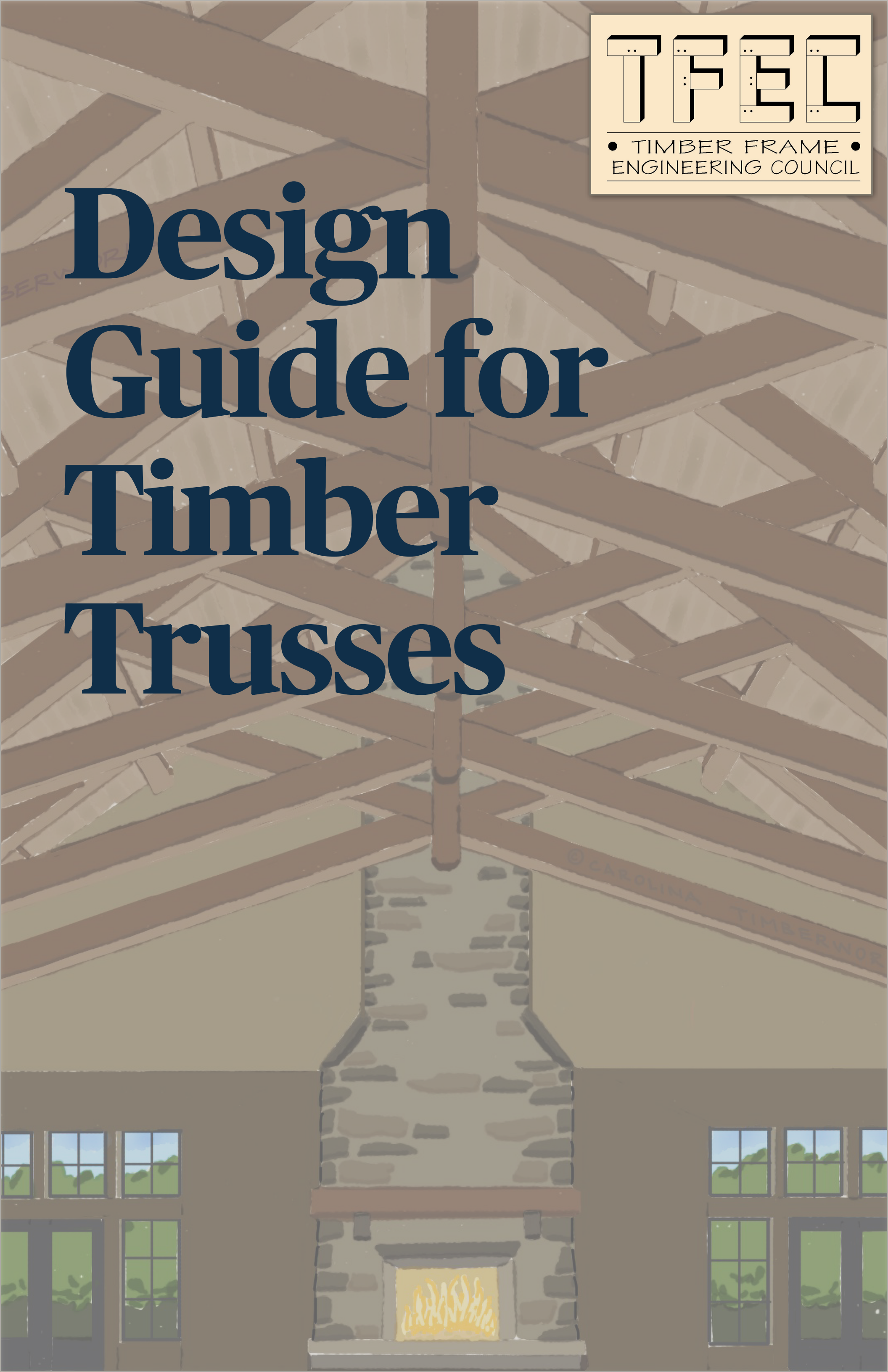
TFEC

• TIMBER FRAME •
ENGINEERING COUNCIL

Design Guide for Timber Trusses



TFEC DG 1

Design Guide for Timber Trusses

December 2021

•TIMBER FRAME•
ENGINEERING COUNCIL

Timber Framers Guild

On the cover, the 151-ft.-span Kicking Horse Pedestrian Bridge in Golden, British Columbia, is the longest free-standing timber-framed bridge in Canada. It was built by the Timber Framers Guild in 2001 as a community development project. Photo by Cheryl Chapman.

TFEC DG 1 Contributors:

Ben Brungraber, PhD, PE
David Connolly, PE
Jeff Hershberger, EI
David Hourdequin, PE
Jan Lewandoski
Jaret Lynch, PE
Duncan McElroy, PE
Leonard Morse-Fortier, PhD, PE
Grigg Mullen, Jr., PhD, PE
Tom Nehil, PE *Coeditor*
Dick Schmidt, PhD, PE *Coeditor*

Unless noted otherwise, illustrations are by the contributors.

Table of Contents

Preface

The first version of the TFEC publication *Design Guide for Timber Roof Trusses* was released in 2020 under the leadership of Jim DeStefano. Jim's spearheading efforts resulted in a document that captured wide attention and praise in that it addressed a topic of vital interest to designers and engineers. The success of that initial offering has prompted its contributors to produce this second version with broader scope and greater depth than the first. This version, designated TFEC DG 1 *Design Guide for Timber Trusses*, is intended as a resource and reference document for structural engineers, designers, preservationists, restorationists, and other professionals in the timber-building industry. It provides guidance in the design, evaluation, and repair of timber trusses.

The scope of the guide is broad in that it applies to truss forms that are principally constructed of wood members (lumber or timber) that likely use some traditional carpenter-style wood joinery but may also contain steel elements including steel tension rods, conventional fasteners, and weldments. About the only truss forms not addressed are metal plate–connected wood trusses. The Truss Plate Institute and ANSI/TPI Standard 1-2014 serve as key resources for those structural components.

The guide was developed by a team of contributors, all active members of the Timber Framers Guild and the Timber Frame Engineering Council. Each chapter was developed by a single author or pair of authors. After technical editing, the entire document was reviewed by all contributors and subsequently approved through a consensus balloting process. In that respect, the guide represents the experience and perspectives of those listed as contributors. Due to time limitations, some relevant topics are discussed only briefly or not at all. The TFEC hopes that this will become a living document that is revised and expanded in the years to come.

Chapter 1 – Introduction

Timber trusses have supported roofs and bridges for over 2,000 years. Romans were the first to perfect the art of spanning wide spaces with timber trusses. During the Middle Ages, European cathedral builders used timber trusses to span their vaulted stone ceilings to support the cathedral roofs above. In a few rare instances, such as London's Westminster Hall, the trusses were embellished with ornate carvings and left exposed. In North America, early meetinghouses and churches were built with timber roof trusses in the European tradition.

The design of preindustrial timber trusses was based on tradition, trial and error, and the carpenter's intuition. There were no engineers or engineering principles to guide the design.

With the industrial revolution and the expansion of the railroads in the second half of the 19th century, engineers began to play a role in the design of major structures. Trusses used for railroad bridges were at first based on patented designs. Some of the patented truss shapes proved to be structurally efficient, smart truss designs that performed well in service. Those are the ones with names that we recognize and still use today: Howe, Pratt, Town, Warren, and Fink. Later chapters discuss the historical development and distinguishing characteristics of these truss configurations.

The industrial revolution also brought mill towns with large factories that manufactured textiles and everything else that a person could desire. The mill buildings had long-span, robust timber roof structures supported on thick brick or stone-masonry bearing walls. The roofs of the mills often featured timber trusses that emulated patented bridge trusses. The floor systems of these buildings were often trussed beams, the only way to use solid-sawn timbers and still achieve the longer spans to support heavy floor loads.

While timber trusses from centuries past were built for function, timber trusses today are just as likely to be designed as architectural elements as they are to be created for their structural advantages. In many cases, their form is not driven by structural efficiency, but by architectural fancy. This can present some challenges for the timber engineer. Certainly, a truss configuration can be conceived that is both structurally elegant and aesthetically striking. However, a poorly designed truss, one that fails to incorporate some of the fundamental principles that define a truss, can suffer both in its ability to effectively serve its role as a structural element and in its aesthetic appeal.

A timber truss stands as expressed structure and performs a vital structural function. It exhibits load path: a truss's form illustrates how it carries load to its supports. The timber truss is often regarded as an essential means of spanning large openings, whether in a building's roof or as part of a bridge. Elegant structures display their aesthetics by relying on elegant load paths.

It is not necessary, or even productive, to address truss designs intended to support a building's roof or floors as separate and distinct from those incorporated in a bridge. Certainly, there are typical distinguishing features of each. But it would be inaccurate to declare any particular truss feature as strictly that of a roof truss versus a bridge truss. In both cases, top chords could be pitched, flat, bowed, or segmented. Both types of trusses can receive loads on their top chords or bottom chords. End or midspan supports are usually walls or posts for roof trusses and masonry or concrete abutments for bridge trusses. But the distinction is minor in terms of how forces are transferred. The emphasis here is on truss behavior, analysis, and detailing, and less on specific application. The recommendations presented in this guide, like the cards dealt in a hand of poker, can be played in any number of ways with varying degrees of success. The use and functionality of bridge trusses require several unique design considerations beyond the scope of this design guide. Readers are referred to AASHTO (2020), Ritter (1990), and USDOT (2005) for information on topics such as impact loading, moving live loads, and loads resulting from flowing water and ice.

This document focuses on truss configurations that may be realized in practice. That is, this guide addresses *real* trusses that can be cut, assembled, and erected in a building or a bridge. So-called *ideal* trusses are those that satisfy a particular set of assumptions such that simple structural analysis methods may be used. Necessarily, ideal trusses form the conceptual starting point. The real trusses considered in this guide are more general in that they violate one or more of the ideal-truss assumptions to make them practical as structural components. Nevertheless, real trusses still possess the fundamental behavior of an ideal truss: they resist their loads *primarily* by axial forces (tension and compression) in their members.

To a large extent, the triangulation of members in a truss is responsible for its efficient structural behavior in terms of both strength and stiffness. As generalizations of the pure axial-force members in an ideal truss, the members of a real truss also carry shear forces and bending moments, which are internal actions associated with frame behavior. This document offers no fixed boundary line to separate a truss (dominated by axial forces) from a frame (in which shear and bending dominate the design considerations). Rather, the following chapters apply to structural configurations that are likely to exhibit some amount of frame behavior while still possessing the essential characteristics of a truss. Identification of the transition point at which a particular assembly of structural members behaves more as a frame than as a truss is left to the judgment of the structural engineer, if such a distinction is actually needed.

References

American Association of State Highway and Transportation Officials (AASHTO). 2020. *LRFD Bridge Specification*, Washington, DC: Federal Highway Administration, U.S. Department of Transportation.

Ritter, Michael A. 1990. *Timber Bridges: Design, Construction, Inspection and Maintenance*, EM 7700-8. Forest Service, U. S. Dept. of Agriculture.

U.S. Department of Transportation (USDOT). 2005. *Covered Bridge Manual.* McLean, VA: Federal Highway Administration.

Chapter 2 – Historical Development

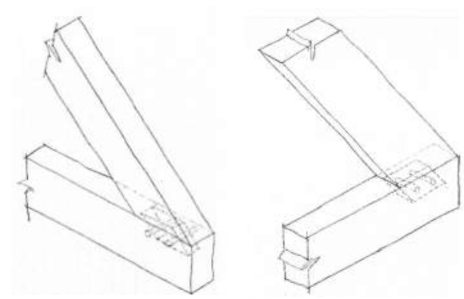
Frame: An assemblage of pieces that create a rigid structure.

Ideal Truss: A frame so constructed and connected that members only incur axial forces.

The first definition can include all historic attempts to span a large room or river. The second describes an idealized version of a truss favored by engineers in the last two centuries. For steel trusses, where gusset plates are employed, the resulting structures may come very close to the ideal. This result is less true for timber trusses, which rarely achieve the ideal condition that all members incur only axial forces. Nevertheless, we will regard the structural configurations considered here and in subsequent chapters as trusses, whether ideal or real.

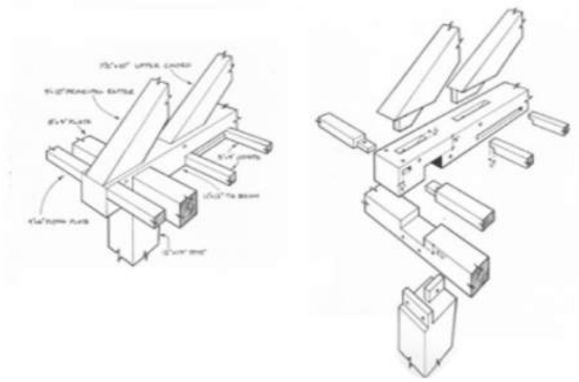
Most historic timber trusses fail to satisfy the ideal-truss definition. When attempting longer clearspans, particularly beyond 40 ft., a successful truss naturally tends toward the idealized form of the modern definition. For the vast majority of spans in the ancient, medieval, and early modern world, including the Americas, Europe, and Asia, 40-ft. clearspans in timber structures were rarely exceeded and thus a huge variety of roof frames could be fabricated to do so (Hoffsummer, 2010). Frequently, arching of single large members, woven arches, or cantilevering (notably in China) were included (Knapp and Miller, 2020).

Regardless of the intent of the framer, the characteristics and limitations of timber as a material and its joinery render the ideal unlikely. Consider a heel joint for a truss with its rafters bearing at the very ends of the bottom chord (Fig. 2.1). This arrangement resolves the forces at a presumed node, with the connection directly aligned over the bearing wall. Unfortunately, as the rafter meets the chord close to the end of that member, this configuration risks failure in the short section of the bottom chord at the end of the tie. Where pegs are used, such a failure may potentially be encouraged by the drying shrinkage checks or splits typical at the end of a stick of timber.



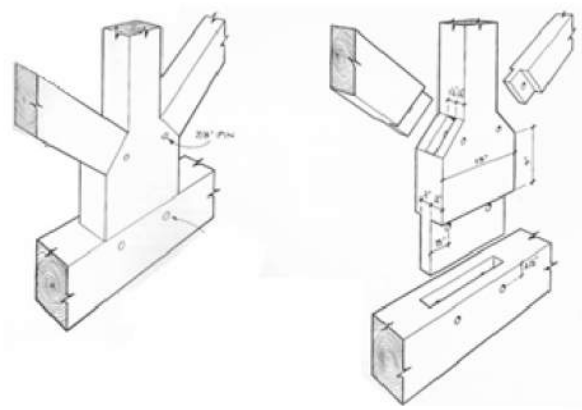
2.1 Truss rafter bearing at the end of the tie beam.

To avoid this end-of-tie failure, kingpost trusses were often double-raftered with a slightly shorter and stiffer inner rafter supporting the kingpost and bearing on the tie slightly inboard of the supporting wall, producing localized but usually acceptable bending in the bottom chord (Fig. 2.2).



Jack Sobon

2.2 Double-rafter truss.

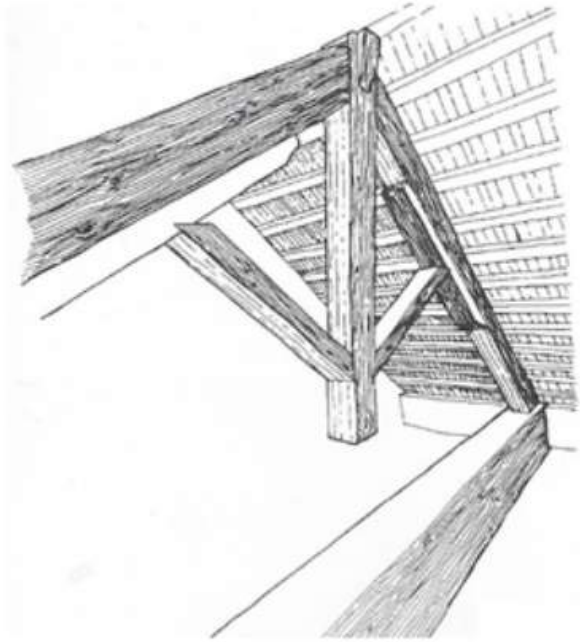


Jack Sobon

2.3 Strut to kingpost connection.

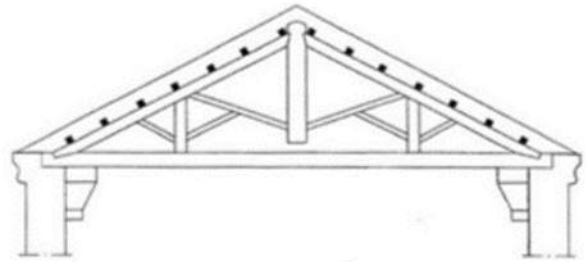
Furthermore, any struts rising from the kingpost to support the midspan of the truss rafters were likely to bear on a shoulder 10 in. to 2 ft. up the kingpost rather than bearing at the node where the kingpost and bottom chord intersect (Fig. 2.3). In this case, the displacement of ideal bearing reflects the difficulty of trying to bring several members together in one joint. Further, if the strut forces are transferred through the bottom chord, it is difficult to transfer the necessary tension force into the kingpost at this joint unless a through tenon is used from kingpost to bottom chord, extending a distance down into the room below. Staggered joinery is a characteristic of historic roof frames and trusses, and need not be seen as lack of knowledge but as reflecting the natural characteristics of the material and the peculiarities of timber joinery. The addition of metal straps and bolts, appearing as early as the Middle Ages, was and is an attempt to get around these limitations (Hoffsummer, 2010).

Timber trusses have their origins in many different places and in many different times. In the ten books of Vitruvius, dating from 30 to 15 BCE, we have descriptions of open rooms as wide as 90 ft. roofed over with timber frames, but the illustrations attached to existing editions of these books date from the 15th century CE and are conjectural. The well-known mid-6th-century king-pendant truss at the St. Catherine's Monastery, near Mount Sinai in Egypt, is the oldest survivor (Fig. 2.4). Stone endures better than wood, and a perhaps earlier 6th-century representation of another truss and crossing struts is represented on a stone tympanum from the Church of Julianos at Barad, in northern Syria (Butler, 1929; see Fig. 2.5). Both of these examples are short spans where the midbottom chord needed no support from the kingpost, but the tying of the rafter feet,strutting of the rafters, and triangulation of the ensemble was accomplished.



Amy Stein

2.4 Pendant kingpost truss (kingpost not joined to bottom chord).



J. Lauffray (1998)

2.5 Church at Barad truss.

Until recently, scholars have not been able to find much evidence for fully realized long-span (greater than 40 ft.) timber trusses in Europe (or anywhere) before the early 17th century, but recent research and dendrochronological dating on the Basilica of Saint-Denis at Liège in Belgium have pushed the date of a kingpost truss, much like what you might find at myriad 18th- and 19th-century churches in New England, back into the 14th century (Blain, Maggi, and Hoffsummer, 2015). Steeped in historical practice, the kingpost truss remains a popular choice to this day.

Later examples of timber trusses can be found in Palladio's treatise *I Quattro Libri dell'Architettura* (1570), where his work included bridge-truss designs. Though there is no record of any of them being built, the designs are convincing and would work, suggesting actual experience. Palladio also shows a variety of kingpost and queenpost trusses in the roofs of buildings, especially in the second book, some with very recognizable joinery, i.e., birdsmouth at bottom of rafters, standard flared kingpost top where rafters meet kingpost, etc. Since the books were printed in 1570, the timber truss tradition must have been in existence for a long time prior.

By the late 17th and early 18th centuries in both Europe and the New World, churches and public buildings were regularly being roofed over with trusses of the kingpost, queenpost (with raised bottom chord variants), scissor, kingpost with flanking prince posts, and eventually kingrod and queenrod trusses where an iron rod or bar takes over the tension functions of some members. While perhaps not meeting modern engineering criteria, these trusses, which typically spanned 36 to 75 ft., including the all-timber examples, were successful enough that tens of thousands of them still exist and are functioning well today all over the landscape (Lewandoski, Sobon, and Rower, 2006).

The fabulously inventive and complex roof frames of the Middle Ages largely disappeared from use. Many of these medieval roof frames only worked under the advantage of the extremely steep roof pitches favored by the Gothic style. These would not have been successful under the lower pitches of the Classic Revivals of the 17th to 19th centuries.

However, the inventiveness of framers did not disappear. Builder's guides began to appear in the 18th century, eventually in great number, and almost all showed examples of trusses suitable for different spans. Nevertheless, few extant historic trusses show exact copying of the published designs (Benjamin, 1806; Nicholson, 1801; Treadgold, 1837; Haupt, 1851). The vast majority of historic trusses were built for churches, town halls, and other public spaces.

The next most common truss types were short kingpost and queenpost trusses crossing a myriad of small rivers and canals throughout Europe and North America. These were usually not covered for protection against the elements and thus didn't survive long, but many still exist in illustrations. The widespread ability of framers to fabricate these short working trusses was so great that cost-benefit calculations were actually made for bridges across the Erie Canal. These calculations showed that it was cheaper to build three successive trusses (the original followed by two replacements) and invest the money saved by not covering the bridge at 5 percent return, than it was to incur the expense of adding a protective cover to a bridge (Whipple, 1847). However, the greatest long-span timber trusses were developed for wooden bridges. Where great expense was incurred, the bridges were wisely covered. By the early 16th century, we have accounts of bridges (such as that at Meissen in Germany) with spans as great as 139 ft. accomplished by complex frames that Philip S. C. Caston refers to as "polygonal arches" and that appear to be combinations of an arched form, multiple kingpost framing, and arch bracing rising from the abutments to the chords (Caston, 2020).

By the late 19th century, the Grubenmann brothers in Switzerland were constructing widely admired wooden bridges with spans approaching (at Schaffhausen) and then exceeding (at Reichenau) 200 ft. (Maggi and Navone, 2003).

At roughly the same time, New World framers working with the great timber resources available and a demand for river crossings where none existed, began experimenting with and inventing trusses capable of commonly spanning 70 ft., often 150 ft., and occasionally 200 ft. or more, with some eventually carrying railroad trains. These large and long trusses involved considerable expense and know-how to frame and erect correctly and were soon protected by patents in the names of their inventors. The vast majority of the large bridge trusses surviving or ever built were under the patents of Burr, Town, Howe, and Long. They were joined by a plethora of others such as Paddleford, McCallum, and Haupt, each possessing properties that their designer felt were useful improvements.

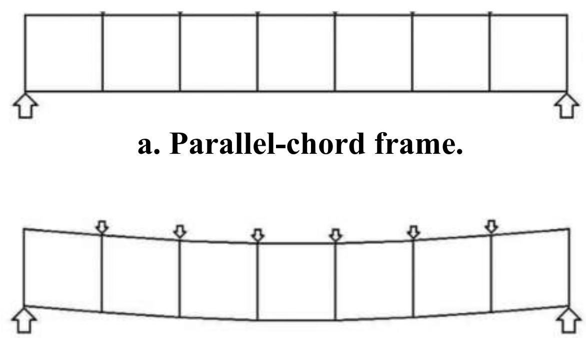
The difficulty of effecting good tension connections with wooden joinery within a constrained space soon led to the incorporation of vertical iron rods or diagonal iron counterbracing, and even iron or partly iron bottom chords in trusses. Bolts, rods, or iron tension splices were found initially in certain Burr, Wernwag, and Whipple trusses, in all Howe trusses, and eventually under the names of Pratt and Warren. Rods and iron members also found their way into bowstring trusses. This same period saw attempts by Whipple, Haupt, and others to quantify timber bridge engineering, following experiments beginning in the late 18th century in England and the European continent on the mechanical properties of timber (Tredgold, 1820; Haupt, 1851; Whipple, 1847; Barlow, 1817).

The Town lattice was the only truss that might include no ironwork at all, and even these utilized some keeper bolts and steel or iron tension splices in the timber lattices such as found in the Cornish–Windsor Bridge (1867), a timber lattice structure including two 204-ft. clearspans that join Vermont with New Hampshire across the Connecticut River. Keeper bolts and steel or iron tension splices are also often found in the double-lattice railroad bridges of the late 19th and early 20th centuries (Fletcher, 1909). Most of these long-span bridges were covered for the protection of the expensive trusses from weather.

The large and picturesque bridges are the most visible of timber trusses today, but driving around the countryside, particularly in the eastern half of North America, if you see three churches dating prior to about 1930 in a village, there are probably between 12 and 20 timber trusses of various, even unique, forms 100 to 250 years old concealed and at work in their roof systems. This record of accomplishment is prevalent, with examples present in town after town.

Development of Truss Forms

As an introduction to truss forms and strategies, Figures 2.6a and 2.6b show parallel chords with vertical members, but no triangulation. If the verticals and horizontals are rigidly attached, a frame of this type is known as a Vierendeel truss. However, a Vierendeel truss relies upon bending in its members and especially in its connections. Therefore, as implied above, such a frame structure deviates substantially from an ideal truss. Where the joints between the vertical and horizontal members are flexible, such a structure is unstable. With the addition of diagonals or other structural elements, the parallel-chord form becomes stiffer and behaves in a truss-like manner. The methods for modifying the form in Figure 2.6 gave rise to the variety of configurations described in the following sections.



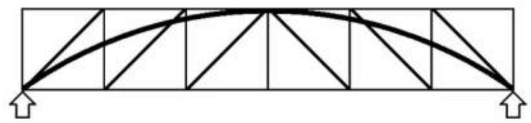
a. Parallel-chord frame.

b. Parallel-chord frame under load.

2.6 Parallel-chord frame (no diagonals).

This overview of truss forms is intended primarily to explain how some of the names came to be associated with these forms. Although there are many designers whose names are not widely known, the following few made contributions that have resulted in their names remaining familiar to practitioners to this day. While many of the truss forms have their origins in bridges, as noted below, these forms still find use in buildings as well. The names may be applied to simplified versions of the original patented form, but they may also reflect an adherence to the original structural concept.

Theodore Burr (1771–1822) Burr combined elements of trusses with timber arches (Fig. 2.7). In the *Covered Bridge Manual*, the authors note that approximately 880 covered timber bridges remain in the United States (Pierce et al., 2005). Of these, 224 are reported to be examples of a Burr arch.

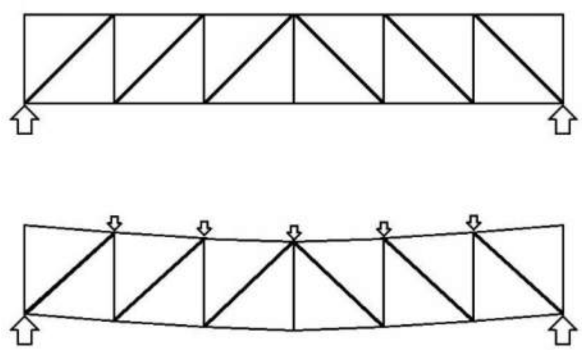


2.7 Burr truss.

Burr reportedly first received a patent in 1804 or 1806. The uncertainty of the date derives from the fact that his and many other patents were destroyed by a fire at the patent office in 1836 (Pierce et al., 2005).

As their prospects for durability were poor in the face of weather and the elements, the timber structures of Burr and others were usually covered with a barn- or shed-like structure. Although principally intended to protect the timber frames, the covers have evolved to become a treasured icon in our American heritage. A later Burr patent in 1817 is associated directly with the Union Bridge, which reportedly spanned the Hudson River between Lansingburgh and Waterford, New York. This bridge, which included four separate spans, was destroyed by fire in 1909 (Griggs, 2014).

William Howe (1803–1852) Howe was among the first to combine wood and metal in bridge trusses. In a parallel-chord truss, if one considers the frame in Figure 2.6a, the strategy for triangulation—and transformation into a truss—can be accomplished in either of two ways. Under load, the rectangular panels distort, with one diagonal getting shorter and the other getting longer (Fig. 2.6b). Howe's trusses use diagonal braces to resist the shortening of one of the diagonals (Fig. 2.8), resulting in compression in the diagonals and tension in the verticals.

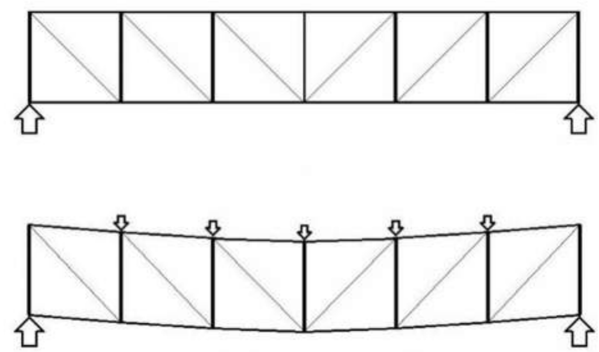


2.8 Howe truss; unloaded (above) and symmetrically loaded (below).

As the diagonals in a Howe truss are in compression, these members are prone to buckling. The diagonals are therefore typically made of heavy timbers, while the vertical tension members can be made of thin iron or steel rods. This emphasis on the diagonals is the principal identifying characteristic of a Howe truss. The construction of a parallel-chord Howe truss results in complex connections between the diagonals and both the top and bottom chords. Wood or cast bearing blocks simplified these connections.

Thomas Willis Pratt (1812–1875) In revisiting the idealized parallel-chord frame of Figure 2.6, one could adopt an alternative strategy for preventing distortion of the rectangular panels. To prevent the diagonals from getting longer, one could tie the corners together, placing the diagonals in tension and resulting in compression in the verticals.

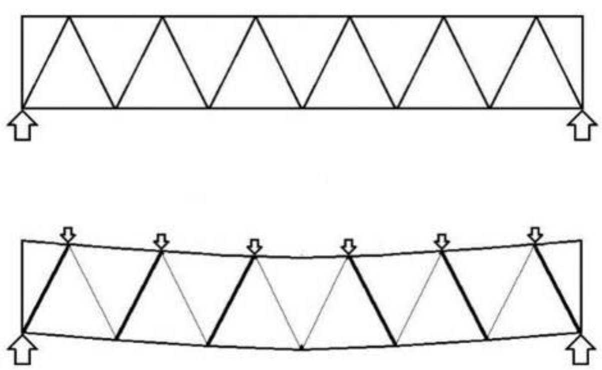
Pratt was an engineer, and his father, Caleb, was an architect. Together, they designed the Pratt truss, notable principally for having vertical members in compression and metal rods for diagonals in tension (Fig. 2.9). As the Pratt truss's verticals function as compression members, these elements can be timber; they are shorter than the diagonals, allowing the use of smaller cross sections and shorter lengths. Further, as the vertical timbers meet the chords at right angles, the joints became easier to manage. The longer diagonals of the Pratt truss required more iron than a Howe truss. As iron was expensive, the Pratt truss was not as commonly used, and at present there are reportedly only eight remaining covered bridges that incorporate Pratt trusses (Pierce et al., 2005). Over time, however, as the cost of iron declined, Pratt trusses gained in popularity and the form was used in all-metal trusses into the early 20th century.



2.9 Pratt truss; unloaded (above) and symmetrically loaded (below).

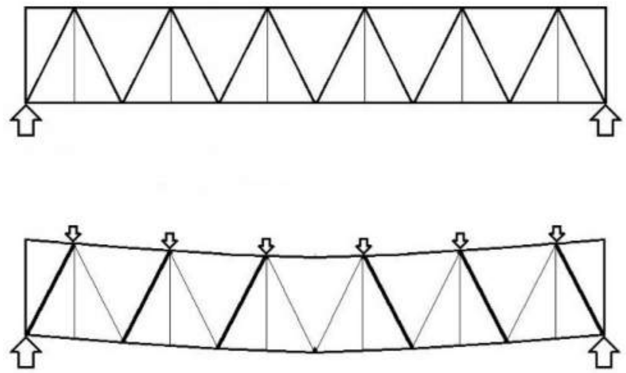
James Warren (1806–1908) A British engineer, Warren patented his truss design in 1848 in England, together with Willoughby Monzani. Warren did not hold an American patent.

A Warren truss deviates from both the Howe and the Pratt trusses by forgoing vertical web members altogether, favoring instead a simple zigzagging diagonal pattern (Fig. 2.10). The Warren truss often included equilateral triangles. In a Warren truss, the outward-sloping diagonals (shown here as thick lines) are in compression and the inward-sloping diagonals are in tension (shown here as thin lines). If steel or iron rods are used for the web members in tension, the resulting aesthetic will reflect this. Warren trusses became common among all-iron or all-steel structures, but as reflected in the *World Guide to Covered Bridges* (2021), only six timber bridges remain, each including variations on the simple Warren pattern shown in Figure 2.10.



2.10 Warren truss; unloaded (above) and symmetrically loaded (below).

Albert Fink (1827–1897) Born in Germany, Fink worked in the US. He gave his name to his patented truss (Fig. 2.11), which seems principally to have innovated by adding verticals within the typical triangular panels of a Warren truss.



2.11 Fink truss; unloaded (above) and symmetrically loaded (below).

The subdivision of the lower triangular panels makes sense for a through-truss bridge design, as the load, whether from vehicular traffic or from railroads, is applied to the bottom chord. This innovation supported the use of more and smaller crossing members because it reduced the load on each crossing beam, and this method reduced bending in the lower chords within each panel.

Ithiel Town (1784–1844) Town patented his lattice truss in 1820. His design is credited as being the first such timber structure to act independently of any arch action (Hayden, 1976). Although the truss is simple, comprising crisscrossing diagonals, it is inherently indeterminate, making structural analysis a challenge. The *Covered Bridge Manual* indicates that there are 124 remaining examples of Town lattice truss bridges, confirming that these trusses represent a significant percentage of the extant covered bridges (Pierce et al., 2005). Interestingly, it seems that Town more actively pursued selling his truss design than building it. Town reportedly collected royalties per foot of span for bridges built in this design. One way of looking at a Town lattice is to imagine superimposing three or more staggered Warren trusses on top of one another, as shown in Figure 2.12. The lattice is made by placing all of the diagonals in one direction in one plane, and the opposing diagonals in a parallel plane. Consequently, for the traveler crossing the bridge, the ply of timbers closest to the traveler is in tension at one end of the bridge and in compression at the other end. As the diagonals in compression are tending to buckle, they bow. With members bowing inward at the end where the interior diagonals are in compression, and the members bowing outward where the exterior diagonals are in compression, the width of the bridge opening differs at each end of the bridge. The end where the interior diagonals are in

Jim DeStefano

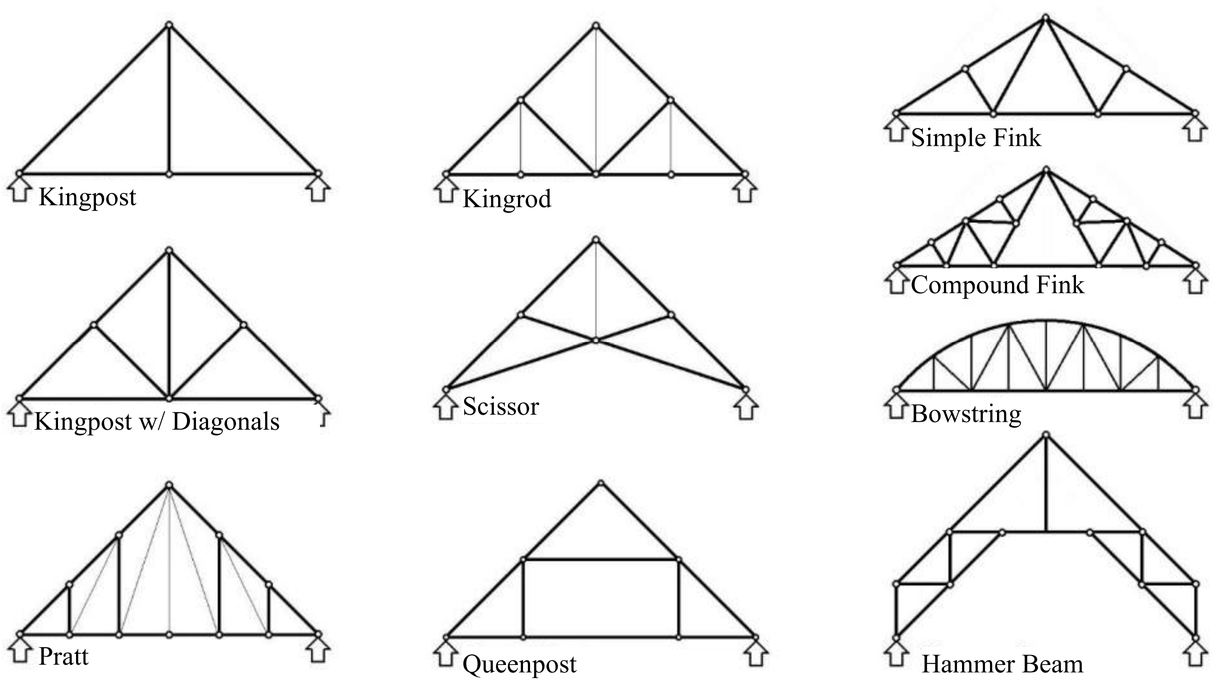
2.12 Town lattice trusses in the Worrall Covered Bridge in Rockingham, Vermont.

compression will measure slightly narrower than the end where the interior diagonals are in tension (Hoffsummer, 2010).

Roof Trusses

In the case of roof trusses with sloping top chords, some of the names given to the patented truss forms are used slightly differently. However, there is some logic to the application of the names, and we provide the following as additional background.

Figure 2.13 presents several versions of roof trusses with sloping top chords. The names assigned to each reflect those aspects of the inventor's patents that seem most descriptive in this context.



2.13 Common roof truss types and names.

For a kingpost truss, there is no inventor name, but these typically include a heavy timber for the midspan vertical. In an ideal truss, there is no force in this vertical. However, in a real truss, the bottom chord has weight and, where a building includes an attic, may support substantial dead and live load as well. Consequently, the kingpost is loaded in tension and could easily be replaced by iron or steel rods. Where the truss top chords support purlins or substantial distributed load along their span, a kingpost truss will typically include diagonals that support the top chord, but which also add to the tension demand on the kingpost. Although some assign the Howe name to a truss with vertical rods in tension, according to scholar Jan Lewandoski, it is more appropriate to call such an example a "kingrod truss." Where tension rods are oriented diagonally, the Pratt name still applies.

A *scissor truss* derives its name from the shape of the sloping, and crossing, bottom chords. The crossing point of the two sloping bottom chords is termed the *crux*. Between the crux and the top chords, the crossing timbers are loaded in compression. Between the crux and the eaves, the lower-chord members are in tension. The result is that significant forces must be resolved at the crux, making scissor trusses challenging to design and fabricate.

A *queenpost truss* may, in certain circumstances, not be a true truss, as the interior is not fully triangulated. Where the sloping truss top chords (principal rafters) are continuous from eave to ridge, the rafters and bottom chord form a simple truss. When the interior verticals and crossing member are added, these provide additional support to the rafters. In this way, the queenpost truss functions as a true truss. However, under nonsymmetric loading, the lack of a diagonal across the interior rectangle leads to bending in the members. As a result, the truss bottom chord must typically resist bending in addition to the axial tie force common to truss bottom chords. Further discussion of the queenpost truss follows in chapter 6.

The *bowstring truss* gets its name from the curved shape of its top chord. Depending upon the details of its web, some folks add to its name. For example, if the web pattern includes vertical web members in tension, some may refer to it as a Bowstring-Howe truss. If the diagonal web members are principally loaded in tension, it may be called a Bowstring-Pratt truss. However, the arched shape typically justifies the bowstring name, and there is no agreement on when that name should be expanded.

The *Fink truss*, shown here in two variants, is notable for subdividing the top chord, thereby reducing the demand at each panel point. When a roof truss is uniformly loaded, instead of supporting purlins and panel points, the principal rafter is loaded as a beam. The Fink truss provides more frequent support along the underside of the top chord, thereby reducing the bending in cases where it functions as a beam.

The *hammer beam truss* shown in this diagram is a conceptual approximation. Hammer beam trusses are characterized by the presence of a diagonal kicker, or knee brace, springing from the supporting column (or wall or buttress), and supporting the end of a short horizontal, a vertical above, and a diagonal that extends up to the crossing tie. In this diagram, the upper diagonals do not meet the crossing member at the same node, though many hammer beam trusses include diagonals that do. In this illustration, the crossing member would be subject to bending.

References

Barlow, P. 1817. *An Essay on the Strength and Stress of Timber.* London: J. Taylor.

Benjamin, A. 1806. *The American Builder's Companion.* New York: Dover Publications.

Blain, S., C. Maggi, and P. Hoffsummer. 2015. "Les charpentes de la collégiale Saint-Denis à Liège." *Archaeologie Medievale*, vol. 45.

Butler, H. 1929. *Early Churches in Syria.* Princeton, NJ: Princeton University Press, 330.

Caston, P. S. C. 2020. "Meissen's Covered Bridges." *Timber Framing* 137. Alstead, New Hampshire: Timber Framers Guild.

Fletcher, R. 1909. “A Neglected Splice for Timber Tension Members.” *Engineering News*, vol. 61, no. 20, 540.

Griggs, Frank Jr. 2014. “Union Bridge: First Bridge Across the Hudson River, 1804.” *Structure Magazine,* Feb.

Haupt, H. 1851. *General Theory of Bridge Construction.* New York: Appleton & Co.

Hayden, Martin. 1976. *The Book of Bridges*. New York: Galahad Books.

Hoffsummer, P. 2010. *Roof Frames from the XIth to the XIXth Centuries*. Belgium: Turnhout.

Knapp, R., T. Miller, and L. Jie. 2020. *China’s Covered Bridges*, Honolulu: University of Hawaii Press.

Lauffray, J. 1998. “Contribution à l'histoire de la charpenterie: une représentation de ferme à poinçon suspendu en Syrie du Nord au VIe siècle.” Beirut: Institut français du Proche-Orient.

Lewandoski, J., E. Levin, J. Sobon, and K. Rower. 2006. "Historic American Roof Trusses." Alstead, New Hampshire: Timber Framers Guild.

Maggi, A., and N. Navone. 2003. *John Soane and the Wooden Bridges of Switzerland*. Balerna Switzerland: Archivio del Moderno, Academia di architettura, Mendriso / Sir John Soane’s Museum.

National Society for the Preservation of Covered Bridges, Inc. 2021. *World Guide to Covered Bridges*.

Nicholson, P. 1801. *The Carpenter’s New Guide*. London: I. and J. Taylor.

Pierce, P. C., R. L. Brungraber, A. Lichtenstein, and S. Sabol. 2005. *Covered Bridge Manual.* Washington, D.C.:USDOT, Publication FHWA-HRT-04-098, April.

Treadgold, T. 1820. *Elementary Principles of Carpentry*, London: E. & F. N. Spon.

Whipple, S. 1847. *A Work on Bridge Building*, Utica, NY, 117.

Chapter 3 – Mechanical Characteristics of Wood

Introduction

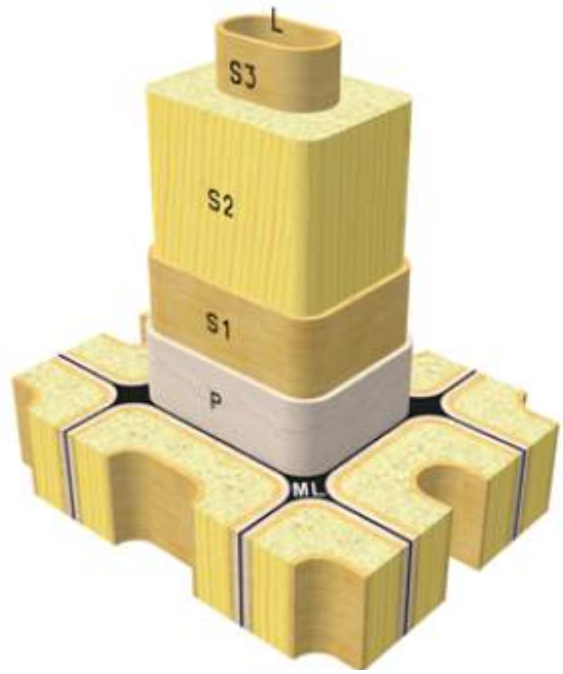
Wood might be the most well known but least understood of all building materials. Virtually everyone has handled a piece of lumber and perhaps used wood in some structural application. But relatively few really understand its unique mechanical characteristics, which evolve from its microscopic attributes and macroscopic growth features. As an organic material, wood has many virtues as a structural material, but it is also one with greater natural complexity than other materials, such as steel and concrete. That means it is critical that someone designing with wood have a sound understanding of its unique characteristics and behavior. The objective of this chapter is to outline some of the most basic mechanical characteristics of wood and how they impact its performance as a structural material, particularly with respect to its use in timber trusses.

Microscopic Characteristics of Wood

Wood is the cellular, organic material produced by the cambium layer of a living tree. The cambium layer lies next to the inner bark. Wood is composed of:

- *Cellulose*: Cellulose is a carbohydrate $(C_6H_{10}O_5)_n$ of long-chain polymers organized into microfibrils. Microfibrils are submicroscopic bundles of elementary fibrils held together by hydrogen bonds. Wood receives its strength and stiffness principally from the high tensile strength of the microfibrils.
- *Lignin*: Lignin is an organic compound that serves to bind the cellulose microfibrils together.
- *Hemicellulose*: Hemicellulose is an amorphous matrix constituent of the wood cell wall. The most important biological role of hemicellulose is its contribution to strengthening the cell wall by interaction with cellulose and, in some walls, with lignin.
- *Extractives*: Extractives consist of organic material that is deposited during heartwood formation. Extractives contain gums, resins, oils, and alkaloids, and they contribute largely to the color, odor, taste, and durability of heartwood.

A schematic of a portion of a normal wood cell showing the layers that make up the cell wall is shown in Figure 3.1. Cell growth starts with the primary wall P and progresses inward with three successive secondary walls: outer (S1), middle (S2), and inner (S3). Inside S3 is a hollow cavity known as the lumen. The boundary between cells, the middle lamella (ML) is rich with lignin to bind adjacent cells together. The middle secondary wall is principally responsible for the strength of the cell, and hence the wood itself, due to its greater thickness and microfibrils oriented along the axis of the cell. Wood cells are generally too small to be visible to the naked eye, but can be distinguished when viewed with a 10x jewelers loupe or a microscope. Cells may be up to 2.3mm long in hardwoods and up to 7.0mm long in softwoods.



after W. A. Côté (1967)

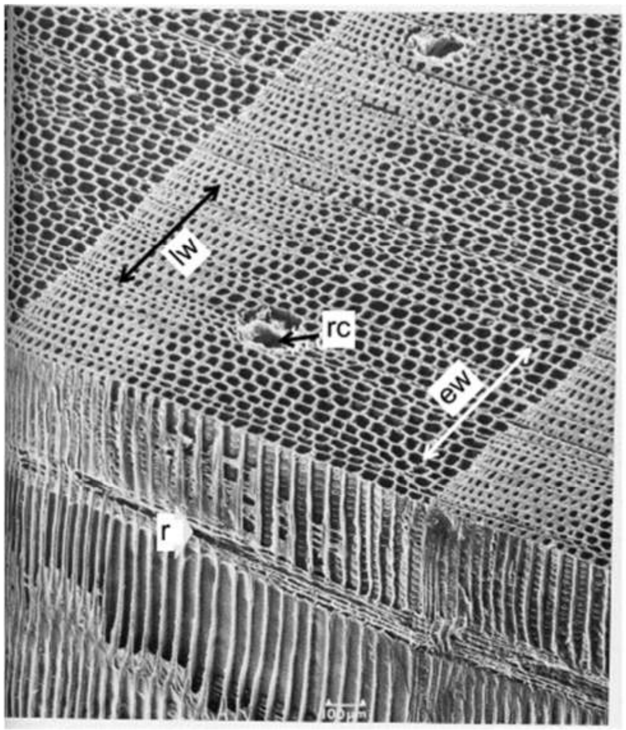
3.1 Wood cell schematic.

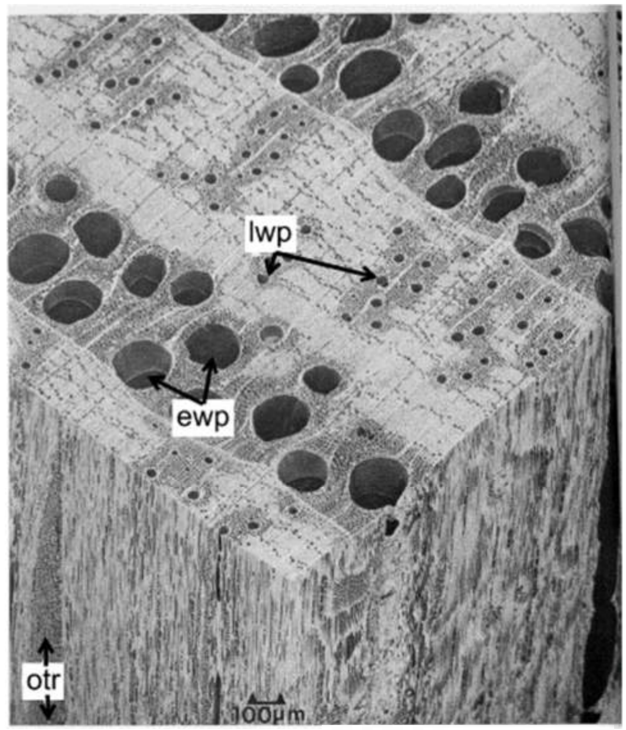
The critical message here is that the bulk of the microfibrils in the cell (those within S2) are generally aligned with the length of the cell and thus make the greatest contribution to strength, stiffness, and moisture stability in the longitudinal direction of the cell. Since most cells in wood are aligned with the stem of the tree, the greatest strength, stiffness, and volumetric stability are also aligned with the stem of the tree. While the relatively flat microfibrils wrapping around the primary and outer secondary walls make the cells themselves relatively strong in the lateral directions, there is no microfibril link binding cells together, only the lignin in the middle lamella. That means wood cells are relatively easily split apart under stresses causing tension perpendicular to grain or shear in planes that stress the middle lamella.

Macroscopic Characteristics of Wood

Timber-producing trees can be classified into two broad groups commonly categorized as hardwoods and softwoods. Hardwoods are angiosperms, flowering plants whose seeds are enclosed in the ovary of the flower. They have broad leaves that are typically shed annually (they are deciduous), and they have relatively complex anatomical structures. Softwoods are gymnosperms, whose seeds are naked. The order of gymnosperms that provides us with lumber and timber is the conifers (cone-bearing plants). Conifers have needle-like leaves that are not typically lost in the winter (they are evergreen), and their wood has a relatively simple cell arrangement. Hardwoods commonly used in timber frame construction include ash, red oak, and white oak. Historically, a wide variety of hardwoods was used for timber frame structures in North America. Timber framing with softwoods involves a broad range of species, including pine, fir, spruce, cedar, Douglas-fir, and southern pine.

Scanning electron micrographs of red pine (Fig. 3.2) and red oak (Fig. 3.3) specimens, shown at roughly the same level of magnification, illustrate the stark differences in their cell types, arrangements, and growth characteristics. For the red pine micrograph, the radial direction is from lower right to upper left; for the red oak, the radial direction is lower left to upper right.





H. A. Core et al. (1976)

3.2 Red pine micrograph. **3.3 Red oak micrograph.**

Important features that can be identified in these micrographs include the following:

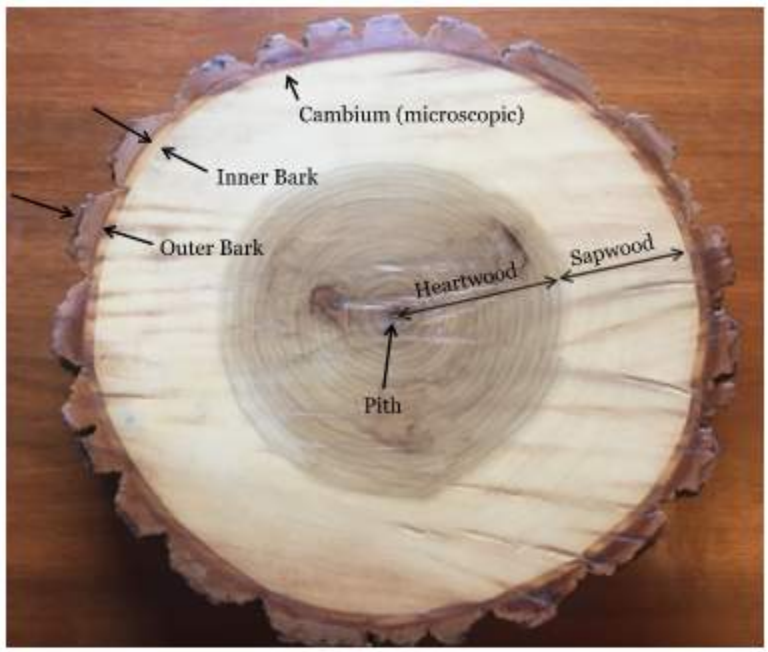
- *Annual growth ring*: The amount of wood cell growth that occurs during one growing season.
- *Early wood*: Large, thin-walled cells that form early in the growing season, a period of rapid growth (denoted *ew* in Fig. 3.2).
- *Late wood*: Small, slow growth, thick-walled cells that form late in the growing season (denoted *lw* in Fig. 3.2).
- *Resin canals*: Longitudinal cavities in some softwoods, two to five times larger than the average wood cell (denoted *rc* in Fig. 3.2).
- *Vessels*: Main arteries for movement of sap in hardwoods. Many times larger than the average cell. Vessels viewed in cross section are termed *pores* (denoted *ewp* for early-wood pore and *lwp* for late-wood pore in Fig. 3.3).
- *Rays*: Horizontal, radial cells that conduct sap from the bark toward the center of the tree in both hardwoods and softwoods (denoted *r* in Fig. 3.2; *otr* for oak-type ray in Fig. 3.3).

Observations regarding the features illustrated in these micrographs include the following:

- The vast majority of the cells in the tree are oriented in the longitudinal (stem) direction, making that direction the primary direction for strength and stiffness.
- Rays extend from the center of the stem to the bark, but no similar cell structure exists in the tangential direction (around the growth rings). Hence, wood responds to stress and changes in moisture content differently in the radial and tangential directions.
- Wood is far from a solid mass of material. In fact, domestic hardwoods and softwoods contain roughly 50–80 percent void space.

Physical Structure of a Tree

From the outside toward the center of the tree, the primary features of a cross section consist of the following (see Fig. 3.4):



3.4 Cottonwood cross section.

- *Outer bark*: Dry, dead tissue, gives protection against external injuries.
- *Inner bark*: Living tissue that carries food from the leaves to all growing parts of the tree.
- *Cambium*: Growth layer that produces bark and wood cells.
- *Sapwood*: Light-colored living and dead wood that carries sap from the roots to the leaves.
- *Heartwood*: Inactive wood, usually darker due to the presence of extractives. Contains no living ray cells.
- *Pith*: Soft tissue about which the first wood growth takes place in newly formed twigs (trunk stems and branches).

Physical Properties of Wood

Some of the key physical properties of wood that influence its behavior in a structure are outlined in this section.

Orthotropy Due to its growth characteristics as a natural material, wood is not isotropic. Rather, wood is orthotropic: its material properties differ in its three principal, mutually orthogonal directions. The three directions are:

- *Longitudinal* (in the height direction of the tree, also parallel to the wood fibers).
- *Radial* (directed from the pith to the bark).
- *Tangential* (tangent to the growth rings, also the circumferential or hoop direction).

Wood's mechanical properties vary substantially between the longitudinal direction and both of the transverse directions (radial or tangential). Some radial and tangential properties are quite similar, but others may be significantly different, as discussed later in this chapter.

Specific Gravity The specific gravity of wood is the ratio of the weight of a volume of wood to an equal volume of water. Specific gravity depends on the moisture content of the wood, which can vary from oven-dry (0 percent moisture) to green (greater than 30 percent moisture). The strength and stiffness of a particular species of wood depend primarily on its specific gravity. The greater the specific gravity, the greater the cellulose content in the wood, and hence the greater the strength and stiffness of the wood. As a natural material, there is variability in the relationship, but the trend is clear. Higher specific gravity generally results in higher strength and higher stiffness.

Moisture Content Wood can hold water in two forms: free water (water held in the lumen, vessels, and other voids by capillary forces) and bound water (water retained within the cell walls by molecular bonds). Free water is easily extracted in warm, dry air, whereas bound water is harder to remove. Under normal conditions in a structure, the free water will be absent from a piece of wood, but the wood will contain some proportion of its maximum capacity of bound water.

The moisture content (MC) of wood is defined as the ratio of the weight of water in a wood sample to the weight of the wood solids, expressed as a percentage. In the *green* condition of a fresh-cut log, the wood will contain both free and bound water. Its MC can even exceed 100 percent in some species. Fiber saturation point (FSP) is the MC at which all free water has been driven off but bound water is at its maximum. FSP varies by species but is generally regarded to be at an MC of about 29 percent. Equilibrium moisture content (EMC) is the moisture content at which the wood stabilizes with respect to its environment. EMC varies with environmental temperature and relative humidity, and can range from 6 to 15 percent, depending on climate and exposure to moisture, either vapor or direct precipitation. Oven-dry wood is wood in which all water, both free and bound, have been driven off. The *Wood Handbook* (FPL, 2010) and other resources list the typical values of specific gravity for various species of wood. These values are generally for oven-dry samples.

Response to Changes in Moisture Content

Shrinkage When the moisture content is above FSP, wood is dimensionally stable. Below FSP, dimensional change in the longitudinal direction with changes in MC is so small that it is regarded as negligible. But wood shrinks and swells in the radial and tangential directions as it loses and absorbs bound water. The rate of shrinking or swelling is roughly linear with respect to the rate of change in MC. Shrinkage from FSP to EMC tends to be the most challenging issue, compared to swelling, so this issue is the focus of the following discussion.

Due to the reinforcing effect of the rays, the rate of shrinkage in the radial direction is around half of that in the tangential direction. Some approximate shrinkage rates from green to oven-dry MC are shown in Table 3.1.

Table 3.1: Shrinkage rates, percent of green dimension, from FSP to oven-dry.

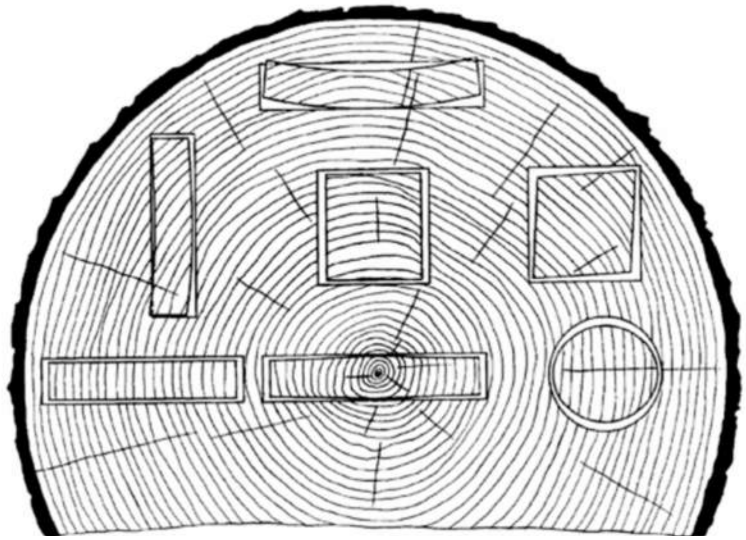
Species	Tangential	Radial	T/R
Ash, White	7.8	4.9	1.6
Oak, Northern Red	8.6	4.0	2.2
Oak, White	10.5	5.6	1.8
Cedar, western red	5.0	2.4	2.1
Douglas-fir (coastal)	7.6	4.8	1.6
Fir, white	7.0	3.3	2.1
Pine, Eastern white	6.1	2.1	2.9
Pine, longleaf	7.5	5.1	1.5
Spruce, Sitka	7.5	4.3	1.7

Source: *Wood Handbook* (FPL, 2010)

Oregon State University includes on its website a calculator that allows the user to input the species of wood, the beginning and ending moisture content, or the associated atmospheric temperature and relative humidity, and estimate the likely swelling or shrinking that will occur with the change in corresponding EMC.[1] If you are working with a known species of timber, and you need to account for the likely changes in dimension that will accompany seasonal changes in EMC, this is a valuable tool.

[1] http://owic.oregonstate.edu/wood-shrinkswell-estimator.

Differential Shrinkage Among other influences, warping is a consequence of differential shrinkage in the radial and transverse directions. Figure 3.5 shows a typical log cross section and includes the likely deformed shapes of members cut at various orientations to the grain. Because of the difference in radial and tangential rates, a member cut "perfectly" from a green log will change cross-sectional shape as it dries.



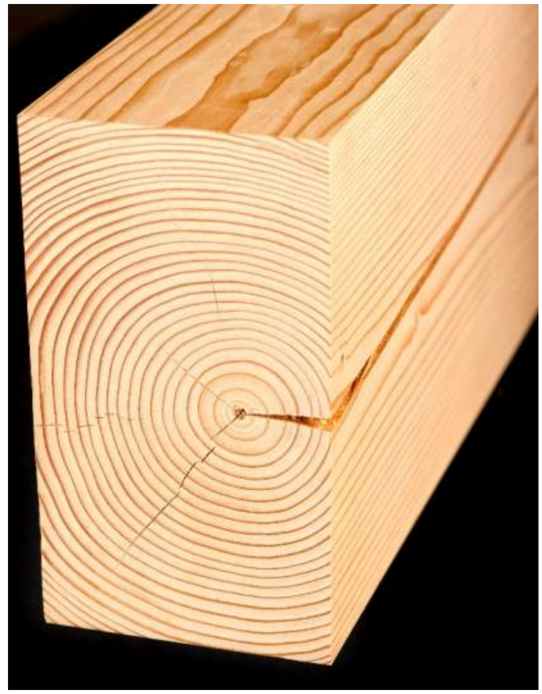
Wood Handbook (FPL, 2010)

3.5 Characteristic shrinkage and distortion of flat, square, and round pieces as affected by direction of growth rings. Tangential shrinkage is about twice as great as radial shrinkage.

Sawmills typically cut the logs, season the lumber—often including kiln drying—and then dress the resulting pieces to restore them to an appropriate cross-sectional shape. One can imagine surface planning each of the rectangular cross sections while still green to create the desired finished shape. In examining this diagram, only the quarter-sawn cross section (the flat board at nine o'clock) retains its shape, even as its dimensions change. The flat-sawn board at the top of the diagram shows cupping as it dries and shrinks. Once a board with this cut is in service, changes in its EMC will cause it to cup or uncup as it goes through seasonal changes.

Of greater relevance to timber trusses are the square or nearly square cross sections in Figure 3.5. As the square cross section at twelve o'clock in the figure seasons, its top and bottom faces will cup as the grain lines tend to flatten out. That will cause the vertical sides to tip inward, toward the top face. The other square section, at two o'clock in the figure, will distort into a diamond shape as it seasons. If either of these sections is used in a truss, the shrinkage distortion could potentially change the contact surfaces at the joints such that bearing areas are reduced, increasing local stresses.

The difference in radial and tangential shrinkage also causes checking, the development of radial cracks in wood. To understand checking, imagine a full-log cross section, such as that in Figure 3.4, in the green condition. It is helpful to first think of allowing a typical growth ring to shrink in circumference as much as it wants. As the ring shrinks, its diameter decreases. However, the wood that is enclosed within this growth ring does not shrink proportionately in the radial direction due to the difference in radial and tangential shrinkage rates. It is as if we allowed the ring to shrink and then stretched it back out so that it could fit around the inner column of wood. This imagined stretching induces cross-grain tension and usually results in radial checking. If a boxed heart timber (one containing the pith) is cut into a perfect square cross section when it was green, the portions of the cross-section that fall outside the outermost complete ring will change shape as they shrink. The final shape of the timber cross section will resemble a square with curved faces (Fig. 3.6). If the square-cut timber were a theoretical one, with the pith at the perfect center and the rings perfectly circular, checks would be expected to form at the midheight of each side. In real timbers, the pith is typically off-center, and the largest check normally forms on or near the face closest to the pith



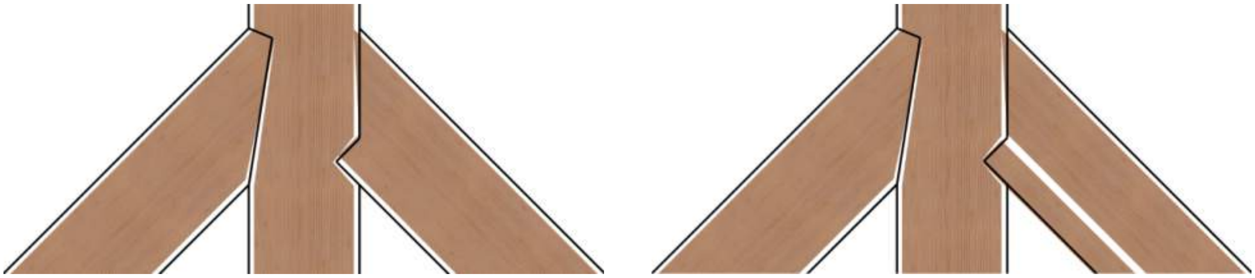
FraserWood Industries

3.6 Check in boxed heart timber.

As another perspective on differential shrinkage, imagine that a fresh-cut disk of white ash begins to shrink in the radial and tangential dimensions at the same time-rate from green to oven-dry. That is to say, for each 1 percent of radial shrinkage, tangential shrinkage would also be 1 percent. When both radial and tangential shrinkage have reached 4.9 percent (see the data in Table 1), the tangential direction still has 7.8% – 4.9% = 2.9% more shrinkage to go. As that remaining tangential shrinkage occurs, the circumference of the disk continues to reduce, but the length of rays does not change accordingly. Thus, tension-perpendicular-to-grain stress is developed, and one or more checks (radial cracks) will open to relieve that stress.

Generally, checking is not regarded as a serious strength-reducing characteristic of a timber. Axial tension and compression loads cause stress in wood fibers in the longitudinal directions. The presence of a check does not impact the timber's ability to carry these stress conditions. According to ASTM D245 (2011), checking is assumed to affect only horizontal shear capacity. A check that develops at or near the critical plane (at the neutral axis) can impact the timber's horizontal shear strength and possibly its bending capacity, since the check may interrupt the transfer of shear stress through the depth of the timber. Adjustments to the design values for horizontal shear are already included in the NDS, so no additional reductions are needed (Cheung et al., 2021).

For design and construction of joinery, the importance of understanding shrinkage derives from the difference between tangential and radial shrinkage when compared to longitudinal shrinkage. As noted above, wood does not shrink appreciably along its length. Therefore, when the end of a timber is cut at an angle, changes in moisture content that result in swelling or shrinkage will change the angle. If a joint between two (or more) members includes angled cuts, the connection may not perform as required if one does not account for the possible change in angle.



a. Prior to full bearing. **b. After full bearing.**

3.7 Distortions of joinery due to differential shrinkage.

The concept of differential shrinkage is directly applicable to the ridge joint of a kingpost truss, where the sloping principal rafters serve as the top chords. In the connections between the kingpost and the rafters, both the top of the post and the tops of the rafters are often cut to fit. Figure 3.7a illustrates two possible ways of cutting the joinery for the rafter-to-kingpost connection. A perfect fit of the joinery is illustrated by the solid outlines of the members, assumed to be green at the time of fit-up. As the members season and shrink, the fit will no longer be perfect, unless both members change geometry in exactly the same way.

For the connection on the right side of the kingpost in Figure 3.7a, both the rafter and the kingpost are notched such that the principal bearing surface, at the heel of the rafter, is at 90^{o} to the lower face of the rafter. As the rafter seasons, the width of that bearing surface is reduced, but its angle does not change. However, the bearing surface in the kingpost becomes steeper, opening a slight gap at the top. Likewise, the plumb cut on the rafter tilts inward toward the kingpost as shrinkage occurs. As load is applied to the truss, the rafter will thrust into the kingpost, bringing the bearing surfaces into tighter contact. As shown in Figure 3.7b, the result could be that the toe of the rafter on the right side of the kingpost becomes the only portion of the plumb cut to contact the kingpost. The resulting tension perpendicular to grain could subsequently lead to splitting of the rafter at the reentrant corner.

For the connection on the left side of the kingpost (Fig. 3.7a), the principal bearing surface is at the toe of the rafter, where both the rafter and kingpost cuts are made to bisect the obtuse angle between the two members. For this case, the change in angle of the two bearing surfaces is identical as shrinkage occurs, so no gap is formed and bearing pressure remains nominally uniform. The gap that opens below the principal bearing surface does not influence load transfer from the rafter to the kingpost.

As discussed above, there will be geometric changes in every case where timbers are worked at a moisture content that differs from their eventual in-service EMC. Depending on how a connection is detailed, joints may distort during seasoning, causing changes in the shape of the bearing surfaces. Where bolts are included in the connection, and in particular where steel side or knife plates are used, additional challenges may arise. If bolts are aligned parallel to grain in a member, then the member is free to shrink without constraint due to the connection. However, if bolts are spaced perpendicular to grain, then the potential exists for restraint of shrinkage, causing tension perpendicular to grain. The result is typically a split in the member that has the two bolts separated across the grain. This phenomenon is reflected in the *National Design Specification for Wood Construction* (AWC, 2018) in the table of wet service factors, C_M for connections. In the most extreme case, where a connection is fabricated at a moisture content above 19 percent and the in-service moisture content will be below 19 percent, the wet service factor may be as low as 0.4, resulting in a 60 percent reduction in connection capacity. Hence, use of bolted connections in trusses, particularly those including steel side or knife plates with multiple rows of fasteners, must be approached with caution. Where feasible, traditional wood-on-wood bearing-type connections are generally preferred.

Changes in moisture content may have beneficial effects as well. The strength and stiffness of wood increases with a decrease in moisture content. In particular, a timber that has reached EMC in service will have stabilized in its cross-section dimensions due to shrinkage, but it will also have increased in strength due to reduced MC. Since the design values for timber in Table 4D of the NDS *Supplement* are for green material, it is appropriate to account for the strength increase in dry material, so long as the adjusted design values are applied to the actual (reduced) cross section of the timber in situ.

Table 10 of ASTM D245 lists the percentage increase in allowable properties (strength and stiffness) above those for green lumber at specific moisture contents. This table applies to lumber (thickness less than 5 in.), but the same concept applies to timber. Research on the effects of seasoning on timber strength is limited due to the expense in testing timber-size members at various moisture contents. Nevertheless, available data (see Anthony and DeStefano, 2018) suggest that as timber seasons from its moisture content when tested nearly green (MC = 22%), its strength increases as given in Table 3.2. As mentioned above, any adjustment in strength must be used in conjunction with the corresponding change in member cross-section dimensions. Also, the strength increases tend to stabilize as MC reaches 10 percent, so no additional increase is taken below that value. No adjustment in F_v, the design value for shear, is made for seasoning.

Table 3.2: Percentage Increase in Timber Property for 1% Decrease in Moisture Content.

Property	Rate of Increase (%)
F_b	4.0
F_t	2.0
F_c	6.0

Compression Set The examples above emphasize the possible—and often likely—impacts of wood shrinkage. However, as wood typically moves with changes in season, there may also be annual cycles of swelling in summer and shrinking in winter. The following issues may arise when wood absorbs moisture and swells: Where moisture absorption can occur and expansion is unrestrained, there will be changes in geometry, and these may or may not present aesthetic or other problems. However, where a member is fully constrained against swelling, there may be crushing in the affected member. Consider a timber diagonal with its end let into the bottom of a principal rafter, possibly including a tenon that is fitted to a mortise that extends through the depth of the rafter: If the diagonal absorbs moisture and swells, the wood that is confined within the mortise will swell, but as its expansion is constrained, the wood fibers at the abutting surfaces will crush slightly perpendicular to grain. When drier conditions cause the member to shrink again, the crushed fibers remain crushed and the tenon no longer fits tightly in the mortise. The result may be an overall loosening of the joint, resulting in additional deflection. This irrecoverable shrinkage is due to a phenomenon known as *compression set*.

Any restraint on the ability of wood to swell as it takes on moisture has the potential to induce compression set, observed as abnormally high shrinkage in the radial and transverse directions. The induced compression restrains expansion with the moisture uptake part of the cycle, and then abnormal shrinkage occurs during the drying portion of the cycle. A particularly common consequence of this behavior occurs in bolted connections in which the bolts are tightened against the timber and subsequently subjected to moisture cycling. Over time, the abnormal shrinkage will result in the bolts becoming loose. Other impacts of compression set may include nail pops and ill-fitting post bases that utilize steel side plates welded to a steel base plate.

Elastic (Engineering) Properties

Wood is a heterogeneous material, due to its growth characteristics at both the microscopic and macroscopic levels, and it is generally regarded as orthotropic in cylindrical coordinates (longitudinal-radial-tangential). Thus, the common assumptions of homogeneity and isotropy do not strictly apply. Whereas axial properties are regarded as most important in member design, the off-axis (radial and tangential) properties play critical roles in connection design. To briefly demonstrate this point, consider the elastic properties for Douglas-fir given in Table 3.3:

Table 3.3: Elastic Properties for Douglas-fir.

Elastic (Young's) Modulus (psi)	Shear Modulus (psi)	Poisson's Ratio
E_l = 1,900,000	G_{lr} = 122,000	ν_{lr} = 0.29
E_r = 130,000	G_{lt} = 148,000	ν_{lt} = 0.45
E_t = 95,000	G_{rt} = 13,300	ν_{rt} = 0.39

Subscripts *l*, *r* and *t* represent longitudinal, radial and tangential directions, respectively.
Derived from data in Tables 4-1 and 4-2 of the *Wood Handbook* (FPL, 2010).

The material stiffnesses in the radial and tangential directions are similar, so wood may be reasonably regarded as isotropic in cross section, but the stark difference in stiffness parallel to grain (longitudinally) and perpendicular to grain (radial and tangential) must be acknowledged in designing connections. Some of the implications are as follows:

- Design of a connection that imposes compression perpendicular to grain is limited by deformation rather than material strength.
- Shear moduli for wood are much lower than for isotropic materials, so shear deformation of wood members can sometimes be a significant contributor to beam deflections.
- Wood's torsional stiffness, represented by G_{rt}, is remarkably low.
- Some strength properties in the radial and tangential directions, particularly tension stress perpendicular to grain and rolling shear stress, are so low that the NDS does not even assign design values to them (an exception is the unavoidable radial tension in curved glulams). Nevertheless, it is common for curved members (natural or glulam) to be used as the bottom chords in timber trusses. Overlooking the radial tension in these situations can be catastrophic.

Durability

The useful life of a wood member is indeterminate, so long as it is not overloaded and so long as it is protected from environmental hazards (decay fungi, insects, fire). Obviously, overload is controlled through proper structural design, construction, and use, so long as some catastrophic event does not occur. Control of environmental hazards is also possible, as described in the following.

Creep Rupture As noted in chapter 2, the vast majority of timber trusses existing today were not engineered. When analyzed with modern engineering methods and building codes intended for new construction, truss members are commonly found to be overloaded. These situations should be approached with careful consideration to discern whether the truss members may be susceptible to the phenomenon known as *creep rupture*. Truss members that exist in a constant state of high stress, with occasional periods of extreme stress when snow or live loads are present, can experience creep rupture, leading to eventual failure at stress levels well below published allowable values.

Decay Fungi Decay fungi grow under suitable conditions of mild temperatures plus sufficient moisture and oxygen—essentially the same conditions that we humans enjoy. The range of temperatures at which decay can occur is roughly 35º to 100º Fahrenheit. If the moisture content of the wood is above the fiber saturation point (> 30%), then decay fungi may be active. If the wood is exposed to air, oxygen will be present.

Different types of fungi attack wood differently, but essentially they consume cellulose and perhaps lignin. Extractives in a timber's heartwood often make it more decay resistant than the sapwood. However, the differences in decay resistance are highly species dependent.

Decay first reduces wood's toughness, its resistance to impact. Further decay reduces static bending strength. Ultimately, all strength properties are compromised. Laboratory test results (see Ibach et al., 2014) on decay include the following:

- A 1 percent loss in weight due to decay corresponded to toughness loss between 6 and 50 percent.
- A 10 percent weight loss due to decay corresponded to strength losses exceeding 50 percent.
- At 10 percent weight loss, decay is detectable only by microscopic examination.

Removing any one of the above three conditions—mild temperatures, moisture, or oxygen—will render decay fungi dormant. Temperature control is not likely feasible, but access to moisture and oxygen can be controlled. Kiln-drying timber to quickly lower the moisture content below the fiber saturation point is effective in preventing the onset of decay. Correspondingly, a common means of protecting fresh-cut logs from blue-stain and decay fungi is to place them under a continuous freshwater spray, thus keeping them soaked, blocking sufficient exposure to oxygen. Above all, the most effective means of preventing decay is to keep wood dry.

Local zones of high moisture content are sufficient to activate decay fungi. Such zones include connections that don't drain freely or post bottoms not elevated or otherwise protected from ground-level moisture. Similarly, truss heel joints and other bearing locations must also be isolated from contact with concrete or masonry foundation elements. Hence, proper detailing of structures exposed to the elements is essential.

Some basic principles to enhance the service life of a timber structure are:

- Build with dry material, free of incipient decay and within grading limits for mold and blue stain.
- Use construction details and building designs that keep exterior components dry and well drained.
- Use heartwood of naturally decay-resistant species or preservative-treated wood for exposure to aboveground decay hazards.
- Use pressure-treated wood for ground or concrete contact.
- Use moisture-resistant membranes below slabs.
- Ventilate attics and crawl spaces; isolate framing from soil or moisture-wicking materials such as masonry and concrete with a moisture-resistant membrane or nonporous barriers (plastics, metals, slate, and so on).
- Control migration of hot, humid air through the building envelope.

Insects A large assortment of insects enjoy dining on wood structures. The list includes bark beetles, powder-post beetles, round-headed beetles (aka: old house borer), termites (subterranean and aboveground), carpenter ants, and carpenter bees. All of these insects damage wood structures by boring through the wood, cutting longitudinal fibers, and creating voids. Treatments to control them include rapid debarking, applying insecticides or fumigants, storing in water or under freshwater spray, heat sterilization (a kiln process), pressure-treating with preservatives, and, of course, keeping wood dry and isolated from ground contact.

Fire Large timber members are naturally fire-resistant in that their outer surface chars and protects the inner volume of wood. The outer char surrounds a heat-affected zone, which does suffer some loss of strength, but the inner core of wood can retain its strength, possibly preventing structural collapse.

In the absence of a fire-suppression system (e.g., a sprinkler system), fire design of timber structures normally involves providing an additional thickness of wood (a sacrificial layer of material) over the cross section required to support the applied loads during the fire event. Based on the required fire resistance (a period of time during which the member is exposed to the fire), an effective char depth is determined for each surface exposed to fire.

Fire design of connections is also critical. Use of exposed metallic fasteners is a serious challenge in designing for fire resistance due to the high heat-conductivity of metals. Sacrificial layers of wood over fasteners are required to protect them from direct fire exposure. However, wood-on-wood joinery connections, without any metal, are more naturally fire-resistant.

Application of fire treatments can enhance the fire resistance of wood, but at some possible loss of material strength. Such an approach is not customary for timber members, but it may be effective for lumber.

References

Anthony, R. and J. DeStefano. 2018. "Effect of Moisture Content on Bending Strength of Timber." TFEC *Technical Bulletin* 9, Timber Frame Engineering Council. Alstead, New Hampshire: Timber Framers Guild.

American Society for Testing and Materials (ASTM). 2011. D245 *Standard Practice for Establishing Structural Grades and Related Allowable Properties for Visually Graded Lumber.* West Conshohocken, PA: ASTM International.

American Wood Council (AWC).2018. *National Design Specification for Wood Construction ASD/LRFD*. Leesburg, VA: American Wood Council.

Cheung, K., R. Anthony, M. Kam-Biron, and B. Yang. 2021. "Seasoning Checks in Timber." *Structure Magazine*, August.

Core, H. A., W. A. Côté Jr., and A. C. Day. 1976. *Wood Structure and Identification.* Syracuse, NY: Syracuse University Press.

Côté, W. A. Jr. 1967. *Wood Ultrastructure: An Atlas of Electron Micrographs*, Seattle: University of Washington Press.

Forest Products Laboratory (FPL).2010. *Wood Handbook: Wood as an Engineering Material, General Technical Report FPL-GTR-190.* Madison, WI: Forest Products Laboratory.

Ibach, R. E., and P. K. Lebow. 2014. "Strength Loss in Decayed Wood." AccessScience, McGraw-Hill Education, https://doi.org/10.1036/1097-8542.YB140275.

Chapter 4 – Structural Analysis

Structural analysis—the determination of loads, member and connection forces, support reactions, and deflections—is an essential component of truss design. There are no span tables for timber frame trusses, unlike those for rafters, joists, and other beam-type members. Except for the most trivial cases, trusses require individual attention to ensure that their design and detailing will be adequate for their intended use.

Two approaches are available for truss structural analysis: classical methods based on graphic sketches and hand calculations, and computer methods normally using commercial software. Classical methods rely on the *method of joints* and the *method of sections*, analysis procedures taught in undergraduate engineering courses. These methods are based on particle equilibrium and rigid body equilibrium concepts. Since the methods are accurate only for ideal trusses, they cannot be used to determine the shear forces and bending moments in members of a real truss. Nevertheless, these methods are useful for preliminary truss analysis and to verify the results from computer models. Appendix A contains brief outlines of the methods as a review of the basic equilibrium concepts.

The objective of this chapter is to outline some of the important considerations for effective analysis of timber trusses using computer software. It is assumed that readers have completed university-level coursework or other training on the fundamental principles underlying their software and are knowledgeable users of that software. Hence, the basic tasks of generating model geometry, assigning boundary conditions, and applying loads are not reviewed. A brief list of commercial packages that have proved to be effective for truss analysis is listed in Appendix B.

Considerations for Effective Truss Modeling and Analysis

Selecting Between 2D and 3D Models More often than not, a truss may be accurately modeled as a two-dimensional (2D) structure. However, there are occasions when three-dimensional (3D) models are needed to accurately predict structural behavior. Some questions to ask when deciding on whether a 2D model is sufficient follow:

- How do loads find their way to a 2D truss? Is the load path through the structure simple enough that structural components out-of-plane to the truss carry their loads directly to the truss? If so, then a 2D model is likely appropriate. Or, is the truss part of a more complex structural arrangement in which basic tributary area methods for proportioning load are less appropriate? In the latter case, it might be more appropriate to build a 3D model of an entire structural system to more accurately predict the distribution of loads.
- Is there load sharing among various building components? If each truss in a building act independently, it is likely to be a candidate for a 2D model. However, if trusses are arranged with each other or with other structural components such that they work together to carry loads, in proportion to their relative stiffnesses, then assignment of loads

by tributary area or other approximate methods is less likely to result in an accurate prediction of behavior, and a 3D model is indicated.

- Do multiple building components provide mutual support? Loads are distributed through structures according to the relative stiffness of the structural components. When building components of varying stiffness that act in parallel to resist loads, a 3D model is likely the only reasonable way to represent the stiffness variations and hence obtain the proper distribution of loads.
- Does a planar truss resist out-of-plane loading? Consider a truss placed against a tall end wall full of glass. Possessing little or no out-of-plane stiffness itself, the end wall must rely on the truss to resist that wind pressure. A 2D model of the truss might be sufficient for gravity loads, but a 3D representation of the truss, including out-of-plane stiffness of the members and the connections, would be essential to ensuring adequate strength and stiffness under wind loading.
- Are truss members, such as a bottom chord, used to brace other framing members? Consider a kingpost truss supporting ridge beams oriented perpendicular to the plane of the truss. Knee braces from the ridge beams down to the kingpost provide longitudinal stiffness to the overall structural assembly, and they also provide support to the ridge beams under gravity load. For unbalanced loading, the kingpost truss must resist out-of-plane thrust from the ridge-beam braces via bending of the bottom chord. In some cases, tension in the bottom chord could be used in a second-order analysis to increase chord stiffness, providing additional restraint against brace thrust. Such a situation would require a 3D model of what might be regarded as a simple 2D kingpost truss.
- When a truss is part of a larger structure such that boundary conditions for the truss don't match the standard options of roller support, hinge support, or fixed support, a 3D analysis may become a requisite. Imagine a series of scissor trusses, uniformly spaced and supported on longitudinal plates. The plates are heavy timbers supported intermittently along their spans. Analysis of the scissor trusses should include the lateral restraint provided by the plates against the outward thrust of the scissor trusses. Treating the scissor trusses as planar with hinge-roller supports will overestimate scissor chord tension. The restraint of the plates is elastic, so hinge-hinge supports for the scissor truss would be incorrect, providing more thrust restraint than is possible. Rather, a 3D model of all scissor trusses with their supporting longitudinal plates would more accurately predict overall structural behavior.

Special Element Characteristics The basic 2D frame element has three degrees of freedom, allowing it to represent member axial force, shear force, and bending moment. In 3D models, the frame element possesses six degrees of freedom, which result in six internal actions: axial force, two shear forces, two bending moments, and torsional moment. Special element types are often available in commercial software to handle nonstandard analysis requirements. Some of those element types are as follows:

- *Tension-only members*: Thin rods intended as tension ties and bracing should be modeled such that they resist tension only. In the actual structure, such members would buckle under relatively low compressive load, essentially eliminating their role as structural components. If in the analysis model such members are subjected to axial compression, then the software must perform an iterative analysis to mathematically remove them from the model.
- *Compression-only members*: Diagonal braces with pegged mortise-and-tenon connections are often used in pairs such that when one brace is loaded in tension, its companion is loaded in compression. To avoid overestimating the stiffness of a pegged joint in tension, it is conservative to model all braces as compression-only, the counterpoint to tension-only members.
- *Unbraced length*: Long compression or bending members might fail a software-based design check for stability (Euler buckling or lateral-torsional buckling) if the unbraced length defaults to the member length. Failure to recognize this oversight could result in unrealistically large member sizes to satisfy stability requirements. Hence, unbraced length specification is an essential part of the definition of a member in a model. Note that brace points must be accurately assigned. For instance, it is not uncommon for the compression chord of timber trusses to be braced out-of-plane only at their crossings with purlins and the ridge beam. Note that unbraced length may differ depending on the direction considered. In the compression chord example above, the in-plane unbraced length usually coincides with the distance between panel points, whereas out-of-plane bracing may be more or less frequent. Also, bracing against Euler buckling might not provide restraint against lateral-torsional buckling. These are important distinctions for modern analysis software that permits members to continue through multiple panel points of a truss.
- *Weak-axis bending*: Flatwise bending of members in a 2D model or biaxial bending in 3D models require appropriate definition of member local axes. The *beta angle* is used to rotate a member about its longitudinal axis so that the proper orientation is achieved. Common situations that require definition of the beta angle include purlins with their wide face lying in the plane of the roof and posts subjected to lateral load in 3D models.
- *Passing members*: Counterdiagonals or crossing scissor chords that extend past each other are attached at the crossing point for displacement continuity, but they are not joined to transfer bending moment. Special modeling techniques must be applied to permit the crossing elements to be hinged together without moment restraint. Such connections are known as *scissor nodes* in some commercial software systems. Without such a special modeling feature, the crossing connection must be fudged by use of element releases or other techniques.
- *Curved members*: The standard frame element is straight, with nodes only at its ends. A curved member, such as an arched bottom chord in a truss, must be given special consideration in an analysis model. If the software does not support curved members as a

special element type, then multiple straight elements must create a faceted curve. The number of straight elements needed depends on the radius of curvature and the length of the arc to be represented. No fixed rule exists for the proper number of facets. Instead, an iterative analysis is recommended in which the number of straight segments is increased until the results stabilize. Be aware that the facet approach is unable to predict the radial stresses in a curved member subjected to bending.

Boundary Conditions Accurate analysis requires that the boundary conditions on a model properly represent how the truss will actually be supported. For a 2D truss model, the support constraint at a node is generally a roller (restraint of one translational degree of freedom) or a hinge (restraint of both translational degrees of freedom). Fixed constraints are rarely appropriate due to the difficulty of connecting a timber member end for moment restraint. As discussed above, sometimes elastic restraint is more appropriate. In such cases, axial or torsional spring elements may be introduced, but their use requires knowledge of the appropriate spring stiffness to use. An alternative to building a 3D model is to analyze a 2D model with two sets of boundary conditions such that the actual restraint is bracketed between two extremes. The nodal deflection at roller supports and elastic supports should be given a reality check before accepting the results. Likewise, the thrust at hinge supports should be checked by considering whether the resulting force can be resisted by the supports. If the model behaves well at both extremes, it is reasonable to conclude that it will also behave well under a more precisely developed model.

Member and Joint Stiffness A fundamental principle of structural mechanics can be expressed as “stiffness attracts load.” That is to say, members in a structure resist load in proportion to their stiffness. Accurate modeling of element stiffness is essential to accurate prediction of both member forces and deflections. Correctly modeling joint stiffness also influences force distributions and overall deformation of the model. A few points to consider in this regard follow:

- Notching a member for joinery does not normally influence the member’s stiffness. For instance, the loss of cross-sectional area at a mortise for a diagonal brace has negligible impact on the overall bending stiffness of a truss chord. Hence, for the purposes of predicting deflections, members can be assumed to maintain their full cross section over their entire length.
- Hinge connections: Moment releases must be used in almost all cases when one member is connected to another at a common node. Generally, the software default is to attach members for full moment transfer, so failure to add moment releases at member ends will result in a model that is too stiff and misrepresents the distribution of forces throughout the elements. When long members are constructed using a splice or scarf, it is important to introduce a hinge connection in the appropriate element of the analysis model. For design of a new truss, the truss should be first modeled without the splice or scarf. Then the scarf should ideally be placed at or near an inflection point for the member. If

evaluating an existing scarf, some consideration of unintentional moment transfer through the connection may be needed, if the scarf is not located at an inflection point.

- Spring connections: Occasionally, a timber connection capable of resisting moment transfer is needed. In such a case, a rotational spring can be used to represent some moment-transfer capacity, yet still allow joint rotation so that deflections are properly modeled. The challenge with this approach is to properly identify the spring stiffness.
- Axial and shear springs: Similarly, extensional springs may be used to model the stiffness of wood-on-wood connections, especially those representing mortise-and-tenon connections subjected to tension where local deformation of and around pegs contributes to joint flexibility.

Finite Joint Size Frame elements in analysis software are regarded as centerline elements with no thickness. They are intended to represent the longitudinal axis of the member in the structure. Consequently, without special consideration, all connections between elements occur along element axes. But in real structures, connections are made between surfaces, offset from the member axes. These offsets induce shear and moment due to joint eccentricity. Modeling software usually includes the ability to add a rigid link or a rigid end offset to an element, which represents the location of a connection that doesn't correspond to element axes. The software predicts the impacts of eccentricity automatically. The obvious trade-off is the convenience of lining up centerlines for ease of analysis versus the reality of choosing the best joint geometry for cutting timber joinery, such as landing the struts in a kingpost truss up several inches on the post.

Loss of Connection Continuity The ability of a structural configuration to act as a truss could be compromised by the loss of force-transfer continuity at connections. For example, a heel joint resting on a post might be interrupted by a continuous eave plate over the post, causing the top and bottom chords to contact the plate rather than each other. Another example could be a continuous ridge beam passing through the joint at the peak of a kingpost truss, interrupting the connection between the top chords and the post. Basic truss behavior is compromised by such situations, in that force transfer through the connections involves more complex load paths, greater potential for crossgrain tension, and likely increases in joint flexibility. This loss of continuity must be recognized and modeled appropriately—or better yet, corrected by redesign of the structural configuration.

Structural Behavior Assumptions If a truss is analyzed with a single assumption or perception regarding its response to applied loads, important behavior considerations might be overlooked. For instance, the load case that controls member design selection, such as dead plus snow on a roof, might not be the load case that controls the design of every connection in the truss. Changes in configuration—such as inclusion or omission of knee braces—may also impact design decisions. Consider a case in which a truss sits on posts with knee braces below the bottom chord. It is common to design the bottom chord to span post-to-post without help from the knee braces, omitting them from the analysis model. But it is also important to evaluate the alternative results, should the knee braces become engaged under gravity loads. Without braces, the truss can just sit on top of the posts; but the introduction of knee braces would create tension under gravity loads where the chord joins the post.

Trusses, while often designed as stand-alone elements, are just as often incorporated into a greater structural framework. If a truss (or at least its bottom chord) is a part of the structure's lateral force–resisting system (e.g., pavilions, covered porches), and these lateral braces engage the truss chord, how could the "simple span" truss design be impacted? In other cases, frame members are added for aesthetics or maybe they only provide stability during frame raising. They might be ignored in the engineering analysis (or considered as tension- or compression-only members), but if these members are assembled using common joinery, they may be subject to (or generate) unintended forces in the frame. The designing engineer must take responsibility in considering and addressing all potential loading conditions and structural configurations for the structure.

Verification of Results

As with any engineering computational process—by hand or by computer—it is essential that results are accurate and that the model's response passes a "sanity check."

- The first tool available for verification is a statics check. Do the support reactions sum up to the applied loads? If not, is the equilibrium imbalance due to the analysis software automatically generating and applying dead load of the truss members to the model? Can a sum-of-moments equation be written to confirm rotational equilibrium as well?
- Are member forces realistic? It sounds obvious that a truss must have both tension and compression members. But if all members are in compression, the model is not behaving as a truss. It's possible that boundary conditions are in error.
- Do deflections and deflected shapes appear reasonable? Any spreading of support points needs to be handled by the supporting components (walls, posts, etc.). In the deflected-shape plot, do any members lose their attachment at nodes, suggesting that too many releases have been applied?
- Excessively high reactions could be a signal that support points are overly constrained.
- Look at the bending moment diagrams to confirm that the member ends aren't transferring moment through the connections.
- If a pair of 2D models are used to study complex 3D behavior (as discussed in the section above on boundary conditions), confirm that the deflections at roller supports and the reactions at hinge supports are appropriate.
- Look for surprises in the torsion diagrams for members in 3D models.
- Make sure roof live load and snow load assignments for principal rafters and purlins are applied correctly. The direction of the load should be in the global gravity direction, but the magnitude of the force per unit length for rafters may need to be scaled down so that the appropriate magnitude is applied to the inclined length of the member. If you think

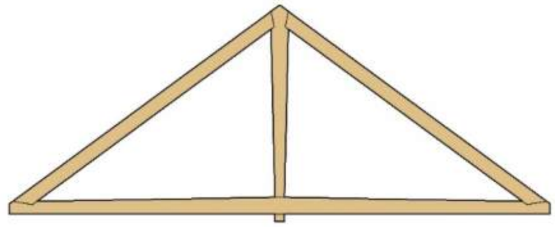
the software is handling this automatically, make a rudimentary model to verify that you understand what the software is doing.

- For 2D models, verify that compression members will be adequately braced for out-of-plane buckling. This is especially important for truss bottom chords that switch to compression when a lightweight roof is subjected to uplift.
- See chapter 10 for special modeling considerations when analyzing repairs and alterations to existing trusses.

Chapter 5 – Common Truss Forms: Advantages and Challenges

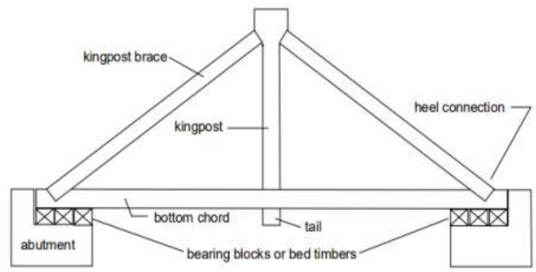
Kingpost Trusses

The kingpost truss (Fig. 5.1) is the most common truss form for short-span applications. Kingpost trusses are those with a principal vertical web member at the center of the span. Some have suggested that a kingpost truss must have a heavy timber in that vertical position, and that if a steel rod is used instead of a timber, it is no longer considered a true kingpost truss.



5.1 Basic kingpost truss.

A kingpost truss used for a short bridge would typically be loaded along the bottom chord, possibly with a transverse beam connected to the base of the kingpost. Taken from a Federal Highway Administration document (Pierce et al., 2005), Figure 5.2 shows a kingpost truss used to carry a roadway. As a result, the bottom chord is supported by the kingpost, either because there is a crossing beam located at midspan, or because the bottom chord is carrying the road deck directly.

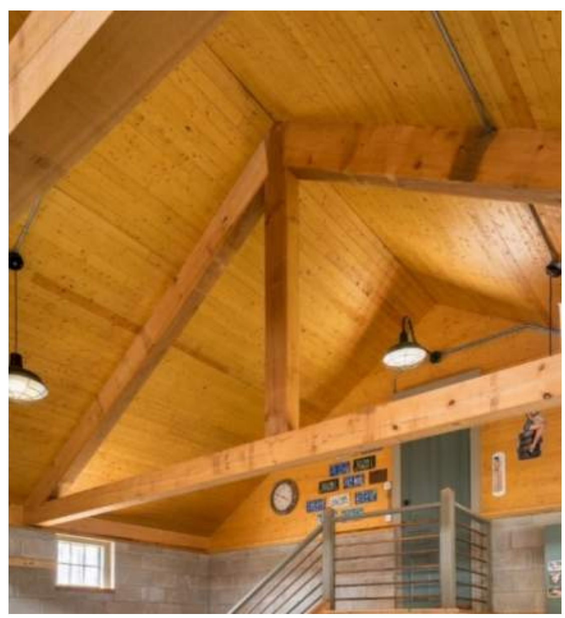


Pierce et al., (2005)

5.2 Kingpost truss for a bridge.

In a roof truss, roof loads are typically applied to the top chord through purlins or structural panels (see Fig. 5.3). In such instances, for a simple kingpost truss, the kingpost would theoretically be a zero-force member. In a real truss, the kingpost carries the dead load of the bottom chord and any additional loads applied to the bottom chord. Load on the bottom chord commonly includes joists to support an attic and ceiling.

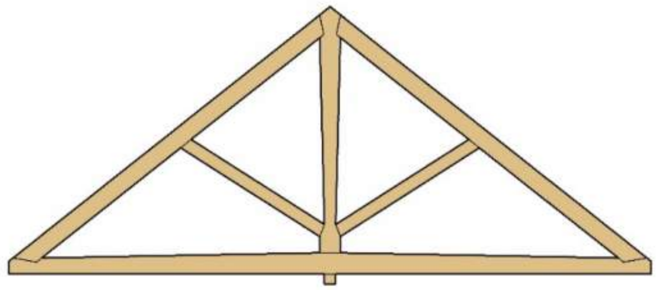
When purlins are added, or when the flexural demand on the principal rafters is significant, the kingpost truss is usually modified to include diagonal struts or braces (see Fig. 5.4). The roof loads applied to the top chord are then transferred through the braces to the bottom chord, to the kingpost, or to the joint where the kingpost meets the bottom chord.



Jim DeStefano

5.3 Kingpost truss supporting CLT roof deck.

Because of the inherent difficulty of bringing the braces, kingpost, and bottom chord all to a common joint, it is customary to connect the braces directly to the kingpost, and usually the connection is located just above the joint with the bottom chord. This arrangement prevents complexity in the joint between the kingpost and the bottom chord, and while there will be some bending and shear induced in the kingpost under asymmetric loading—as would be the case under strong wind, possibly accompanied by unbalanced snow loads—the kingpost can be sized to meet this demand. Joining the brace directly to the kingpost also avoids:



5.4 Kingpost truss with braces.

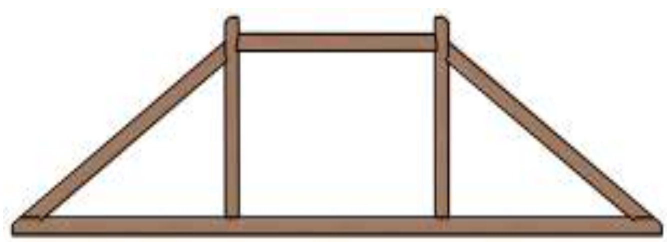
- transfer of brace compression into the bottom chord,
- introduction of shear and bending in the bottom chord, and
- an increase in the tension force transmitted from the bottom chord to the kingpost.

The modified version of a kingpost truss, as shown in Figure 5.4, is typically called a kingpost truss with braces.

In Figure 5.4, the kingpost is a timber, and the top has been shaped to fit between the top chords in a manner resembling the keystone in an arch. As noted, adding diagonal web members reduces the span of the top chords and either provides direct support for purlins or simply reduces bending if the roof is loading the principal rafters directly. Adding the diagonals increases tension in the kingpost and compression in the top-chord members.

Queenpost Trusses

With similar origins in bridges, the queenpost truss (Fig. 5.5) is notable for having two principal verticals instead of the single for which the kingpost gets its name. However, since the interior rectangle is not subdivided into triangles, a queenpost truss deviates from what is generally termed an ideal truss. The crossing member between posts serves as the top chord of the truss.



5.5 Basic queenpost truss.

Since this form lacks triangulation, shear and bending capacity in the bottom chord must be developed to maintain stability under nonsymmetric loading. Clearly, the biggest advantage is allowing the occupants of the attic greater access to the interior space without the intrusion of truss web members. For this reason, queenpost trusses have been and remain a popular choice. Their disadvantage lies principally in the resulting bending in the truss lower chord and in the principal rafters under unbalanced loading.

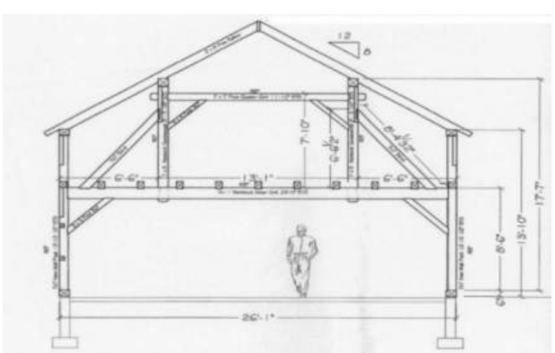
Alternate versions of the queenpost truss may have continuous rafters from the tie, over the post, and meeting at the ridge. In Figure 5.6a, the two vertical posts meet the top chords and the crossing member joins the posts with a vertical offset. Shear and bending are introduced in the posts due to crossing-member compression, but since the continuous rafters and bottom chord form a triangulated system, this compression is generally small. It is also common to allow the midlevel crossing member to connect to both sloping top chords, but to hold the connection between the verticals and the crossing member offset slightly toward the truss interior (Fig. 5.6b). This configuration introduces some bending in the crossing member, which needs to be designed for this load. With modern tools of analysis, it has become easier to size the members of a queenpost truss to meet the demands of several different load cases, including those with significant asymmetry.

a. Crossing member between posts.

b. Full-width crossing member.

5.6 Queenpost trusses with continuous rafters.

The form of queenpost truss shown in Figure 5.5, lacking full triangulation, can impose significant shear and bending demands on the lower chord under unbalanced loading. However, the truncated form does not generally act alone, but serves as the primary load-carrying system for an over-framed roof. For instance, Figure 5.7 illustrates a queenpost truss supporting the second floor and roof of a barn to provide a column-free space. Purlins supporting roof rafters rest atop the queenposts to complete the pitched roof structure. It is common to have purlins aligned with the



Jim DeStefano

5.7 Queenpost truss supporting second floor, providing column-free space below.

queenposts supporting a system of common rafters above that extend to the ridge. This arrangement facilitates preassembly of the truss and transportation to the site, with the common rafters added on-site.

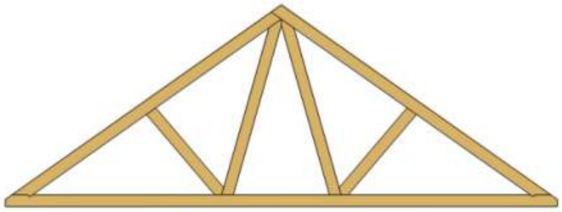
As noted above, a queenpost truss may be desired where there is to be an open area under the roof—a loft or attic. As such, there is likely a distributed floor load acting on the bottom chord, which must be sized accordingly for bending. Where the principal rafters extend to the peak, and under uniform and symmetric loading, the presence of the mid-height crossing member helps support the sloping top chords but increases the tie force in the bottom chord. Under wind load or asymmetric snow load, however, the lack of diagonals in the interior rectangle causes an increase in bending in the bottom chord. The lack of interior diagonals (corner to corner) can be somewhat rectified by the addition of knee braces in the interior rectangle, providing some stiffness against distortion (Fig. 5.8).

Jim DeStefano

5.8 Queenpost trusses without ceiling or attic loads on bottom chord.

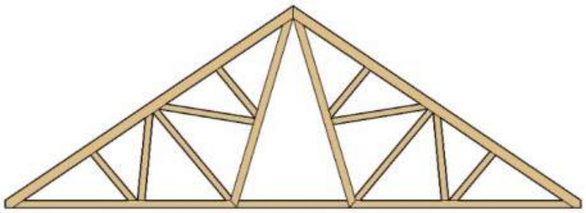
Fink Trusses

The Fink truss (Fig. 5.9) is an interesting and practical truss form for long-span applications. For gravity loading on the simple configuration, only the rafters and their support struts are in compression. The braces down from the ridge and the bottom chord carry tension. Hence, the truss can be assembled from appropriate combinations of timber and steel tie rods. The innermost diagonals may create a fairly large enclosed space, appropriate for attic storage.



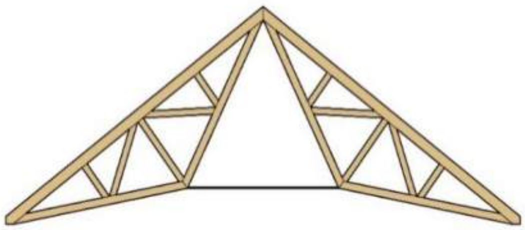
5.9 Simple Fink truss.

A compound Fink truss (Fig. 5.10) is formed by the addition of web members that sequentially reduce the effective unbraced length of the principal rafters (top chords).



5.10 Compound Fink truss.

One advantage of a Fink is that each half of the truss can be fully assembled in the shop, and these can be shipped to the site for final assembly. In some instances, the bottom chords can be sloped similarly to a scissor truss, with an elevated tie that connects them. The resulting form may add architectural drama (Fig. 5.11 and 5.12).



5.11 Compound Fink truss with elevated tie.

Jim DeStefano

5.12 Compound Fink truss consisting of two smaller trusses joined in the field.

Scissor Truss

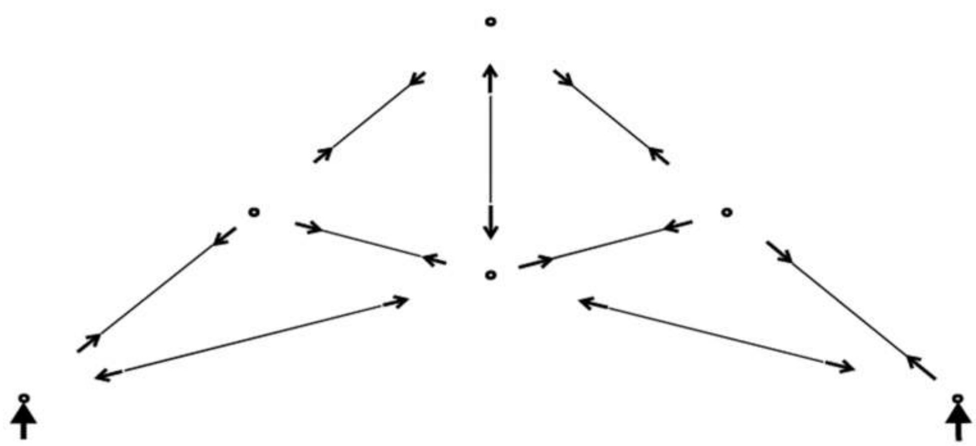
The scissor truss (Fig. 5.13) is a structurally inefficient truss form that remains popular nonetheless.

Characterized by sloping bottom chords that cross at midspan, a scissor truss allows for an elevated (vaulted) ceiling. In early Gothic cathedrals, where the masonry vaulting extended above the height of the side walls, the scissor truss made room for the vaulting by liberating the space where a horizontal, eave-level, bottom chord would have interfered. There is much attention required at the crux—so named because it is where the lower members cross and where the principal challenges lie in designing and fabricating a scissor truss.

5.13 Scissor trusses with kingrod, Trinity Evangelical Lutheran Church, Laramie, Wyoming.

The September 2003 issue of *Timber Framing*, Jan Lewandoski provides a comprehensive history of the scissor truss. The interested reader is directed to that work. The following reviews the special characteristics important to consider in the design and detailing of scissor trusses.

To understand the behavior of a scissor truss, the most important attribute is likely the change in loading in each of the crossing timbers at the crux.
An exploded view of a scissor truss analysis model (Fig. 5.14) illustrates the change in member force as the lower ties pass by the crux. For designing and building a scissor truss, it is also important to consider the tolerances that will be required when it is assembled. It is common to build a scissor truss with continuous crossing members, each carved to allow passage by and connection to the alternate member. As a result, if there is any flexibility in the truss due to loose-fitting joinery, there will be bending concentrated at the crux and at the joints where the crossing members connect to the top chords.



5.14 Exploded analysis model showing axial forces on members.

A vertical tie from the ridge to the crux is essential, as in its absence, the resulting bending in the crossing members and rafters is the only means by which the scissor truss can function. Absent the vertical tie, the crossing members are in tension over their full length, compounding bending in the top chord as well as requiring a challenging connection between the two. If the crossing members are curved and bolted, instead of half-lapped, unbalanced loads on the roof can cause crossgrain tension at the crux, a demonstrated cause of failures in industrial revolution–era scissor trusses.

Considering how much bending can occur at the crux, where it is common to remove at least half of each crossing member to make them pass one another with a lap joint, it is understandable that dealing with this joint is the crux of the problem. Aside from the fact that the vertical element is essential to develop truss behavior, it is typically important to provide some adjustment in the vertical element to allow alignment of the crossing members with a minimum of eccentricity. That is to say, the bearing surfaces between the two crossing members must be as close to coplanar as possible to minimize weak-axis bending at the reduced cross section.

Making the connection between the crux and the vertical can be challenging when the vertical is also a timber. Between the principal rafter and the crux, the portion of each crossing member is in compression. Between the crux and the heel joint, the portion of each crossing member is in tension. When drawing a free-body diagram of the crux, this results in a significant need for an upward vertical force to counteract the four converging forces, each of which includes a downward-acting component (see Fig. 5.14). The result is a confluence of forces at the crux, where it is common to remove portions of both crossing members to allow them to pass each other and remain coplanar. Considering the additional need to support the crux with the mid-span vertical, it may be perceived as necessary to remove more wood at this critical connection.

There are three approaches to handling the force transfer at the crux:

- Tension rod as vertical (including a variant where the rod is inserted through the middle of a timber kingpost).
- Triple dado, where the kingpost is dadoed symmetrically, to pass through the crux joint, and the wood to either side of the dado is loaded in compression from above by the intersecting and dadoed crossing members. This also requires that enough of the kingpost extends below the crux to adequately transfer the load in shear from the bearing surfaces into the kingpost.
- A clasping kingpost, where two halves are dadoed and fitted around the crux from the outside. Again, the kingpost must extend below the joint far enough to adequately transfer the load through shear in the clasping element.

In these latter two cases, the vertical kingpost will extend below the crux. As this is often preferred aesthetically, the extension of the kingpost is not usually a problem.

Hammer Beam Trusses

To quote timber engineer and architect Jim DeStefano, “Hammer beam structures are by far the most structurally inefficient, problematic, and celebrated of all timber roof types.” Their behavior bears very little relation to that of other truss types, as their members are subjected to significant bending moments and shear. Hammer beam trusses are typically associated with stone Gothic churches, where the outward thrusts at the tops of the walls are resisted by massive stone walls with buttresses—even flying buttresses.

Westminster Hall in London (Fig. 5.15) is not really a cathedral. It is part of a campus of buildings that make up the Parliament; it is where British state events, such as the coronation of kings, have been held. The timber hammer beam roof structure was commissioned by Richard II. It was built by master carpenter Hugh Herland and completed in 1397.

The Times (London), 26 Sept. 2017, Times Newspapers Ltd.

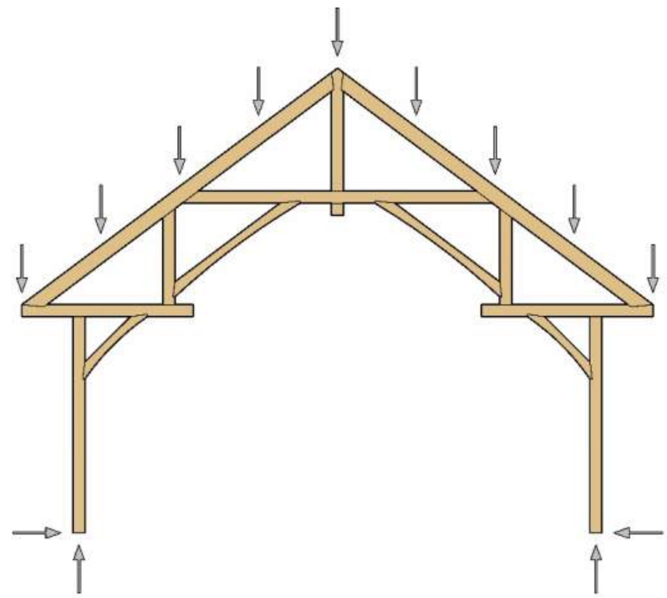
5.15 Westminster Hall, London, England.

The exposed timber structure was actually budget driven, since it was a fraction of the cost of a vaulted stone ceiling. The timbers were ornately carved to resemble stone. Westminster Hall is the largest and most famous medieval hammer beam structure in Europe.

The immensely thick exterior stone masonry walls were repurposed from an earlier Anglo-Norman hall built in 1097 during the reign of William II. The stone masonry walls are over 6 ft. thick and are reinforced with buttresses on the exterior. The horizontal thrust is resisted by the masonry walls, and the timber hammer beam structure behaves much like an arch, with all of the members and joints loaded in compression.

When hammer beam structures are adapted to modern applications, such as that in Figure 5.16, with the massive masonry walls replaced by slender timber columns, the structural behavior is profoundly different.

Where a computer model is employed in analyzing and designing a hammer beam structure, the results of the analysis are sensitive to assumptions about the supports at the column bases. These supports must both be treated as pins (or hinges), providing both vertical and horizontal reactions (Fig. 5.16). In a statically determinate analysis, it is common (and erroneous) to treat one of the bases as a roller, allowing for a simple analysis but resulting in a completely inaccurate interpretation of the structural behavior.



5.16 Hammer beam truss schematic.

In considering the portion of the hammer beam truss that extends above the eave line, one may view it as a pair of trussed rafters, bearing on the walls, and leaning against each other at the ridge. However, when rafters lean against each other, an eave-level tie is needed, or some other means must be provided to resist the outward eave-level thrust. In Figure 5.16, the roof loads and eave-level thrusts are transferred to the columns and knee braces, placing significant demands on these elements. The exterior posts need to be designed for the combined action of axial loads and bending induced by the compression in the knee braces. The knee-brace joinery typically results in a reduced section in the post at the point of maximum moment. Post sizing needs to take both strength and stiffness into account to produce a frame that can resist the thrust of the trussed roof rafters.

In Figure 5.16, the uppermost horizontal member will be in tension, but it is clearly not positioned to resist the entire thrust of the roof framing. Without stiff posts and properly sized knee braces, the tension in the horizontal tie would be very large, as would be the bending in the other frame elements. Although inconsistent with most aesthetic preferences for a hammer beam truss, adding an eave-level tie improves their performance greatly by introducing a bottom chord that enhances truss behavior. A steel rod with a turnbuckle could serve as the tie, but, as noted above, there is rarely an appetite for the aesthetics of a crossing tie. Because they are needed, and because they are often despised, it is not uncommon for ties to be added later, after the eaves have spread, the posts have bowed out, or both. The photo in Figure 5.17 shows a chain used as a tie element.

As noted, masonry buttresses would historically resist the outward thrust. These may be built into the wall or as flying buttresses spanning over side aisles. Whenever possible, such a strategy may still be exploited, as when there are side aisles or perpendicular wings that abut the space over which the hammer beam trusses span.

Jim DeStefano

5.17 Hammer beam truss with a harped chain tension tie.

The inward end of the hammer beam, that is, the lowest horizontal member projecting from the wall, has a tendency to buckle out-of-plane as a result of compression in the hammer post (the first interior vertical) and the knee brace, requiring some form of lateral bracing. This bracing can be accomplished with vertical knee braces from purlins to the hammer posts (as shown in Fig. 5.18), with horizontal knee braces from the wall plates to the hammer beams or with horizontal bridging running the length of the building and anchored at suitable locations in the end walls.

Jim DeStefano

5.18 Modern hammer beam truss installation, Saint Patrick's Church, Redding, Connecticut.

While a symmetrically loaded hammer beam structure may appear fully stable and within reasonable bounds in terms of member forces, moments, and stresses, hammer beam structures are very sensitive to asymmetric loading, and especially to wind. Hammer beam structures make visually appealing end-wall frames, but they perform poorly in those situations. Because of the lack of either continuous vertical or horizontal members to transfer out-of-plane wind loads to adjacent resisting elements, they do not perform well when subjected to positive or negative air pressure. As a result, hammer beam structures should not be used on exterior walls without adequate in-plane and out-of-plane bracing. Placing that bracing in the wall cavity can be one option, but that too can be a challenge when the wall consists largely of windows to support dramatic views.

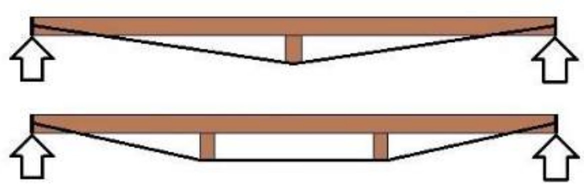
Nevertheless, in spite of the challenges presented in analyzing, designing, and constructing hammer beam trusses, their aesthetic appeal cannot be denied, and they provide great opportunities to demonstrate skillful execution of details by the joiner (Fig. 5.19).

Jim DeStefano

5.19 Detail of hammer beam trusses from Saint Patrick's Church.

Inverted Trusses

Old mill buildings often include inverted kingpost or queenpost trusses (Fig. 5.20), commonly referred to as *trussed beams* or *trussed girders*. These may also be thought of as strengthened beams, where iron or steel rods extend from one end to the other, with enough vertical offset to provide a vertical reaction at the point where the strut meets the beam. At a shallow angle, the resulting axial load in the beam goes up, even as the bending stress goes down. Figure 5.21 shows an example of wood struts and iron rods configured as an inverted queenpost truss. This method of strengthening beams became so popular that an industry developed to produce cast-iron hardware for this use.



5.20 Inverted trusses: kingpost (above) and queenpost (below).

In an example of strengthening, a wood-framed roof over a riding arena proved to be inadequate in bending as the lumber used was not of the grade intended by the designer. The frame included metal-plate-connected dimension lumber bundled to form larger elements. Rather than supplement each frame with an entirely new steel truss, as recommended by one engineering firm, the repair employed a version of inverted kingpost trusses, together with a raised bottom chord to strengthen each frame and allow the building to retain some of its openness (Figs. 5.22 and 5.23). Rafters along one slope are now inverted kingpost trusses. The crossing ties restrain rafter thrust and act as bottom chords, but are raised to provide greater headroom for horses and riders.

5.21 Inverted queenpost truss. (Storage of a snow shovel was not likely intended by the inventor.)

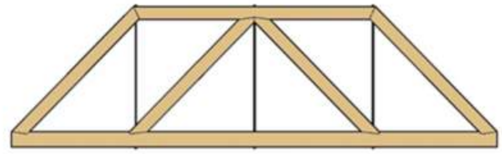
5.22 Indoor riding arena: before (left) and after (right).

5.23 Indoor riding arena roof strengthened with inverted trusses and tie rods.

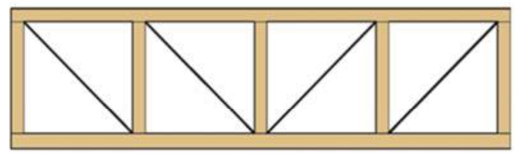
Parallel-Chord Trusses

Parallel-chord trusses are occasionally utilized in timber structures. For example, parallel-chord trusses in residential and commercial buildings may be used as ridge beams or purlins, spanning parallel to the ridge line, or they may form the uppermost portion of a timber wall frame. This discussion considers trusses with spans less than 100 ft., although longer spans are possible. This discussion applies equally to timber and glulam components.

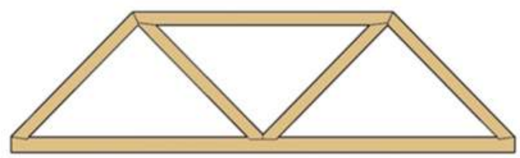
Configurations Parallel-chord truss configurations may be driven by the span and spacing, aesthetics, support conditions, and environment. In general, there are three main configurations (Fig. 5.24), one of which lends itself ideally to heavy timber design and the other two by either aesthetics or support conditions.



Howe truss



Pratt truss



Warren truss

5.24 Common parallel-chord trusses.

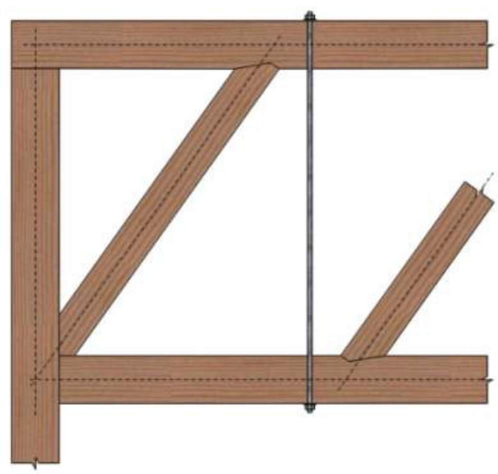
Howe Trusses The Howe truss is a desirable configuration where bottom-chord bearing is desired. In this configuration, the diagonal members are in compression and the vertical members are in tension. The vertical members lend themselves to the use of steel rods, while the diagonal members lend themselves to timber. The (more or less) parallel chords are typically timber. The diagonal timbers rise from the point of bearing upward and inward toward the center of the span. The truss is symmetrical about the transverse centerline, and the configuration of the diagonals is mirrored about this centerline. A common configuration for the panels (spacing between vertical members) is where the width of the panel and the depth of the truss are more or less equal. Under heavy loads, the panels are typically taller than they are wide. For an even number of panels, as shown in Figure 5.24, opposing diagonals meet at the center vertical tension rod. With an odd number of panels, the center panel may have crossed diagonals or diagonals may be omitted entirely (though this changes the structure and requires adequate flexural strength and stiffness in the chords to resist unbalanced loading).

Vertical member joinery for the Howe truss is straightforward. The tension rods pass through holes in the top and bottom chords and are secured with washers and nuts. Tension rods are typically ¾ in. or 1 in. in diameter. Larger sizes may be required for longer spans and heavier loads. Washers are sized for the compression perpendicular-to-grain loading.

Ogee washers on the bottom chords are often used for aesthetic purposes where exposed to view. Where the induced compression stresses from these washers exceed their allowable limits due to the diameter of the ogee washer, larger plate washers can be embedded flush with the face of the timber and capped with the ogee washer. Extending the rod well beyond the nut allows for later utility use of the rods, if the trusses are exposed from below.

For a flush bottom surface on the truss, the rods can be welded into holes in the plate washer and recessed into a shallow mortise in the bottom chord, relieving the need for a rather deep mortise to accommodate the plate washer, nut, and thread extension beyond the nut. The tension rod connection in the top chord may need to be recessed if flush, tongue-and-groove decking is applied directly to the top chord. Otherwise, plate washers and nuts are usually acceptable and are hidden by overframing.

Alignment of the truss web members is important in all cases, but alignment may become critical in a Howe truss. Figure 5.25 illustrates a small but significant eccentricity between the lines of action of the timber diagonal and the vertical steel rod. This offset causes the web-member forces to be transferred in shear and bending through the top and bottom chords, which could overstress the chords as these effects are combined with the axial forces in the chord members.



5.25 Misalignment of centerlines in a Howe truss.

Trying to adapt timber tension members to a Howe truss is possible, of course, but is neither the most desirable nor the most economical approach. Connections can be accomplished with metal plates secured with pins or bolts, embedded steel rods inserted through longitudinal drilled holes in shorter members or split members in longer timbers, and proprietary connectors such as Timberlinx.

Compression-member joinery can be accomplished with birdsmouth notched connections into the interior faces of the top and bottom chords secured with extended tenons, bolts, or screws. Reinforced bolsters may be required at bearing points if the induced horizontal shear forces from the birdsmouth to the end of the timber are excessive. Bolsters and special considerations at the bearing ends of the trusses are often required for longer-span, more heavily loaded trusses. (See chapter 7 for additional guidance on connection detailing.)

Pratt Trusses In the Pratt truss configuration, the forces in the members are reversed from those in the Howe truss: the diagonals are in tension and the verticals are in compression. This configuration is useful where top-chord bearing is desired. The diagonals go down and inward from the bearing point, making them, in effect, part of the bottom chord (they drag bottom-chord tension up to the top chord). Any vertical end members of square-ended trusses with top-chord bearing are usually just floating and have no induced loads, other than that from their own dead weight.

The timber verticals are usually secured with simple mortise-and-tenon joinery. However, the connections of diagonal tension members become more complex and expensive than the typically simple birdsmouth compression joinery of the timber diagonals in the Howe truss. Timber diagonals may be connected with plates and pins or bolts, tension tie rods, or proprietary connections such as Timberlinx. Steel rod diagonals require long holes drilled at an angle through the top and bottom chords and use of beveled washers or notched-in plates, all of which require greater time and precision than that required for the rods in a Howe truss.

Like the Howe, the typical configuration of the Pratt truss maintains a 1:1 relationship (more or less) between panel widths and the truss height. If a Pratt truss is designed and fabricated to allow post-tensioning of the diagonals, it may be possible to true the geometry should shrinkage or loading cause the truss to sag or induce camber to oppose dead-load deflections.

Warren Trusses A Warren truss is based on a configuration of triangles. This type of truss is more typically used in bridges and is seldom seen in building construction. Diagonal forces alternate from tension to compression. Joint detailing then becomes a combination of the diagonal compression or tension joinery employed in the Howe and Pratt trusses.

Special Considerations

Camber and Splices An important consideration in the design of parallel chord trusses, especially Pratt trusses with timber diagonals, is deformation of the connections that induces sagging of the truss. Oversized drilled holes can allow unacceptable deflections. Holes in timbers should be drilled the same size as the bolt or pin such that they need to be driven through the timber with light blows of a mallet. They should not be forced too tightly into the hole either. Withdrawing and inserting the drill a few times will help clean out the hole and facilitate insertion of the bolt. Soaping the bolt or pin will also help. If possible, timbers should be kiln-dried or radio frequency (RF) cured to permit full-strength bolted or pinned connections where two rows of connectors are required to secure a pair of steel side plates.

When the timber members season and shrink, timber connections loosen and steel tension ties can become slack, resulting in undesirable deflections. It is crucial that the nuts on the tension rods be accessible and periodically tightened until the timbers have fully seasoned.

Long-term deflections under sustained dead loads can produce unacceptable deflections and cause “ponding” conditions in roof trusses. For this reason, cambering the truss should be considered. Natural cambering of top and bottom chords of glulam members is accomplished during the initial layup of the members. In solid timber trusses, center-splice connections can accomplish the same thing; it is virtually impossible to bend top and bottom timber chords. Setting the camber in a solid timber truss is as much an art as it is a science, and is normally done in spans over 35 ft., when it becomes impractical to obtain longer members. In the latter case, center-point splices should be considered. A target camber of approximately one and a half times the total target deflection normally provides compensation for truss sagging due to connection distortion and long-term deflections.

Stress Reversal In high-wind regions, it is important to check possible truss uplift and stress reversal as early as possible in the analysis phase of design. Avoid getting overly committed to a given design concept until this condition has been evaluated. The inducement of stress reversal may mandate that panels contain counterbraces, that all truss members be timber or steel tubes to provide both tension and compression resistance, and that connections accommodate both loading conditions. Stress reversals in the bottom chord may also present the need for bridging or bracing against out-of-plane buckling.

Bracing and Bridging Long-span trusses, say those requiring splices, or over about 40 ft., should be evaluated for lateral stability. Building sidewalls subjected to out-of-plane wind loads that require lateral support from the crossing trusses may also induce the need for permanent bridging and bracing to prevent out-of-plane flexural failure of the bottom chord. Parallel-chord trusses lend themselves to being assembled in pairs on the ground with installation of permanent bridging and bracing. These units can then be safely raised onto the structure. This system avoids the necessity of temporary bracing and improves the safety aspects of the installation process. (See detailed comments later, in chapter 8.)

Depth-to-Span Ratios There are no hard-and-fast rules regarding parallel-chord truss depth-to-span ratios. One to ten (1:10) is usually a good starting point. Lower ratios (1:8, 1:6) will reduce the member and connection forces, but may increase the number of panels and the vertical web-member lengths. Also, higher compression-member slenderness ratios should be evaluated for possible increases in size due to the potential for buckling. Higher ratios (1:12, 1:14, etc.) will increase the member and connection forces, but may decrease the web member lengths. Architectural considerations will be important as well. Costs impacts associated with these variations are difficult to generalize, so they must be evaluated on a case-by-case basis.

References

Lewandoski, J. 2003. “Historic American Trusses I: Scissor Trusses.” *Timber Framing* 69, 4-14. Alstead, New Hampshire: Timber Framers Guild.

Pierce, P. C., R. L. Brungraber, A. Lichtenstein, and S. Sabol. 2005. *Covered Bridge Manual.* USDOT, Publication FHWA-HRT-04-098. April.

Chapter 6 – Variations from Common Truss Forms

Introduction

Earlier chapters have identified a truss, more specifically an ideal truss, as a frame so constructed and connected that members incur *only* axial forces. From a structural analysis standpoint, the only way to achieve this ideal condition is through a triangulated frame structure with all member forces acting concentrically through frictionless pin (or hinge) connections. The truss members are assumed to be weightless, and member-end connections are assumed to be free to rotate as the truss responds to loads applied exclusively at the joints. Most of the common truss forms presented in the previous chapter easily lend themselves to this type of analysis, and the resulting member forces serve as a great starting point for the design of the truss. Preliminary member sizes can usually be determined at this stage, but this ideal analysis is just that: a starting point. It is vital that the truss designer recognizes common conditions that often result in significant changes in member forces and truss behavior. These conditions are often created through fabrication logistics, but more often they are the result of the desired aesthetic. Modern timber trusses are typically left exposed, so the designer should be prepared for appearance to trump efficiency. The final product should be an agreeable balance between aesthetics and engineering. This chapter will address some of the most common conditions that can significantly complicate the final design.

Member Continuity

During an examination of the analysis diagrams in the previous chapter, the first thing to note is that each member stops and starts at each connection point or *node*. This is rarely the case in truss design, as top and bottom chord members are often continuous across multiple nodes or even over their entire length. Continuous members will incur bending and shear stresses at these joints, while other connecting members may carry only tension or compression. When a mortise-and-tenon connection is utilized, the mortise pocket reduces the available cross section of the continuous member at a location where the combined axial, bending, and shear stresses are their greatest. This condition is often resolved by increasing the timber size or modifying the connection.

Modifying timber sizes can be the simplest solution; however, the designer should remain sensitive to how a larger member relates to the rest of the structure. Increasing member size has a direct effect on every joint for which the member is a part. This is especially true if the member width is increased. For example, increasing the width of a truss top chord can directly affect joinery detailing at the heel and the peak, often leading to increasing the width of those adjacent members as well. Such adjustment may be needed to avoid offset surfaces that may affect aesthetics, complicate joint layout and production, and create issues for application of finishes. Increasing a truss member’s depth is usually more easily accommodated, but, again, attention should be given to how the increase will affect adjacent connections.

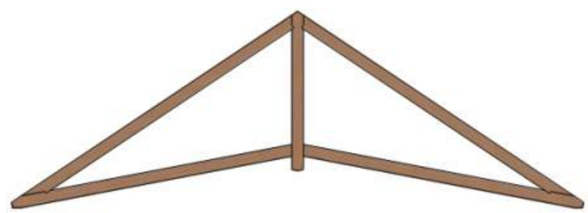
Modifying connections is typically a function of trying to remove less material at the net section of an overstressed member. The designer may choose to eliminate a traditional pegged tenon, thereby reducing the amount of material removed for a mortise or housing. This often results in the need for mechanical hardware (threaded fasteners, steel plates, etc.) to compensate for the eliminated tenon.

Finite Joint Size

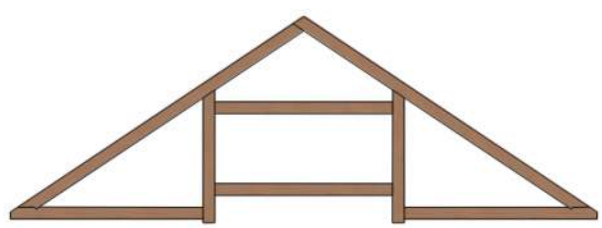
The second item to note in any ideal-truss analysis diagram is that each member is represented as a simple line connecting to a node. This works great on paper, but when the design results in three large timbers all requiring connection through one common point, accommodations are typically required. The engineer may start by aligning the centerlines of all connecting members through a common working point, only to find that the resulting joint is impossible, impractical, or ineffective. Often, it is all three. When members are moved to accommodate joinery, the member forces, assumed to be acting along the member centroid, no longer pass through the working point, the theoretical node. The resulting load eccentricity and the potential bending or shear stresses induced on adjacent members must be considered in the joint design. Again, these conditions can result in increased timber sizes or alternate connection choices.

Combined or Mixed Truss Forms

The truss forms presented in chapter 5 reflect some of the most common forms encountered throughout history. They are “common” because, for the most part, they are simple and effective, at least in their original form. For many clients, “common" is just too plain or simple, so truss designs begin to evolve into variations or combinations of these common forms. A kingpost truss might have the bottom chords sloped, joining to the bottom end of an extended kingpost (Fig. 6.1a). A queenpost truss might have the center segment of the bottom chord raised higher than the outer segments (Fig. 6.1b). Even these two forms might be combined to have the outer segments of the bottom chord slope upward to the queenposts, with a horizontal chord in the center (Fig. 6.1c).



6.1a Kingpost truss with sloped bottom chords.



6.1b Queenpost truss with broken bottom chord.

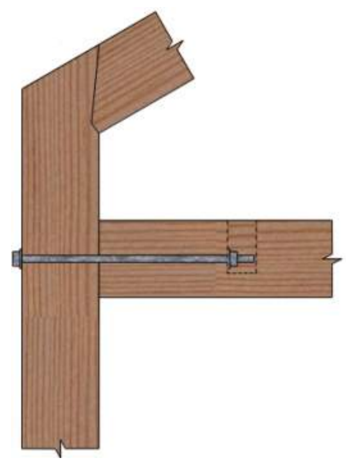
Regardless of the variations, the designer will have to consider the practicality and constructability of the desired form. Almost any configuration can be analyzed, whether by hand calculations or through the use of sophisticated engineering software; the question lies in whether it can (or should) be constructed, and at what cost. When truss forms are mixed or modified, what is typically a preferred compression joint (with wood-on-wood bearing) can quickly turn into a more complex tension joint. Bending and shear stresses can also be introduced that lead to significant deviations from common joinery.

6.1c Queenpost truss with broken and sloped bottom chord.

Lower Chord Position

Simply put, the lower chord of a truss is typically in tension while the top chord is in compression. Following the standard rules for heel joints, it is most common to engage sloped top chords into the top surface of the bottom chord to transfer loads through a compression joint. This connection transfers the compressive force of the top chord, into the bottom chord through bearing, resulting in tension in the bottom chord as it resists outward thrust. Variations in truss or frame layout can easily inhibit this most desirable configuration.

Bottom Chord Dropped In many cases, the vertical posts intended to support a truss at the heel end up integrated into the truss. This design variation typically calls for the bottom chord to be dropped below the truss's heel node, tying into the interior face of the post. The top chords then engage the top of the post, and the segment of post between them acts as a vertical member at the truss heel (Fig. 6.2). When this occurs, the "truss" has technically become a frame, but the truss-like configuration is still relied upon to transfer applied loads to the posts. The result is that the top chords push out on the top of the post, and the bottom chord must prevent the posts from spreading apart. The ends of the bottom chord must now be attached to the post using tension joinery. A conventional mortise-and-tenon joint will occasionally (but rarely) work; however, a wedged-dovetail through tenon can often solve the problem. In areas with higher roof loads, shallow roof pitches, and longer spans, hardware must often be introduced to transfer these tension loads. The most common hardware solution is to embed a threaded steel tension rod in the joint. The rod can extend completely through the post, with a plate washer on the backside. In the tension chord, another bearing plate is installed in a small mortise (a washer pocket) in the top of the member. This connection often provides enough



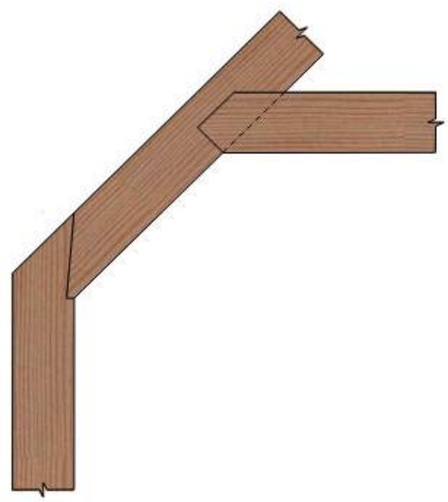
6.2 Truss heel joint with dropped bottom chord.

flexibility to locate the rod to act through the centroid of the member or slightly above the centroid with modest eccentricity.

Another important issue created by this configuration is the bending stress introduced into the post. Considering that the post is already carrying a significant axial load, the combined axial and bending stresses can quickly lead to a post failure. To compensate, the post size is typically increased, particularly in the direction of the applied bending stress.

Finally, the high compressive force in the top chord induces shear stress in the segment of the post between its connections to the top and bottom chords. Joinery requirements that reduce the post's cross section may cause stress concentrations that subsequently can challenge the post's ability to resist shear stress, possibly resulting in a brittle failure mode.

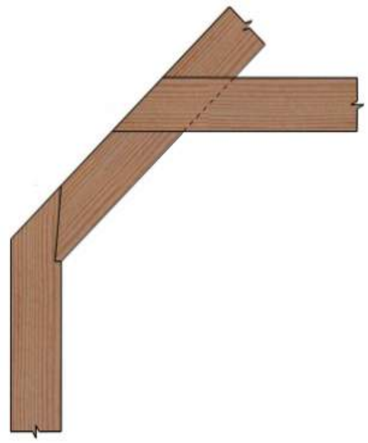
Bottom Chord Raised The opposite of the dropped lower chord is a variant where the chord is raised above the common heel joint (Fig. 6.3). In this case, the bottom chord is generally regarded as a collar tie, and the overall truss configuration is sometimes referred to as a *top-chord* or *tail-bearing* truss. As previously mentioned, the most common truss configurations allow for lower-chord tension to be developed through top-chord compression joinery at the heel. If the lower chord is raised, it is forced to engage the bottom face of the top chord.



6.3 Truss heel with raised bottom chord.

In short-span trusses supporting steeply pitched and lightly loaded roofs, this connection can sometimes be achieved with a traditional mortise-and-tenon joint. For this condition, edge distance and spacing are critical, and pegs are typically staggered to avoid having all the pegs acting along a common grain line in the top chord.

As spans increase, and for low-pitched or heavily loaded roofs, the tension force in a raised lower chord can quickly exceed the capacity of a traditional mortise-and-tenon joint. Other traditional joinery alternatives can include a wedged-dovetail through tenon (Fig. 6.4a), half lap (Fig. 6.4b), dovetailed half lap (Fig. 6.4c), or a dovetailed free tenon. Each of these options will cause a significant reduction in top-chord cross section. Through-tenon options may easily

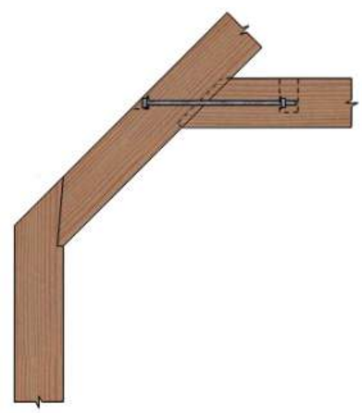


6.4a Wedged-dovetail through tenon.

6.4b Half lap.

6.4c Dovetailed half lap.

accommodate an increased top-chord breadth; however, in half-lapped joints, it is best practice to keep the shear interface centered in the truss breadth, so both top and bottom chords should be the same breadth.



6.5 Tie rod with plate washers.

If these options cannot be configured to adequately resist the design forces, the next step will likely include adding hardware. The free or centered through-tenon options can be replaced with a threaded steel tension rod through this joint with plate washers at each end. The plate washers then bear against the square face of a narrow mortise pocket in the top of both the top chord and lower chord (Fig. 6.5). A small locator tenon maintains alignment of the members. The threaded rod option removes the least amount of material from each member and allows for relatively accurate placement acting through the centroid of each member. This option also permits future tightening of the connection.

Another hardware option involves use of a kerf (knife) plate slotted into the rafter and tie. Through bolts or dowel rods secure the plate to both members. If the members are wide enough, the fasteners can be recessed and the counterbores plugged to hide the hardware. If more than a single line of fasteners is needed, the potential constraint of member shrinkage must be considered.

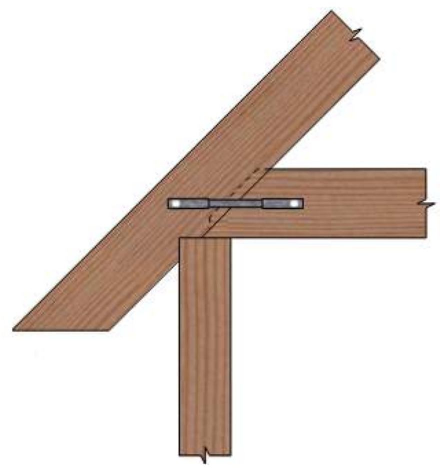
Half-lapped joints typically already incorporate through bolts, but this joint can be further strengthened by adding split-ring shear connectors or shear plates to the through bolts. Careful detailing is required to ensure proper engagement of these items, and the designer must consider the amount of material that is removed to incorporate them. An alternative to the half lap is to utilize two lower-chord members with each member lapping the outer faces of the top chord.

This configuration benefits from the through bolts acting in double shear, but it significantly changes the aesthetic of the finished product.

As the lower chord is shifted higher and higher up along the top chords, further from the top chord support points, greater and greater bending moment in the top chords develop. For a simple three-member truss (two pitched top chords and one tension chord under uniformly distributed roof load and supported by a hinge and roller), the maximum moment in the rafter occurs at its joint with the lower chord when the lower chord attaches at or above $^{3}/_{10}$ of the rafter length. At this point, the moment in the top chord is twice as large as its base value, that with the lower chord at the heel. When the lower chord is halfway up the rafter, the bending moment is three times its base value. The combination of this rapid increase in bending demand with the reduction in top-chord cross section due to joinery can seriously compromise the structural integrity of the truss. By contrast, a truss spanning between fixed abutments (hinged supports at the ends of the top chord) benefits from the collar tie being raised to the middle of the top chords. For additional details on this behavior, see Schmidt and Miller (2018).

Rafter Tails at Heel Connections

The previous section focused on the effects of raising the bottom chord above the truss-heel support, leaving the truss supported only by the top-chord member. Similar joinery conditions appear when the designer configures the truss such that the top chords extend beyond the truss heel to form rafter tails, while the bottom chord remains supported by the post or plate (Fig. 6.6). This condition is typically found in a common rafter frame, where a truss is introduced and the top chord is in the roof plane with intermediate common rafters. The designer's intent is to have evenly spaced exposed timber rafter tails. While this creates an appealing aesthetic, it again eliminates the ability to transfer top-chord thrust through compression joinery. Joinery options are similar to those presented in the previous section; however, the designer must now also consider the connection of the truss to the top of the supporting post or plate.

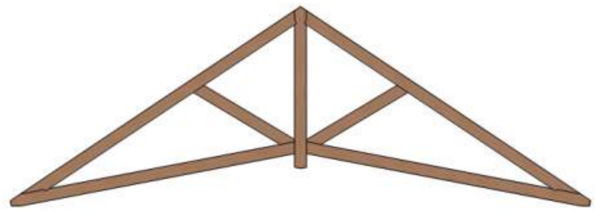


6.6 Heel with rafter tail.

Sloped Bottom Chords

Sloped bottom chords are most commonly found in a scissor truss, previously presented as a common truss form (Fig. 6.7). In a true scissor truss, the heel joint is relatively similar to that for a kingpost or queenpost truss. Further, the crux joint benefits from the continuity of the chords as they cross (with a lap joint) and extend to the bottom faces of the opposing top chords.

6.7 Scissor truss.



6.8 Scissor truss with broken bottom chords.

Variations from this form result when the sloped bottom chords are designed to stop (or change direction), typically at an intersection with a vertical member (Fig. 6.8.). With the kingpost and the bottom chords being in tension, the material discontinuity will need to be reestablished through tension joinery.

On very rare occasions, offset lapping tenons in a through mortise can resolve the tension loads, but most often, the loads far exceed the pegged or keyed tenon capacity. Geometry may allow for a hardwood through-spline connection, which can be configured to include additional dowel-type fasteners (pegs, steel pins, or bolts). The additional dowels, spline strength, and loading orientation provide increased joint strength. More often than not, this condition is resolved using previously discussed hardware applications. Steel knife plates (embedded) or side plates (surface-mounted) can be configured with through bolting. In lieu of plating the connection, the designer may also elect to use threaded steel tension rods through the joint with plate washers at each end. The plate washers bear against the square face of a mortise pocket in the top of the discontinuous chord members. In extreme conditions, the steel rod (typically slotted into the top of the chord) can extend continuously over the full length of the member, passing through the top chord with a bearing plate. The timber then serves only to conceal the rod and maintain the geometry and appearance of the truss. As previously noted, threaded rod options remove the least amount of material from each member, allow for relatively accurate placement acting through the centroid of each member, and permit future tightening of the connections.

Curved Members

When building structural frames and trusses, straight milled timbers will always provide the most efficient use of the material. While naturally appealing in this form, the client or architect will often seek to soften the appearance of these exposed structures by adding curved members. A favorite explanation was once provided by a feng shui master, who insisted that the masculinity of the heavy timber structure required feminine balance, achieved by introducing curves.

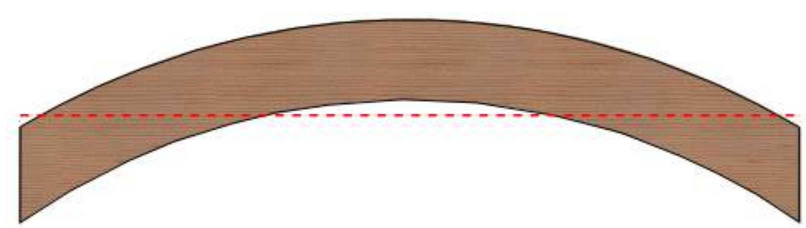
Curved members are often created using glued-laminated (glulam) timbers formed to the desired radius, but they can also be cut from a solid milled timber or found in naturally curved form. The resulting structural characteristics of each approach are unique and need to be addressed accordingly.

Solid Milled Timbers When a relatively straight log is milled into a straight timber, grading requirements confirm that the characteristics of the wood along the length of that member are adequate to meet the published design values. When that straight timber is cut into a curve, some important guidelines must be followed to maintain structural integrity. Following these guidelines will result in the most stable curved member conditions while producing the least amount of wasted material.

The greatest consideration for curved structural timbers is continuity of grain. Ideally, the minimum required rectilinear timber cross section will still exist within the final curved member (Fig. 6.9). The remaining wood, outside of this cross section, simply creates the desired shape. For tension-only members, just the continuous straight cross section should be considered for calculations. For curved members subjected to high compression loads, the curve influences the stability of the member, increasing the likelihood of buckling due to beam-column behavior. For members subject to bending, the useful cross section should be measured from the bottom of the internal straight member to the top of the arch. This cross section will be greatest at the center, but if the maximum bending stress occurs elsewhere along the member, the reduced cross section at that location should be used.

6.9 Rectilinear section in a curved member.

If a full-length rectilinear timber cannot be visualized within the curved member, then that member must absolutely not be used in a structural capacity. Such a member is so severely curved that a horizontal line can be drawn, passing through both the top and bottom faces of the member (Fig. 6.10). The slope of grain (grain runout) induced by cutting such a timber will result in high shear stress and tension perpendicular to grain, leading to failure under even moderate loads. Alternatives to cutting such a severe curve are presented below.



6.10 Curved member with severe grain runout.

At member ends, it is acceptable to use the full-depth cross section to transfer shear loads, but it is important that the entry and exit cuts for the curve do not create short-grain runout corners at the ends of the member, such as that in Figure 6.11a.

6.11a Short-grain runout at bearing.

This surface is typically a crucial bearing surface, most commonly occurring where curved bottom chords are raised (housing into the bottom face of a top chord) or where curved braces are housed into a post or beam. The short-grain runout condition can often be alleviated by using milder curves, moving the entry or exit point of the curve cut a few inches away from the intended bearing surface, or both (Fig. 6.11b).

6.11b Curve entry point shifted from bearing.

Where deep curves or long curved members are desired, the design should incorporate interrupted timbers to break the curve into segments. The best example of this is a kingpost or Howe truss with an arched bottom chord. The kingpost should be allowed to pass through, with the curved segments coming into each side (Fig. 6.12). Each half of the bottom chord can then be cut from a shorter timber and grain continuity is more easily obtained. These members can then be evaluated as straight, pitched members, and proper connections can be employed through the kingpost.

6.12 Curved bottom-chord segments joined at kingpost.

Another common curved member condition in trusses is a full-bay arch extending between two vertical posts (i.e., queenposts) while also connecting to the bottom of a horizontal tie beam. In this case, a pair of curved segments can be joined with scarf-style joinery and the entire arch can then be joined to the horizontal and vertical members. Scarf joint selection and location should be configured such that once the arch is attached to the horizontal member, the scarf joints can be considered well braced. The outer segments of the arch will typically act as compression struts, therefore midspan hinge points should be avoided. Another option to segment this arch is to install a timber pendant centered in the bottom of the horizontal member. This central member mocks the keystone in a classic masonry arch and provides alternative joinery options for the curved members (Fig. 6.13).

6.13 Curved bottom chord broken with keystone pendant; scarf joints outlined for clarity.

Glued-Laminated Curved Timbers Glued-laminated curves are created by physically bending layers of wood into the desired radius or shape and then gluing them together under pressure. The finished products are available in a variety of surface finishes to closely match other, solid-sawn timbers in a project. Grain-matched curved glulams are produced using very thin laminations cut from a single timber. The grain lines of each lamination are carefully aligned to create a finished product that is nearly indistinguishable from a solid-sawn timber. In some cases, grain-matched glulams can even appear too perfect when contrasted by the natural knots and checks of the solid-sawn timbers in the structure.

Fortunately, the design of curved glulams is well supported by the NDS, which includes adjustments to straight glulam design values to account for the effects of curvature. These effects include the residual stresses in the laminations associated with the fabrication process, the nonlinear bending stress distribution through the cross section, and the radial stress developed due to bending.

Naturally Curved Members A timber cut from a tree that grows in the form of a gentle curve may be attractive for use as a structural member. In fact, such members are commonly used to form the principal structural elements in cruck frames. In trusses, the use of a naturally curved member avoids the problems associated with cutting the curve from a straight timber or the expense of forming a curved glulam. However, a naturally curved member is not free of other challenges. It still obeys the same fundamental mechanics principles as any curved member: nonlinear bending stress distribution and radial stress under bending.

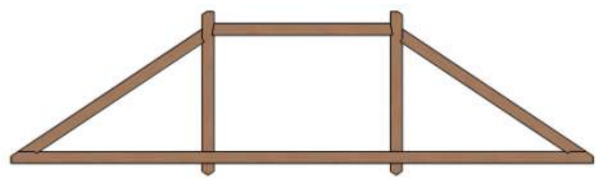
In addition, though, the wood in naturally curved members is abnormal compared to that in straight timber. Curved trees develop reaction wood as they grow. As new wood fiber is established, it is subjected to bending stress due to gravity and wind loads on the curved tree. The reaction wood in hardwoods is known as *tension wood*, as it develops on the tension side of the stem. The reaction wood in softwoods is called *compression wood*, since it forms on the compression side of the stem. Tension wood and compression wood are distinct in their cell structures but exhibit similar strength-reducing characteristics. Specifically, both forms of reaction wood experience higher longitudinal shrinkage—five to ten times higher—than normal wood fiber. Tension wood has reduced compression strength, and compression wood has reduced tension strength. Hence, both result in a reduction in bending strength of the member.

So, if a naturally curved member were to season in place and be confined by joinery at its ends, the abnormal shrinkage causing warping and the reduced fiber strength could result in failure of the member, even in the absence of imposed loads. Caution and good judgment must be exercised when selecting naturally curved members for use in high load-demand situations.

Discontinuities in Members and Connections

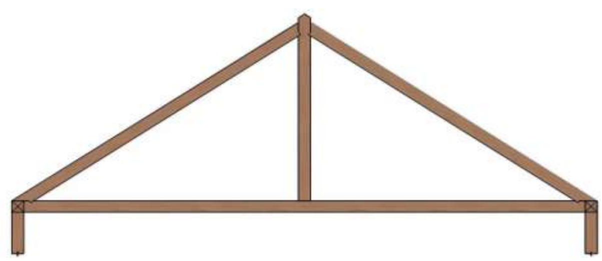
One of the more challenging aspects of truss design is dealing with discontinuities: breaks in a member along its length or nontruss members passing through connections. These discontinuities are often the results of design choices intended to express an architectural feature or misunderstandings of the importance of a systematic load path through a truss.

For instance, a designer might wish to emphasize pendants on the tops and bottoms of the kingposts and queenposts in a truss, thus running the posts through the top and bottom chord members (Fig. 6.14). Breaks in the bottom chord can be handled with threaded rods, knife plates, or other means, but not without additional cost of hardware and labor. Breaks in the top chord interrupt the flow of compressive forces, possibly causing crossgrain crushing of the posts at the connections. The more likely and serious outcome is a loss in axial stiffness of the top chord. As the posts season and shrink, along with possible crossgrain compression deformation, the top chords effectively shorten, causing the entire truss to sag.



6.14 Queenpost truss with interrupted top chords.

Another common design error is to rest continuous eave plates across support posts and attempt to join the top and bottom truss chords through that location (Fig. 6.15). Top-chord thrust, tending to roll the plate outward, will need to be carried back through the plate and into the lower chord, generally by some specialty hardware. Also, shrinkage and cross-grain compression of the plate lower the stiffness of the connection, leading to unintended sag of the truss. Use of offset tenons and steel straps can help facilitate the attachment, but neither of these are really suitable connection details for that arrangement. A more appropriate configuration might include placement of the plate atop the top chord or spanning the plate from post to post with just end-bearing on the post.



6.15 Kingpost truss with continuous eave plates interrupting heel joint.

Closing Considerations

Regardless of which variations from the standard truss forms or joint configurations are selected, it is important to consider these additional effects on truss behavior:

- Materials with a high moisture content will cause changes in member size (shrinkage) and shape (distortion) as they season. These effects may result in changes in connection bearing surfaces and overall truss stiffness.
- The trusses discussed here are two-dimensional configurations, but of course, they must act in three-dimensional structures. Out-of-plane stability must be maintained during both erection and in-service conditions.
- The finite sizes of the members joined at a connection generally result in some eccentricity caused by an inability to pass all member centerlines through a common working point. Hence, the effects of secondary shear forces and bending moments on member size must be considered, in addition to the transfer of forces from one member to another.

References

Schmidt, D., and J. Miller. 2018. "Behavior of Statically Determinate and Indeterminate Rafters." TFEC Technical Bulletin 12, Timber Frame Engineering Council, Alstead, New Hampshire: Timber Framers Guild.

Chapter 7 - Joinery and Connections

Introduction

Designing joinery and connections in timber trusses involves choosing among numerous options for the joints likely to be encountered. However, there are fundamental rules of joinery design that apply to the best of these possibilities. The authors have attempted to illustrate many of the suggested connections with photos and sketches. These are by no means intended to dictate any one particular design solution, but rather to suggest a range of possibilities that resolve the forces in a particular connection. For mechanical connections, an attempt has been made to use conventional, readily available, off-the-shelf hardware in these illustrations. However, there are numerous proprietary connectors that are readily available and should be employed in the arsenal of the well-equipped timber engineer.

Structures might best be described as "connections held to one another with members." It has often been said that a timber truss is really nothing more than a bunch of truss joints that happen to be separated by chords, struts, and ties. This focus on connection design is especially true for timber design, even more than for steel and concrete.

As discussed in chapter 3, wood is composed of cellulose in a bundle of straw-shaped cells, making this material unique in terms of joinery design considerations. Within timber structures, trusses may represent the highest reliance on connection design. Trusses can be innately efficient, so the few interconnections must inevitably transmit a lot of force, and, since the trusses are (typically) statically determinant, those connections must all perform if the truss is not to fail. Roughly half of the connections are transmitting tension—the most challenging of member forces to transfer in timber structures.

The trick and art to designing a timber truss is how you put the timbers together. The rest of it is easy. Timber trusses with practical connection details are usually cost-efficient and easy to fabricate and assemble, whereas poorly conceived connection details will often result in an overly costly structure plagued with difficulty.

Fundamental Rules of Joinery Design

When designing timber truss joinery, there are a few fundamental rules (as presented in DeStefano [2011]) that should be kept in mind:

Rule #1: The geometry of the joint should have mating surfaces that allow all structural loads to be transferred in bearing of one member against the other. Pegs are better used to hold a joint together (during assembly and installation) rather than to resist structural loads. Bolts or steel rods should be used to resist tension rather than shear at the joint. Let the geometry of the joint do the work, not the fasteners, whenever practical.

Rule #2: The wood removed to create the joint should not unduly weaken any of the timbers being joined. The timber section of all members connected at a joint must be reduced in some fashion to create the joint. The challenge is to strategically remove wood only from the portion of each member that is not highly stressed.

Rule #3: The geometry of the joint should not be altered by shrinkage of the wood, and bearing surfaces should remain in tight contact after seasoning. This is the rule that is most often forgotten and probably the toughest to follow.

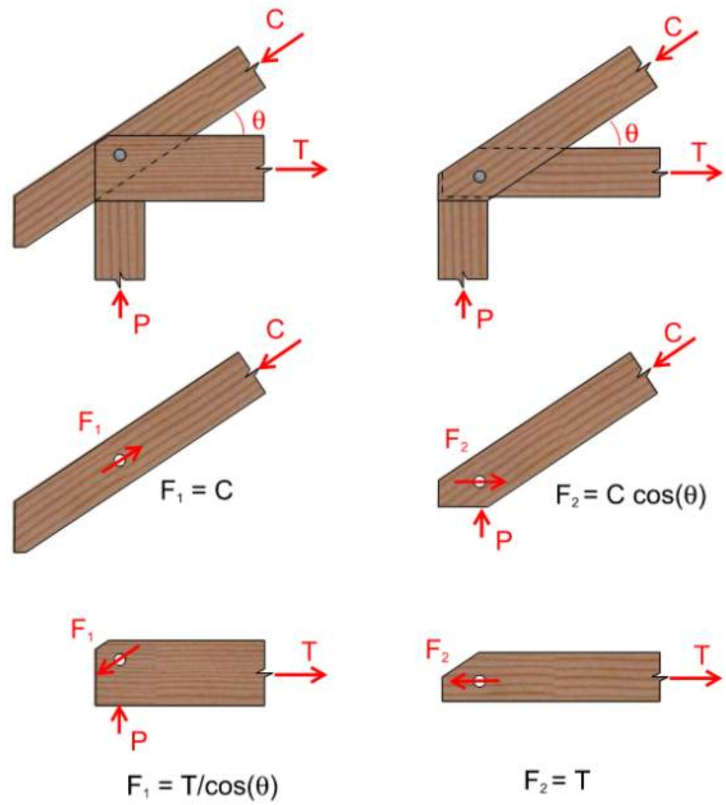
Rule #4: Anticipate all potential modes of failure and provide sufficient strength to resist each potential failure mode. This is a rule that naturally applies to any structure, not just to timber truss joinery. The challenge here is that you must think of everything. Failure to anticipate a potential failure mode can have dire consequences.

The engineering of timber truss joinery is not a cookbook process of following overly prescriptive codes and standards. It requires no small measure of ingenuity, creative energy, and experience. These rules are not limited to truss connections, but actually apply to all types of timber joinery.

Free-Body Diagrams vs. Finite Element Analysis Software

Many trusses can be analyzed using a powerful, basic analytical tool known as the *free-body diagram.* Refer to *Wood Technology in the Design of Structures* (Hoyle and Woeste, 1989) for additional discussion on this topic. An assembly of pieces can be envisioned as "disassembled" into components, and the forces applied to and among those "bodies" can be evaluated in terms of the body's equilibrium. Any free body that is sitting still does so only because all the applied forces cancel each other out against moving the body and are arranged so as not to spin the body about any point of our choosing. In addition to free-body diagrams of joints, judiciously selected sections through the truss may be analyzed in the same fashion. Free-body diagrams are also invaluable in seeing how steel side plates, or gussets, may be inducing cross-grain splitting.

As an example of the value of free-body diagrams, consider the heel joints illustrated in Figure 7.1. In Figure 7.1a, the lower chord rests on the post, whereas the upper chord rests on the post in Figure 7.1b. In both cases lower-chord tension is T, upper-chord compression is C, and post reaction is P. For simplicity, a single fastener joins the two chords together. In case a, fastener force is clearly $F_1 = C$, but in case b, fastener force is $F_2 = T$. If the distinction between the two support cases is not recognized, or demonstrated via free-body diagrams, the resulting fastener force could be erroneously determined.



a. Bearing on lower chord. b. Bearing on upper chord.

7.1 Free-body diagrams reveal how forces within a connection vary with detailing of heel joint.

Trusses that are more complex may be easily modeled and analyzed using current finite element analysis software. (See Appendix B for some of the more common software versions.) Nevertheless, once (and however) the design forces and stresses on the members and joints are established, detailed joinery design still involves using free-body diagrams and analysis to properly detail the connection to resist the applied forces and stresses.

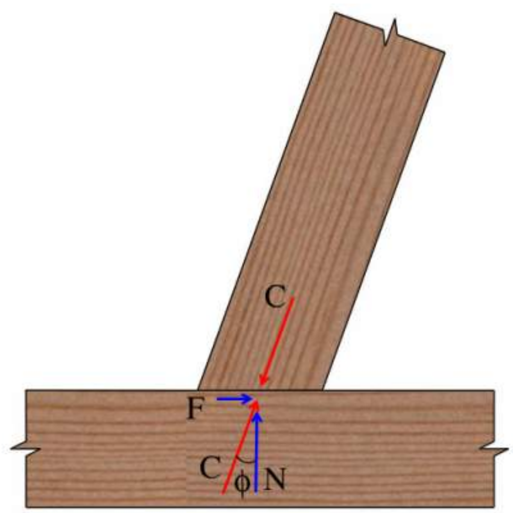
Friction in Joinery

Any bearing face that is not perpendicular to the axis of the force being transmitted through bearing is inevitably relying on friction—a historically unreliable phenomenon to maintain connection stability—if no positive mechanical interlock is provided to resist slip. Brungraber (1976) has noted that "Although, starting with Leonardo da Vinci, some very famous scientists have studied friction over the centuries, it still remains one of the most familiar and yet least understood facets of mechanics." Reliance on such an unpredictable phenomenon for structural stability should be avoided. However, some traditional joinery details do indeed rely on some degree of friction, and some modern practitioners seem dedicated to those details, so a discussion of friction's role in connection design is warranted.

The generally accepted theory of *dry friction* is attributed to Charles-Augustin Coulomb, circa 1785. About the best information we have on the nature of Coulomb friction between pieces of wood is from the researchers E. H. Messiter and R. C. Hanson, in their 1894 work on the topic (Jacoby, 1909). They found coefficients of friction that ranged from 0.215 for sanded pieces of wood to 0.365 in rough-sawn timbers. They found roughly the same values for sanded pieces sliding face-to-face along the side grain and ones that were sliding end-grain-to-end-grain. Among researchers conducting valid testing, it is rare to find a coefficient of friction much above 0.4 between timbers.

Relying on friction in a structure seems unwise, but it is accepted practice in modern high-strength, steel-bolted connections. The highly tensioned bolts induce such reliable clamping that they do not bear on the hole edges; rather, the friction between the connected pieces transfers the loads. If the bearing surfaces do slip, the bolts can still work in shear when they become engaged as the surfaces slide under the applied load. Similarly, many sloped bearing faces in timber joinery also incorporate tenons in mortises (or clamping bolts that can work in shear, if the bearing fails) that could "catch" the sliding members. The amount of slip needed for the tenon to bear on its end, as well as the possible rolling shear (shear stress acting in the radial-tangential plane) induced in the tenoned member, make this a different connection scheme to check. (In such cases, it may be better to use an angled clamping bolt as discussed in "Mechanical Connections" below.)

What is this measure of stickiness known as the *coefficient of friction*? This discussion will deal solely with the static coefficient of friction; however, it is noteworthy that scientists now understand that dynamic friction coefficients are a function of the speed of sliding—friction is not a simple dichotomy of static and moving phenomena—and that dynamic friction coefficients can be much lower than static friction at low speeds.



7.2 Normal force N and friction force F in wood-on-wood bearing.

Consider a compression strut bearing with axial force C on a plate, where sliding of the strut on the plate is impending (Fig. 7.2). The components of force C are the normal force N and the friction force F. At its simplest, the coefficient of static friction μ is the ratio of the two forces, F and N. F is the largest force parallel to the bearing surface capable of resisting sliding, while N is the normal (perpendicular) force pressing the two surfaces together.

One aspect of this ratio is that it measures ratios of forces, not pressures. This means, among other things, that the available force to resist any tendency to slide is *not* a function of the bearing area between the two bodies. According to Coulomb friction theory, increasing or decreasing the area of the bearing surface between two timbers does not change the friction resistance to sliding. This means that the friction force remains the same even if imperfections in the mating surfaces develop, either as a result of drying shrinkage or minor changes in the angle $\emptyset$ while in service.

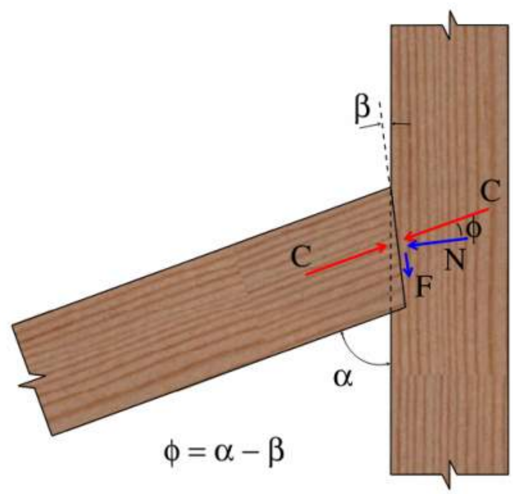
The coefficient of friction can also be viewed geometrically as the angle ϕ, the angle between normal force N and axial force C, at which an axially loaded member will begin to slide along the surface of the member it bears on. A higher coefficient of friction will permit a larger angle between the line of force C and a line perpendicular to the bearing surface.

If the coefficient of friction varies from 0.25 to 0.40 for wood-on-wood bearing, then the maximum friction force lies between 0.25N and 0.40N. Prudence demands that a factor of safety be applied to the lower limit. So, if a factor of safety of 2.0 is applied for design purposes, the maximum friction force must be taken as F = 0.125N. The corresponding angle is

$$\emptyset = \tan^{-1}(F/N) = 7^{\circ}$$

Hence, the strut in Figure 7.2 should be nearly perpendicular to the plate. The strut must make an angle of 83° or more with the face of the plate in order for friction to resist sliding as a design approach. In most cases, it is impractical, if not simply reckless, to rely on friction alone to resist connection forces.

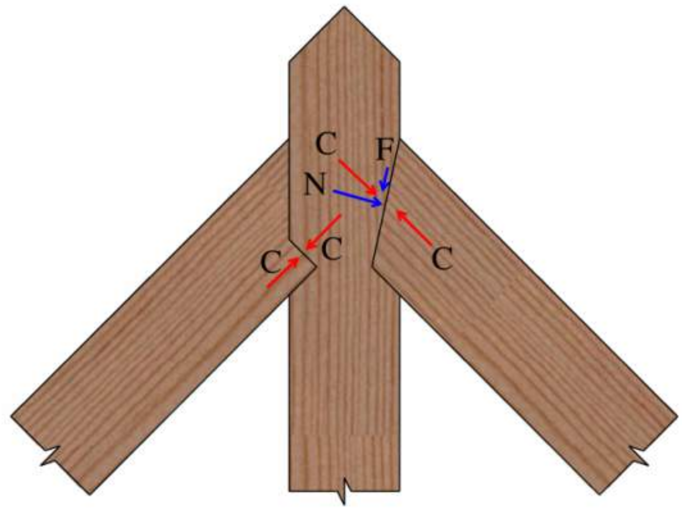
The bare angle joint in Figure 7.2 is clearly ill advised. A bearing surface perpendicular to the axis of the strut force is reasonable. So, any beveled bearing face, such as that in Figure 7.3, is, at its root, a move from reckless toward reasonable: a situation much like positioning collar ties (smart: minimum tension for collars placed down at the eave; reckless: infinite tension for collars placed at the peak).



7.3 Connection with sloped bearing surface.

For comparison purposes, consider the opposing connection details in Figure 7.4. On the right side of the kingpost, force transfer at the bearing surface is more complex compared to a bearing surface that is perpendicular to the line of force. This detail requires the friction component F, working with the bearing component N, to resist the chord thrust. On the left side, force transfer is simpler and more secure. The chord-force application is only slightly eccentric from the member centroid, and the notch could be shifted up to eliminate the eccentricity, if need be. Reliance on friction for load transfer is avoided. In either case, the chords must be secured to the kingpost with fasteners of some description to facilitate raising and to handle any bearing-surface changes due to shrinkage or shifting loads.

So, the question becomes: How close to reckless do you care to get? This is the decision being made by designers who slope the bearing table but stop well short of getting to a notch with a bearing face perpendicular to the compressed timber axis. To sum up the friction physics and factors of safety decisions: Designers who use bearing faces more than 10° from perpendicular to the compressed timber axis do so at their own peril. Designers are well advised to seek some form of mechanical interlock for any joint in which the bearing face is not perpendicular to the line of the force being transferred.



7.4 Notched vs. sloped bearing options.

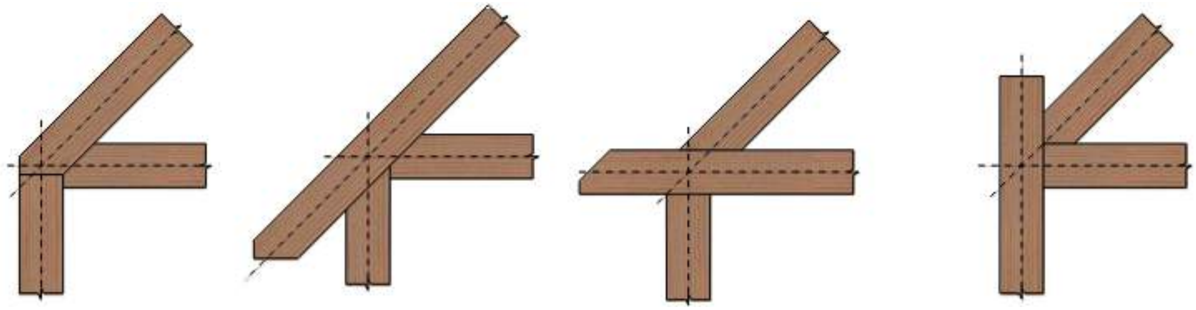
Wood Joinery - Traditional Methods

Notches Force transfer between timbers has involved notches since ancient times. Notches are configured primarily to transmit axial forces, although notching is also a component in joints intended to transmit shear and even bending forces. Notches offer perhaps the most vivid illustrations of the art of compromise in joinery design: how to accommodate, minimize, and balance the innate damage done to the members in cutting the notched bearing faces.

There are several issues to consider when designing and detailing notches. Notch location, bearing-face angle, allowable stresses on the bearing faces, double notching, clamping, and reinforcing are just a few concerns that will be covered herein.

Notch Location Many times, a designer will only realize after some effort that there simply is not enough available material in the notched members to withstand the induced damage and generate the design capacity. The designer may, in some instances, simply upsize timbers that were presumably satisfactory otherwise. Another approach, though, is a considered, modest adjustment in how the timbers meet that will make significant differences in the available joinery options. In order to provide adequate, solid areas of timber in which to cut joinery it may be possible to extend or shift one of the timbers. For instance, consider a connection including a post, bottom chord, and top chord. Options for adjustment are:

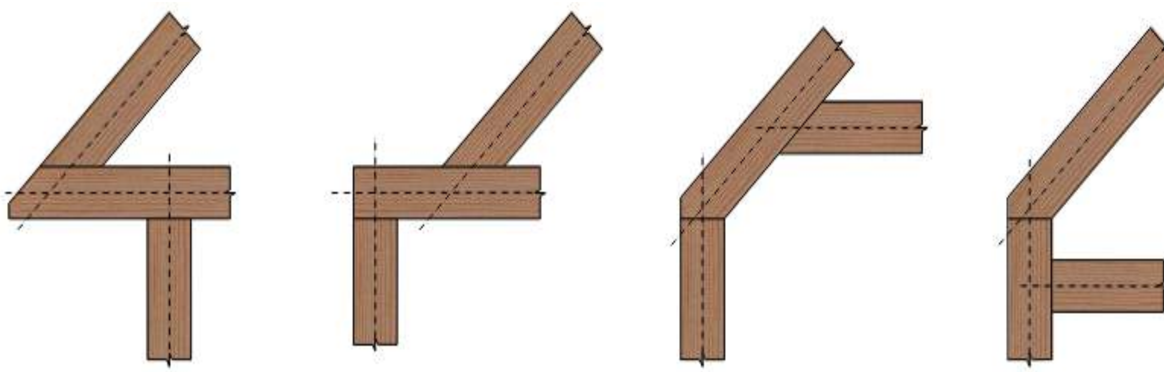
1. Extend one of the three—post, bottom chord, top chord—to have a zone of solid timber in one of them, into which the other two are joined (see Fig. 7.5).



a. Base condition. **b. Extended top chord.** **c. Extended bottom chord.** **d. Extended post.**

7.5 Extending a member to make room for joinery.

2. Shift any one of the three sideways, one way or the other. These shifts will change the single three-way joint aligned between three separate members, into two two-way joints between the three (see Fig. 7.6). The downside to this shifting is that the connections now are eccentric: the three forces no longer pass through a common working point. This, in turn, introduces bending and shear in the members at locations where notching may be weakening the members. This needs to be checked.



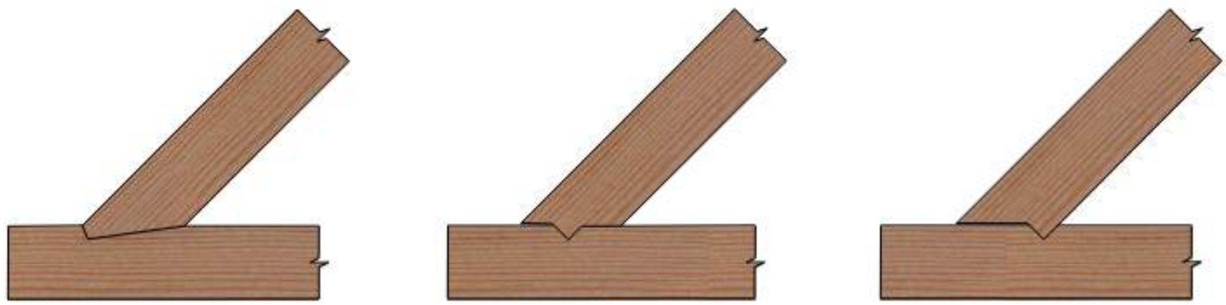
a. Post in. **b. Rafter in.** **c. Bottom chord up.** **d. Bottom chord down.**

7.6 Shifting members to provide adequate solid timber for joinery.

3. Combine a shift with an extension.

A prime location for using notches in timber trusses is at the intersections between top and bottom chords. Typically, the compressed top chord is trying to spread the walls and is being prevented from doing this by the tension induced in the bottom chord. If the top chord might be anthropomorphized as an engineer backing up to the compression in the top chord, digging in his or her feet to prevent being shoved off the end of the bottom chord, the choices for locating the notch—along the contact surface of the two chords—varies from the outer toe to the inner heel, or anywhere along the sole between those two extremes.

The outermost option for notching the bearing faces between top and bottom chord is to place the notch at the *toe* of the top chord. This option has one big advantage: minimizing the possibility of bending failure in the bottom chord due to stress concentrations at the notch. If there is enough end-grain relish in the bottom chord to resist the shear stresses outboard of the notch, this is the place to notch it. (See Fig. 7.7a.)



a. Toe. **b. Sole.** **c. Heel.**

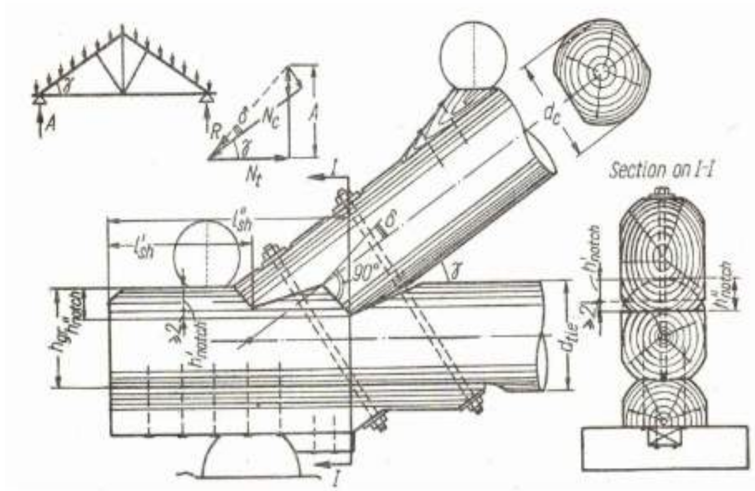
7.7 Options for locating notch at top-chord-to-bottom-chord joint.

If, on the other hand, the available relish past the notch in the bottom chord is in short supply, moving the notch inward, toward the *heel* of the compression chord, can increase the capacity. The inverse of the toe joint still holds, though the notch in the bottom chord can induce bending stresses that require additional reinforcement to resist, such as a corbel or gunstocked post. (See Figs. 7.7c and 7.8.)

7.8 Corbelled post to extend support for heel-notched bottom chord.

As usual, when discussing the limits of options like notching at the toe or the heel, there are options to be explored between the two extremes. In this instance, the compromise joint location arises along the *sole* of the available interface between the two chords. The cut on the top chord is a bit more complex, but the force is applied more centrally to that compression member, helping to minimize any secondary bending influence around the joint. (See Fig. 7.7b.)

Double Notching The double-notch joint (Fig. 7.9) is a complicated form that generates a larger bearing area to deal with large compression loads without unduly reducing net section in the notched member. The geometric challenges and required excruciating tolerance make this joint a near act of design desperation. In order not to introduce a fragilely short shear plane beyond the notches, the inner notch should be deeper than the outer notch by at least ¾ in. The designer must not add the two shear capacities beyond the two notches when calculating shear resistance of the bottom-chord relish. Horizontal shear resistance is limited solely to that beyond the deeper inner notch, since all the upper notch shear must pass through that lower shear plane as well. This double notching does not much improve the resistance to shearing off the end, but it does mitigate the damage done the notched member by a single deep notch. The need for the two bearing surfaces to both be in contact at the same time to avoid sequential failure demands the highest precision in crafting all involved members. For good reason, double notches are only rarely used and only in well-seasoned stock. For more thorough discussion of geometric specifics and performance disclaimers, see Karlsen (1967), pp. 126–28.



Karlsen (1967)

7.9 Double-notch joint with reinforcing bolts.

Clamping and Reinforcing Reinforcing the notch with bolts can help to pull the connection tight during assembly. The bolts help during raising to hold the joint together, if the truss is lifted with a commonly used centered strap. If a shear failure does occur in the connection shear plane, the bolts (and washers) can serve a life-safety role: the truss has "failed," and may sag as that joint relaxes, but the structure may not collapse.

Reinforcing the shear plane (relish) beyond the notch, even using modern fully threaded screws is not a straightforward proposition—much as the steel designer avoids mixing bolts and welds to share load in a connection. The designer should not simply add the available shear capacity in the relish to the resistance of the added screws. The two load-resisting mechanisms act in parallel and carry load proportional to their relative stiffnesses, and the two failure modes occur at different magnitudes of strain. The shear plane in the relish deforms only imperceptibly before sudden, brittle, and complete shear failure occurs. Only then is there enough potential sliding along that failure plane to meaningfully engage the screw threads. In short, all the shear is in the wood until it fails; thereafter, all the load is in the screws. A proper design provides screws to resist the entire sliding shear force.

Allowable Bearing Stresses Notches ultimately rely upon bearing between two surfaces crafted into the timbers meeting at the notch. That bearing pressure is applied at various angles to the grain of the connected timbers, and the designer needs to recognize the impact these angles play in determining the notch's load capacity. Most wood species tend to be more prone to crushing perpendicular to the grain than parallel to the grain. Strength and stiffness vary with moisture content, and both properties increase as the wood dries. The allowable compressive stress perpendicular to the grain is the sole example of a design stress limited by deflection rather than capacity, that is, by strength. Crushing a piece of wood across the grain may be the definitive example of a tough property: the wood can undergo deformation without the deflection having a significant detrimental impact on the capacity of that mechanism.

A thorough study that compares calculation models from various European standards to results of experimental tests is found in Ali, Hussain, and Kamali (2014). Note that when using Eurocodes, one must be careful translating the information for use in the US, since the factors involved are considerably different and must be taken into account.

Designers almost always use the tabulated design values in the NDS Table 4D for timbers in dry service conditions. Although the tabulated values are based on green timbers, a 50 percent increase for compression perpendicular to grain and a 10 percent increase for compression parallel over unseasoned material are already included in the table values. This assumes that though the trusses might be fabricated from unseasoned timbers, those timbers will be used in dry service conditions and have a chance to dry (and strengthen) before being subjected to maximum design loading. If the trusses will be subjected to heavy loading prior to seasoning, use of the wet service factor C_M may be appropriate. (Refer to NDS Supplement, "Table 4D Adjustment Factors."

Compression perpendicular to grain and the modulus of elasticity are the only common design properties with allowable values based on mean values obtained from the small clear specimen tests of ASTM D143. They are little affected by strength-reducing characteristics (principally slope of grain and knots) that affect other properties. The other allowable properties are based on a factor of safety applied to the 5 percent exclusion value to deal with the variability in wood properties. Refer to Anthony and Nehil (2018) for more information. The different reasoning also means that neither property—compression perpendicular nor modulus of elasticity—is subject to the time-honored influence of the load duration factor C_D. This distinction can contribute some confusion when applying Hankinson's formula, as discussed below.

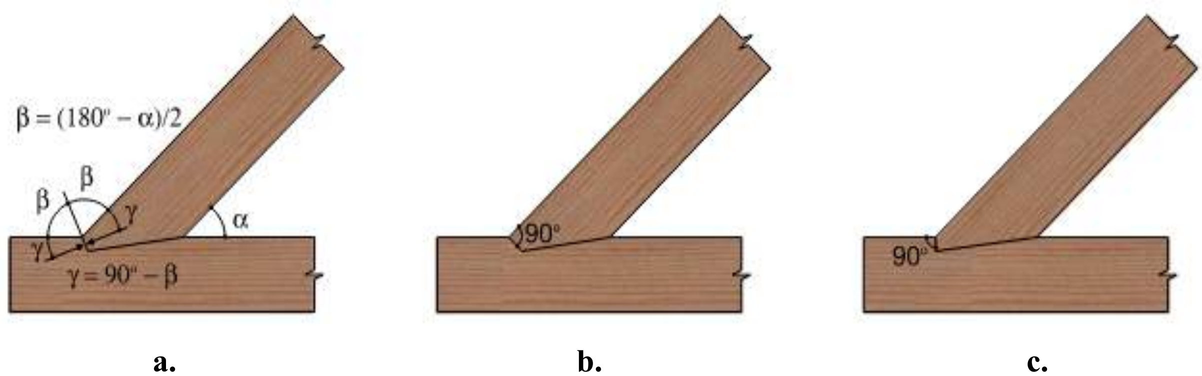
Until the 2001 edition of the NDS, designers had access to F_g, the allowable bearing value parallel to grain. That value was intended for end bearing of compression members and was strictly a function of species; stress grade was not a factor, and there were only two size groups: lumber and timber. These design values were often notably larger than compression parallel to grain in the common grades of timbers, but smaller than the allowable compression parallel to grain in better grades of lumber of some species. So, when evaluating existing structures designed under a pre-1991 version of the NDS, some consideration should be given to the appropriateness of using F_g for end-grain bearing or the current NDS requirements.

An interesting aspect of the allowable bearing stress parallel to grain applies when two pieces of wood bear end-to-end on each other. The exposed edges of the two opposing masses of individual cell walls will pierce through each other as they bear at the notch face, leading to unexpectedly high deflections. The NDS limits this behavior by requiring a steel bearing plate when the end-grain bearing stress was above 75 percent of the allowable stress. These added bearing plates can even serve to "snug up" the truss connections. Proper detailing is required to prevent the metal plates from falling out during erection or when subjected to load reversals.

What about the vast majority of notches used to interconnect truss members that rely on bearing faces that are neither parallel nor perpendicular to the longitudinal axes of the truss members? Welcome, now, the entry of one of the hoarier traditions of timber design: Hankinson's formula. This classic is applied to interpolate between any two timber properties, depending on the angle of load other than perpendicular-to-grain or parallel-to-grain orientation. Connection design inevitably engages Hankinson's regularly when establishing individual connector capacities.

The designer needs to be careful when applying load duration factors to allowable bearing stresses. C_D applies only to compression parallel to grain, not to compression perpendicular to grain. One ought not to calculate the allowable bearing stress for a given species and bearing angle, and then apply adjustment factors to that value. Allowable stress values are to be adjusted as appropriate to the load case prior to using them in Hankinson's formula.

Regarding the angle of the notch, where at least one of the bearing surfaces will be at an angle to the grain, Eurocode 5 regarding design of timber structures is pretty specific about the recommended angle to cut in notch joinery. The notch in Figure 7.10a presents the same angle to grain for both the rafter and the tie, so attendant allowable stresses are the same. This allows the designer to optimize the joinery design.



7.10 (a) Notch angle for equal bearing stress; (b) Notch perpendicular to rafter grain; (c) Notch perpendicular to tie grain.

The notches in Figures 7.10b and 7.10c have one member with bearing parallel to grain and the other with bearing at a higher angle to grain, mandating a lower allowable bearing stress given by Hankinson's formula. Note that if the timber species are not the same at any notched joint, the designer will need to modify the bearing angle accordingly to equalize the bearing stresses.

Impact of Crossgrain Shrinkage Angles cut at joinery in green timbers become more acute as the wood shrinks across the grain but not along it. (See chapter 3 for additional discussion.) This distortion from the green, as-cut shape shifts the bearing stress distribution along those faces. A certain amount of compliance is inherent in timber bearing connections: overly loaded areas will crush, and the load will be spread across wider surfaces.

Nevertheless, drying distortion can have other deleterious impacts on notch-joinery performance. The floating toe beyond a heel notch can induce splitting if it runs afoul of the other member with the shrinkage. The Eurocode is helpful in recommending that the designer and fabricator build in a gap to allow for this movement while preventing splitting. The goal is to make certain that forces are transferred between members at their mating bearing faces, not at inadvertently bearing faces that might develop with timber shrinking and angles changing. The Eurocode is clear on allowing a ⅛-in. gap between the top of bottom chord and the level cut past heel joints to minimize splitting tendency. Note the gaps indicated in Figures 7.7b and 7.7c. Deliberate gapping of the nonbearing faces associated with the notches should be done to avoid unwanted bending stresses and splitting. Deliberate gapping beyond a heel notch prevents splitting the top chord as it shrinks and the tip of the toe comes into contact with the other member.

The Eurocode is less clear, but the concern is just as real about potential for bearing at the nominally nonbearing faces of the notch in toe-cut notches. Should the top chord deflect under load while bearing at the heel, prying at the toe could induce tension perpendicular to grain at the relish shear plane, an obvious risk. A gap will tend to open along this nonbearing face with drying of green timbers, but the gap should be provided with any dry, stable timber members.

Deflections Computer analyses tend to treat truss joints as perfectly pinned or perfectly fixed. None of them are either, of course, but what are the translational- and rotational-spring constants, and are they linear? The localized deflections in the connections can be a significant contributor to overall truss deflection but are only rarely included in even the most sophisticated computer models. Sloppy joints make for weak, flexible springs, at least until unevenly cut bearing faces crush into some sort of mating condition or an initial gap closes. One source of recommendations on deflection calculations (TECO, 1956) advocates simply doubling the deflection due to member shortening and lengthening under load. This recommendation was intended to account for the deflection contribution from joinery slip and crushing. While that guidance may be somewhat unscientific, it may be a realistic way to incorporate the sag due to deflection in the less-than-rigid connections. Trusses, in general, tend to be remarkably stiff, but the careful designer, even using potent analytic tools, will acknowledge the likelihood of deflections larger than predicted by most models.

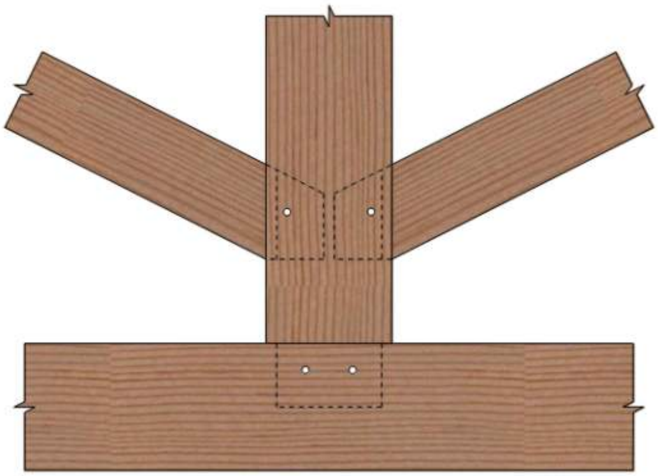
Failure Modes at Notches One advantage of the bearing surfaces at notches is that they are ductile and can absorb a lot of distortion (crushing) and deflection without breaking. Ultimately, however, failure can come abruptly and completely. One example is direct shear failure parallel to grain from the bottom of the notch out to the end of the timber in the bottom-chord relish. Increases in allowable shear stresses first presented in the 2001 NDS have been helpful in making relish design less conservative, but this aspect of notched bearing design should be approached with caution, especially where end-grain splitting may occur due to crossgrain shrinkage during the curing of green timber, or in kiln-dried timber when the connection is cut in an end that may already exhibit checks and splits. Experienced fabricators may be aware of this, but it is always wise to add a cautionary note to the details of this connection to make sure the end is sound. Alternatively, fully threaded screws may be utilized to reinforce the relish; note that the design in that case should provide for the entire shear force to be resisted solely by the screws and not by the shear capacity of the timber relish.

Through Tenons with Wedges, Keys, and Pegs Through tenons are useful for securing straining beams and tie beams against tension. They are especially reliable because the fit and behavior of the joint can be easily inspected—unlike concealed tenons, which can tear out without being observable. Wedges, keys, and pegs are used to secure through tenons. Other possible applications for through tenons are any conditions where a tenon on a tension member can extend through and out the far side of the receiving member. Wedges can also be used to snug up joinery at critical bearing faces. They have even helped to introduce camber in trusses. A wedged kingpost bottom, for instance, would allow for the assemblers to draw the connections into tight bearing by driving the wedges snug. The Long truss purports to rely on them for prestressing the truss in ways that reduce the maximum member forces under design loads. Using wedges in truss connections is a rich topic, indeed. Considerations for design of wedged through tenons may be found in Shields and Hindman (2018).

If at all, wedges or keys should be judiciously combined with pegs in the same joint, because wedges on the far side of a timber with a through mortise can work against any pegs in the same joint. However, pegs may be used alone with the through tenon to resist tension. Since wedges or keys may need periodic reseating due to crossgrain shrinkage in the post, pegs may be advantageous because they are more mechanically stable. Also, if the tenon is allowed to extend slightly beyond the outside face of the post, any possible shear failure of the tenon can be easily observed and the joint reinforced or repaired as necessary. Ideally, through tenons should not be left exposed to weathering or used in locations that are vulnerable to impact damage, such as from agricultural or commercial vehicles.

Mortise-and-Tenon Joints Conventional mortise-and-tenon joints are used to connect members in compression and, to a lesser extent, can be used in joints with moderate tensile forces. These joints are also entirely capable of transmitting shear forces. While shear forces are minimal in idealized trusses, they are common in timber trusses, especially where diagonals, bearing on chords, are restrained against sliding along the face of the mortised chord members. Excellent references for the design of these joints may be found in Miller (2010) and TFEC 1 (2019).

A typical example of mortise-and-tenon joints in a truss is shown in Figure 7.11. If this configuration is part of a common kingpost truss under uniform load, the struts carry compression into the kingpost, and the kingpost provides minimal if any uplift support to the tie. Thus, tension capacity of the pegged joints is generally not a concern, unless the bottom chord is used to support midmember gravity loads, such as an attic floor or a finished ceiling. The struts are fully housed into the faces of the kingpost to transfer their axial compression on the shoulders of the struts, rather than on the ends and bottom faces of the tenons. The housing depth is determined by the vertical component of the strut force and the angle-to-grain bearing capacity of the strut. The struts join the kingpost above the tie to provide sufficient relish to resist the vertical component of the strut force in block shear. Raising the struts above the kingpost-to-tie connection also eliminates transfer of strut force into the tie. If the struts were to bear on the tie, that thrust must be carried in shear by the tie and then transferred to the kingpost via a more robust tension connection, likely requiring steel hardware.



7.11 Mortise-and-tenon joints for truss connections.

Mechanical Connections

This section covers the following uses of mechanical hardware to facilitate connections:

- Rods to replace timber members in tension.
- Bolts to resist the sliding shear force in top-chord-to-kingpost connections.
- Tie bolts with plate washers or barrel nuts for joints in tension.
- Metal side plates and knife plates.
- Screws to resist tension-induced shear forces typically found in heel connections.
- Heel stirrups with knife plates or side plates.
- Truss heel shoes with dado bars.
- Specialized proprietary connectors for shear (Simpson Strong-Tie, Ricon, Timberlinx, etc.).
- Shear plates and split rings.

The NDS (AWC, 2018) and CSA O86 (CSA, 2017) provide the technical requirements for dowel-type fasteners, split rings, and shear plates. The AISC *Manual for Steel Construction* (AISC [2017]) and the CISC *Handbook of Steel Construction* (CISC [2017]) contain provisions for design of bolts, plates, tie rods, turnbuckles, nuts, and washers. Proprietary connection design relies on code evaluation reports and design guides for manufacturers' products.

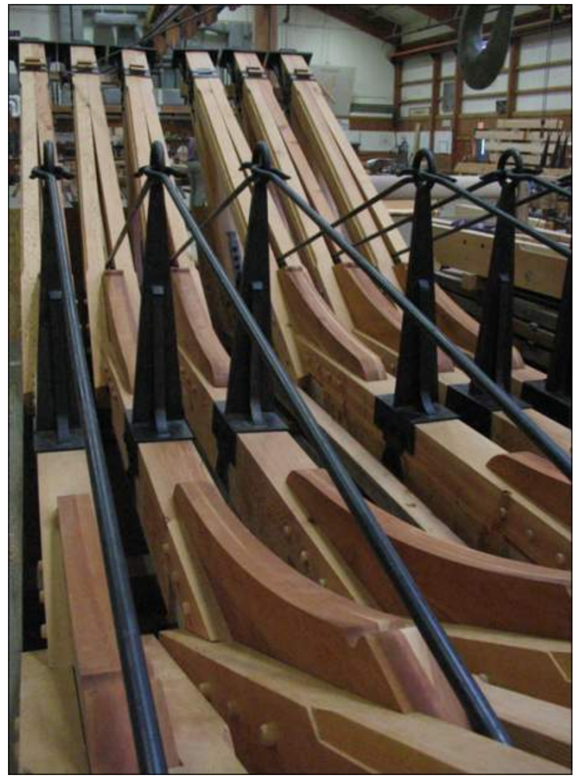
Several companies and organizations provide connection design software modules including Simpson Strong-Tie, Woodworks, and the American Wood Council. This freeware is especially handy for the design of bolted connections of all types.

Steel Tension Rods The low cost and weight of steel tension rods make them highly efficient tension members. Tension rods can be readily adjusted for easy assembly, since tightening the rods will bring the bearing surfaces into contact. They are useful to precamber a truss. They can help the truss stewards to remove sag that can develop as a result of loading and timber seasoning. Connecting rods to timbers can minimize loss of cross section due to joinery, compared to notching for wood-on-wood bearing connections. Concealed rods in holes or troughs with plate washers in mortise pockets can provide tensile resistance where it is impractical to do so with timber alone. Supplementary rods may also be used to take the sag out of existing trusses.

Examples of the use of steel rods as tension members include a simple kingpost truss using a steel kingrod rather than a timber kingpost (Fig. 7.12), a Fink truss (Fig. 7.13), and harped bottom-chord tension rods (Fig. 7.14). Note that the trusses pictured in Figure 7.14 are upside down.

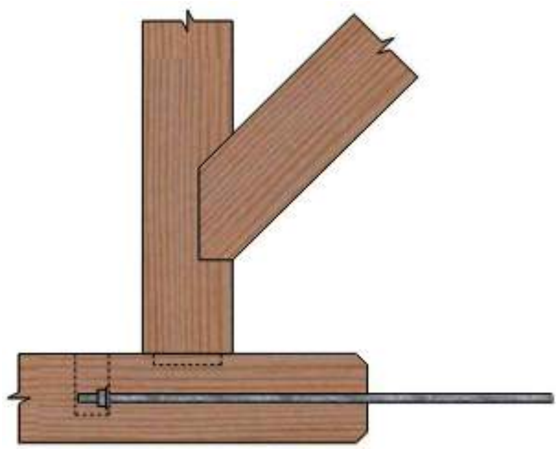
7.12 Simple truss with steel kingrod in lieu of a timber kingpost.

7.13 Steel tie rods used as truss tension members.



7.14 Compression struts with harped bottom-chord tension rods.

Steel in Hammer Beam Trusses There are a number of opportunities for reducing cost or changing the aesthetic character of a truss by replacing or supplementing timber members in tension with steel rods. Hammer beam trusses often incorporate steel tie rods (Fig. 7.15). This truss style is frequently used, but it originated when the trusses were supported by thick, often buttressed masonry walls capable of resisting thrust, thereby eliminating all tension in the truss members. In modern applications, buttressed walls are typically not feasible, but sometimes the thrust can be resisted by shear walls in adjacent areas to the plane of the hammer beam. However, when this approach is not appropriate, posts are incorporated as part of the truss, usually leaving the hammer beam, upper braces, and upper ties in tension and producing large in-plane bending and shear loads in the posts. A tie rod



7.15 Hammer beam with tie rod.

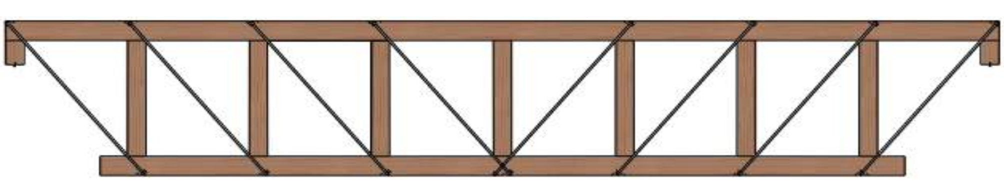
between the interior ends of the hammer beams can eliminate the tension joinery requirements for the other members and relieve the bending in the supporting posts.

In hammer beam trusses, tie rods connecting the interior ends of the hammer beams usually terminate within a washer pocket accessible from above, so it is concealed from view below. Plate washers are sized to keep compression parallel to grain loading within acceptable limits. Turnbuckles are often used to adjust the rods for length. Small length adjustments may also be made within the mortise itself by tightening or loosening the nut. Longer rods often incorporate hanger rods at midspan to reduce sag in the tie rod. Belleville washers may be employed to maintain tension without the need to regularly tighten a nut or turnbuckle as the timber shrinks.

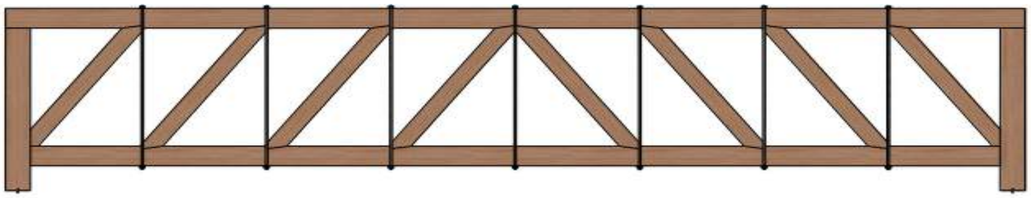
Scissor Trusses The bottom chords and kingpost of scissor trusses are generally in tension and can be replaced by steel rods with a junction plate at the bottom end of what becomes a kingrod truss (Fig. 7.16). The upper end of the kingrod usually terminates in a steel fitting that straddles the joint between abutting top chords. At the heel, the scissor rods can terminate in a mortise with a plate washer and nut accessible from above. This approach results in a variation of the classic scissor truss that eliminates the complexity of passing bottom chords.

7.16 Kingrod truss with harped tension chords.

Pratt and Howe Trusses In a Pratt truss (Fig. 7.17), the diagonals are in tension and can be replaced with steel rods. Similarly, Howe truss vertical members are in tension and are often replaced with rods (Fig. 7.18). Refer to examples and more detailed discussion in chapters 2 and 5. Pratt parallel-chord trusses remain among the most efficient truss forms. With equal panel widths, the rod loads generally increase toward the supports. The designer can more nearly equalize the rod tensions by varying the panel spacing. Alternatively, increasing rod size from midspan out toward the ends expresses the behavior of the structure. As another approach, shorter panels at the ends result in steeper rods that are more efficiently oriented to handle the higher tensile forces.



7.17 Pratt truss with tension rod diagonals.



7.18 Howe truss with tension rod verticals.

7.19 Top-chord-to-kingpost connection with shear rod.

7.20 Top-chord-to-king post connection with double-shear tie rods.

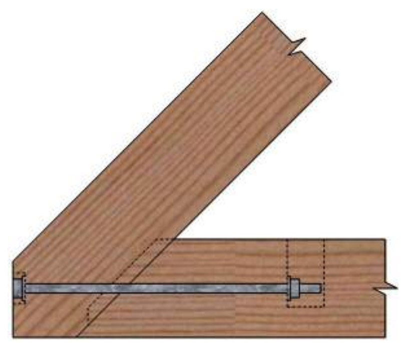
Top Chord to Kingpost Connections
Diminished housings with or without mortise-and-tenon joinery are often used for simpler fabrication in top-chord-to-kingpost connections (Figs. 7.19 and 7.20). However, the connection shown has little or no relish above any tenons on the top-chord ends to participate in transferring tension into the kingpost. Without supplemental help, the diminished housing would have to rely on friction to transfer the vertical component of the top-chord axial force, and that approach is not advisable at best. An effective means to resist these sliding forces is by using shear bolts to connect the upper ends of the top chords with the kingpost. The double-shear capacity of the bolts and the required bearing areas in the top chords and kingpost are easily determined to size the bolts. Typically, one or two bolts are required with standard washers.

Top Chord to Bottom Chord Connections High compression loads in the top-chord-to-heel connection may require connection reinforcement with bolts that are in both tension and shear to resist the sliding forces. These are often used in conjunction with notches, where the direct shear due to tension parallel to grain from the bottom of the notch to the outer end of the bottom chord is at or in excess of the allowable shear stress. Because a failure in this mode is abrupt, the shear bolts and washers must be sized to resist the full load.

Fully threaded screws may also be used in applications where sufficient relish to resist block shear is not available. The heel joint in Figure 7.21 is such an example. The red dotted line identifies the shear-failure plane. If that surface is insufficient to transfer the horizontal component of the rafter force as tension into the tie, then supplemental reinforcing is required. The vertical component of the rafter force goes directly into the post. Thus, the screws are needed only for the horizontal component. Since they are inclined relative to the tie, they resist both withdrawal and lateral load. The NDS and O86 standards do not contain provisions for this combined loading, but use of Hankinson's formula is appropriate, as it is endorsed by the Eurocode and the code evaluation reports provided by screw manufacturers. Since the screw is oriented at an angle to the grain other than 90°, an adjustment to its withdrawal capacity should be made, for which Hankinson's formula is again appropriate. Remember that capacities may be adjusted by C_D , the duration of load factor. Once the number, size, and length of the screws have been determined, a trial layout is required to ensure that proper spacing is maintained.

7.21 Overhanging top-chord-to-bottom-chord heel connection with reinforcing screws.

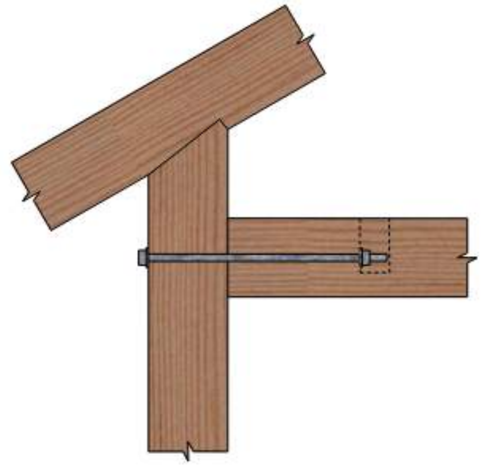
Bottom Chord to Top Chord at the Heel Another way to transfer the direct tension from the bottom chord to the top chord at the heel is to use a single or a pair of tie bolts (Fig. 7.22). The bolts are secured in the top chord with plate washers and nuts in a mortise pocket from the top. If the bottom chord is raised above the plate or post support, such as in Figure 7.6c, bending stresses will be induced in the top chord, which may require an increase in depth of that member to offset the loss of effective cross section at the location of the mortise. This loss of section might be lessened by using a barrel nut located in the centroid of the connection on the neutral axis of the top chord.



7.22 Heel connection with tie rod.

An alternate tie bolt–type connection in this situation could utilize a single- or double-pinned Timberlinx. Depending on the species of timber, these connectors are limited in their capacity to approximately 3.0 and 5.0 kips respectively for the single-pin and double-pin versions.

Upper Struts to Queenposts or Top Chords Raised ties and horizontal struts may use similar tie bolts and connections. However, ties to queenposts often extend to the outside face of the post, where a plate washer may be used to transfer the tension into compression perpendicular to grain on the side of the post (Fig. 7.23). Mortises to contain the plate washer are optional. Timberlinx connectors are often used in this application as well.



7.23 Tie-beam-to-queenpost connection with tie rod.

Metal Side Plates Exposed metal side plates are often specified by the designer. Compared to knife plates, which are fully concealed within the timber, exposed plates are more expensive. Typically, side plates are a minimum of ¼-in. in thickness, which will handle most building loads. Their capacity must always be checked for bolt bearing stresses, tension on the net section, and appropriate end and edge distances. Appendix C offers a brief overview of the design requirements.

These connections can develop high capacities, but their design depends strongly on the moisture content of the timber at the time of fabrication and in service. For a connection with two rows of bolts in a common plate and timbers above 19 percent moisture content at the time of fabrication, but below 19 percent when in service, the wet service factor is $C_M = 0.40$. This reduction in capacity accounts for the potential splitting of the timber caused by restrained crossgrain shrinkage of the timber by the plates; see NDS Table 11.3.3.

Most heavy timber frames and trusses are fabricated from timber that is green at the time of fabrication. Therefore, in unseasoned timber the design of these metal-plated connections should be limited to single rows of bolts in a single plate. When more than two rows of bolts are needed, it might be possible to use two narrow plates, side by side, to realize the connection. Once the design load requirements get to about five bolts in a row, the group action factor C_g causes the allowable load per bolt to reduce rapidly and requires an increase in bolt size to keep the number of bolts at a reasonable level.

The differing thermal properties of timber and steel can also become problematic, especially in untempered environments. Even when protected from rain, timbers in open mill buildings will decay behind the plates and around the heavy bolts, and the steel will rust as moisture condenses on the chilled steel each morning. This condition may warrant the consideration of hot-dip galvanized or stainless-steel plates and bolts to protect the steel. However, condensation can still promote decay in the timber. This is especially problematic since the decay is concealed from view behind the plates until well after it reaches an advanced state.

Of major concern with using steel plates on timbers is how to get all the holes to line up within tolerances, given the reality of drilling holes through timber and the convenience of predrilling the steel plates. It is all too easy to ream the wood in order to get the reluctant bolt to fit through the far hole, thereby reducing the bearing capacity of that bolt significantly.

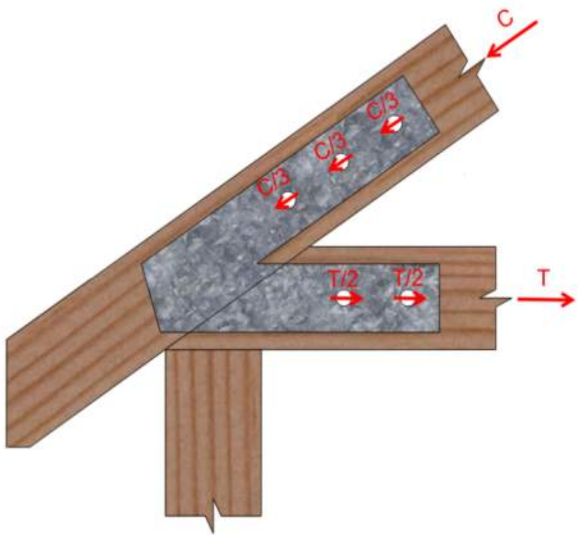
Holes in the timber are normally drilled the same diameter as the bolt to ensure a snug fit, and the bolt may need to be driven through the timber with a mallet. Holes in the plates may be sized no more than $^{1}/_{16}$ in. larger than the bolt diameter to ensure a good fit. Sloppy fits with bolts in metal plates can result in severe sagging of trusses and should be avoided. The heads of bolts should always face in the same direction for best appearance. The use of all-thread rod instead of bolts eliminates any concern for irregular appearance from the floor level. Vertical bolts should ideally be inserted from above, so that the nut and washers are the only things that can fall to the floor over time.

It is important to avoid or minimize eccentric forces at the centroid of the connection. Eccentric connections are avoided by ensuring that the centerlines of the timbers all pass through a common working point. Lines of force and resistance must act through this point. Generally speaking, it is best to avoid eccentric, moment-resistant connections. On the other hand, deliberate moment connections in timber construction are occasionally desired to avoid the use of knee braces and struts. The design of these connections involves balancing the need for rotational restraint of the joint against the induced tension perpendicular to grain and the restraint on crossgrain shrinkage. Maintaining tight fits and edge distances is essential for good performance.

Many side-plate connections between top and bottom chords induce eccentric load transfer in at least one of the members. This situation is equivalent to columns with side loads and eccentricities—a topic covered in the NDS Section 15.4. The combined stresses from eccentricity, induced bending, and the axial force can further reduce the capacity of these joints. The distribution of shear forces within a pattern of bolts in a side plate is analytically complex. The centroid of the bolt pattern needs to be located so the moment-induced shears can be determined. The magnitude of those shears is proportional to the distance from the centroid to a given bolt. The direction of the shear component is perpendicular to the radial line from the centroid to the bolt. These varying components are superimposed with the evenly shared components induced by the axial force. Additional information on this connection challenge is presented in Stalnaker and Harris (1989), pp. 191–93.

In moment connections, free-body diagrams are essential to analyze and assess the flow of forces between and among the bolts and timbers. The axial force (and possible shear) in one truss member is transferred by bolts into the plates, and then the forces are transferred back out through bolts into the other connected member. Consider the side plates as separate bodies and assume that the plates are neither moving nor spinning about any point. Analyze the forces being transferred to the plates by the bolts that allow for this lack of motion—that is, establish equilibrium among the applied forces and moments.

As an example of the importance of proper free-body diagrams, consider the connection shown in Figure 7.24. We assume that the connection is in an ideal truss, such that the rafter and tie carry only axial force: C in the rafter, T in the tie. A carelessly drawn free-body diagram of the side plate might suggest that each bolt in the rafter carries shear force C/3 and each in the tie carries T/2. However, the arrangement of shear forces clearly does not satisfy equilibrium: the vertical component of rafter force C is not resisted. Since equilibrium is not satisfied, the assumed directions and magnitudes of the bolt shears must be wrong. Figure 7.25 offers another example of an improper connection detail, compounded by the two rows of bolts that restrain crossgrain shrinkage as well as chord rotation at the joint.



7.24 Improper free-body diagram of loads on side-plate connector.

7.25 Example of an eccentrically loaded truss-heel connection to be avoided.

Fortunately, there are other approaches to side-plate connections that result in effective member force transfer with minimal or no eccentricity. Figures 7.26 and 7.27 are such examples.

Figure 7.26 shows a hand-forged variation on steel side plates as a tension connection between the kingpost and the arched bottom chord.

In Figure 7.27, each truss member uses a side plate with a single line of fasteners. The plates are then pinned together with a single fastener where they lap to eliminate any eccentricity and rotational restraint.

7.26 Kingpost to arched glulam bottom chord connection with creative steel strapping.

7.27 Scissor truss with well-designed side-plate connections.

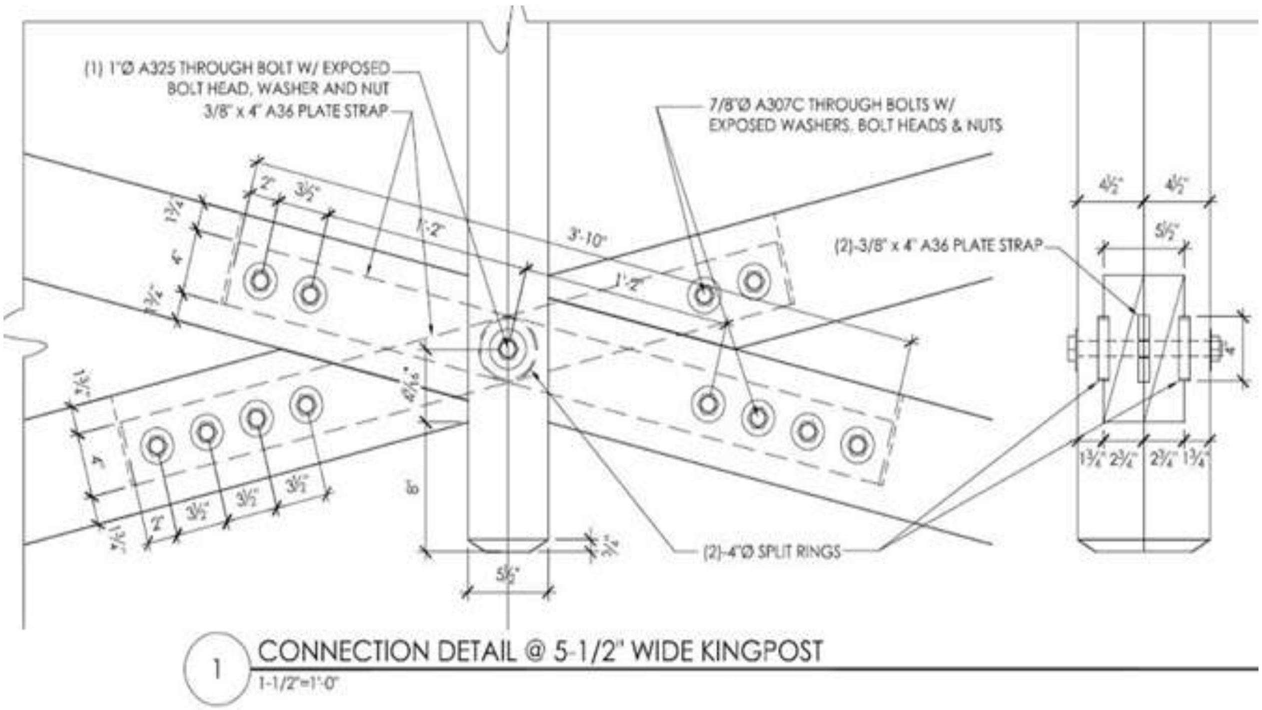
Metal Knife Plates All of the NDS requirements pertaining to metal side plates also apply to metal knife plates. Metal knife plates are typically ⅜-in. thick (or thicker depending on the loads and forces involved) in a ½-in.-wide slot cut with a chain saw from one edge or completely through the timber. Normally, the intention is for the metal in the connection to be concealed, so cutting the slot from the blind side of the timber is desirable. Since these plates are always oriented vertically, the slots can be cut from the top edge of the timber.

Completely concealed connections may be achieved by using steel pins, which are typically ¾- or 1-in. in diameter to accommodate end plugs to conceal and stabilize the pin in position (see Fig. 7.28). Setting the plugs in epoxy will ensure that they remain in place. Typically, the pins are the thickness of the timber less about ⅝–¾-in. recess from the face of the timber to accommodate the plugs. The recess for the plug on each side reduces the pin's penetration length in the side member, which might impact its design capacity. NDS Section 12.3.11 limits the capacity of a connection with drift bolts or drift pins to 75 percent of the normal bolt capacity of the same diameter and side-member thickness. It is advisable to use a ¾-in.-long by 2-in.-thick stub tenon to ensure that the joint does not twist and induce cracks in the timber. Alternatively, twisting can be resisted by using ⅜-in. bolts with 1-in.-diameter washers that are recessed and plugged to make sure that the plate stays in place.

7.28 Eccentrically loaded, moment-resistant connection using steel knife plates with plugged pins.

While there may be some benefit to having a completely concealed connection, using steel drift pins instead of bolts reduces the capacity by 25 percent. If higher loads are to be resisted, bolts with washers should be used to develop the full capacity of the connection. It is desirable to create a shallow recess (⅛-in. and slightly larger than the diameter of the washer) to avoid visible crushing of the surface fibers when the nuts are tightened. Typically, standard washers are used, and the exposed parts are often painted flat black to add contrast to the timber and add interest to the connection.

An example of a truss connection using knife plates with bolts is shown in Figure 7.29. The tension in the kingpost is resisted by the split rings. In the bottom chord, the tension is resisted by the four bolts. In the web, the compression is transferred by the two bolts. The webs could have

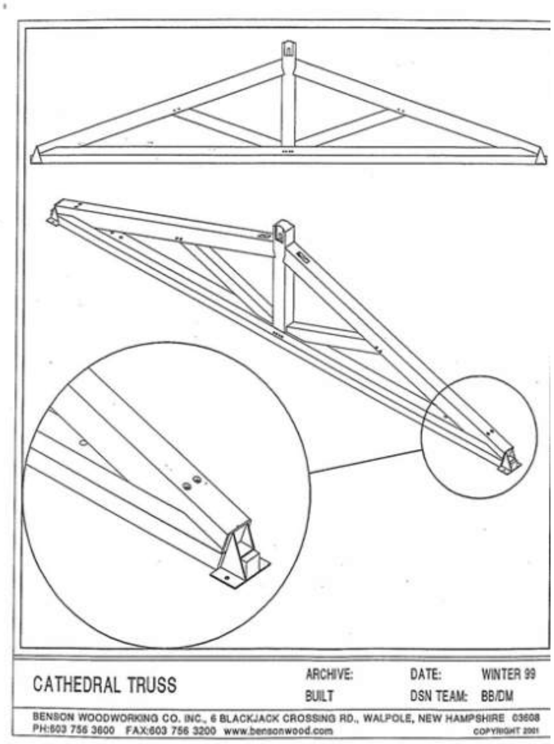


7.29 Scissor truss connection at bottom of kingpost with knife plates and bolts.

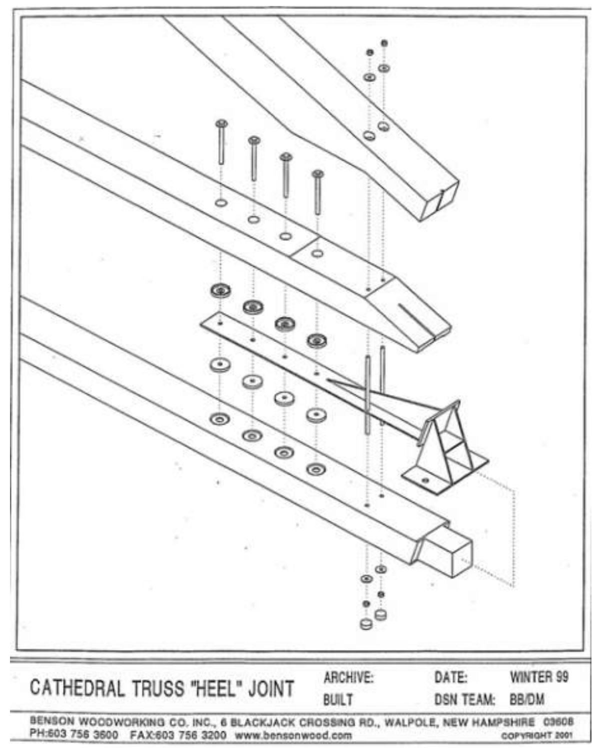
been seated with a stub tenon and screws to reduce cost, but aesthetic considerations in this case dictated the use of the exposed bolt heads, washers, and nuts.

Knife-plate connections may also be realized using small-diameter, self-drilling dowels instead of bolts or pins. These proprietary fasteners are available from distributors including SFS Intec, MTC Solutions, and Rothoblaas. Wet service factor reductions in capacity do not apply to these small-diameter fasteners, and such connections can be used with some efficiency. One possible negative aspect of these connections is the visual evidence of the drilled hole in the surface of the timber. However, viewed at some distance, the small holes should hardly be of practical concern.

Heel Stirrups with Knife Plate Heel stirrups (or shoes) against the vertically cut end of the top chord with a knife plate extending into the bottom chord can carry high tension forces into the bottom chord. The stirrup itself must be sized in area to resist compression at some angle to grain and thick enough to resist bending and uneven distortion of the wood fibers in compression. The knife plate acts as a gusset to stiffen the stirrup against bending due to top-chord thrust. A good rule of thumb for initial sizing is ⅛-in. of thickness for every inch of width required, assuming a square stirrup plate. Normally, connections requiring loads of high magnitude require glulam timber or radio-frequency dried Douglas-fir to ensure that the moisture content is well below 19 percent, so that two rows of bolts may be used. Steel pins would not be suitable for this application due to their reduced capacity (see Figs. 7.30 and 7.31).



7.30 Steel shoe with concealed knife plate into the bottom chord.



7.31 Concealed portion of the steel shoe detail with shear plates for the tension connection to the bottom chord.

Heel Shoes with a Dado Bar A steel shoe with a dado bar works similarly to the vertically oriented stirrup with a knife plate discussed above, except that the tension plate is placed on top of the top chord (out of sight) and secured with a steel bar in a dado slot cut in the top edge of the top chord.

Split Rings and Shear Plates Split rings and shear plates are sometimes used to join timer members to timber members in lapped top-chord-over-bottom-chord heel connections, timber-to-tube-steel connections, spaced chord splices, and splices using plates. The NDS Chapter 13 has detailed requirements and illustration for designing these connections. The Timber Construction Manual (AITC, 2012) has excellent suggestions regarding design considerations and types of connections to avoid.

Proprietary Steel Connectors The use of Timberlinx for concealed tension connections is common, and some possibilities for their use are included above. Simpson Strong-Tie makes a variety of straps, hangers, and concealed joist ties that are often useful to make timber connections. Ricon and Rothoblaas make high-capacity shear connections that are often utilized to connect purlins to principal rafters or top chords to avoid cutting housings and reducing the bending capacity of those members. Fully threaded screws by ASSY, Rothoblaas, and MTC Solutions are useful for resisting tension, as discussed above. In some cases, the literature that accompanies these products contains design values based on Eurocode requirements. For US applications, exercise caution in making sure that all differences in dimensional units, material properties, and design basis are properly considered. The truss engineer should request and follow guidance provided in the code evaluation reports for proprietary fasteners. These documents often contain additional engineering data and requirements (such as spacing, edge distance, and end distance values) beyond that published in marketing materials.

References

Ali, K. H., T. Hussain, and A. Kamili. 2014. "Compression Perpendicular to Grain in Timber: Bearing Strength for a Sill Plate". Masters thesis. Växjö, Sweden: School of Engineering at Linnaeus University.

American Institute of Steel Construction (AISC). 2017. *Steel Construction Manual*, 15th Edition. Chicago: American Institute of Steel Construction.

American Institute of Timber Construction (AITC). 2012. *Timber Construction Manual*, 6th Edition. American Institute of Timber Construction. Wiley Inter-Science.

Anthony, R., and T. Nehil. 2018. *Determining Allowable Design Stresses for Timber Using ASTM Standards D2555 and D245*. TFEC *Technical Bulletin* 11. Timber Frame Engineering Council, Alstead, New Hampshire: Timber Framers Guild.

American Wood Council (AWC). 2018. *National Design Specification for Wood Construction*. Leesburg, VA: American Wood Council.

Brungraber, R. J. 1976. *An Overview of Floor Slip Resistance Research with Annotated Bibliography*. NBS Technical Note 895. Washington DC: National Bureau of Standards.

Canadian Institute of Steel Construction (CISC). 2017. *Handbook of Steel Construction.* 11th Edition, 3rd Revised Printing. Markham, Ontario: Canadian Institute of Steel Construction.

Canadian Standards Association (CSA). 2017. *Engineering Design in Wood*, O86-17. Toronto: Canadian Standards Association.

DeStefano, J. 2011. *Rules of Timber Joinery Design*. TFEC *Technical Bulletin* 3, Timber Frame Engineering Council, Alstead, New Hampshire: Timber Framers Guild.

Hoyle, R. J., and F. E. Woeste. 1989. *Wood Technology in the Design of Structures*. Iowa State University Press.

Jacoby, H. S. 1909. *Structural Details: Or Elements of Design in Heavy Framing.* New York: John Wiley & Sons.

Karlsen, G. G. 1967. *Wooden Structures*. Moscow: MIR Publishers.

Miller, J. F. 2010. *Capacity of Pegged Connections*. TFEC *Technical Bulletin* 1, Timber Frame Engineering Council, Alstead, New Hampshire: Timber Framers Guild.

Shields, L., and D. Hindman. 2018. *Keyed Through Tenon Joints—Structural Design Guide*. TFEC *Technical Bulletin* 8A, Timber Frame Engineering Council, Alstead, New Hampshire: Timber Framers Guild.

Stalnaker, J. J., and E. C. Harris. 1997. *Structural Design in Wood*, 2nd Edition. Boston: Kluwer Academic Publishers.

Timber Engineering Company (TECO). 1956. *Timber Design and Construction Handbook*. New York: McGraw-Hill Book Co.

Timber Frame Engineering Council (TFEC). 2019. TFEC 1-2019 *Standard for Design of Timber Frame Structures and Commentary*. Timber Frame Engineering Council, Alstead, New Hampshire: Timber Framers Guild.

Chapter 8 - Bridging and Bracing

Introduction

Handling and stabilizing long-span trusses for pickup and movement toward their final position on the structure can be challenging. Keeping them stabilized in place while other trusses are set and before additional structural elements can be added offer additional challenges for the erector and timber engineer. Finally, permanent bracing may also be required to maintain stability under load. Hence, the design of both temporary and permanent bridging and bracing of these trusses is an essential part of the truss-design process.

For the purposes of this discussion, bridging and bracing members are defined as follows:

> *Bridging* consists of horizontal members oriented perpendicular to the main axis of the truss. They may be in the plane of the top or bottom chords or offset downward and upward respectively on a vertical web member. Bridging members may serve the dual purpose of purlins that are flush with the top or bottom chords. Ridge purlins often serve this dual purpose. Top-set purlins should not typically serve as bridging members due to the impracticality of developing the required tension or compression capacity in the bridging member with shear connections to the top chord. Bridging members can be utilized along the bottom chord of long trusses (spans greater than 50 ft.) to maintain precise alignment of the bottom chord.
>
> *Bracing* consists of inclined or diagonal members oriented in a plane perpendicular to the axis of the truss. They are typically secured at or near the ends of bridging members or may be secured directly to the top or bottom chords of the truss or to any vertical web member. Bracing members can be full-depth, partial-depth, paired as K-braces or chevrons off the center of the upper bridging member, or full-depth cross braces. The various configurations will depend upon the forces involved as well as aesthetic considerations.
>
> Taken together and properly assembled from truss to truss, the bridging and bracing members can provide permanent lateral stability to the structural timber roof framing system.

Safety is the primary concern during these operations. Having permanent, integral bridging and bracing that is part of the timber-truss design and having those elements installed as part of the erection sequence can minimize and mitigate safety concerns. Bridging and bracing can also create three-dimensional load paths to provide load sharing between trusses and minimize stresses that may arise from heavy localized loads.

Long-span gable trusses are especially challenging due to their height at the ridge, even with modest slopes. Such trusses become especially cumbersome and difficult to brace, even temporarily, due to the height and weight of the trusses.

Bridging and bracing may be required to resist out-of-plane wind load on end walls and to carry that load into the roof diaphragm. Typically, diaphragms alone are insufficient for handling large compression loading due to wind at end walls.

In long-span gable trusses or other tall trusses, buckling of slender web compression members may be a reason for special bracing, or at least, increasing the cross-sectional dimensions of these members to provide adequate stiffness and stability.

Because lateral bracing of compression chords is required to prevent buckling of the member, it is imperative that stress reversals be considered when trusses are subjected to high wind loading. In these instances, it is as if the truss is *turned upside down*. Bottom chords can go into compression and top chords can carry tension. This possibility is a universal and mandatory design consideration for timber trusses in coastal regions and island areas, and also for those in inland areas where wind Exposure Category D could be a consideration.

The lateral stability of short-span trusses (under 40 ft.) is achieved through various combinations of purlins, rafters, structural decking, and structural insulated panels. Occasionally, a designer may include bridging and bracing elements for aesthetic reasons, but these can easily be incorporated into the overall timber-truss design to facilitate stability during installation.

The design and detailing of temporary bracing during the erection of timber trusses is beyond the scope of this section. Nevertheless, many of the references contained herein have excellent suggestions and considerations for the design of those systems.

Literature Review

A variety of helpful resources is available to provide guidance and recommendations for design of bridging and bracing. A brief review of selected publications is offered here.

Scofield and O'Brien (1963) briefly address lateral support of compression members, which is primarily concerned with the slenderness ratio of the top chord of the truss based on how the roof is framed, i.e., with purlins or joists.

Ambrose (1994) addresses truss bracing in several sections of his book. He comments that purlins are usually adequate for stabilizing the compression chord, and goes on to say, "However it is also necessary to brace the truss generally for out-of-plane movement throughout its height." In his parallel-chord truss example, he explains that this support is provided by X-bracing at every other panel point. (Note: X-bracing is a form of cross bracing. Cross bracing could also include K, V, chevron, and single diagonal braces.) Ambrose also mentions that such bracing could be installed in alternate bays, since each set of X-braces stabilizes two trusses. In the case of the Shenandoah Services Center (see Fig. 8.1), this technique was utilized to preassemble pairs of 72-ft. parallel-chord trusses on the ground with their included X-bracing. This approach enabled lifting the entire assembly as a single unit of two trusses and setting it in place on the structure. Lateral stability (resistance to rollover) was achieved automatically and greatly improved safety during installation.

Ambrose continues his discussion by suggesting that this same X-bracing could be used as part of the lateral force–resisting system for other elements of the structure. One example would include bracing along the top edge of an end wall running parallel to the trusses. Trying to achieve this same wall bracing in larger structures by directly loading the deck diaphragm in compression is often not feasible. Ambrose also suggests horizontal X-bracing to stabilize the bottom chords of several trusses, and he recommends installation of horizontal struts to engage several trusses with a single set of horizontal bracing. The use of X-bracing will be most familiar to those designing preengineered metal buildings, where the bracing scheme is used in the plane of the roof to provide wind bracing for end walls (see bracing schemes B and C below).

AISC's *Design Guide 7* (2019) suggests use of a 2.5 percent P-force as the design lateral force for the bracing scheme, where P is the axial force in the member to be braced. In earlier versions of this document, P-forces from 5 to 10 percent have been suggested. A 5 percent P-force is recommended for the design of permanent bridging and bracing of timber trusses. That is to say, the bracing member is designed for an axial force of 5 percent of the force in the member to be braced.

Of course, considerations for the design of bridging and bracing of steel trusses include more considerations for slenderness and stiffness than typical timber trusses. Nevertheless, the guidelines found in this publication are often useful.

Meeks (1999) offers some good suggestions for lateral bracing concepts for stabilizing gabled end walls. There are schemes for taking lateral loads in the bottom chords into several sets of trusses parallel to the wall with diagonal bracing to transfer the loads up into the deck diaphragm. The WTCA (Wood Truss Council of America) and TPI (Truss Plate Institute) also have a requirement for permanent bridging and bracing for metal plate connected wood trusses with spans in excess of 60 ft.

Interestingly, neither the Timber Engineering Company (TECO) books nor those by the American Institute of Timber Construction (AITC) address permanent bridging and bracing of timber trusses. Structural timber-design books dating from the early to mid-20th century offer little if any information on the subject.

Permanent Bracing Considerations

Generally speaking, timber trusses with spans up to the 40-ft. range are fairly easy to stabilize during the erection phase using purlins on the top chord and temporary diagonal bracing to the nearest floor. However, once the wall heights exceed one and one-half to two stories, and for pitches ranging up to 12:12, even these trusses can become unwieldy and deserve consideration for permanent bracing schemes that can be installed as the trusses are erected.

High compression in web members may drive considerations for bridging and bracing in long-span, high-pitched gable trusses. Normally in a typical kingpost truss, the compression in the diagonal web is stabilized against buckling by the well-braced compression chord at the top end and by the connection at its lower end to the kingpost, which is in turn stabilized by the tension chord to which it is connected. The distance from the lower end of the web to the bottom chord is usually less than 1 ft. or so. However, as spans start to exceed 40 ft. and this web member gets longer and carries higher loads, the ability of the tension chord to hold things in alignment becomes reduced to the point where bridging and bracing can become necessary. Additionally, the timber engineer must check the slenderness of the bridging and bracing members to verify that their allowable buckling stresses are not exceeded.

As truss spans increase, the required minimum size of timber sections does not necessarily have to increase, since laterally stabilized compression members can resist high loads with fairly nominal sizes, in the 8x10 to 8x14 range. Sizes of compression members are more often dictated by bending stresses between panel points, especially when uniformly loaded with longer-span purlins or structural decking.

Likewise, tension members in the same size range are capable of handling loads with fairly long spans, ranging up to 70 ft. or more. The longer the bottom chords become, the more prone they are to sweeping and curving out of alignment. Bottom-chord bridging and bracing will maintain the alignment of these long tension members, provide additional end fixity to long web compression members, and make the truss system much safer to erect if they are installed during assembly, prior to raising.

As the spans increase beyond 50 ft., member sizes begin to look more and more slender aesthetically, and for tension members, despite the higher loads, keeping members straight may become problematic. Thus, bridging members between bottom chords and periodic bracing may be warranted for both practical and safety reasons.

Points of discontinuity due to splices may drive considerations for bridging and bracing. Although the complexity of steel-reinforced connections can be minimized by moving splices off panel points, bridging and bracing can significantly reduce handling stresses by using pre-assembled truss pairs on the ground, where the individual trusses brace each other. This eliminates the necessity of temporary strongbacks and stiffeners, and improves safety. This notion is illustrated in the parallel chord trusses used in the Shenandoah Social Center (see Fig. 8.1)

Hammer beam trusses demand bridging and bracing of the hammer post to prevent out-of-plane buckling due to the typically high compression loads in that member. Tension forces change direction at the outer end of the hammer beam as they move into the upper diagonal. Those tension members are not capable of stabilizing the compression-loaded hammer post. This requirement becomes more significant as the loads increase, as trusses are spaced farther apart, and as spans increase.

Member Sizes, Configurations, and Connections

Sizes for bridging and bracing members can be smaller than those of the trusses they brace due to the smaller axial loads used in their design. Sizes are often driven by aesthetic considerations, and picking sizes in proportion to those of the main truss members may become the most important consideration. As the sizes are reduced, it is important to remain mindful of possible lateral buckling on any long compression members in the bracing scheme.

A special precaution is warranted for cross bracing of long-span bowstring trusses (100 ft. or more). Cross bracing is commonly connected to the web members of these trusses. These web members were typically modest in size due to the efficiency of this truss form when unbalanced loads are neglected. Relatively small differences in the deflections of adjacent cross-braced trusses can lead to failure of the web members in the direction of the minor axis.

Bridging and bracing of bowstring trusses can also have unintended consequences. Failure of a single bowstring truss bottom chord may bring down a 20-ft. swath of the building if the roof purlins are simply supported. If bridging and bracing are present, the collapse often progresses through the building as the neighboring trusses are dragged down and forced to carry more than their tributary width of roof loads.

The options for the configuration of bridging and bracing schemes are numerous, and safety should be the foremost consideration. Top- and bottom-chord bridging with X- or single-diagonal bracing, forming a bridging truss, is the most conventional. The entire bridging-truss elevation view should be examined to yield the most pleasing pattern. For example, in multiple-truss bays, single diagonals can be arranged as chevrons, symmetrical about the center of the bridging truss. In addition to safety and aesthetic considerations, the focus should be on maximizing function and minimizing cost. To that end, stabilizing trusses in alternate bays is very effective. This also enables the two-truss pick mentioned above.

The spacing and number of bridging trusses is more or less proportional to the span. At 40 to 50 ft., a single row of bridging and bracing along the centerline of the truss may be sufficient. In gable trusses, bracing is normally accomplished using a ridge purlin and chevron bracing from the lower ends of the kingposts up to the center of the purlin. This approach induces shear forces between the kingpost and bottom chord that must be considered in the design of that connection. Diagonals from the top chord to the bottom chord and bridging between the bottom chords may be desirable. For spans greater than 40 or 50 ft., the plane of intermediate vertical webs would be ideal locations for the bridging truss. At 60-ft. spans, two sets of bridging trusses would be appropriate, and beyond that, consideration should be given to three rows of bridging trusses.

Harmony Timberworks

8.1 Shenandoah Social Center, Frederick County, Virginia.

In parallel-chord trusses of the same length, two rows of bridging trusses would facilitate the two-truss pick and provide adequate stability up to 60 ft. Beyond 60 ft., three or more bridging trusses might be appropriate if the two-truss pick is being considered. The trusses for the Shenandoah Social Center (Fig. 8.1) were lifted in sets of two and set on the structure, requiring no temporary bracing.

Other bracing schemes are inherent in the overall design concept for the structural system, such as crossed trusses that can be preassembled on the ground and set with a lift. Once in place, they are stable and do not typically require supplemental bracing. In the Horizon Community Church in Cincinnati, Ohio, curved bottom-chord glulam crossed trusses spanning over 80 ft. were set in a single lift (Fig. 8.2). Interestingly, this truss system not only supported the roof of the structure but a 20-ton steeple as well.

Three-dimensional grids offer another self-supporting framing concept, such as that chosen for the Belmont Abbey in Belmont, North Carolina (Fig. 8.3). The 5x5 grid of 6x6 timbers was

Harmony Timberworks

8.2 Horizon Community Church, Cincinnati, Ohio.

assembled with Timberlinx connectors and formed an inherently stable framing system without the need for temporary bracing within the roof systems. Overall structural stability was achieved with diagonal stainless-steel rod bracing.

8.3 Belmont Abbey, Belmont, North Carolina.

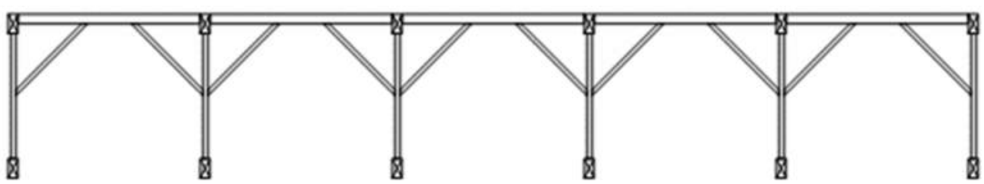
For concealed connections, mortise-and-tenon joinery is not typically adaptable to right-angled intersections of bracing members to main chord members without weakening main member sections. Typically, mechanical (steel) connections are utilized. Proprietary T-type knife-plate connections, such as the CJT hanger by Simpson Strong-Tie, can be appropriate and easily handle the tension and shear forces involved. Custom connectors using face plates can also be designed to handle the loads involved, subject to the limitations and considerations presented in the preceding chapter.

Since connections should be at panel points, not between them, consideration must be given to avoiding conflicts with joinery connections in order to avoid introducing lateral bending stresses in the principal chords. Fin plates extending perpendicular to face plates are a simple detail. Concealed connections can be accommodated by selecting fasteners to the main member that either pass all the way through the main member and into a similar fastener on the other side or that terminate before interfering with any embedded fastener in the main truss.

Bracing Schemes

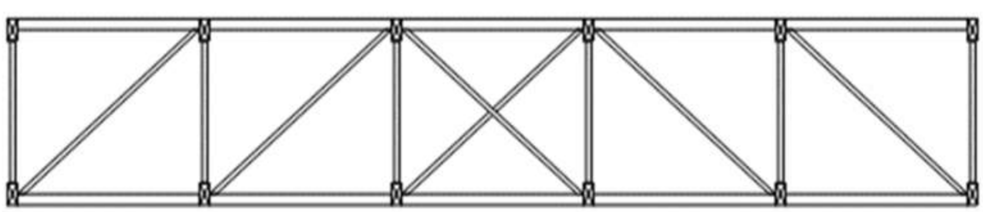
The following bracing schemes illustrate some options that may be considered. In these illustrations, the trusses are shown in cross section and the bracing schemes are shown in elevation.

Scheme A Shallow chevron bracing, also called K-bracing, that does not extend the full height of the truss to be braced is best for shorter spans (Fig. 8.4). One common application is in the plane of the kingpost on a gable truss utilizing a truss-to-truss ridge purlin. The braces are relatively easily and economically added to the design and provide a high degree of installation safety. These are suitable for trusses ranging in span up to approximately 40 ft. The braces are typically oriented at 45 degrees to the plane of the truss. It is important to verify that induced shear forces due to tension and compression in the braces are resisted by the kingpost and purlin or bridging connections.



8.4 Chevron bracing.

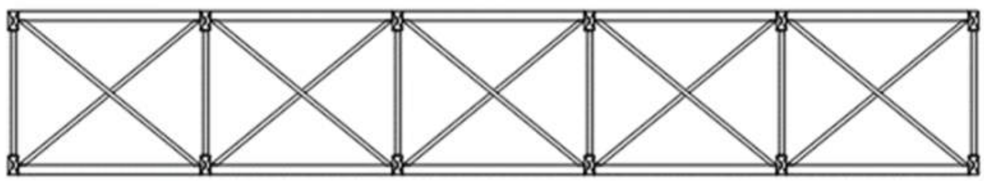
Scheme B Single-diagonal bracing (Fig. 8.5) is suitable for trusses in the 50–60-ft. range and provides greater lateral stability than Scheme A. If the bracing in the end bays starts from the lower truss chord, horizontal struts or connections to end walls can be used to transfer wind loads from the walls up into the roof diaphragm and reduce the unbraced height of the wall. Ideally, bridging members are connected directly to the bottom and top chords, and diagonals are connected to the ends of the bridging members. Connections from the bridging members to the



8.5 Single-diagonal bracing with counterdiagonals in center panel.

truss chords must resist the induced shear caused by the tension and compression from the diagonal braces. For configurations with an odd number of bays, the center panel may use X-bracing, as shown, or be left unbraced.

Scheme C X-bracing (Fig. 8.6) is commonly utilized for spans in excess of 70 ft. due to the larger lateral loads to be resisted. Connections from the bottom chord to end walls can be used to transfer wind loads up into the roof diaphragm in a similar manner to that suggested for Scheme B. If there is an odd number of bays, as in the illustration, the center bay can be left open or X-braced depending on overall strength requirements. Aesthetic considerations often play into the orientation of the braces. Diagonal bracing connections are similar to those in Scheme B (at the ends of the bridging members). The designer is cautioned against cross-bracing long-span trusses to short-span trusses or masonry, gable end walls without making provision for differential vertical deflection. Careful consideration of the truss deflection relative to adjacent structural elements is needed to avoid failure of the bracing.



8.6 X-bracing.

References

American Institute of Steel Construction (AISC). 2019. Design Guide 7 *Industrial Building Design*. Chicago: American Institute of Steel Construction.

Ambrose, J. 1994. *Design of Building Trusses*. New York: John Wiley and Sons, Inc.

Meeks, J. E. 1999. *Commentary for Permanent Bracing of Metal Plate Connected Wood Trusses*. Madison WI: Wood Truss Council of America.

Scofield and O'Brien, 1963. *Modern Timber Engineering,* New Orleans, LA: Southern Pine Association.

Chapter 9 - Construction: Fit-up and Raising

Fit-up

One of the authors, Grigg Mullen, has spent more than 20 years managing two community service timber framing projects a year, staffed primarily with volunteers. Joinery is cut using hand tools and portable power tools. Crew experience ranges from complete neophyte to seasoned professional. Opportunities abound for a frame not to fit on initial assembly. Discussed below are solutions developed over the years to avoid misfits during a frame raising. The process described is basic to timber framing with mortise-and-tenon joinery, and remains essentially the same regardless of scale of the project.

Little brings a raising to a screeching halt faster than joinery not fitting. Some assembly hangs on the crane that can't be set in place and released. Correct the problem in the air, or bring the assembly back to the ground? It all takes time, effort, and expense in a difficult situation that can be avoided by proper fit-up.

Fit-up itself follows the classic engineering problem solving process: break the problem into small parts; solve the individual small parts; then combine the individual small components into the solution of the larger problem, checking progress along the way. As an example, follow the process of fitting up a modest-scale gazebo with queenpost trusses incorporated in the roof framing.

The truss shown in Figure 9.1 is composed of a series of mortise-and-tenon joints, some more complex than others, but each tenon and each mortise has a designed set of governing dimensions. Checking that the mortises and tenons are cut to dimension is the first step in fitting the truss.

Tenons are checked for thickness, width, and length. Slotted plywood gauges or two framing squares clamped together are common examples of tenon checkers (Fig. 9.2). The gauge slides onto the tenon until it meets resistance. The thick or wide area is marked with a pencil. A plane or a slick can be used to remove excess material as marked, the gauge is tried again, and the process repeats until the gauge slides against the tenon shoulder.

Paul Magann

9.1 Assembled queenpost truss.

Once the reference side of the tenon is flat and parallel to the reference face, care must be taken that material is only removed from the off-reference side or edge of the tenon. Tenon length is also checked. Common practice is to cut the tenon ¼ in. shorter than the specified mortise depth, avoiding the possibility of the tenon bottoming out before the tenon shoulder makes full contact.

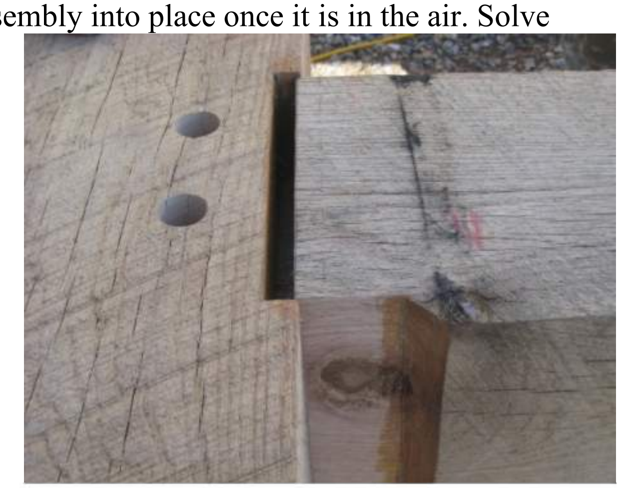
Cindy Mullen

9.2 Tenon checker in use.

Similarly, mortises are checked for depth, width, and length. Depth can be checked with a combination square. The 2-in. blade or the 1 ½-in. tongue of a framing square or a 2-in. or 1 ½-in. chisel are useful for checking mortise width. A tape measure checks length.

The most common difficulties with mortises are sides sloping inward, not being cleaned completely square to full depth, and insufficient length, particularly at the back side of angled braces. Often a small bit of wood in the bottom of the mortise is what keeps the joint from going home.

Finally, a perfect 2-in. tenon does not actually fit into a perfect 2-in. mortise. There must be a bit of clearance to allow the joint to slide home. A couple of plane shavings from the side of a tenon are all that is needed. If the joint cannot be completely assembled by hand (Fig. 9.3) during fit-up, it is too tight. It only gets harder to beat the assembly into place once it is in the air. Solve these problems on the ground.



Cindy Mullen

9.3 Mortise and tenon sliding home by hand.

Once the individual mortises and tenons have been checked for dimension, the joints are test fitted individually (as in Figs. 9.4–9.7) before the entire truss is assembled. Trying to fit the entire truss together before ensuring that the individual joints fit leads to frustration. Something is wrong, but which of eight or ten possible joints is the culprit? Work through the entire assembly joint by joint to confirm that each one fits. Then test fit the whole truss (Fig. 9.8).

Cindy Mullen

9.4 One queenpost fitted, another being tested.

Cindy Mullen

9.5 Test fitting a straining beam between queenposts.

After the entire assembly fits, overall dimensions are checked (Fig. 9.9). It is possible that some adjustment of squareness will be needed. Possibly a timber is slightly off in length, causing the assembly to rack.

Paul Magann

9.6 Test fit of strut in truss bottom chord.

Paul Magann

9.7 Test fit of a strut to a queenpost and bottom chord.

Paul Magann

9.8 Test fitting the entire truss.

Once the overall fit of the truss is confirmed, the various pieces are fit to the rest of the frame. The truss pieces are then used in the process of test fitting the rest of the frame. The usual procedure is to fit all parallel sections of the frame, such as the walls, and then fit the assemblies perpendicular to the walls (bents). Fitting can then be accomplished in two dimensions on the ground or on sawhorses, instead of in three dimensions in the air, making the process much easier and safer. Assemblies must be shimmed so the various pieces are coplanar. Any misalignment between pieces complicates fitting the joinery.

Cindy Mullen

9.9 Checking overall dimensions.

Figure 9.10 shows the purlin line that runs perpendicular to the queenpost truss previously fitted. The queenposts from the four trusses in the frame are now being fitted with their braces into the purlin line. Again, joints are tested for fit individually before the entire section is fitted.

Cindy Mullen

9.10 Test fitting queenpost truss into a full-length roof purlin assembly.

A successful raising assembles the frame in a smooth series of lifts, with each assembly fitting easily into place (Fig. 9.11). Done properly, every piece should only be lifted once and easily set into place. If a piece returns to the ground for further touch-up, then the test-fitting process was not entirely successful. The raising should be boring but satisfying (Fig. 9.12). Drama means something went wrong.

Paul Magann

9.11 Prefitted assembly setting easily into place.

Cindy Mullen

9.12 All fitted.

Rigging and Raising

The desired result of any raising is to assemble the frame safely without damaging the crew, the components of the frame, or creating any excitement. Raising done well is boring. The crew is positioned for effective work while also staying safe. Assemblies are rigged to lift without destructive stresses, and everything fits the first time because pre-fit has been done carefully.

Crew safety is a fairly standard, but extremely important, process no matter what is being raised. Coverage of crew safety can be found in references such as Rosnagel, Higgins, and McDonald (2009).

Truss raisings differ from raising individual pieces in that an entire assembly is picked up. Trusses in their final position in the structure primarily resist downward (vertical) loads while being supported at the lower, outer ends of the truss. Members and joinery are sized to resist the vertical loads, with much of the joinery typically working in compression. Commonly, a truss is raised by lifting from a point or points near the center of the upper part of the truss. Raising often is the only time that the compression joinery in the truss endures tensile loads. Joinery details or temporary rigging must be sufficient to resist these one-time load reversals. Frame sections are also commonly assembled lying flat on the ground or on sawhorses and must safely be rotated to vertical prior to lifting.

Primary information for planning a lift is the total weight of the assembled component and its center of gravity (CG). The lifting apparatus must handle the weight safely at the maximum height and radius of the lift. Rigging must be located in proper relation to the center of gravity, so that the load hangs in the proper position to be easily and safely set on the frame.

Both weight and center of gravity can be estimated from the frame drawings and materials list. Total weight is the sum of the weights of the individual members in the truss. Individual member weight can be calculated as volume multiplied by wet unit weight of the material, assuming unseasoned material. The weight calculations can be set up in a spreadsheet, such as that shown in Figure 9.13.

Once the weight spreadsheet is developed, some additional calculations can easily be performed to find the CG of the assembly. Back to basic statics, location of the CG from a reference axis can be calculated as:

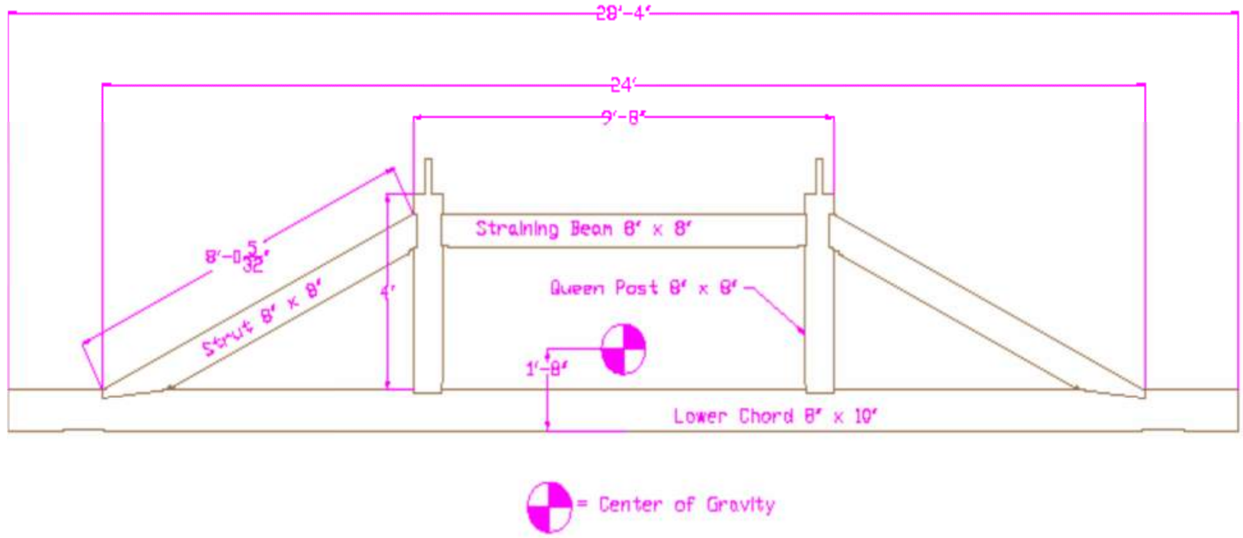
$$\frac{\sum[(\text{individual piece weight}) \text{x } (\text{distance of CG of the piece from a reference axis})]}{\sum(\text{weight of all pieces})}$$

The summations of weight, the (weight x distance), and the final division can be added to the weight calculation spreadsheet if desired.

As an example, Figure 9.13 illustrates the weight and CG calculations for the queenpost truss used as an example in the previous discussion of fit-up. A wet unit weight of 55 pounds per cubic foot was used, based on measurements from the actual timbers. The reference axes for the calculations were set at the lower left corner of the truss lower chord. Results estimate truss weight at 1,656 lbs. The CG is located 170 in. to the right of the left end of the lower chord and 20 in. above the bottom of the lower chord. The horizontal centroid is at the center of the assembled truss, which is half the length of 28 ft., 4 in.; not surprising as the truss is symmetrical about its central Y axis.

Queenpost Truss						
Weight and Centroids						
#1 white oak						
rough sawn, full dimension						
white oak @	55	lb/ft^3				
Weight calculation						
Piece List (individual pieces by location)						
name	**number**	**width**	**depth**	**length**	**Volume**	**Weight**
		(in)	**(in)**	**(ft)**	**(ft^3)**	**(lb)**
Lower Chord	1	8	10	28.33	15.74	866
Left side strut	1	8	8	8.00	3.56	196
Left side queenpost	1	8	8	4.00	1.78	98
Straining Beam	1	8	8	8.33	3.70	204
Right side queenpost	1	8	8	4.00	1.78	98
Right side strut	1	8	8	8.00	3.56	196
				Total weight, lb		**1656**
Centroid Calculation						
Y distance measured up from bottom face of lower chord						
X distance measured from left end of lower chord						
name	**Weight**	**Y center**	**Weight x**	**X center**	**Weight x**	
	(lb)	**(in)**	**Y center**	**(in)**	**X center**	
			(in-lb)		**(in-lb)**	
Lower Chord	866	5.00	4,329	170.00	147,174	
Left side strut	196	31.50	6,160	69.00	13,493	
Left side queenpost	98	34.00	3,324	116.00	11,342	
Straining Beam	204	49.00	9,981	170.00	34,628	
Right side queenpost	98	34.00	3,324	224.00	21,902	
Right side strut	196	31.50	6,160	271.00	52,996	
Totals	**1656**	**(lb)**	**33,279**	**(in-lb)**	**281,535**	**(in-lb)**
		Y centroid	**20.1**	**X centroid**	**170.0**	
		(in)			**(in)**	

9.13 Spreadsheet calculation of weight and center of gravity for a queenpost truss.



9.14 Queenpost truss dimensions and center of gravity.

As shown in Figure 9.14, the vertical centroid, at 1 ft., 8 in. above the bottom of the lower chord, is closer to the bottom of the truss than to the 4 ft., 10 in. height from the bottom of the lower chord to the top shoulder of the queenposts. Again, this makes sense as the bottom chord accounts for greater than 50 percent of the total truss weight.

Lifting the truss from the center of the straining beam will ensure that the CG is below and centered under the pick point, thereby properly orienting the truss for installation. But, because the final loading of the truss puts the struts and straining beam in compression, those members only have 1-in.-long locator tenons on each end. The load reversal (tension) from the upper, center lift point could cause the short tenons to disengage and the assembly to fall apart.

Several items both during fabrication and as temporary reinforcement for raising prevent the truss from falling apart during installation. Primarily, the two yellow ratchet straps around the top of the straining beam and under the bottom of the lower chord shown in Figure 9.15 ensure that the queenpost cannot disengage from the lower chord and release the straining beam and struts. Secondly, the pegged mortise-and-tenon joints at the bottom of the queenpost provide some withdrawal resistance. However, in the example project shown, the rigger was unwilling to rely solely on the tension capacity of the pegged joints. Finally, the struts were cut ¼ in. longer than the calculated dimension. The extra strut length induced 1 in. of upward camber in the lower chord, meaning that the struts were already in compression due to the bend in the lower chord. All that said, the ratchet straps provide cheap and easy peace of mind.

Cindy Mullen

9.15 Rigging the queenpost truss.

Several other notes on the lift. The center pick point makes it easy to orient the truss as it spins on the lifting sling. To control the spin of the truss, tag lines are attached to each end of the bottom chord. The orange ropes are visible in Figure 9.15. Also, the tag lines keep the crew members orienting the truss safely out from under the load. One interesting lesson to instill in the tag line crew is that rope only works in tension and only in a straight line. It sometimes takes a while for that lesson to sink in.

As another example of rigging considerations in raising an assembled truss, consider the kingpost truss shown in Figure 9.16.

9.16 Kingpost truss dimensions and CG.

As discussed above, weight and center of gravity calculations can be performed in a spreadsheet. This calculation is a bit more complex, as numerous members are not square-ended. Length measurements were taken along timber centerlines. Center of gravity measurements for each timber were set at midlength of the centerline. The reduced-section kingpost is treated as three separate timbers. The whole process is a geometric averaging of the timber volume.

The weight of the kingpost truss calculates to 1,868 lbs., with the horizontal CG again on the centerline of the truss and the vertical CG 42½ in. above the bottom of the bottom chord.

Although the hole in the top of the kingpost looks like a convenient place to rig a sling, doing so would completely reverse the design loading of the truss members, so the following approach to rigging was used. The vertical CG of the truss is well below the horizontal line connecting the upper ends of the struts.

Lifting from below the struts (Fig. 9.17) compresses the rafters into the top of the kingpost. This is the same loading that the upper section of the rafters and the kingpost see in service. Also, the struts have 4-in. tenons into the underside of the rafters. This connection provides more than adequate shear resistance to prevent the slings from sliding up the rafters as the truss is lifted. The bottom chord is still creating tension in the lower sections of the rafters and the kingpost. The tension capacity of the pegged joints may be sufficient to resist the tension, but the 1,500-lb. capacity of the comealongs rigged to either side between the kingpost top and the bottom of the lower chord more than adequately support the 770-lb. bottom chord. Again, notice the tag line rigged for control at the right side of the truss.

Paul Magann

9.17 Rigging the kingpost truss.

Figure 9.18 shows two kingpost trusses in place on the wall plates.

Paul Magann

9.18 Kingpost truss set in place.

Both of the previous examples have dealt solely with the use of the vertical location of the center of gravity. The horizontal location of the CG was obvious, as the two trusses were symmetrical about the centerline. Knowing the horizontal location can be useful when dealing with a nonsymmetrical load. Figure 9.19 shows a nonsymmetrical assembly being lifted above the horizontal centroid so that the assembly will fly level, allowing easy positioning.

The through tenon at the left end of the girt and the through spline connecting the main girt to the fly girt provide plenty of bearing to safely lift the posts. And a tag line is not needed as the bent

9.19 Rigging a nonsymmetric assembly.

9.20 Nonsymmetric assembly (picnic shelter) complete.

is being set on the ground, remaining under hand control of the crew. The completed assembly appears in Figure 9.20.

There are times when reinforcement beyond additional strapping or comealongs is needed to stabilize an assembly for lifting. Pictured in Figures 9.21 and 9.22 is a wall section rigged to be lifted as a unit. The scarf joints in the top plate over each interior post provide natural hinge points in the top plate. Although the scarf joints are more than adequate to resist the vertical load of the common rafters, they are inadequate to resist the one-time horizontal load during the raising. A similar situation can occur in larger trusses where joints are required in the upper or lower chords.

Paul Magann

9.21 Strongback on wall section.

Paul Magann

9.22 Strongback seen from the other side.

Two strongbacks have been strapped across the scarf joints in the wall section to stiffen the scarf joints in the wall plate. The slings are attached near the ends of the strongback to make full use of the additional stiffness.

Notice in the photos that two purple slings are prepositioned near the center of the strongback. Once the wall assembly is stabilized, the forklift can be repositioned to catch the purple slings, taking the weight of the strongback before the ratchet straps are released.

Following are additional notes and details to be considered in planning a raising:

- Be sure the final destination is clear and prepared to receive whatever is being lifted. In the words of one of the authors, Ben Brungraber, "Never pick up anything until you know where you're going to put it down!"
- Again, in Ben's words, "When lifting trusses longer than forty feet, spinning a crane more than forty degrees or both, a quick walkthrough of the path with the operator, connectors, and tag-liner(s) is in order."
- The first bent of a frame is unstable when first set down. Before lifting and setting the first frame in an assembly, plan for how it will be temporarily braced. Common methods include framing lumber braced off to the ground or opposing cables to anchor points. An outgrowth of the need for temporary bracing is the desire to make the frame stable within itself as quickly as possible. Often, after two bents are set, connected, and pegged, the assembly is a stable, self-supporting box ready to accept and support additional pieces without the need for additional bracing.
- Once a load is set, how is the rigging to be released? Plan for appropriate scaffolding, person lifts, or even radio-controlled hook releases.

One other detail before concluding: In Brion Toss's *The Rigger's Apprentice* (1998), he spends a fair amount of time on the proper treatment of the soft parts of a rig. Fiber rope and fabric slings are particularly susceptible to dirt. In his words, each grain of dirt is a small knife just waiting to cut the fibers in the rope or sling. When rigging is not in the air doing its job, it must be kept off the ground and out of the dirt.

Paul Magann

9.23 Expedient rigging rack.

Two different approaches to keeping the rigging clean are illustrated below. Figure 9.23 shows a simple, expedient rigging rack made by clamping a 2x4 between two posts. A more involved timber-framed rigging rack that can be disassembled and moved from job to job is shown in Figure 9.24.

An additional note about Figure 9.24: What looks like a common galvanized trash can in the lower left corner is actually a safe way of protecting 300 ft. of fiber rope strung as a five-part block and tackle. The can allows the tackle to be moved without dragging the rope in the dirt. And, as the rope is pulled to shorten the tackle, the tail rope is collected in the can, again without touching the dirt.

Also needed is crew training. A common crew reaction to unrigging a completed lift is to remove the slings and drop them on the ground. Proper practice is to immediately return unused rigging to the rigging rack.

9.24 A more involved, portable rigging rack. Note cans used to keep rigging and tackle off the ground.

In summary, rigging and raising can induce one-time stresses in trusses that the truss will never see again during its service life. Think through the process to determine what needs reinforcing for a safe lift. Be sure that the lift setup safely addresses the lifting stresses, proceed calmly and deliberately, and always, always put the safety of the crew first.

Recommended Reading

Beemer, W. 2016. *Learn to Timber Frame.* North Adams, MA: Storey Publishing.

> Will Beemer provides clear and concise instruction on the whole process of producing timber frame joinery, from initial layout to final fitting. A section of the book also covers joinery fit-up in useful detail.

Rosnagel, W.E., L. R. Higgins, and J. A. McDonald. 2009. *Handbook of Rigging for Construction and Industrial Operations*, 5th Edition. New York, NY: McGraw-Hill.

> This book provides a detailed look at rigging and safety practices in the industrial arena. Information is based on various building codes and industrial standards.

Toss, B. 1998. *The Complete Rigger's Apprentice*, 2nd Edition. Camden, ME: International Marine/Ragged Mountain Press.

It is highly recommended but not necessary to obtain a copy of *The Complete Rigger's Apprentice*. The wit, wisdom, information, advice, and humor contained in the book covering all phases of ropes, rigging, and knot work will last a lifetime. Brion spoke at a number of TFG conferences over the years and was always enjoyable. Unfortunately, Brion passed on in June of 2020.

The United States Army 1995. *Army Field Manual FM 5-125 (Rigging Techniques, Procedures and Applications)*. Washington, DC: Department of the Army.

Army field manuals are not at the cutting edge of industrial practice. The nature of military service is that personnel move through positions on a roughly three-year rolling basis. Manuals are written from the approach that while people move on, the procedures remain the same. The intent is that the new member of the unit can follow the manual and be just as safe and successful as the experienced member who recently transferred out. Procedures set forth in *FM 5-125* are safe and effective for the range of rigging situations described. And, the manual is a free download.

Chapter 10 – Repairs and Alterations

> *Understanding how the various loads are handled by the timber frame is crucial to designing the repairs. A good working knowledge of timber joinery is also essential both for understanding the configuration of joints in situ and designing the repairs.*
>
> —Jack A. Sobon (2011)

The purpose of this chapter is to provide useful guidance to engineers, architects, and timber framers who need to carry out work on existing trusses with a focus on considerations unique to trusses. Earlier tasks like assessment of the existing conditions and development of a repair philosophy are covered in TFEC 3 *Guide to Structural Evaluation of Existing Timber Structures*. Analysis of truss members and joinery are covered elsewhere in this design guide. The focus here is on tangible repairs and alterations, with examples taken from actual jobs (such as the Kicking Horse River Bridge, Golden, British Columbia Fig. 10.1) showing what has worked well and what has not.

Repairing timber truss forms, like other timber frames, requires a working knowledge of timber frame form, function, and joinery. This knowledge is a prerequisite for this chapter. Early in the project, a trusted timber framer who is capable of repairing timber trusses should be added to the team if such expertise is not already present.

Robin Zirnhelt

10.1 Kicking Horse River Bridge, Golden, B.C.

Although these concepts are applicable to a wide variety of truss forms, for clarity and simplicity they will be presented with the following assumptions:

- A traditional truss form with compression in the top chord or principal rafter.
- A truss that bears on a plate or the top of a bearing wall.

Repairs

This chapter presents an introduction to timber-truss repair concepts and best practices. All examples shown should be checked by the licensed design professional for the specific requirements of their project. Repair concepts should be discussed with the contractor to review access requirements and installation methods before designs are finalized. *Repair*, as defined by the *International Existing Building Code* (ICC, 2017), is the reconstruction or renewal of any part of an existing building for the purpose of its maintenance or to correct damage. Repairs are commonly needed for trusses that have been damaged by a variety of causes both natural and manmade. With certain exceptions, damaged elements are permitted to be restored to their pre-damage condition regardless of the extent of the damage (Martin, 2017).

By definition, truss repairs are related to changes in the capacity of the truss. Decreases in the capacity of the truss members or connections are normally the concern, and are typically the result of damage from a variety of causes, both natural and manmade. Frequent causes of damage include wood-decay fungi, insect attack, fire, vehicular impact, chemical attack, and cutting, notching, drilling, or boring of the truss members or connections. Trusses are commonly damaged by more than one of these causes acting at the same location in a given truss. Long-term water leakage often results in elevated moisture content in the timber that allows decay fungi and insect attack, such as that shown in Figure 10.2. It is usually helpful to understand the root cause of the damage so that the source of the problem can be eliminated. For example, repairing the decay in the top chord of a truss is nearly pointless if the roof leak above is not found and fixed. Similarly, a tenon with relish failure should not be replaced by another tenon of the same design.

10.2 Truss decay found during reroofing at Center Church on the Green, New Haven, Conn.

The events that cause the decrease in capacity can occur over widely varying periods of time. The duration of the causal events can range from an explosion lasting a fraction of a second to wood-decay fungi active occasionally over centuries. Long-duration events are typically harder to appreciate and are more likely to be overlooked during in situ assessment. TFEC 3 contains additional information about in situ assessment, nondestructive evaluation, and analysis of existing timber structures.

The strength of the wood itself is also time dependent. Wood can resist higher short-term stresses than long-term stresses. Wood trusses that have stood for decades or centuries can fail unexpectedly, absent external loads on the roof. After ruling out decay, insect attack, chemical attack, and other latent mechanisms, the engineer should analyze the stresses in the members and connections. Wood members and connections subjected to high stresses over extended periods of time may experience *creep rupture* (also known as *damage accumulation*). Creep rupture is not regarded as damage in the same sense as that from decay, insect attack, or impact, but rather as a normal effect of long-term loading regimes (Searer, 2011). The probability of failure increases for truss members with dead-load-only demand-to-capacity ratios (DCRs) at or above 1.0. Damage accumulation models for wood structural members under stochastic live loads indicate that "practically all damage occurs when the live load intensity is equal or nearly equal to the nominal live load" (Murphy, 1987). Historic allowable stresses for wood are often unconservative, leading to truss members that exist in a constant state of high stress with occasional periods of extreme stress when snow or live loads are present. This is especially true for allowable tension parallel to grain stresses for structures designed prior to 1971. Current NDS provisions should be used to calculate DCRs, as they more accurately reflect the relationship between allowable and ultimate strength and are therefore superior when assessing the probability of failure (Martin, 2018).

Alterations

Alterations, for the purposes of this guide, are defined as planned modifications to timber trusses necessitated by the desire to increase the capacity of the truss. Alterations are usually linked to increases in the demands placed on trusses that are otherwise performing normally. Alterations are frequently voluntary. Note that IEBC requires the licensed design professional to categorize the project as a repair or alteration for the purposes of permitting; however, truss projects frequently require both repairs and alterations. In these cases, judgment is required to select an appropriate course of action for each situation. The guiding principle when making repairs or alterations is that the work cannot make the building less compliant than it was before the project. Alterations typically must meet additional requirements for new construction. Although IEBC differentiates between repairs, alterations, additions, and changes of occupancy, the concepts in this chapter are generally applicable to work on existing timber trusses whatever the reason. As usual, the designer of record should consult IEBC or their governing building code to confirm compliance with local requirements.

Increasing the strength or stiffness of an existing truss is commonly required when the loading on a truss is expected to exceed its capacity or the expected deflections exceed serviceable limits. Commonly encountered situations include changes that increase the amount of snow that collects on the roof, adding rooftop equipment, changing the occupancy of the floor or attic space supported by the truss, and correcting latent defects in the truss.

Changes that increase the loading on existing trusses deserve careful consideration but are frequently overlooked. Usually, more snow will be present on cold roofs than on warm roofs, as is reflected in the thermal factors of ASCE 7 (2016). Adding insulation to the roof or in an attic tends to decrease the wintertime temperature of the roof and thereby increases the snow that accumulates on the roof. Switching single-pane glazing to insulated glazing units in large rooftop skylights can cause a similar effect. The potential for snowdrifts on lower roofs can be created by the addition of a higher portion of the same structure or an adjacent structure. Addition of large rooftop equipment or parapets can also cause new snowdrifts to form in the shadow of the obstruction that is created. Adding snow retainage devices to sloped roofs or switching from a slippery to a nonslippery surface (e.g., metal roofing replaced with asphalt shingles) can increase the snow loading on existing roof trusses.

Shoring, Jacking, and Preloading Considerations

Shoring the top chord is sometimes infeasible—for example, when it would be in the way of the repair work, when the owner's operations cannot be interrupted, or if the owner desires to preserve an ornate plaster ceiling to the greatest extent possible. If the bottom chord is to be shored, supplemental lateral bracing of the bottom chord may be required to prevent the web members from tilting.

Lack of redundancy in trusses often precludes localized removal and replacement of individual truss members without significant shoring or external stabilization. Trusses should ideally be shored at their top chord. Shoring should be designed so that it is as close to the truss panel points as possible without getting in the way of the repairs (Fig. 10.3).

John Cocca

10.3 Truss shoring continued to top-chord panel points, Poughkeepsie Train Station, N.Y.

The line between shoring and jacking is often blurred, but the distinction is important to understand and discuss with the project team when repairing trusses. Shores that are tightened snugly may be enough to arrest further displacement or collapse of a truss that has failed (Fig. 10.4), but jacking is typically required to return the broken truss to its rightful position before repairs are installed (Fig. 10.5). The rightful position is theoretically the position the truss would take if lying flat on the sawhorses.

10.4 Shoring a scissor truss.

10.5 Ellis screw posts used to jack metal plate connected trusses.

For repairs performed in situ, this position is above the resting position of the truss by an amount equal to the long-term dead-load deflection of the undamaged truss. A straightforward way to approximate this condition is to evenly jack the top chord until the truss lifts slightly off its bearings. This process resets the forces in the truss sufficiently that the repaired truss members and joinery take their share of the load when the jacking forces are removed. Jacking the trusses back to this position may be unfeasible or undesirable if damage is likely to result, in which case the truss with its failed member removed from the model should be analyzed for the deflection presently imposed. The resulting forces and moments can then be used as the initial step for modeling the completed truss carrying all subsequent loads. This type of analysis can get complicated quickly. At a minimum, the bending stresses in the top chord should be calculated based on the deflection (crook) that will be locked into the repaired truss. Curved truss members should be evaluated considering the secondary stresses resulting from the centroid of the member deviating from the stick model of the truss.

Careful consideration should be given before any truss-jacking operation is begun. These considerations include:

- Is the resulting damage to interior finishes acceptable?
- Are there mechanical, electrical, plumbing, or fire protection systems that may be damaged?
- Will the truss heels be lifted off their bearings? Would this leave any leaning masonry walls free to collapse?
- Will there be stress reversals in the truss? Special attention should be paid to truss members that may buckle when flipped from tension to compression.
- How will the jacking force be monitored? Ideally, from a safe distance or location.

- How will jacking progress be monitored? The benchmark for elevation measurements must be independent of the jacking operation.

The engineer should be on-site for the jacking operation if it will be complex or potentially dangerous. There should be clear channels of communication, and any member of the project team should have the authority to pause the jacking operation.

Methodology

Repairing timber trusses differs from repairs elsewhere in post-and-beam frames. These differences are generally due to the nonredundant nature of the truss form and the higher demand-to-capacity ratios frequently found in truss members and joinery. The risks are generally higher when repairing trusses as compared to post-and-beam frames. Failure of individual truss members or joints can lead to substantial deflection or collapse of the truss. Failure of principal tension members or their joinery may leave the principal rafters, i.e., top chords, free to thrust their abutments apart. This can cause shifting of the wall plate or lean in the wall itself. Progressive collapse may result if the adjacent trusses are left without lateral bracing along their compression chord or are unable to carry the additional loads of the roof and purlins left hanging above the failed truss.

Higher demand-to-capacity ratios in truss members as compared to other frames typically rule out many of the traditional joinery repair techniques such as scarfs and patches. Truss members are also more likely to carry high axial loads along their full length, limiting the amount of material that can be removed for repair joinery and shutting out the possibility of locating joinery in regions of low stress. Scarf joints in the top chord may be possible if they are carefully located at inflection points and the shifting of the inflection point has been analyzed for unbalanced loading, such as snow drifting across the ridge. Repairs that introduce joinery along the length of compression members should be kerf-cut if possible and should carefully consider the influence of that joint on the buckling strength of the member. Kerfing the joint is a process whereby two pieces to be mated are brought to bear and a saw is run down the joint between the two, thus providing two matched surfaces for good, even bearing. Long scarf joints, including bolts of lightning or fishtail splices, are found in the lower chords of covered bridges, however, and there are more practical alternatives for repairing tension chords if additive repairs, especially metal, are permitted.

> *The assistance of metal at tension joinery in trusses is both venerable and desirable.*
>
> —Jan Lewandoski

Trusses favor additive repairs and alterations, thereby avoiding the need to dismantle the truss or install shoring or falsework in most cases.

Additive measures include:

- Sistering members with full-length supplemental members.
- Adding fishplates either in wood or metal to splice a broken tension member, carry load past a large knot, or augment an existing splice.
- Installing supplemental tie rods along bottom chords or between the heels of raised chord trusses.[2]
- Installing supplemental steel strapping, barrel nuts, and threaded rods at connections.
- Sistering blocking to compression members to enlarge bearing interface.
- Adding check braces or sistered blocking opposite compression diagonals.
- Adding fasteners to reinforce shear planes parallel to grain (post joggles, truss heels, tenon relish, corbels, bolt tear-out, or block shear concerns).
- Adding fasteners to resist splitting or tension perpendicular to grain (stitch screws at notches, splits, and areas of high beam shear).
- Adding U-straps at the connection of king- or queenpost to the bottom chord.
- Adding bolts or external stirrups to anchor the top chord to the bottom chord at the truss foot.
- Adding kingrods to scissor trusses lacking a center vertical.[3]
- Installing wedges or blocking to take up gaps at compression joints.
- Adding trusses between existing trusses.
- Adding bridge trusses perpendicular to the existing trusses.[4]
- Addition of wind bracing in the plane of the roof.
- Addition of a diaphragm truss.[5]

Adding metal hardware and fastenings is often the most efficient way to repair trusses; however, metal may be inappropriate or damaging in the following circumstances:

- When the appearance would be objectionable.
- If other, more reversible, repairs are possible in structures of historic significance.
- If the perpendicular-to-grain spacing between the outer rows of the fasteners is large enough to induce splitting in members because of restraint of drying shrinkage or volume change due to seasonal changes in moisture content.[6]

---

[2] Stacks of Belleville disc springs or conically shaped washers can be used to maintain tension in the rod with the changing of the seasons.
[3] See chapter 2, "Historical Development."
[4] Bridge trusses provide redundancy and can permit two-way spanning of the space.
[5] This unusual intervention was added to an 1876 synagogue for wind bracing of the 16:12 pitch roof. A bolted steel truss was laid flat and spanned 72 ft. between gable end walls. (Adding buttresses to the masonry bearing walls may have been a more direct solution but was ruled out.)
[6] This is typically not a problem if the distance is less than 5 in. or the moisture content of the member remains relatively stable.

- When condensation (e.g., dew) can form on the metal and become trapped against the timber or pool on horizontal surfaces of the timber. [7]
- When exposed metal fastenings would violate required fire-resistance ratings of the structure.
- When the spacing of the fasteners parallel to grain is large enough to induce perpendicular-to-grain splitting of the member when eccentric forces are applied to the fastener group.[8]

Other recommendations for metal hardware are covered in the next section.

Dos and Don'ts of Truss Repair

Although every truss repair project is unique, common themes can be identified by observing what has and hasn't worked on past projects. The concepts below are general and applicable to a variety of truss forms. They are not strict rules meant to address all possible truss configurations.

Do:	**Don't:**
	
Use repairs that minimize cutting, notching, and drilling of the existing truss members.	Drill holes that leave the remaining cross section inadequate to carry the expected loads. The photograph shows a vertical through bolt intended to repair a longitudinal split in the bottom chord of a bowstring truss. Instead, the misguided repair caused a brash tensile fracture in the bottom chord as a result of high tensile stress on the net section.

[7] This is typically only a concern in unconditioned spaces.
[8] This problem commonly presents itself when relative rotations between members are ignored or are unnecessarily restrained. Large metal gusset plates are the bane of timber trusses.

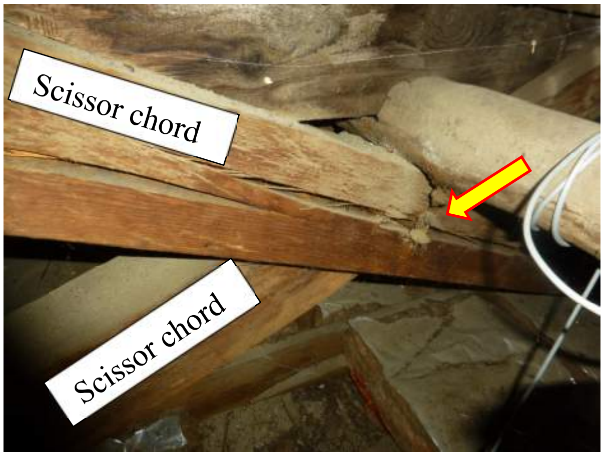
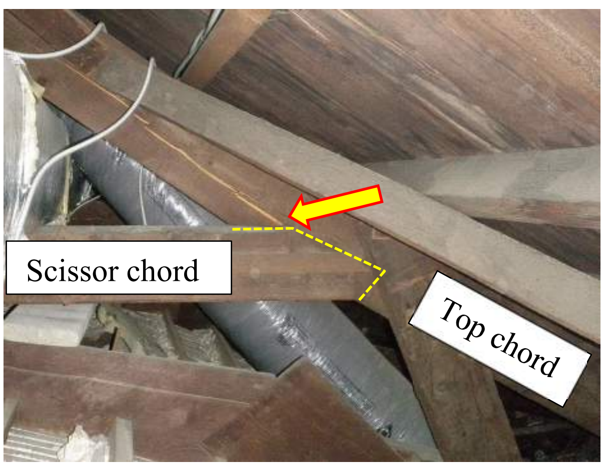
Do:	Don’t:
Use reversible repairs in historically significant structures. Steel should be padded off the timber if possible, as shown in the example below.	Use epoxy consolidants or spray foam in locations prone to moisture or decay, at seasoning checks, in bending members, in historic structures, or in structures that require fire ratings.
	

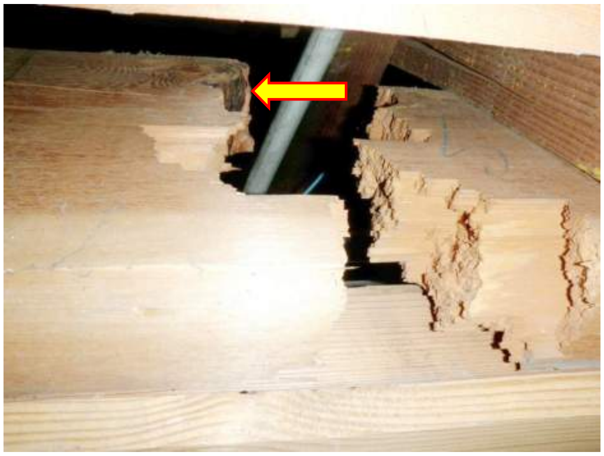
Do:	Don’t:
Consider the use of deformed (Bellville) washers on solid tension rods to maintain tension with changes in moisture content of the timber.	Use wire rope as tensile reinforcement. The initial stretch, elastic elongation, and end terminations make it more difficult to use effectively than solid rod with threaded ends.
	

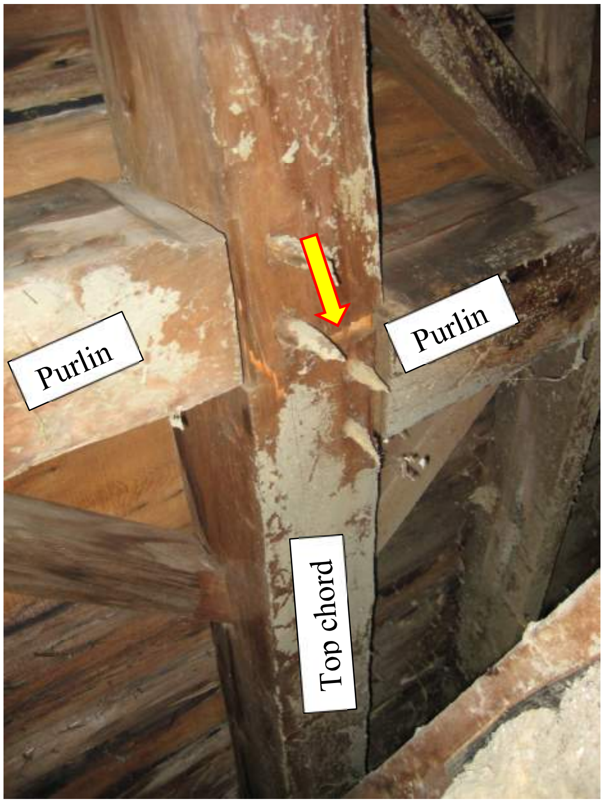
Do:	**Don't:**
Consider tension perpendicular to grain caused by fastener groups that extend along the length of the member. Consider member rotation and eccentricity of work points.	Ignore member rotation and eccentricity of work points. Don't use wide spacing between rows of fasteners when bolting steel to timber, or green timber to dry timber.
	

Common Problem Areas

Included below is a list of truss forms and joinery that are often problematic. The engineer should pay special attention to these areas if encountered.

	• Scissor trusses without a center vertical usually split at their crossing (as shown in the adjacent photo) and fail at their connections to the top chords.
	• Scissor trusses can fail at their connections to the top chord, or cause failure of the top chord in bending. • This risk is increased if notches are present in the underside of the top chord, as shown in the adjacent photo.
	• Bowstring trusses were commonly designed for balanced loading only. When subjected to unbalanced loading, such as snow drifting across the ridge, unanticipated forces in the web members develop. • Bowstring trusses commonly have web members that do not meet at a common working point. This configuration causes crossgrain tension in the chords when the truss is carrying unbalanced load.

	• Due to unconservative parallel-to-grain tensile stresses found in structures designed prior to 1971, brash fractures in the bottom-chord members are common at knots, slope of grain, and splice connections. The adjacent photo shows a brash fracture originating at an edge knot near to the top of the piece. Bowstring truss chords should be visually graded along their full length with no tolerance for below-grade members.
 	● Truss heels buried in exterior masonry walls are at a higher risk for moisture-related decay. This decay is often hidden by the masonry and more severe than anticipated. Sounding and resistance drilling may be used to check truss bearing locations for decay.

	● Truss chords with large housings for purlins or ceiling joists.
	● Trusses concealed in the ceiling of natatoriums or other high-humidity spaces.

	● Truss heels with short relish distances. ● Failure can result from horizontal shear stresses as well as tension perpendicular to grain.

Alternatives to Repair

Repairing or altering trusses are undertakings that usually require considerable planning, expertise, time, disruption, and money. Prior to the construction phase, the project team should periodically ask the engineer if there are any alternatives to repair. If the engineer refuses to answer this question a reasonable number of times, it is time for the owner to find another engineer. The design of repairs should be completed only after thoughtful consideration of the alternatives, discussion with the team, and elimination of those alternatives. Consideration should be given to the following, ranked generally from least to most invasive:

- *Better analysis:* This should be the first alternative considered. Reducing assumptions and refining the dead load calculation are wonderful ways to sharpen the analysis. Identification of the wood species seems basic but is frequently overlooked. Keep in mind that trusses often utilize different species in the same truss. Visual grading of the timbers is fundamental to establishing appropriate allowable stresses for in situ timber. Close examination and visual grading of the portion of the truss member that is highly stressed can result in a locally higher grade. In certain cases, allowable stresses published for green lumber may be increased for in situ (dry) timber. TFEC 3 includes a discussion of this topic.
- *Removing unnecessary dead loads:* This can be an effective alternative, especially if there are multiple layers of asphalt shingles or built-up roofing, abandoned plaster ceilings above drop ceilings, or storage loads in the attic.
- *Load posting, conditional occupancy:* This can include having a proactive plan to limit the live loads on the structure, safely remove snow from the roof, or evacuate when snow or wind loads are forecast to exceed allowable limits. Temporary shoring or bracing can be installed if plans are made well before the storm.
- *Load testing:* A good discussion of load testing of timber structures can be found in TFEC 3.
- *Cocooning:* This refers to reducing or eliminating environmental loads on the problematic structure by building a protective structure around or above it. It is an exotic

intervention that is usually relevant only if systemic defects exist in a historically significant structure.

- *Long-term shoring:* This is an enticing alternative if the root cause of the deterioration can be corrected and use of the structure is not impeded. Long-term shoring in buildings that are to remain occupied can severely limit use of the structure, impede fire egress, and violate fire ratings of the structure.
- *Abandonment:* This should be the option of last resort, since the structure is vulnerable to ongoing deterioration and eventual collapse. Access to the structure must be prevented so that trespassers are not injured.

References

International Code Council. 2017. *International Existing Building Code*. International Code Council.

Martin, Z. 2017. "Current Code and Repair of Damaged Buildings." *STRUCTURE Magazine*, February, 22–24.

Martin, Z. 2018. "Examination of Wood Truss Failures by Creep Rupture," *Proceedings*, Eighth Congress on Forensic Engineering. 930–939. Reston, VA: American Society of Civil Engineers.

Murphy, J. 1987. "Damage Accumulation in Wood Structural Members Under Stochastic Live Loads." *Wood and Fiber Science* 19 (4), 453—63.

Searer, G. R., T. F. Paret, and R. J. Kristie. 2011. "Contextualizing the Load Duration-Dependent Strength of Wood Members," *Proceedings*. Structures Congress. Reston, VA: American Society of Civil Engineers.

Sobon, Jack A. 2011. "Joinery Repair Techniques" *Timber Framing Fundamentals*, Alstead, New Hampshire: Timber Framers Guild.

Postscript

The timber trusses built between 1820 and 1920 are often awe-inspiring examples of craft, intuition, and construction savvy, having been designed and built in a time we would consider today to be decidedly “low tech.” Though they were utilitarian structures, they inspire us with their demonstrated level of skill, their powerful effectiveness, and their unselfconscious aesthetic. Having these historical prototypes as a basis for our study and benefiting from improved analytical capabilities, new connection technologies, and construction equipment, we are able to design and build trusses today that can also be works of art and craftsmanship.

When a timber truss is designed as a collaboration between an experienced engineer, a visionary architect, and an accomplished timber framer, the result can be a masterpiece. Conversely, if one attempts to design a timber truss or structure that is unconventional, while ignoring history, craft, and engineering principles, the outcome can be a failure in terms of both economics and performance.

While “form follows function” may no longer be architecturally in vogue, when it comes to designing a timber structure, “form follows function” is crucial. A timber truss can’t be delicate, elegant, and beautiful unless it is also structurally efficient.

We close this design guide with some thoughts that extend well beyond the scope of timber trusses but may serve to illustrate the perspectives of those who contributed to its development.

On Wood

Wood is beautiful, plentiful, and sustainable. Even as we use heavy timbers for our trusses, we can imagine they will eventually be replaceable by the next growth of timber, depending upon how big they are and how long we are willing to let their replacement grow. The following excerpt from Stewart Brand’s book *How Buildings Learn* (1994) is credited to anthropologist and philosopher Gregory Bateson:

> It seems that an entomologist took a penknife into the attic of the great dining hall at New College, Oxford, a building that dated from the late 14th century. The dining-hall ceiling featured large oak timbers, recounted as being two-feet square in cross section and 45 ft long. Digging with his penknife, the scientist found the timbers to be full of beetles.
>
> The College Council met to ponder the magnitude of this problem and wondered where they might get beams of that caliber. A Junior Fellow suggested that the College lands may include some oaks. The College had been endowed with pieces of land all across England, so the Council called the College Forester to ask about oaks.

> The College Forester reportedly expressed some wonder that they hadn't asked earlier. It seems that when the College was founded, a grove of oak trees had been planted for the express purpose of becoming, eventually, replacements for the dining-hall ceiling timbers when they eventually and inevitably became beetly. This plan had been passed down from one generation of forester to the next, each charged with the explicit instruction: "You don't cut them oaks. Them's for the College Hall."

Brand reportedly inquired as to whether a new grove was planted, in anticipation of a future need, but no one seemed to know.

For each of us writing and reading this book, consider that we can plant replacements for whatever timber we use.

On Craftsmanship

We have a long catalogue of efficient and attractive truss configurations to choose from, so it is seldom necessary to conceive a new form, unless solely for the sake of originality. We offer the following interpretations and excerpts from Richard Sennett's *The Craftsman*:

- At best, this document can only represent a portion of the explicit knowledge held by its contributors. Limitations in time and space restrict what we can and should present. More important and more vital to the success of any structural project is the tacit knowledge developed through experience and dedication to task. As a carpenter serves an apprenticeship, so too does an engineer serve an internship, both periods of learning and development being essential to development of his or her own tacit knowledge.
- "Ruskin sought to assert the claims of work that is neither amateur nor virtuoso. This middle ground of work is craftsmanship . . . The good craftsman is . . . absorbed in doing something well, unable to explain the value of what he or she is doing."
- "It might be better to focus on what makes an object interesting. This is the craftsman's proper conscious domain; all his or her efforts to do good-quality work depend on curiosity about the material at hand."
- "The craftsman constructing an object that seems simple and honest is as thoughtful—might we say as cunning?—as the craftsman contriving a fantasy."
- "The craftsman is a more inclusive category than the artisan; he or she represents in each of us the desire to do something well, concretely, for its own sake."

On Beauty

Finally, as the great French engineer Gustave Eiffel wrote, "Can one think that because we are engineers, beauty does not preoccupy us or that we do not try to build beautiful, as well as solid and long-lasting, structures? Aren't the genuine functions of strength always in keeping with unwritten conditions of harmony?"

References

Brand, S. 1994. *How Buildings Learn: What Happens After They're Built*. New York, NY: Viking Press.

Sennett, R. 2008. *The Craftsman.* New Haven and London: Yale University Press.

Appendix A – Analysis of an Ideal Truss

Since the primary load-carrying mechanism in a truss is axial force in the members, it is possible to make certain assumptions about a truss to simplify its analysis. A truss that satisfies these assumptions is known as an *ideal truss*. These assumptions permit us to use relatively simple concepts from statics alone to analyze a truss. Without use of these assumptions, truss analysis requires procedures that are not practical without the aid of a computer. This discussion is limited to two-dimensional (planar) trusses. The ideal-truss assumptions are:

1. All members of a truss are straight and can be represented by lines (they have no width).
2. Joints occur only at the ends of the members. Joints can be represented by points (they have no size).
3. All joints are formed by frictionless pins.
4. The weight of each member is applied at the ends of the member, or the member weight is negligible.
5. Only concentrated loads can be applied, and they are applied at the joints.
6. All members and loads lie in the same plane.

With these assumptions, each member of an ideal truss is subjected to axial force alone.

The analysis of a truss involves the determination of the forces in the members, the forces acting on the joints, and the forces exerted on the truss by its supports. Two analysis methods are widely used: the *method of joints* and the *method of sections*. The method of joints employs the concept of particle equilibrium, and the method of sections uses the concept of rigid-body equilibrium. The method of joints and the method of sections are not mutually exclusive. A combination of the two methods may result in the most efficient solution procedure. Additionally, each of these methods can be an effective tool for verifying computer-analysis results of trusses that don't satisfy the ideal-truss assumptions.

The Method of Joints

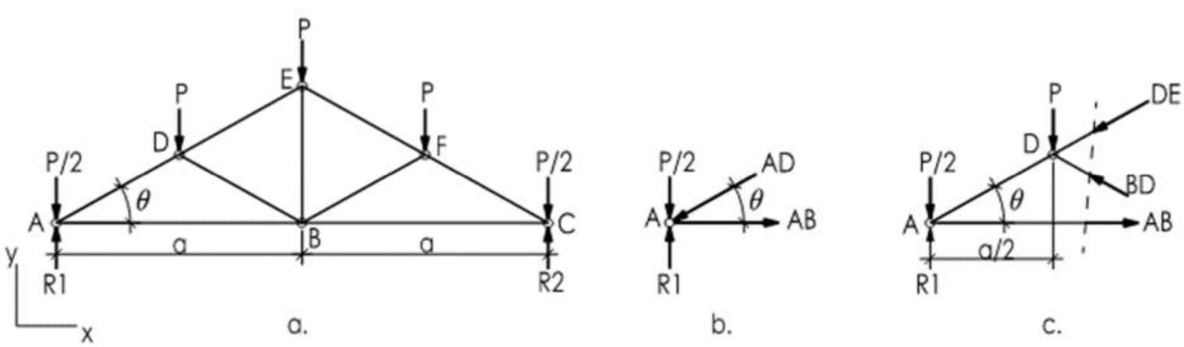
The method of joints is a method of analyzing a statically determinate truss by the application of the equations of equilibrium to the joints of the truss. There are two independent equations of equilibrium for each joint of a simple truss, such as the sum of all horizontal forces and the sum of all vertical forces. If these equations are sufficient to determine all the member forces and all the support reactions, the truss is statically determinate. If not, the truss is statically indeterminate.

In the method of joints, free-body diagrams of the joints of the truss are drawn, and the equations of equilibrium written for each joint, with the joints treated as particles. In a statically determinate truss, the solution of these equations yields all the forces acting on the joints, and hence the forces in the truss members.

Truss analysis by the method of joints involves the same techniques used to solve problems of particle equilibrium. Each joint in the truss is considered to be a particle. Since, the force in each member is aligned with the axis of the member, the force exerted by a member on a joint is directed along the axis of the member. Forces acting on the joint can be the result of the actions of members, the actions of a support, and the effects of concentrated loads. To analyze a truss using the method of joints:

1. Draw an accurate free-body diagram of the truss, showing all important dimensions and quantities, including angles. If possible, identify the zero-force members by inspection of the truss geometry and the loadings.
2. Select a joint to begin the analysis. If possible, choose a joint at which only two unknown member forces act. Recall that, for each joint, the equations of equilibrium provide for solution of only two unknowns in two-dimensional problems. Hence, it is convenient to select the joints in a sequence that allows you to evaluate one or more of the unknowns immediately, rather than collecting the complete set of equations for all the joints and solving them simultaneously.
3. If you cannot identify a suitable joint at which to start the analysis, it might be helpful to use rigid-body equilibrium equations to find the support reactions for the truss first. With the support reactions known, one of the supported joints might be a suitable starting point.
4. Write and solve the equilibrium equations for the selected joint.
5. Proceed to the succeeding joints and repeat step 4 for each until you have determined the member forces of interest.

As an illustration of the method of joints, consider the kingpost truss in Figure A.1a. The truss is symmetrically loaded, so reactions are easily determined as $R1 = R2 = 2P$. In Figure A.1b, joint A is isolated, and the forces acting on the joint are shown.



A.1 (a) Kingpost truss for analysis; (b) Free-body diagram of joint A; (c) Free-body diagram of section through members AB, BD, and DE.

The associated equilibrium equations are:

$$\sum F_x = AB - AD\ \cos\theta = 0$$

$$\sum F_y = R1 - P/2\ - AD\ \sin\theta = 0$$

With $R1$ known, the second equation gives the force in member *AD.* Then the first equation gives the force in member *AB.* Repeated use of the methods at joints *D, E, F,* and *C* gives the remaining member forces. Examination of joint *B* is not needed, since the reactions are known at the outset. Also, due to symmetry, it would be sufficient to examine just joints *A, D,* and *E* to analyze this truss.

The Method of Sections

If a truss is in equilibrium under the action of a set of coplanar forces, any part of the truss is also in equilibrium. In the *method of sections*, we divide the truss into two or more parts, each containing at least one member, by sectioning (cutting) through certain members of the truss. This sectioning does not disturb the state of equilibrium. Since each part of the truss is in equilibrium, it may be treated as a separate rigid body. Thus, we may draw a free-body diagram of a part of the truss, showing the forces in the members at the cut section. The problem-solving technique for the method of sections follows:

1. Draw a free-body diagram of the entire truss and write the equilibrium equations. Solve these equations to determine the support reactions.
2. Locate the members in the truss for which forces are desired.
3. Pass a section (a cut) through the truss to separate it into two parts. If possible, the section line should pass through no more than three members with unknown member forces. The section line need not be straight. It must merely separate the truss into two appropriate parts. In some cases, it may be necessary to cut more than three members to separate the truss into two parts. Each part of the truss must contain at least one complete (uncut) member. Otherwise only a single joint would be isolated, as in the method of joints.
4. Select one of the truss parts sectioned in step 3 and draw a free-body diagram. Unless you know otherwise, assume that the unknown member forces are tensile.
5. Write the equilibrium equations for the truss part selected in step 4. If it was necessary to cut more than three members with unknown forces in step 3, you may have to consider additional sections of the truss (or perhaps truss joints) in order to determine the unknowns. Hence, draw appropriate free-body diagrams of the additional parts or joints and write their equilibrium equations to obtain the same number of independent equilibrium equations as the number of unknowns.
6. Solve the set of equations derived in step 5 to determine the unknown forces.
7. Repeat steps 3–6 as required to complete the analysis.

To illustrate the method of sections, consider again the truss in Figure A.1(a). Again, symmetry reveals the reactions to be $R1 = R2 = 2P$. The dashed line in Figure A.1(c) cuts members *AB*, *BD,* and *DE* to isolate member *AD*. The corresponding equilibrium equations, determined from the free-body diagram in Figure A1(c), are:

$$\Sigma F_x = AB - BD\ \cos\theta - DE\cos\theta = 0$$

$$\Sigma F_y = R1 - P/2\ - P + BD\sin\theta - DE\sin\theta = 0$$

$$\Sigma M^D = AB(a/2\tan\theta) - (R1 - P/2)(a/2) = 0$$

Taking moments about joint *D* is convenient, as it results in only one unknown, *AB*, in the equilibrium equation. After *AB* is found, *BD* and *DE* are determined by simultaneous solution of the two force equations.

Appendix B – Software for Timber Frames and Trusses

The following reviews and comments are intended to provide basic information on appropriate software options for those considering purchasing a computer-aided analysis and design program. They are based on the cost (as of January 2021), applicability, and ease of use of the various programs and modules as they apply primarily to timber frame analysis and design.

In general, where there is a base price for a 2D program, a 3D program is also available at approximately twice the price of the 2D program. Additionally, where a base price is indicated, there are usually a number of add-on modules available for connection design in timber and steel, design in other materials, design of foundations, etc.

The list of programs is arranged in order of ascending price, as noted. All of the listed programs contain the full NDS database plus composite and glulam sections as well as ASCE7 and various code-required load combinations.

These remarks are intended to be introductory in nature and should invite the deeper exploration of the features and tutorials on each website. The list is not exhaustive but presents some of the most commonly used programs that are useful for working with timber frames and trusses.

Member Sizing and Simplified Frame Analysis

ClearCalcs (clearcalcs.com)

> ClearCalcs is a relative newcomer to the structural design market and includes a variety of modules for all aspects of smaller structural, low-rise project design including beams, columns, footings, walls, etc. It is an ideal replacement, and more, for the now-defunct StruCalc software. Each module has a simple, consistent input format. There is an extensive variety of customizable truss and frame designs, and you can create your own design by entering node coordinates and creating members by joint-to-joint definition. Their in-person customer service is excellent, and they are willing to modify, improve, and add to their design modules based on customer request and preference.

Enercalc (enercalc.com)

> Enercalc was started in 1983 as a Lotus 1-2-3 design package and has evolved nicely since then. The software has a somewhat broader scope and capability compared to ClearCalcs, with perhaps a bit more flexibility with an extensive number of well-developed modules. Their frame analysis modules are based on entering node coordinates and creating members by joint-to-joint definition, and they have a wide variety of basic standard trusses and frames that can be modified for analysis. One advantage over ClearCalcs is that line drawings of frames can be imported and then defined. Like ClearCalcs, the frame analysis is a bit tedious compared to VisualAnalysis, RISA 2D, and R-STAD. However, at three

times the price of ClearCalcs, it will be time well spent to do a thorough evaluation of these two packages before making your decision to purchase.

Comprehensive 2D and 3D Frame Analysis

VisualAnalysis (iesweb.com)

Popular with timber frame engineers, this software is well supported and includes a student edition. Like all of the listed comprehensive software, VA is easily utilized for beam, rafter, joists, and post design. It includes the NDS database and covers concrete and steel design as well. Additional modules, including connection design and foundations, are available. There are numerous pdf and video tutorials, and the software is relatively easy to learn. The more robust 3D packages are highly recommended for those looking to analyze all but the simplest truss forms.

RISA-2D (risa.com)

RISA and Woodworks have teamed up to support each other with an emphasis on modeling, analysis, and design of wood-framed structures as compared to heavy timber frames and trusses. Although there is an extensive pdf design guide, video tutorials are less plentiful than those found for VisualAnalysis. Subsequently, it may take a bit longer to learn. Similar to other software programs, upgrading to the 3D suite can often create a bit more diverse analysis capability. Straightforward 2D analysis can still be completed by adjusting the boundary conditions of the model.

RSTAB (dlubal.com)

For the purpose of timber frame and truss design, this software, albeit extremely powerful, may be a bit of an overkill, especially considering the cost. However, by incorporating their plate and surface analysis package, RFEM, this could be an ideal platform for mass timber structures and larger, complexly shaped structures.

SkyCiv (skyciv.com)

A subscription, cloud-based structural analysis software, with the ability to log into projects from any computer off one seat. Modeling is surprisingly smooth, even with basic internet connection, and a significantly better option than trying to work remotely through a single seat using a VPN. The workflow of the program is a bit unique when compared to the others listed here. But the intuitiveness of the modeling tools along with the plethora of video tutorials, make for a steep learning curve. The reporting options along with load-generation modules are also extremely useful for those who need to create detailed calculation summaries.

Appendix C – Design of Steel in Timber Frame Connections

Whereas the focus of this design guide is on timber trusses, principally using wood-on-wood joinery, use of steel connection hardware is unavoidable, especially for long-span trusses, such as those used in commercial applications. This appendix is intended as a review of some of the basic requirements for design of connections using steel apart from those covered in the NDS.

ASD vs. LRFD

Both the NDS and AISC 360 support the use of either allowable stress design (ASD) and load and resistance factor design (LRFD). In the timber engineering field, ASD is the more common approach, due in part to familiarity with the approach and the simplicity of the calculations. However, LRFD is more commonly used in the design of steel structures. LRFD methods for both steel and timber were calibrated to ASD designs, generally for components and structures subjected to a live-load-to-dead-load ratio of $L/D \approx 3.0$. When $L/D > 3.0$, ASD will generally result in a greater capacity (a more conservative design), and when $L/D < 3.0$, LRFD results in greater capacity (more conservative). When properly applied, either method is acceptable and should result in safe, economical designs.

Bolted Connections

For timber connections utilizing steel plates, once the size and number of bolts for a particular connection are determined based on the allowable loading for the timber species, the steel plates must be designed. There are two typical configurations for these connections:

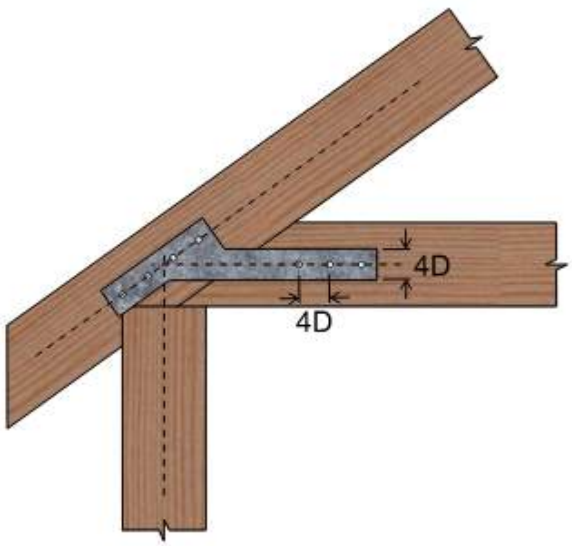
1. Knife plates (sometimes referred to as kerf plates) as the main member with bolts in double shear.
2. Side plates (also known as fish plates) with a timber main member, also with bolts in double shear.

The accepted specification for steel design, AISC 360 (2016) sets ASD limits on the steel tension member capacity for the following limit states:

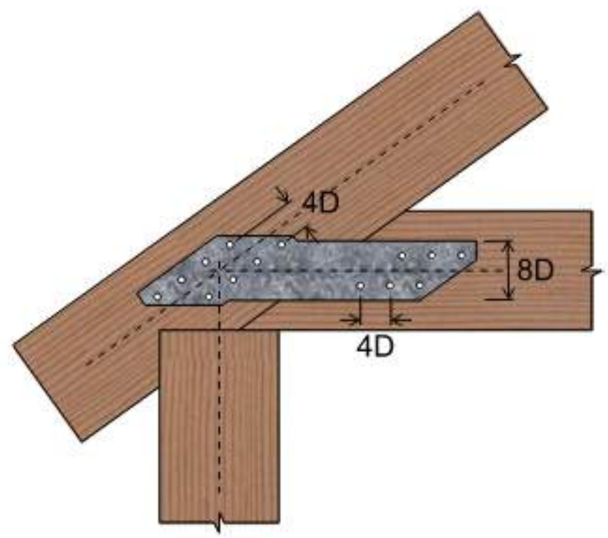
- Gross section yield: AISC Sec. D2(a)
- Net section rupture: AISC Sec. D5.1(a)
- Shear rupture: AISC Sec. D5.1(b)
- Bearing: AISC Sec. J7(a)

The strength of the bolt itself is not typically a consideration with timber, since the allowable loads on the bolt or tightly fit pin are generally controlled by the capacity of the wood, although the bolt stiffness does play a role in determining the timber failure mode. Nevertheless, bolt shear capacity should be checked to confirm that bolt size and grade are adequate. A307-grade bolts are typically adequate unless the loads are extremely high, as may be the case for truss connections in long-span, shallow trusses. The AISC section on bolted connections addresses the difference in bolt shear strength based on whether the threaded or the unthreaded portion of the fastener lies in the shear plane.

When selecting the preliminary size of plates containing a single row of bolts in green timber, try four times the bolt diameter in width and an end distance of two bolt diameters (Fig. C.1). For double rows of bolts (in timber with a moisture content at or below 19 percent), try eight times the diameter in width (leaving four diameters between rows) and an end distance of two bolt diameters (Fig. C.2).



C.1 Trial size of plate for one row of bolts in green timber.



C.2 Trial size of plate for two rows of bolts in dry timber.

Following this preliminary sizing suggestion, the above limit states must be checked. A review of Section D5 in the Commentary to the AISC *Specification* is helpful in understanding the terms and limiting dimensions of pin-connected members. To clarify terminology, AISC 360 regards a pin-connected member as one using bolts, rivets, or other connecting devices, whereas the NDS regards a drift pin or a drift bolt as a smooth dowel-type fastener without a head, nut, and washer.

For bolted connections with kerf plates, using 2-in.-diameter washers on 1-in.-diameter bolts with a $^{1}/_{8}$-in.-deep by $2^{1}/_{4}$-in.-diameter drilled counterbore is a nice architectural feature and prevents visible crushing of the wood fibers around the edge of the washer. (Note: Using standard washers should prevent visible crushing of the wood fibers so the counterbore may be just a precautionary detail and not actually required.) All-thread rod cut to a length with nuts on each end to provide two or three exposed threads is uniform in appearance from any view below, compared to bolts where the orientation of the heads must be considered.

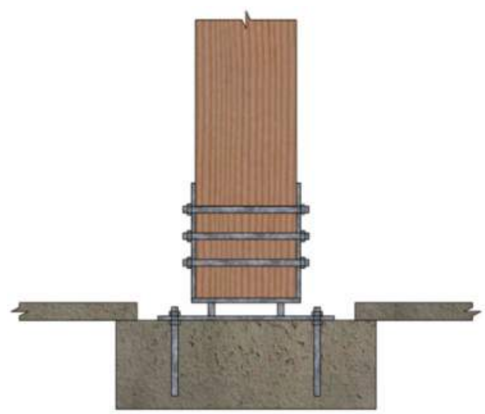
If metal side plates are to be used, the bolt shear capacity may be limited by the root diameter of the bolt (or all-thread) if the threads are not excluded from the shear plane.

Since the diameter D is a factor in the numerator of the yield equations, the limiting value for the nominal size can be adjusted proportionally.

An alternate to single or double rows of larger bolts in steel kerf or side plates involves the use of single or multiple kerf plates utilizing self-drilling screws. Connections of this type avoid the strength-reduction requirement of multiple rows of fasteners in a common plate in green timber (over 19 percent moisture content). They are also more forgiving of changes in moisture content and do not require the great precision in the predrilling of holes that must line up when using larger bolts or pins. And finally, the reduction in capacity for multiple bolts in a row is not a factor.

Welded Connections

Welded connections are often required for prefabrication and assembly of post bases. This requirement is especially true where a leveling plate, tube-steel stand-off, bearing plate, and side plates are used to develop moment-resisting connections (see Fig. C.3). The interconnections of the side plates, bottom plate, stand-off tube, and base plate require fillet welding to join the assembly.



C.3 - Single-axis post-base moment connection.

Critical design checks include the thicknesses of the base plates for the anchor bolts and the bottom plate below the timber post; side-plate or knife-plate tension capacity; anchor bolt tension capacity and, for the purpose of this discussion, the side-plate tension capacity at the connection to the post bottom plate.

If possible, welds should be limited to single-pass fillets, due to the higher cost required of larger, multiple-pass welds. Normally, $^{3}/_{16}$-in. welds for $^{1}/_{4}$-in. plates, $^{1}/_{4}$-in. welds for $^{5}/_{16}$-in. plates, and $^{5}/_{16}$-in. welds for $^{3}/_{8}$-in. plates are commonly used. Note that the maximum weld size is $^{1}/_{16}$-in. less than the plate thickness. If the plate is less than $^{1}/_{4}$-in. thick, the maximum weld size may be the same as the plate thickness.

The weld size is the width of the weld leg on the joined plate. The allowable loads are controlled by the throat dimension, which is cos(45°) times the weld size. For A36 steel plate with an ultimate stress of 58ksi, E70 electrodes are used to ensure that the capacity of the weld material is equal to or greater than the base metal.

References

American Institute of Steel Construction (AISC]. 2016. *Specification for Structural Steel Buildings*. Chicago: American Institute of Steel Construction.

Index