to center lines and sighting along framing squares. The lack of power tools and machinery also made the rigging and cutting exercises very uncommon."

"For that remote site," adds Andrea Warchaizer, "we could have brought in a gas generator and power tools. It would have been noisy and less safe. We couldn't have had the children at the site because of the confusion. We would have been fighting over the tools and dragging them around with cords tangled everywhere. Instead, we opted to use hand tools. You really could cut through a 12-in. log with the two-person saw in the time that it would take to oil a chainsaw. The 4-in. mortises were easily drilled out with T-augers and a little time. I never felt as if the tools that we were using were holding us up. We fell into a different rhythm."

Whether it was because of the refreshing feeling of teamwork while using a double-ended saw, the meditative opportunities in cutting mortises and tenons with handtools, or simply the revelling in the aura of a timber frame so filled with character and entirely free of manufactured square-edged timber, everyone left Green Island reluctantly, but with a new enthusiasm for the timber framing trade. —SAM KIRBY

Sam Kirby is an architect and writer.

Tying Joint Variation X

IN the late 1700s, the "English" Barns of Western Massachusetts sustained changes to better adapt them to local needs and conditions. In Adams (Berkshire County) we find a 1770s vintage 28-ft. x 36-ft. barn of the standard three-bay side entrance plan but with transitional tying joints. The two intermediate cross-frames have the later dropped ties with a through-wedged dovetailed mortise and tenon joint (see TF 37, page 11, for assembled views of both types, and TF 26, page 16, for an exploded view of the wedged dovetail version of the dropped tie). The end-wall tying joints, however, retain the older form brought from England, at least in appearance. Tapered posts with two top tenons support the junction of tie beam, plate and rafter as one would expect. But instead of the lapped dovetail (or half-dovetail) that was a standard in England for several hundred years and New England for at least 100 years, there is a cogged joint.

There are two inherent problems with the classic English tying joint. First, shrinkage in the members makes the dovetail lap ineffective and susceptible to withdrawal. If the post head has a drying check between the two tenons, a typical location, the jowl may split off when the dovetail withdraws. Many English-American barns (including this one) utilized posts tapered full-length rather than jowled to lessen the splitting phenomenon. Our greater extremes of temperature and humidity here are undoubtedly rougher on timber joints than in England. Roof materials and pitch also changed here from steep, thatched English roofs to lesser-pitched shingled roofs—and as the roof pitch lessens, the outward thrust of common rafters on the plate increases. The roof loads increased as well (wet snow). In addition to principal rafters, this barn has intermediate common rafters with collars (at about the midpoint), tenoned into a full-length ridge beam. Since intermediate rafters bear on the plate, there is outward thrust force transferred to the tying joints.

The second problem occurs on the gable tying joints. For the dovetail to work, there must be sufficient relish, 6-8 in. at least, in the plate beyond the dovetail to avoid splitting out. Thus an essential part of the joint is protruding past the siding to the weather. It is a great nesting spot for birds, and a place for snow to collect.

The builder's choice of this joint was an obvious attempt to alleviate those two problems. The only shrinkage that affects the joint is that of the cog in the plate. At only 1½ in. wide, it is hardly a concern. For a failure to occur from roof thrust, the post must split in half down its length and either the interior face of the plate or the relish in the tie beam (a block having a shear area of about 5½ x 7) must break away. Neither has occurred in the four joints in this barn. Because the vertical 1¼-in. sheathing planks (up to 21 in. wide) covered the joint, there was no weather damage. Though the plate does protrude the thickness of the sheathing, it is covered by the upper gable sheathing.

Of the four surviving 18th-century barns in Adams, no two had the same combinations of tying joints, an indication that the builders were in a period of experimentation. By the 19th century, most barns in the area were being built with dropped ties.

—JACK A. SOBON