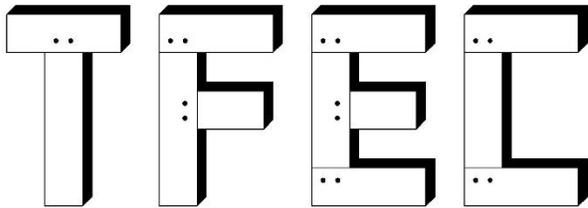


TFEC 4-2020

Design Guide for Timber Roof Trusses

August 2020



• TIMBER FRAME •
ENGINEERING COUNCIL

This document is intended to be used by engineers to provide guidance in designing and evaluating timber roof truss structures. Do not attempt to design a timber roof truss structure without adult supervision from a qualified professional (preferably an experienced timber engineer). The Timber Frame Engineering Council (TFEC) and the Timber Framers Guild (TFG) assume no liability for the use or misuse of this document.

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Background

Man has been building with timber trusses for over 2,000 years. The Romans were the first to perfect the art of spanning wide spaces with timber trusses. During the Medieval age, European cathedral builders used timber trusses to span over their vaulted stone ceilings to support the cathedral roofs above. In a few rare instances, such as 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 all pre-industrial 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. Often, the carpenter's intuition was flawed, leading to irrational or mongrel trusses, some of which have managed to survive.

With the industrial revolution and the expansion of the railroads in the second half of the nineteenth century, engineers began to play a role in the design of major structures. Many of the early engineers were West Point trained Civil War veterans. Trusses used for railroad bridges were at first based on patented designs. Every inventor or amateur engineer raced to patent his own unique truss design in hopes of making his fortune off of the railroads. Naturally, each one of them named their truss design after themselves to add fame to their anticipated fortune. Some of the patented truss shapes proved to be structurally efficient, smart truss designs that actually worked. Those are the ones with names that we recognize and still use today – Howe, Pratt, Town, Warren, and Fink. Other patented truss bridge designs were not so lucky and ended in catastrophic train wrecks.

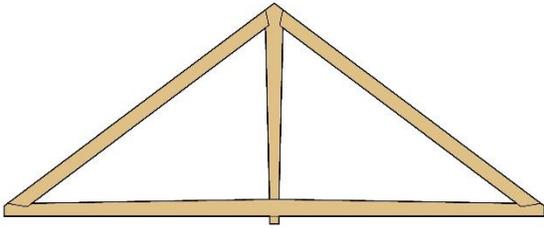
The industrial revolution also brought mill towns with large factories that manufactured textiles and everything else that a person could desire. The mill buildings had robust timber structures supported on thick brick or stone masonry bearing walls. The roofs of the mills often featured timber trusses that emulated patented bridge trusses.

While timber trusses from centuries past were built for function, today, timber trusses 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.

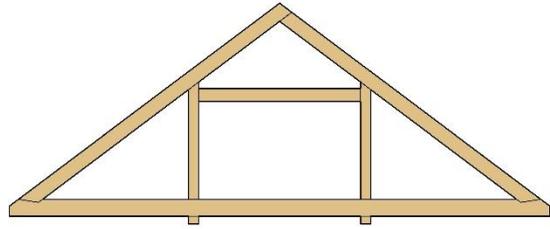
This document is intended to provide guidance to engineers designing, evaluating, and repairing timber roof trusses.



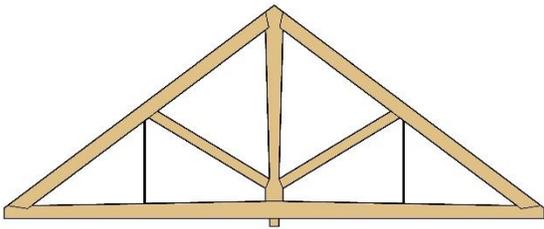
THE ARCHITECT LOUIS KAHN ALLEGED TO HAVE ONCE HAD A CONVERSATION WITH A BRICK. AS THE STORY GOES, HE ASKED THE BRICK “WHAT DO YOU WANT BRICK?” AND THE BRICK REPLIED “I LIKE AN ARCH.” HAD HE ASKED THE SAME QUESTION OF A TIMBER, THE REPLY WOULD MOST CERTAINLY HAVE BEEN “I LIKE A TRUSS.”



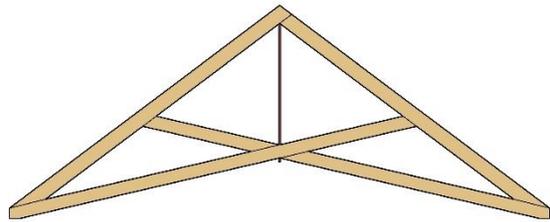
King Post Truss



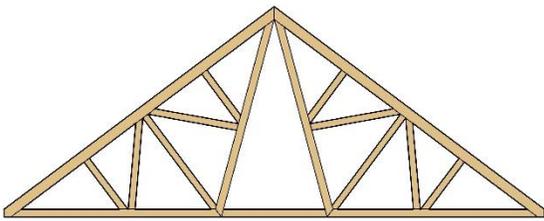
Queen Post Truss



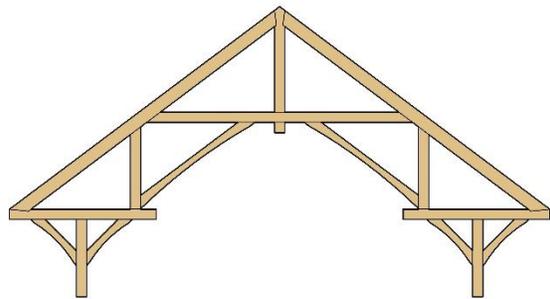
Howe Truss



Scissor Truss



Fink Truss



Hammer-Beam Truss

Common Timber Truss Types

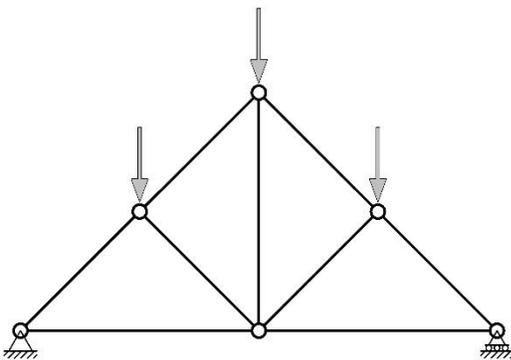
Truss Analysis

Ideal Trusses

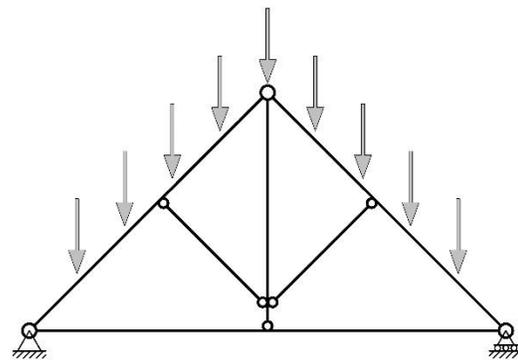
Trusses are structural assemblies that respond to applied loads with pure axial compression or tension in their members. The top and bottom truss members are called *chords* and the members between the chords are called *web members*. Web members that are in axial compression are called *struts*. Web members that are in axial tension are called *ties*.

In an ideal truss, members meet at *nodes* or *joints* (also called *panel points*) that are idealized as hinges or pins that are incapable of transmitting bending moments. Loads are applied to an ideal truss only at its nodes. Applying loads (and supports) only at nodes keeps the truss members shear-free. It helps, too, that ideal truss analysis tended to neglect member self-weight. The centroidal axes of all truss members meeting at a node converge to a discrete point.

Classical methods of analyzing trusses are only valid for ideal trusses, and real-world timber trusses are not very idealistic. Real trusses respond to applied loads with a combination of axial stress, bending moments, and shear in their members. Real trusses often have continuous chords that are not pinned at the joints and loads are often applied along the length of the chords. It is also common to intentionally introduce eccentricities into truss joints for more efficient joinery.



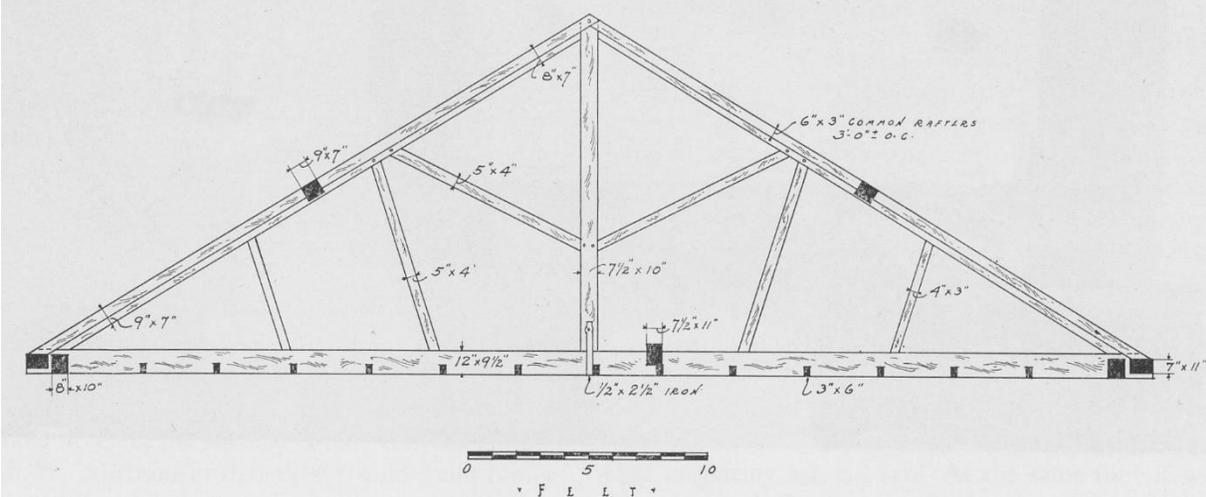
IDEAL TRUSS



REAL TRUSS

Not all structural elements that look like trusses actually perform as trusses. To qualify as a true truss, it must be capable of responding to loads applied at the nodes predominately with axial stresses in its members. If significant bending moments and shear forces are introduced into the members, it is not a true truss. For instance, a hammer-beam truss behaves very little like a truss. Similarly, a queen-post truss is not always a true truss

The truss shown below from the roof of the First Congregational Church in Canterbury, Connecticut, is not a true truss - it is a mongrel. The diagonal struts transfer roof loads to the bottom chord where they are resisted by bending. This is a blatant case of a carpenter's failed structural intuition.



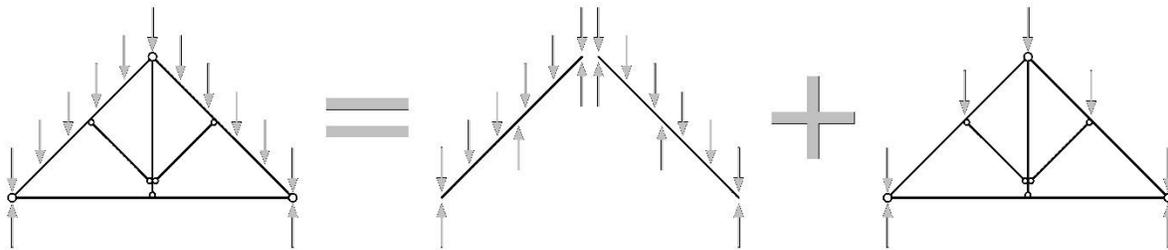
Source: *Early Connecticut Meetinghouses* by J. Frederick Kelly, 1948

Classical Methods

It has been said that an engineer only truly understands the behavior of a structure when he can analyze it on the back of a napkin or envelope. Classical methods of truss analysis are simple and reliable. All that you need is a pencil, a calculator, and of course, a napkin.

There are actually two classical methods for analyzing trusses – the *Method of Joints*, and the *Method of Sections*. Sadly, few engineering schools still teach these methods to their students. The principles are similar for both methods. A free-body diagram is constructed for each individual truss joint or a portion of a truss. The truss axial forces are algebraically solved to satisfy static equilibrium.

Classical methods are only valid for ideal trusses, so if you happen to be engineering a real truss with a continuous top chord supporting distributed loads, it becomes a two-step process. The first step is to analyze the top chord as a continuous beam with each truss joint treated as a rigid support. This will reveal the bending moments and shear in the top chord. As the struts and other web members are themselves elastic structures, this represents an approximation; an excellent one, but nonetheless imperfect. The second step is to apply the beam reactions from step one to an ideal truss as point loads at the joints. Then analyze the truss by one of the classical methods.

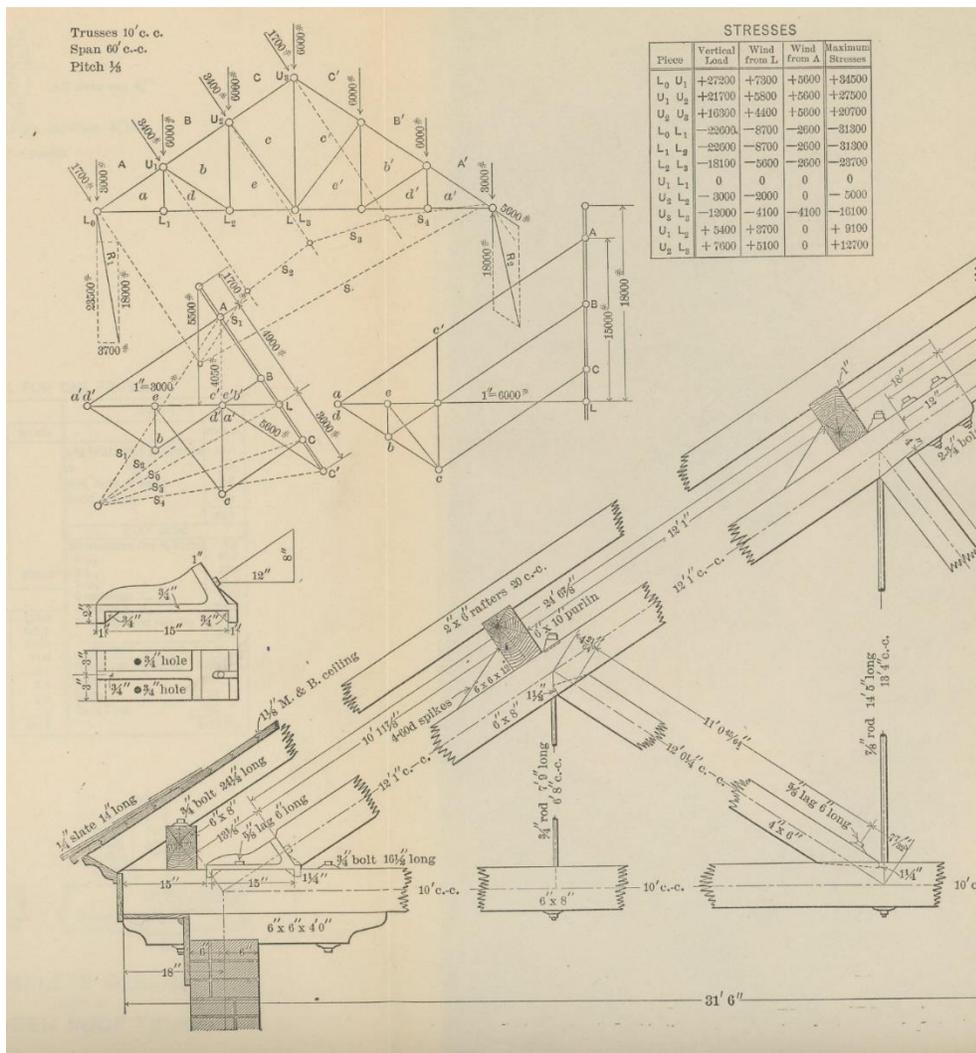


For a statically determinate truss, the analysis seldom takes an experienced engineer more than 10 minutes. As engineering practice has evolved to take advantage of computer methods, those idealizations can be set aside with models that incorporate the elastic properties of the members to provide more realistic answers. Nevertheless, hand calculations are a good check on a computer model and will quickly identify if there was an error in the modeling.

Graphical Method

At one time, the most common method of analyzing a truss was a graphical method referred to as a *Maxwell Stress Diagram*. This method harkens to an age when engineers drafted the drawings for their structures and sat at drafting tables. The stress diagram for each load case would be included on the truss drawings and was often required by the Building Official. The axial force in each truss member could be determined by scaling the length of the line representing a particular truss member in the stress diagram.

The Maxwell stress diagram was a very quick and efficient method once you mastered the technique. The popularity of the method faded in the 1970s with the introduction of the handheld calculator. Once personal computers and cheap analytical software were introduced in the 1980s, the method became extinct.

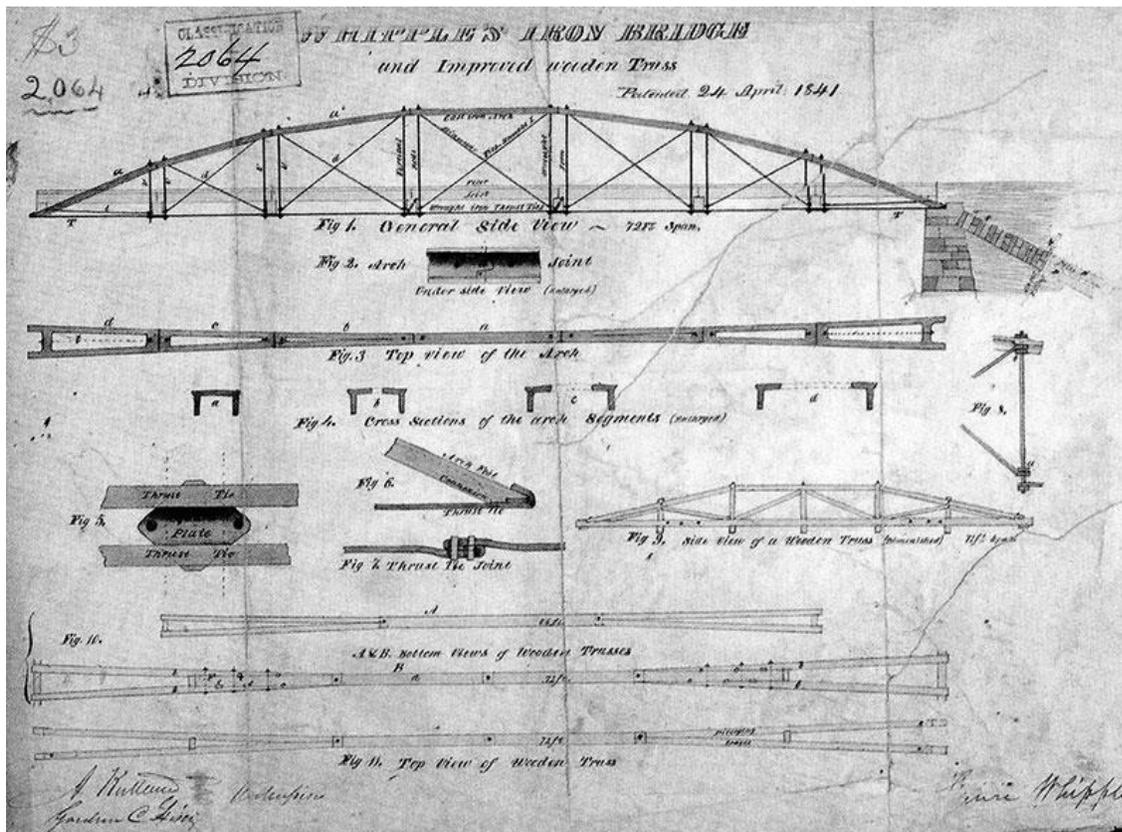


Timber Roof Truss drawing with Maxwell stress diagrams. Source: *The Design of Simple Roof Trusses in Wood and Steel* by Malverd A. Howe, 1903

Squire Whipple (1804-1888)



No authoritative treatise on trusses can fail to pay homage to Squire Whipple, who wrote the book on rational calculation of forces and stresses in trusses. In 1830, after graduating from Union College, Schenectady, NY, Whipple conducted surveys for several railroad and canal projects and made surveying instruments. In 1840 he invented a lock for weighing canal boats. In the next years he turned his attention to bridges and invented two new truss designs employing iron as well as timber. In 1853 he completed an iron railroad bridge of 146-foot (44-metre) span near West Troy (now Watervliet), NY. In the following year appeared his **Work on Bridge Building**, the first significant attempt to supply a theoretical means for calculating stresses in place of the rule-of-thumb methods then in general practice. The book, which he expanded and personally printed in 1869 under the title **An Elementary and Practical Treatise on Bridge Building**, facilitated the rational use of wrought and cast iron and was widely used in railroad engineering for decades.



Computer Modeling

There are currently several software options available for truss analysis, many of which also incorporate more sophisticated Finite Element Analysis (FEA) and even dynamic and non-linear modeling. The intense modeling involved in most FEA software tends to be overkill for the vast majority of timber trusses, particularly when compared to using method of sections to quickly find member axial loads by hand. Much as with drafting, analytical effort performed on the computer does aid in quick revisions, detailed record keeping, and investigating the structural complexity that commonly arises in modern design.



A truss can be easily and accurately modeled in two dimensions, but three-dimensional analysis is helpful when several similar trusses resist more than just vertical gravity loads - a common situation in open frame pavilions or where a truss serves to drag a lateral force between two shear wall segments. Ultimately, truss analysis software should be viewed as a tool available for developing proper member sizing and axial loads, which can be used for detailing joinery and specifying fasteners. Like all tools, operator knowledge, experience, and attention to detail are paramount for accurate and meaningful structural analysis with computer software. As with any powerful tool, in the wrong hands, structural analysis software can be rather dangerous.

Determining the boundary conditions supporting the model are an extremely important first step in analyzing any timber truss. A true truss resolves all outward thrust or horizontal forces internally; not relying on support reactions. This means that gravity loading usually results in only vertical load reactions at the truss bearing points. Just as with performing a beam analysis, the truss is usually modeled as sitting on a pin support on one end and a roller support on the other. Setting the model on a pin/pin connection suggests that the truss is sitting within two figurative canyon walls, which would yield erroneous member forces, particularly for a scissor truss.

Once the member is either modeled within the software or imported from a BIM model, care should be taken to confirm that all members are the correct species, grade, size and orientation. It is also a good idea to confirm that the software's preset design characteristics for a selected species are accurate. Review of the preset material unit density is important, since it can significantly affect the self-weight dead loads.

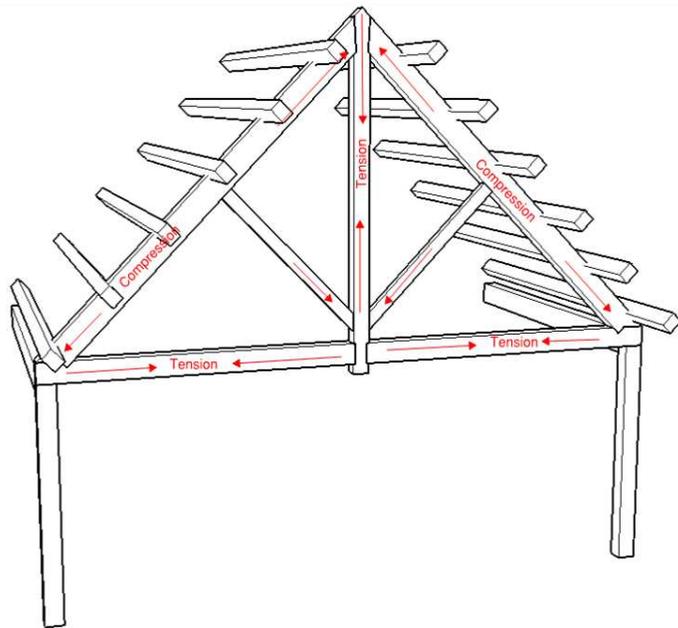
Similar to structural boundary conditions at supports, member end fixities should be reviewed as well. As mentioned earlier, all joints are usually analyzed as a hinge or pinned connection, even though most mortise and tenon joints do provide some moment fixity, and might most accurately be modeled as

springs. There is some variation in how connections are labeled between software platforms, but ultimately all joints should be free to rotate. Continuous members that pass-through nodes will also need to be identified at this point. Some software makes each member segment independent, requiring the analysis to fix any midspan nodes to represent a continuous member. Other software recognizes continuous members if the member center line is drawn continuously through the node. The latter method can become an issue if the center line model is imported as a CAD file, as continuous lines are typically lost in the importing process, resulting in segmenting all members at every node.

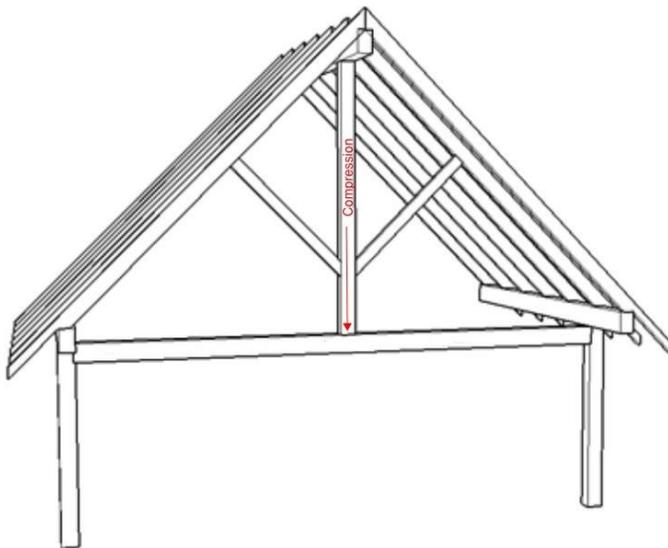
Similar to analytical calculation methods performed by hand, computer software generally requires that diagrams be drawn as center line models if drawn within the program itself. This certainly aids in simplifying the analysis and future modification of member dimensions, but it can be problematic in creating an accurate depiction of the realistic load paths through the truss. This is typically a bigger hurdle when performing three-dimensional analysis of full timber frames that incorporates trusses, rather than in the simple review of a single two-dimensional truss. Stacked timber connections or continuous perpendicular members bisecting truss elements, are common sources of this dilemma, and often require finessing of the node locations with an understanding of probable axial loading.

Many analysis programs provide a member offset function to handle the stacked timber connections, while maintaining proper member planes for area loading. But bisecting members, such as continuous eave and ridge beams which have become more common place in modern designs, particularly for supporting gable overhangs, present a significant trap during analysis. Modeling software typically has a difficult time distinguishing when truss axial load path has been broken, and often will continue to analyze the elements as an idealized or real truss even when members are not even joined together. But because the center line modeled members share a common node, the software has a difficult time distinguishing a break in the truss continuity. This tendency is likely a result of most software's multiple material platform, as these types of axial load transferring through bisecting members is much less problematic in steel and concrete, than when dealing with the nuances and design capacities of wood.

A common example to better illustrate this point is a king-post truss versus a structural ridge supported by a segmented central post and a full span header. When initially modeled in a structural software both frames will look and analyze similarly unless additional steps are taken. In this scenario the structural ridge will likely simply load the mid-post in compression, but the analysis might often continue to show the post in tension, similar to a king-post in the true truss. This can be simply resolved by restricting members, such as the mid-post to compression only. Failing to recognize this flawed model glitch would create a significant misrepresentation of the realistic load path in the analysis. Part of the key to truly understanding and performing accurate software analysis is to clearly understand when a labeled truss isn't really a truss at all.



Example of a real king-post truss supporting a distributed purlin roof load. The offset and non-bisected joinery aids in computer modeling. The upward thrust of the rafters opposing the compression of the diagonal struts forcing the king-post into tension.



An example of structural ridge supported by a segmented post and a full span header. Many computer programs have a difficult time distinguishing this as a non-truss, particularly if the header is brought up to the same plane as the eave plate. A key warning sign is the axial loading of the segmented mid-post. If the program suggest that it is tension a closer look and resulting manipulation is required.

The move toward better integration between Building Information Modeling (BIM) design models and structural analytical software might reduce these types of misrepresentations, but will also inevitably create new hurdles to be discovered and cleared. As with all tools, a good prior understanding of the expected results will go a long way toward proper use and accurate analysis. Being able to step back and take a deeper dive into a result that does follow truss expectations is key to truly unlocking the full potential of the various software platforms.

Aside from proprietary engineering templates created by individual firms, there is no commercially available software that will analyze or design timber connections. When engineering timber trusses, the computer modeling or hand calculations to determine member forces is the easy part. The design of the truss connections is the real challenge. Connection engineering must be done by hand and requires no small measure of experience and ingenuity.

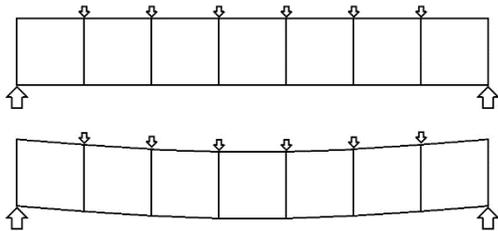
Truss Deflection and Camber

Methods for calculating truss deflections were developed that could be readily done “by hand.” They relied on the concept of “virtual work” and could isolate one particular deflection, at one particular node, corresponding to a “virtual load” applied at that node. While computers generate a much wider array of truss deflections under load, almost none of the software includes any distortion at the connections. Modeling the connections as springs adds enough work and uncertainty as to make it very rare. This is not to say, though, that slip and crushing at connections do not influence truss deflection. Some older texts acknowledge this and offer unsatisfying, but real, suggestions to double the deflection caused by axial stretch and shortening in the members.

Cambering trusses is an arcane but crucial topic. Early truss patents – notably the Long – included this ability as a significant feature. Wedges and shims were intended to prestress the internal components, reducing their forces when under design loading. Threaded rods, in Pratt trusses, make for ready adjustment of the truss sag – even as it increases while dead load is being added. The recommended truss camber offered in *Heavy Timber Construction* (1963) is $= K_1L^3/H + K_2L^2/H$, where the camber is measured in inches, at the center of truss. L is span in feet; H is height of truss in feet, at center; $K_1 = 0.000032$ for any type of truss; $K_2 = 0.0028$ for flat or pitched trusses or 0.00063 for bowstring trusses (that is trusses without splices in upper chord).

Development of Truss Forms

As many truss designs have their origin in early timber bridges, their names are better explained first by starting a discussion of parallel-chord trusses.

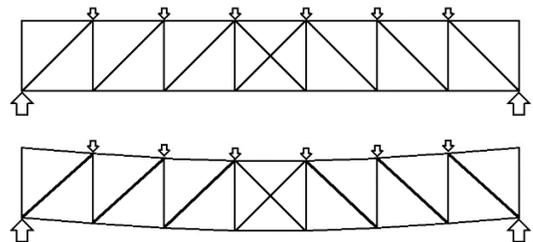


To illustrate, we start with two parallel chords, with verticals that subdivide the span into generally identical rectangles.

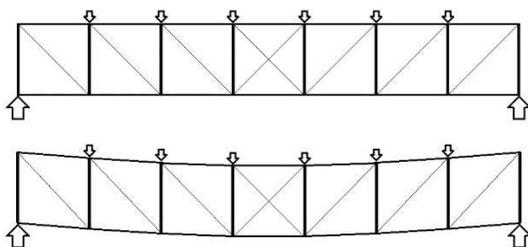
Under load, each of these rectangular sections distorts, with one diagonal becoming shorter and the other becoming longer. In this case, if we assume all of the applied loads are the same, only the middle bay remains

undistorted. If there IS a middle bay. Even numbers of bays avoid this.

Two truss types – Howe and Pratt – resolve to prevent this distortion in opposite ways. In a Howe truss, diagonal web members are introduced to prevent the opposing corners of each bay from getting closer to one another. The result is diagonal members in compression. As the middle bay will only be loaded – and tend to distort – if the loading is unbalanced, it is typical to see a Howe truss with only an even number of bays, or with diagonals in both directions at the center span:

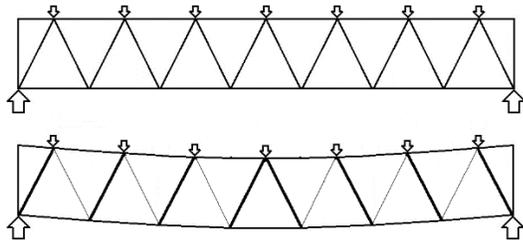


Where the diagonals will be in compression, these members must be stout enough to resist buckling, and so timber trusses of the Howe type include timber diagonals. As this results in the vertical members acting in tension, with the advent of iron and steel, it is common to see steel or iron rods as verticals, and this results in an aesthetic that emphasizes the diagonals.



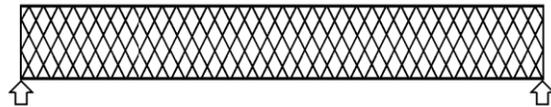
Conversely, one could prevent the corners of each square bay from getting farther apart by tying the corners together with web members that would be in tension. This is how the Pratt Truss works. Further, as diagonal web members in tension can be thin – they do not have to resist buckling – the Pratt Truss often employs steel or iron rods as diagonal web members.

This results in compression in the vertical members, requiring timber members stout enough to resist buckling, and the resulting aesthetic emphasizes the verticals with light-weight diagonals. As an added bonus, the verticals are shorter than the diagonals, so shorter timbers are permissible. Further, as the vulnerability to buckling is decreased for shorter members, the verticals can typically be smaller than they would have to be if they were oriented diagonally. Finally, it is fairly straightforward to detail a joint between wood members when they meet at right angles. For these reasons, a Pratt Truss is often a good choice where a parallel-chord truss is desired.



A Warren Truss deviates from both the Howe and Pratt by foregoing vertical web members, favoring instead, a simple zig-zagging diagonal pattern. In a Warren Truss, the outer diagonals are in compression and the inner ones are in tension. If steel or iron rods are used for the web members in tension, the resulting aesthetic may reflect this.

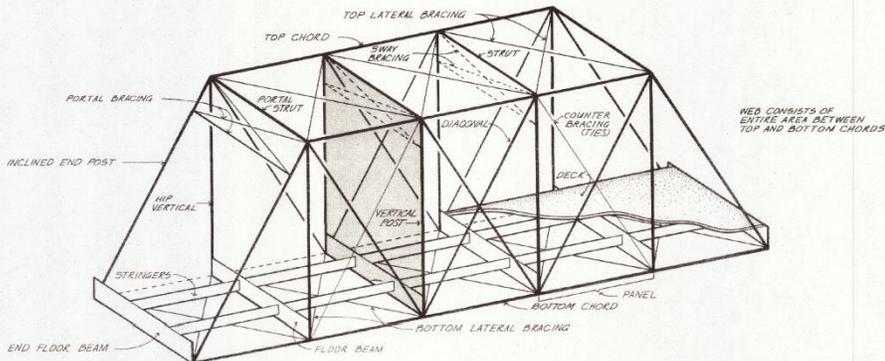
Finally, and especially for those who look at covered timber bridges, the most familiar patented truss is likely the Town Lattice Truss. In addition to their widespread use in covered bridges, these trusses can be found supporting town hall and other building roofs. At present, approximately half of the extant covered bridges in Vermont rely upon Town Lattice Trusses. One way of looking at a Town Lattice is to imagine superimposing three or more Warren Trusses on top of one another as so:



With this as background, in the case of roof trusses with sloping top chords, some of the names given to the truss forms are slightly different from those assigned to the parallel-chord versions described here. However, this may serve as background for later chapters on each roof truss type.



Worrall covered bridge in Rockingham, Vermont. The Town Lattice truss was patented in 1820 by Ithiel Town, a prominent architect from Connecticut. Town never actually built a bridge himself, but he sold the rights to use his design to covered bridge builders. He charged a royalty of one dollar for every foot of bridge span. If he caught someone building a Town Lattice bridge without having purchased the rights to the design, he would impose a penalty of two dollars per foot of span.



TRUSSES

A STUDY BY THE HISTORIC AMERICAN ENGINEERING RECORD

A TRUSS IS COMPOSED OF STRUCTURAL TRIANGLES JOINED TOGETHER WITH PINNED OR RIVETED CONNECTIONS. THE MAIN TYPES OF MEMBERS MAY BE EITHER STIFF HEAVY STRUTS, POSTS OR THIN FLEXIBLE BARS. IT IS THE ARRANGEMENT OF THESE MEMBERS THAT DETERMINES THE SPECIFIC TRUSS TYPE.

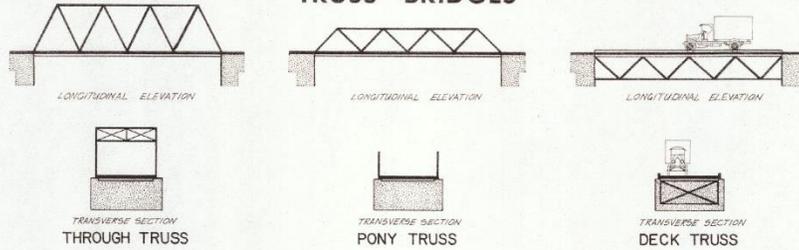
STRUCTURAL MEMBERS RESIST FORCES IN TWO PRIMARY WAYS — COMPRESSION AND TENSION. HEAVY RIGID MEMBERS MAY RESIST BOTH COMPRESSIVE AND TENSILE FORCES BUT THIN RODS CAN ONLY RESIST TENSION AND THESE CHARACTERISTICS ARE MAJOR CLUES IN TRUSS IDENTIFICATION. NOTE THAT THE MAIN STRUCTURAL MEMBERS OF A TRUSS PANEL MAY BE SUPPLEMENTED BY THIN DIAGONAL TIES. RECURVE TRUSS TYPES ARE DETERMINED BY THEIR MAIN STRUCTURAL MEMBERS. THESE COUNTER BRACES (INDICATED BY DASHED LINES) ON THE IDENTIFICATION SHEET MAY BE IGNORED AFTER WATCHING THE STRUCTURAL OUTLINE OF THE TRUSS IN QUESTION WITH THE DIAGRAM IT MOST RESEMBLES. CHECK TO MAKE SURE THE ARRANGEMENT OF HEAVY COMPRESSION AND LIGHT TENSION MEMBERS IS COMPATIBLE WITH THE DIAGRAM. IF THERE IS AGREEMENT, THEN

THE BASIC TRUSS TYPE IS IDENTIFIED.

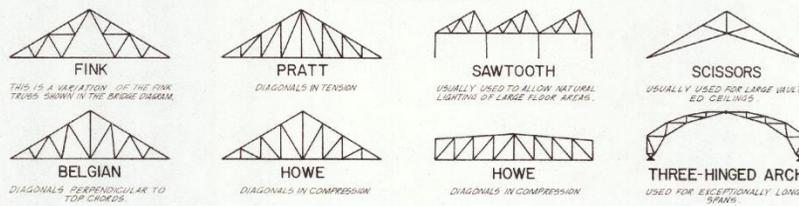
THE SHEET OF TRUSS DIAGRAMS PRESENTS ONLY THE STANDARD FORMS OF THE MOST COMMON TRUSSES. THERE ARE ALSO MANY UNUSUAL TRUSSES THAT DO NOT FALL INTO EASILY DEFINED CATEGORIES. IN SUCH CASES IDENTIFICATION SHOULD BE MADE AS CLOSELY AS POSSIBLE IN TERMS OF THE STANDARD DESIGN. ADDITIONALLY, TRUSSES OFTEN ARE INVERTED, CREATING OUTLINES QUITE DIFFERENT FROM THE ORIGINAL — TENSION MEMBERS BECOMING COMPRESSION MEMBERS AND VICE VERSA. BEWARE ASSUMING A TRUSS IS NOT REPRESENTED ON THE DIAGRAM. CHECK TO SEE IF IT IS AN INVERTED FORM.

MOST BRIDGE TRUSSES ARE OF THREE BASIC TYPES. IF THE DECK AND/OR RAILS ARE LEVEL WITH THE BOTTOM CHORDS, IT IS A THROUGH TRUSS. A PONY TRUSS IS A THROUGH TRUSS WITH NO LATERAL BRACING BETWEEN TOP CHORDS. A DECK TRUSS CARRIES ITS TRAFFIC LOAD LEVEL WITH THE TOP CHORDS.

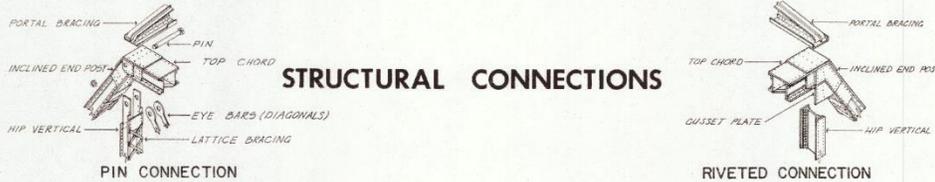
TRUSS BRIDGES



ROOF TRUSSES



STRUCTURAL CONNECTIONS

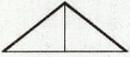
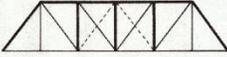
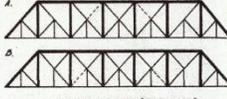
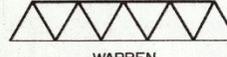
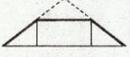
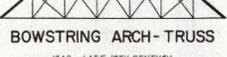
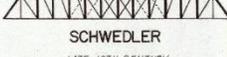
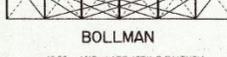
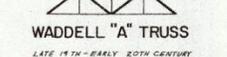
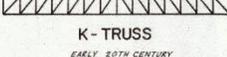
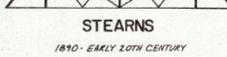


OFFICE OF ARCHITECTURE AND HISTORIC PRESENTATION
TECHNICAL INFORMATION PROJECT
UNDER DIRECTION OF THE NATIONAL PARK SERVICE
UNITED STATES DEPARTMENT OF THE INTERIOR

TRUSS IDENTIFICATION: NOMENCLATURE

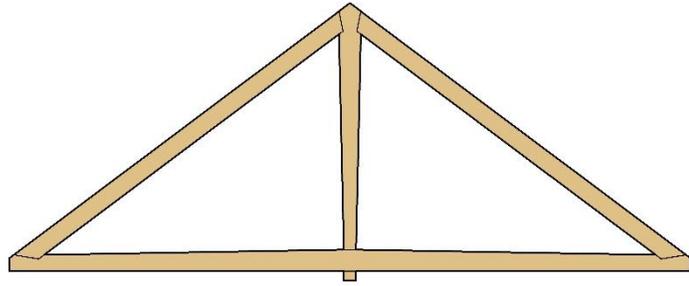
RECORD NO. HAER TH-1
HISTORIC AMERICAN ENGINEERING RECORD
SHEET 1 OF 2 SHEETS

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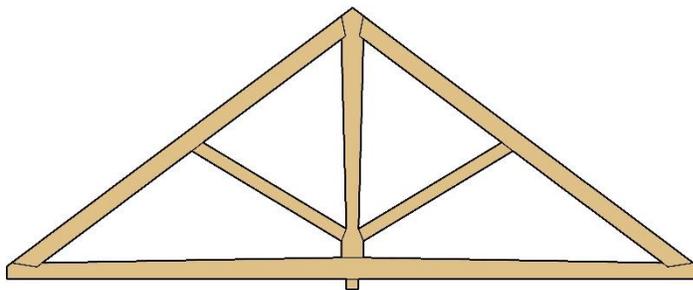
 <p>KING POST (WOOD) A TRADITIONAL TRUSS TYPE WITH ITS ORIGINS IN THE MIDDLE AGES. LENGTH: 20-60 FEET 6-18 METERS</p>	 <p>PRATT 1844 - 20TH CENTURY DIAGONALS IN TENSION, VERTICALS IN COMPRESSION (EXCEPT FOR HIP TRUSS) (ALS ADAPTED TO INCLINED END POSTS) LENGTH: 30-250 FEET 9-75 METERS</p>	 <p>BALTIMORE (PETIT) 1871 - EARLY 20TH CENTURY A. A PRATT WITH SUB-STRUTS B. A PRATT WITH SUB-TIES LENGTH: 250-600 FEET 75-180 METERS</p>	 <p>WARREN 1848 - 20TH CENTURY TRIANGULAR IN OUTLINE THE DIAGONALS CARRY BOTH COMPRESSIVE AND TENSILE FORCES. A TRUE WARREN TRUSS HAS EQUILATERAL TRIANGLES. LENGTH: 50-400 FEET 15-120 METERS</p>
 <p>QUEEN POST (WOOD) A LENGTHENED VERSION OF THE KING POST. LENGTH: 20-60 FEET 6-24 METERS</p>	 <p>PRATT HALF-HIP LATE 19TH - EARLY 20TH CENTURY A PRATT WITH INCLINED END POSTS THAT DO NOT HORIZONTALLY EXTEND THE LENGTH OF A FULL PANEL. LENGTH: 30-150 FEET 9-45 METERS</p>	 <p>PENNSYLVANIA (PETIT) 1875 - EARLY 20TH CENTURY A. A PARKER WITH SUB-STRUTS B. A PARKER WITH SUB-TIES LENGTH: 250-600 FEET 75-180 METERS</p>	 <p>WARREN WITH VERTICALS MID 19TH - 20TH CENTURY DIAGONALS CARRY BOTH COMPRESSIVE AND TENSILE FORCES. VERTICALS SERVE AS BRACING FOR TRIANGULAR WEB SYSTEM. LENGTH: 50-400 FEET 15-120 METERS</p>
 <p>BURR ARCH TRUSS 1804 - LATE 19TH CENTURY (WOOD) COMBINATION OF A WOODEN ARCH WITH A MULTIPLE KING POST ARCH ALSO COMBINED WITH LATER WOODEN TRUSSES. LENGTH: 50-125 FEET 15-50 METERS</p>	 <p>TRUSS LEG BEDSTEAD LATE 19TH - EARLY 20TH CENTURY A PRATT WITH VERTICAL END POSTS THAT DO NOT HORIZONTALLY EXTEND THE LENGTH OF A FULL PANEL. LENGTH: 30-100 FEET 9-30 METERS</p>	 <p>LENTICULAR (PARABOLIC) 1878 - EARLY 20TH CENTURY A PRATT WITH BOTH TOP AND BOTTOM CHORDS PARABOLICALLY CURVED OVER THEIR ENTIRE LENGTH. LENGTH: 50-360 FEET 15-110 METERS</p>	 <p>DOUBLE INTERSECTION WARREN (LATTICE) MID 19TH - 20TH CENTURY STRUCTURE IS INDETERMINATE. MEMBERS ACT IN BOTH COMPRESSION AND TENSION. TWO TRIANGULAR WEB SYSTEMS ARE SUPERIMPOSED UPON EACH OTHER WITH OR WITHOUT VERTICALS. LENGTH: 25-400 FEET 7.5-120 METERS</p>
 <p>TOWN LATTICE 1850 - LATE 19TH CENTURY (WOOD) A SYSTEM OF WOODEN DIAGONALS WITH NO VERTICALS. MEMBERS TAKE BOTH COMPRESSION AND TENSION. LENGTH: 50-125 FEET 15-44 METERS</p>	 <p>PARKER MID-LATE 19TH - 20TH CENTURY A PRATT WITH A POLYGONAL TOP CHORD. LENGTH: 40-250 FEET 12-75 METERS</p>	 <p>GREINER 1899 - EARLY 20TH CENTURY PRATT TRUSS WITH THE DIAGONALS REPLACED BY AN INVERTED BOWSTRING TRUSS. LENGTH: 75-250 FEET 23-75 METERS</p>	 <p>PEGRAM 1847 - EARLY 20TH CENTURY A HYBRID BETWEEN THE WARREN AND PARKER TRUSSES. UPPER CHORDS ARE ALL OF EQUAL LENGTH. LENGTH: 150-430 FEET 45-135 METERS</p>
 <p>HOWE 1840 - 20TH CENTURY (WOOD, VERTICALS OF METAL) DIAGONALS IN COMPRESSION, VERTICALS IN TENSION. LENGTH: 50-250 FEET 15-75 METERS</p>	 <p>CAMELBACK LATE 19TH - 20TH CENTURY A PARKER WITH A POLYGONAL TOP CHORD OF EXACTLY FIVE SIDES. LENGTH: 100-300 FEET 30-90 METERS</p>	 <p>DOUBLE INTERSECTION PRATT 1847 - 20TH CENTURY (WHIPPLE, WHIFFLE-MURPHY, LINVILLE) AN INCLINED END POST PRATT WITH DIAGONALS THAT EXTEND ACROSS TWO PANELS. LENGTH: 70-300 FEET 21-90 METERS</p>	 <p>POST 1845 - LATE 19TH CENTURY A HYBRID BETWEEN THE WARREN AND THE DOUBLE INTERSECTION PRATT. LENGTH: 100-300 FEET 30-90 METERS</p>
 <p>BOWSTRING ARCH-TRUSS 1840 - LATE 19TH CENTURY A TIED ARCH WITH THE DIAGONALS SERVING AS BRACING AND THE VERTICALS SUPPORTING THE DECK. LENGTH: 50-150 FEET 15-45 METERS</p>	 <p>CAMELBACK WITH IRONWOOD PANELS LATE 19TH - EARLY 20TH CENTURY A. A PENNSYLVANIA TRUSS WITH A POLYGONAL TOP CHORD OF EXACTLY FIVE SIDES B. SAME AS A WITH HORIZONTAL STRUTS. LENGTH: 100-300 FEET 30-90 METERS</p>	 <p>SCHWEDLER LATE 19TH CENTURY A DOUBLE INTERSECTION PRATT POSITIONED IN THE CENTER OF A PARKER. LENGTH: 100-300 FEET 30-90 METERS</p>	 <p>BOLLMAN 1851 - MID-LATE 19TH CENTURY (RARE) VERTICALS IN COMPRESSION, DIAGONALS IN TENSION. DIAGONALS RUN FROM END POSTS TO CENTER PANEL POINTS. LENGTH: 75-100 FEET 23-30 METERS</p>
 <p>WADDELL "A" TRUSS LATE 19TH - EARLY 20TH CENTURY EXPANDED VERSION OF THE KING POST TRUSS. USUALLY MADE OF METAL. LENGTH: 25-75 FEET 8-23 METERS</p>	 <p>KELLOGG LATE 19TH CENTURY A VARIATION ON THE PRATT WITH ADDITIONAL DIAGONALS RUNNING FROM UPPER CHORD PANEL POINTS TO THE CENTER OF THE LOWER CHORDS. LENGTH: 75-150 FEET 23-30 METERS</p>	 <p>K-TRUSS EARLY 20TH CENTURY SO CALLED BECAUSE OF THE DISTINCTIVE OUTLINE OF THE STRUCTURAL MEMBERS. LENGTH: 200-800 FEET 60-240 METERS</p>	 <p>FINK 1861 - MID - LATE 19TH CENTURY (RARE) VERTICALS IN COMPRESSION, DIAGONALS IN TENSION. LONGEST DIAGONALS RUN FROM END POSTS TO CENTER PANEL POINTS. LENGTH: 75-100 FEET 23-30 METERS</p>
 <p>WICHERT 1755 - MID-LATE 20TH CENTURY IDENTIFIED BY A CHARACTERISTIC PIN-CONNECTED SUPPORT SYSTEM OVER PIER. TRUSS IS COMBINED OVER PIERS. LENGTH: 400-1000 FEET 125-305 METERS</p>	<h2>TRUSSES</h2> <p>A STUDY BY THE HISTORIC AMERICAN ENGINEERING RECORD</p> <p>• BOLLER, ALFRED F. PRACTICAL TREATISE ON THE CONSTRUCTION AND DESIGN OF BRIDGES. NEW YORK: JOHN WILEY & SONS, 1910. 1-18. • BOHANNAN, H. L. AND BUTTERLAND, J. H. STEEL ARCHES AND ARCHES. NEW YORK: JOHN WILEY & SONS, 1910. 1-18. • COMMITTEE ON BRIDGE AND MARINE DESIGN. BRIDGE DESIGN. NEW YORK: JOHN WILEY & SONS, 1910. 1-18. • HOBAN, JAMES H. AMERICAN BRIDGE AND MARINE DESIGN. NEW YORK: JOHN WILEY & SONS, 1910. 1-18. • JACOBY, JEREMY S. AND HEALING, AMANUS. THE BRIDGE. NEW YORK: JOHN WILEY & SONS, 1910. 1-18. • KILPATRICK, J. H. BRIDGE ENGINEERING. NEW YORK: JOHN WILEY & SONS, 1910. 1-18. • WADDELL, J. L. BRIDGE ENGINEERING. NEW YORK: JOHN WILEY & SONS, 1910. 1-18.</p>		 <p>STEARNS 1890 - EARLY 20TH CENTURY SIMPLIFICATION OF FINK TRUSS WITH VERTICALS OMITTED AT ALTERNATE PANEL POINTS. LENGTH: 50-200 FEET 15-60 METERS</p>
<p>OFFICE OF ARCHAEOLOGY AND HISTORIC PRESERVATION TECHNICAL INFORMATION PROJECT UNDER SUPERVISION OF THE NATIONAL PARK SERVICE UNITED STATES DEPARTMENT OF THE INTERIOR</p> <p>TRUSS IDENTIFICATION: BRIDGE TYPES</p> <p>RECORD NO. TH-1 HISTORIC AMERICAN ENGINEERING RECORD SHEET 2 OF 2 SHEETS</p> <p>IF REPRODUCED, PLEASE CREDIT: HISTORIC AMERICAN ENGINEERING RECORD, NATIONAL PARK SERVICE, NAME OF DELINEATOR, DATE OF THE DRAWING</p>			

King-Post Trusses

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



A king-post 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 king post.



a transverse beam connected to the base of the king post. In a roof truss, however, roof loads are typically applied to the top chord through purlins or structural panels. In such instances, the king post is a zero-stress member and only serves to carry the dead load of the bottom chord unless there are loads applied to the bottom chord. If the king-post truss is modified to include diagonal

struts or braces, the roof loads applied to the top chord are transferred to the king-post which becomes a tension member. The modified version of a king-post truss – as shown at left – may at once be called a king-post truss with braces, but may also be referred to as a Howe truss.

In the diagram shown above, the king post is a timber, and the top has been shaped to fit between the top chords in a manner resembling the keystone in an arch.

King-post pony truss bridge. The roadway is supported by a cross beam that is suspended from the king-post.

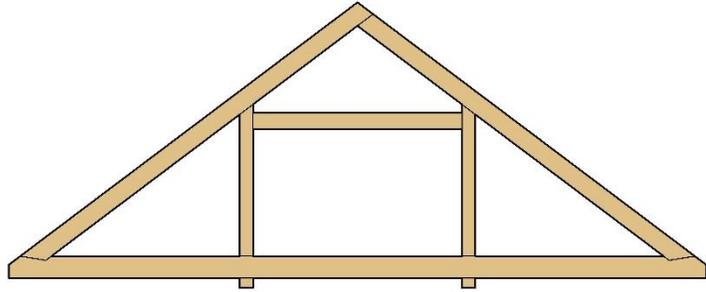




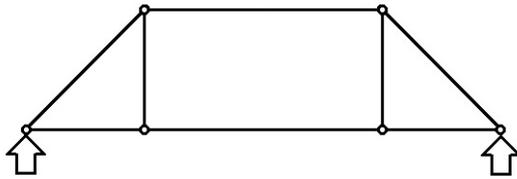
Example of a king-post truss supporting a CLT roof deck

Queen-Post Trusses

With similar origins in bridges, the queen-post truss is notable for having two principal verticals instead of the single for which the king-post gets its name.



However, unless the interior rectangle is braced with one or two diagonals, the queen post truss is not really a truss, but a frame, where the interior rectangle can distort under



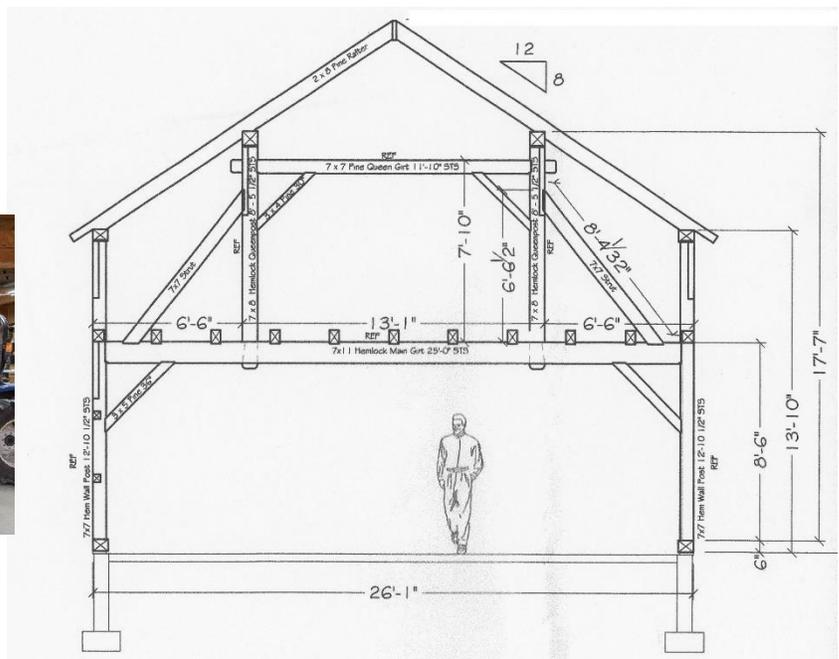
asymmetric loading. Although this diagram shows a queen post in a gabled-roof shape, it is not uncommon to find a queen post truss without the sloping upper chords, as shown at left. It is common to have purlins aligned with the queen-posts supporting the roof structure.

Typically, a queen post 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 load acting on the bottom chord and this 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. It is this induced bending that explains why a queen post truss is not really a truss.



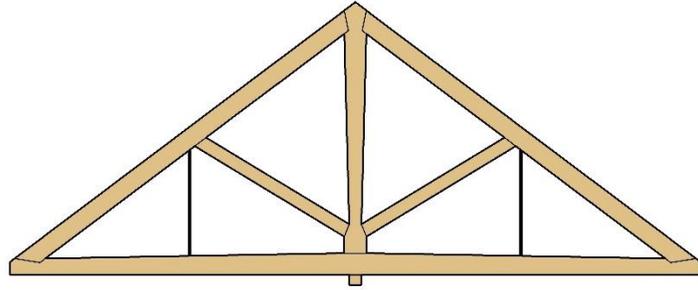


Example of a queen-post truss supporting the second-floor structure of a barn to provide a column free space below. Purlins supporting roof rafters rest on the queen-posts.



Howe Trusses

The Howe truss, patented by William Howe in 1840, is the most popular, practical, and efficient truss form for moderate span applications. In its simplest form it is really a modified king-post truss. The Howe truss is characterized by diagonal compression struts and vertical tension members.

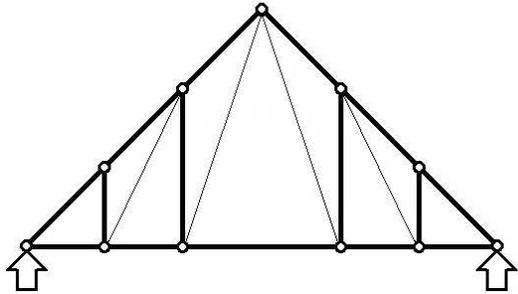


The advantage of a Howe Truss for use in a roof derives from the efficiency of using steel rods as the tension members. However, in the diagram above, the tension members to both sides of the middle theoretically carry no force unless the bottom chord is loaded – for example, by a ceiling or an attic floor. Even in the absence of loads applied directly to the bottom-chord, these rods may help support the self-weight of the bottom chord. For structural efficiency, the slope of the diagonal struts should match the roof slope.



Pratt Trusses

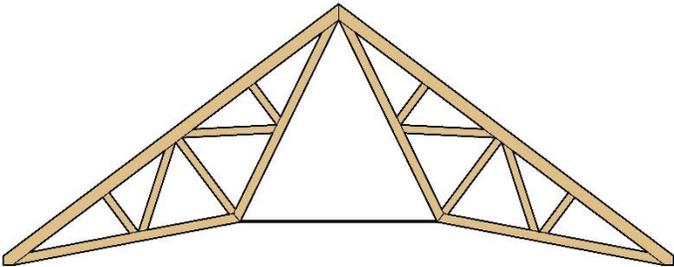
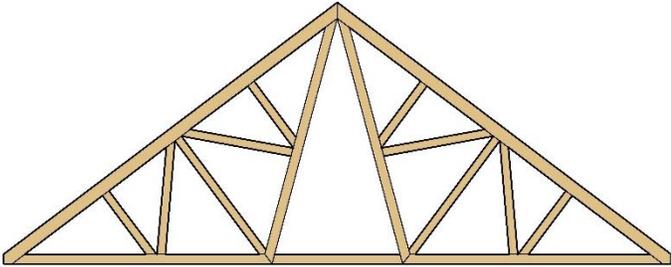
As noted above, a Pratt Truss resolves the need for triangulation through the combination of diagonal members in tension and vertical members in compression.



The advantage of a Pratt Truss for use in a roof derives in part from the efficiency of using steel rods as the tension members. Further, by shortening the compression web members, these elements can generally be smaller in cross-section than those used in a Howe Truss. Unfortunately, and especially for a roof truss with sloping top chords, the vertical web members meet the top chord at an angle, making the design of these joints complicated. It is likely that a Pratt Truss is better suited to use as a parallel-chord structure.

Fink Trusses

The Fink truss is a practical truss form for long-span applications. The major advantage of a Fink is that each half of the truss can be fully assembled in the shop and shipped to the site for final assembly. In some instances, the bottom chords can be sloped similar to a scissor truss for architectural drama.

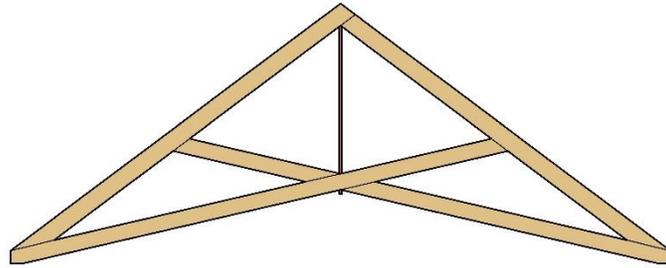


Compound Fink truss made up of two smaller trusses joined in the field.

Scissor Truss

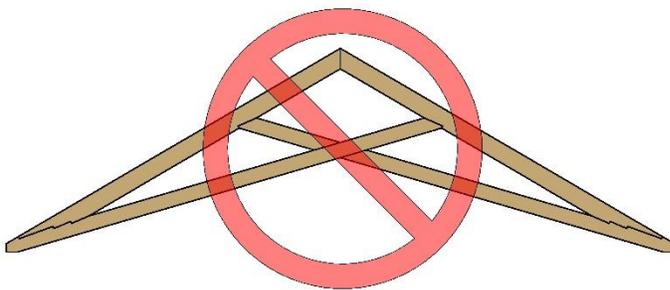
The scissor truss is a structurally inefficient truss form that is very popular with architects and timber frame patrons. Much like the reviled and troublesome “raised bottom chord truss” (known as “rafter buster” more colloquially), scissor trusses can add to

the open feeling of a space, by raising the lowest member above the most effective location – at the eave line. Characterized by sloping bottom chords that cross at mid-span, a scissor truss allows for an elevated 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 with the perceived stone ceiling.



In considering the behavior of a scissor truss, the most important attribute is likely the change in loading in each of the crossing timbers. Between the top chord and the crux - so-named because of its importance and because it occurs at the center of the truss (and because most all the tough design decisions are made here) - 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. The essential vertical member at the center is commonly referred to as the king, possibly reflecting its importance, but also it resembles the king post in a king-post truss.

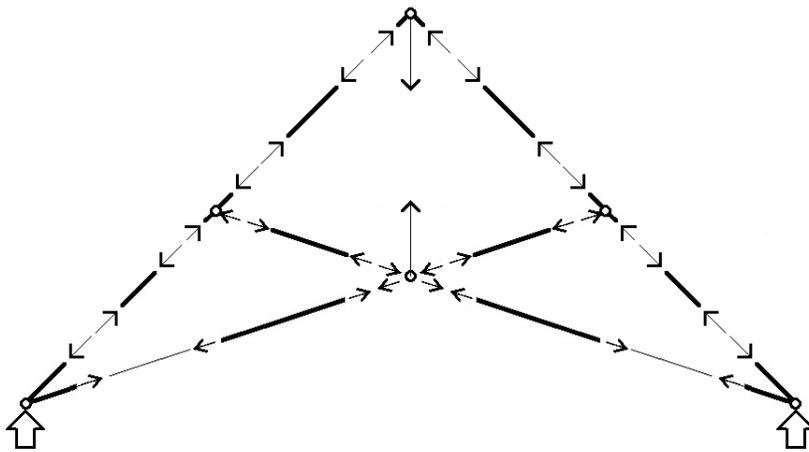
Another way of have explaining the tension in the king is that it restrains the kink in the bottom chord that “wants” to run straight, from eave to eave. The king is the critical member in a scissors truss. Trying to omit it, for a “king-free” scissor, leaves the designer with a frame, not a truss, and a brutalized



Do not omit the king from a scissor – ever!

frame, with severe bending in the diagonals right where they are often halved by dados, and lame, sloped bearing face tension transfer between the diagonals. Do not leave out the king for spans greater than ten feet. We are, also, assuming the diagonals are half-lapped at the crux. They can, in fact, also be cut off and all pasted back into the nodal mess with cover plates.

Scissor trusses exhibit the annoying habit of deflecting horizontally at the truss heels, causing the support posts or walls to spread. As the slope of the bottom chord increases (and approaches the slope of the top chord), the horizontal deflection and the chord forces increase dramatically. When modeling a scissor truss, it is crucial that the supports be modeled as a pin and roller. Otherwise, the calculated member forces will be erroneous.



Exploded View of the Scissor Truss with Forces acting at the crux. Generally, the lower diagonal forces are substantially larger than the uppers.

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 overall truss, there will be bending concentrated at the crux, and in the absence of a vertical tie, the resulting bending is the only means by which the scissor truss can function. Considering how much bending can occur at the crux, and given that it is common to remove half of each crossing member to make them pass one another, it is typically wise to allow for some adjustment in the vertical hanging element. Further, making the connection between the crux and the vertical can be challenging when the vertical is also a timber.

One strategy, if all timber faces must end up flush with one another, is to dado the three timbers into thirds as they pass through the crux, with the king third occupying the center path. This at least allows for the king's tension to be applied as a compression bearing on the undersides of the diagonals, without doing further damage to the third-lapped diagonals. Another method that extends this scheme involves upsizing and splitting the vertical to wrap around the crux and effectively support the center from below. This accentuates the vertical and relieves the designer of the need to make a hole through the crux, where each of the crossing members has already typically lost half its cross-section. The added surface relief between the king and the other members allows for discrete movement and misfits. An example of this approach is shown in the section on connections.

King post trusses with curved bottom chords can be viewed as a specialized form of scissors trusses. The good news about using curved bottom chords is that the overlapping surfaces between upper and lower chords are longer, allowing for more effective transfer of the shear force there, between the tensioned bottom chord and the compressed upper chord. One downside is that the curved chord will straighten as it is stretched, further increasing deflection (ridge sag and eave spread) in a truss form particularly prone to deflection. The curved bottom chord, itself, offers challenges to the designer and fabricator (dealt with in another section of this work). The clasping king concept can be applied in these trusses, too, in ways to nearly eliminate joinery damage to the crucial center of continuous (glulam we fervently hope) bottom chord.

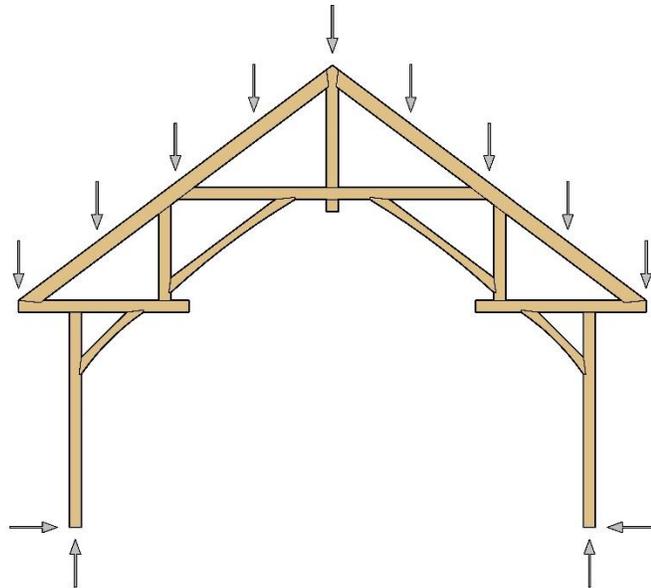


Example of timber scissor trusses

Hammer-Beam Trusses

Hammer-beam structures are by far the most structurally inefficient, problematic, and celebrated of all timber roof types. They behave very little like trusses and their members are subjected to significant bending moments and shear, particularly the columns.

When modeling a hammer-beam structure, it is crucial that the column bases be restrained in both the X and Y direction. If one of the bases is inadvertently modeled as a roller, the calculated moments in the columns will be unrealistically huge.



In considering the portion of the hammer-beam truss that extends above the eave line, it is reasonable to think of the structure as similar to rafters that bear on the exterior walls, lean against one another at the ridge line, and require a crossing tie or some other means of resisting the natural tendency to thrust outward at the eaves. Just as raising the height of the necessary tie increases the demand on the tie, it likewise alters the demand on the other intersecting elements. In the diagram above, the eave-level thrusts are transferred to the columns and knee braces, placing significant demands on these elements. As described below, historically, masonry buttresses would typically resist the outward thrusts. 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.

Out of plane bracing is essential. The hammer-beam has a tendency to buckle laterally. Bracing can be accomplished with vertical knee braces from purlins to the hammer posts, or with horizontal knee braces from the wall plates to the hammer beams. Hammer-beam structures should never be used on exterior walls subject to transverse wind loads if it can be avoided.

Tied hammer-beam trusses perform much better than a common hammer-beam structure. A steel rod with a turnbuckle is commonly used as the tie element, but that is seldom an architecturally acceptable solution. The photo to the right shows a chain used as a tie element.

Often ties are added later, after the eaves spread or the posts bow out – or both.





Westminster Hall in London 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 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 immensely thick exterior stone masonry walls were repurposed from an earlier Anglo-Norman Hall built in 1097 during the reign of William II.

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 stone masonry walls are over six feet thick and they 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 with the massive masonry walls replaced by slender timber columns, the structural behavior is profoundly different.

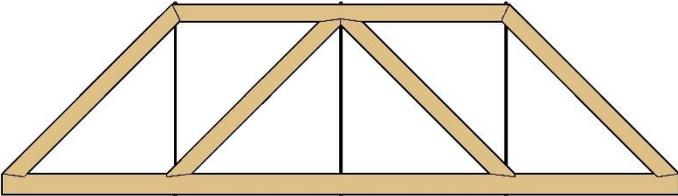


Example of a contemporary hammer-beam structure. Saint Patrick Church, Redding, Connecticut.

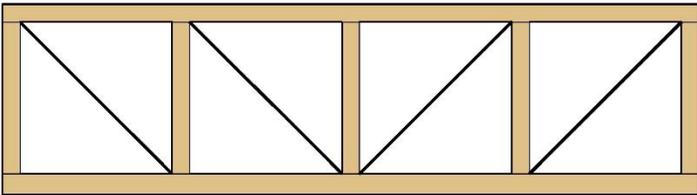


Parallel Chord Trusses

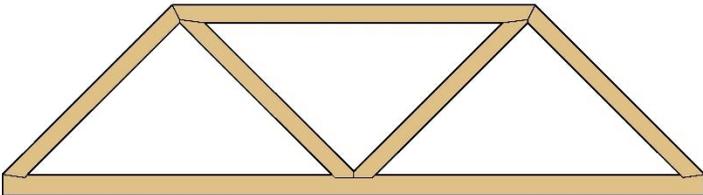
In considering trusses, we will be principally concerned with roof trusses that support gabled roofs. These trusses span transverse to the ridge line with top chords that slope, and where the top chords may support purlins, or they may support the roof sheathing (or panels) directly. However, roofs may also be built with parallel-chord trusses, where the trusses span parallel to the ridge line, or where they may form the upper-most portion of a timber wall frame.



Howe Truss



Pratt Truss



Warren Truss

Common parallel chord truss types are Howe, Pratt, and Warren.

The Howe truss is by far the most popular. The diagonals are compression struts and the verticals are tension ties. Often steel rods are used for the verticals. When the timbers season and shrink, the tension ties can become slack, resulting in unanticipated deflections. It is crucial that the nuts on the tension rods be accessible and periodically tightened until the timbers have fully seasoned.

With a Pratt truss, the verticals are the compression struts and the diagonals are the tension ties.

With a Warren truss, the diagonals alternate between compression and tension, making for very challenging connections.

Parallel chord trusses are often used for long span conditions in excess of 50 feet. It is often necessary to camber the trusses to manage deflections and to allow for adjustment with drying/shrinking and settling with dead load.

When proportioning a parallel chord truss, the depth to span ratio is normally in the range of 1:10 to 1:12.



The photo above shows a space truss for a church in Brookfield, Connecticut. Similar to a space frame, a space truss is a three-dimensional structure. Four glulam timber trusses (two in each direction) clear span 92 feet. The trusses intersect at four points.

Truss Joinery and Connections

The trick and art to designing a timber truss is how you put the timbers together. The rest of the stuff is easy. 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. 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.

Many engineers who are inexperienced at timber engineering will design truss connections that rely on steel gusset plates and bolts to transfer axial forces at a joint. While this type of connection is efficient in structural steel construction, it is a poor choice for a timber truss. Besides the aesthetic issues associated with exposed steel plates and bolts, these types of connections sometimes perform poorly. The steel connection plates tend to restrain the seasoning shrinkage of the timber and can cause the timber to split. If bolts pass through oversized holes that are not tight fitting, unanticipated truss deflection can result.

When designing timber truss joinery, there are a few fundamental rules that you should keep 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 best used to hold a joint together rather than to resist structural loads. Bolts or steel rods should be used to resist tension rather than shear. Let the geometry of the joint do the work, not the fasteners whenever practical.

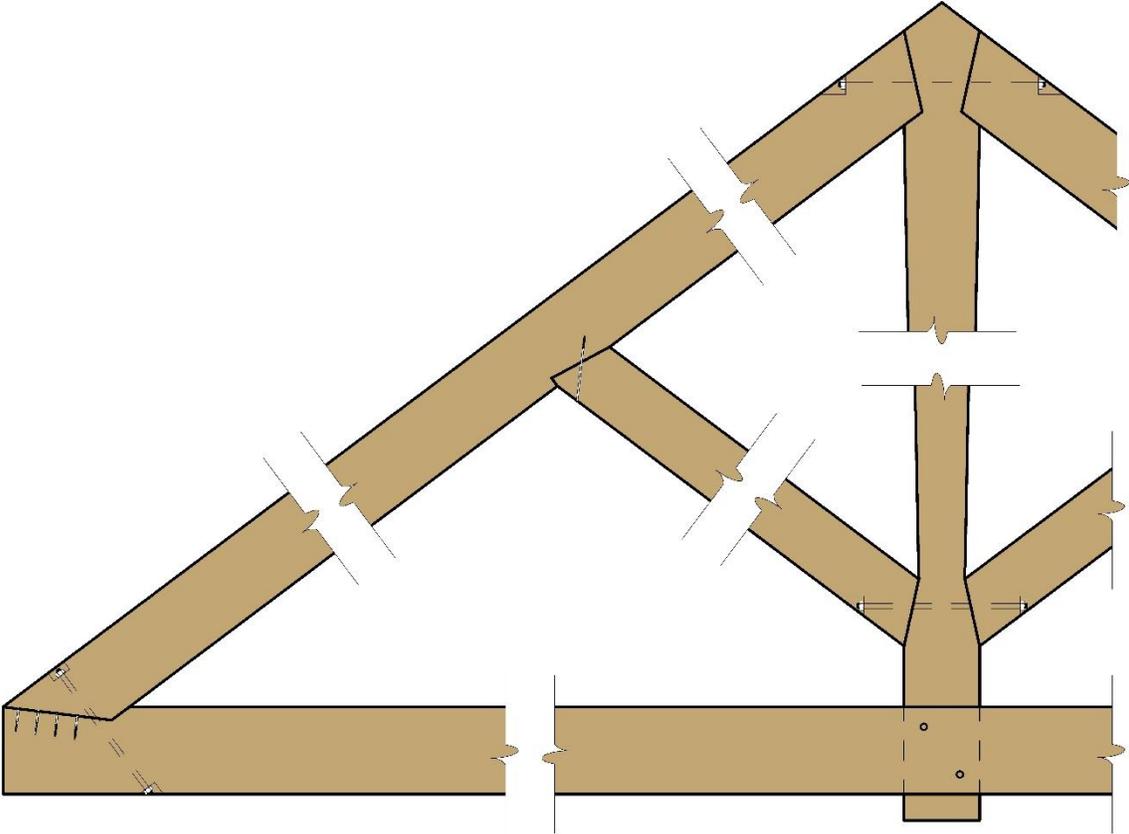
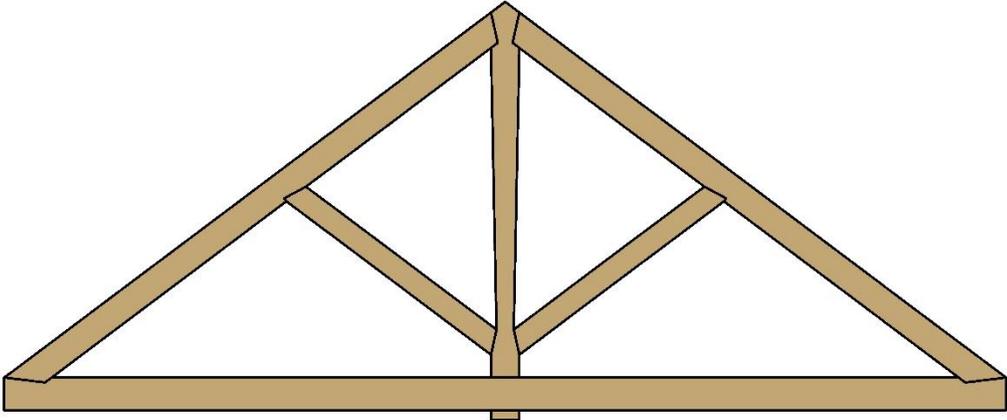
Rule #2 - *The wood removed to create the joint should not unduly weaken any of the timbers being joined.* The timber section of all members connected at a joint must be reduced in some fashion to create the joint. The challenge is to strategically remove wood only from the portion of each member that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

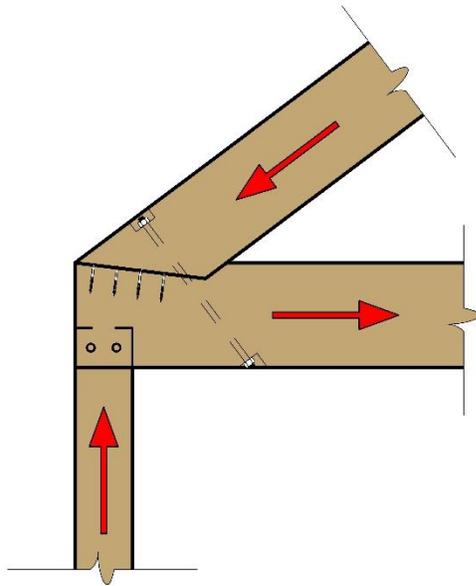
Rule #4 - *Anticipate all potential modes of failure and provide sufficient strength to resist each potential failure mode.* This is a rule that naturally applies to any structure not just timber truss joinery. The challenge here is that you must think of everything. Failure to anticipate a potential 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 joinery, but actually apply to all types of timber connections.

Howe Truss Example

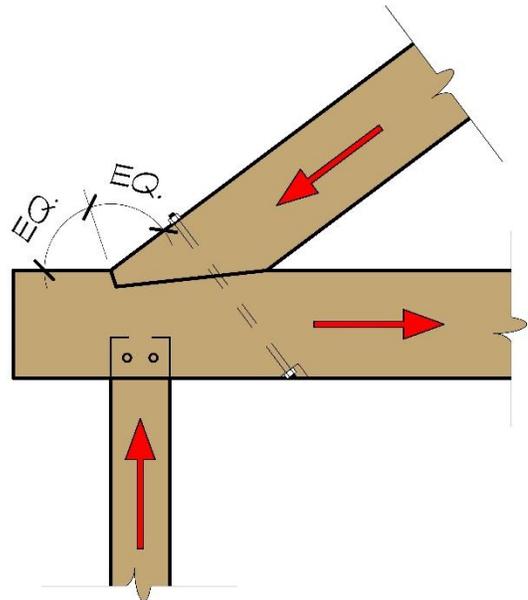


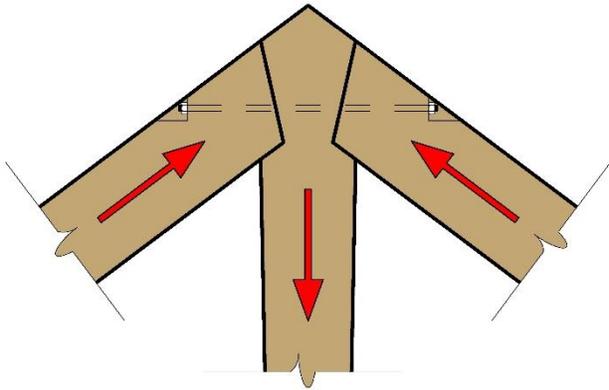
The trick is to configure the joints so that tension forces are resisted by notches and shoulders that are in compression. You cannot always make that trick work, but it is great when you can. To illustrate, here are some common truss joints for a timber Howe truss.



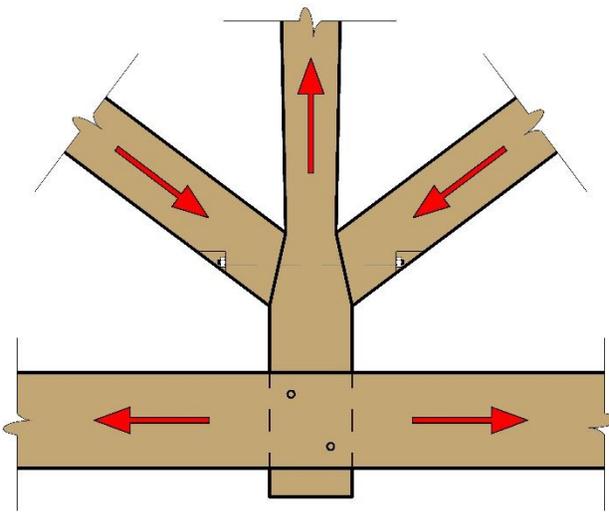
The heel joint is the most challenging joint in any truss because the member forces in the timber chords converging at the heel are substantial. The top chord of the truss is in compression and the bottom chord is in tension. The top chord bears against an inclined notch in the bottom chord (see rule #1). Since the bearing surface is not square to the axis of the top chord, there is a component of the top chord force acting parallel to the bearing surface which exceeds the frictional resistance of the bearing surface. Bolts acting in tension resist that component of the force. The shallow notch in the top of the bottom chord leaves sufficient net section to resist direct tension (see rule #2). The shape of the joint will not change significantly when the timbers season and shrink (see rule #3). The notch in the bottom chord must be far enough from the end of the timber so that the end of the timber does not fail in block shear (see rule #4).

An alternative heel joint detail is shown to the right. This detail only works if the bottom chord can run long and extend beyond the support post. The bearing notch allows more direct transfer of the top chord force to the bottom chord. Consequently, the tension force in the bolts is very small. The notch must be deep enough that the compression stress on the bearing surfaces is not excessive. The bottom chord must run long enough to provide adequate resistance to block shear.



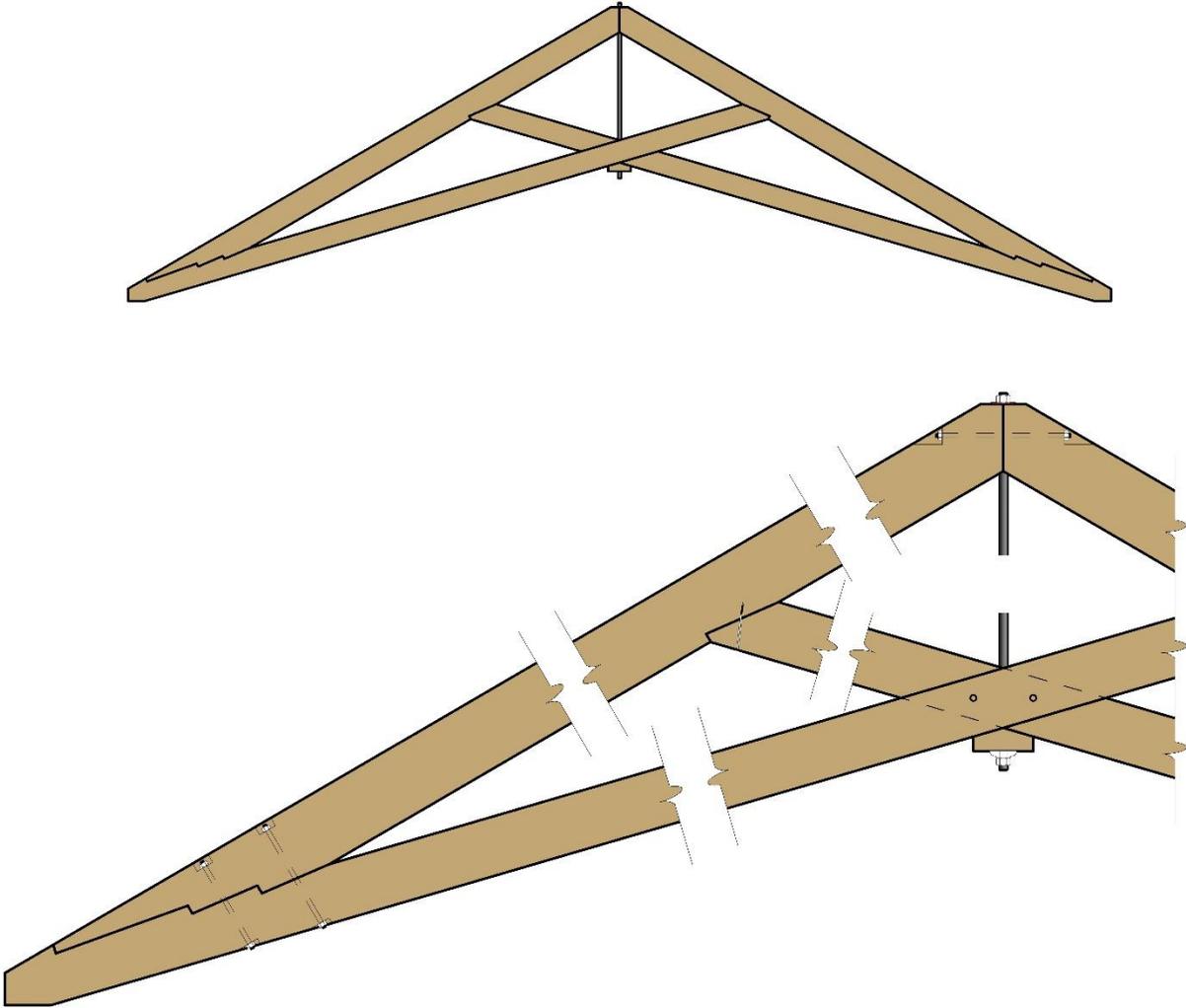


The joint where the top chords join to the king post is similar to the heel joint. The top chords bear on inclined notches in the side of the king post. The bolts clamp the joint together, preventing the bearing surfaces from slipping. The net cross section of the king post at the notches must be capable of resisting the tension in the king post. Resistance to block shear in the king post often controls the connection design.

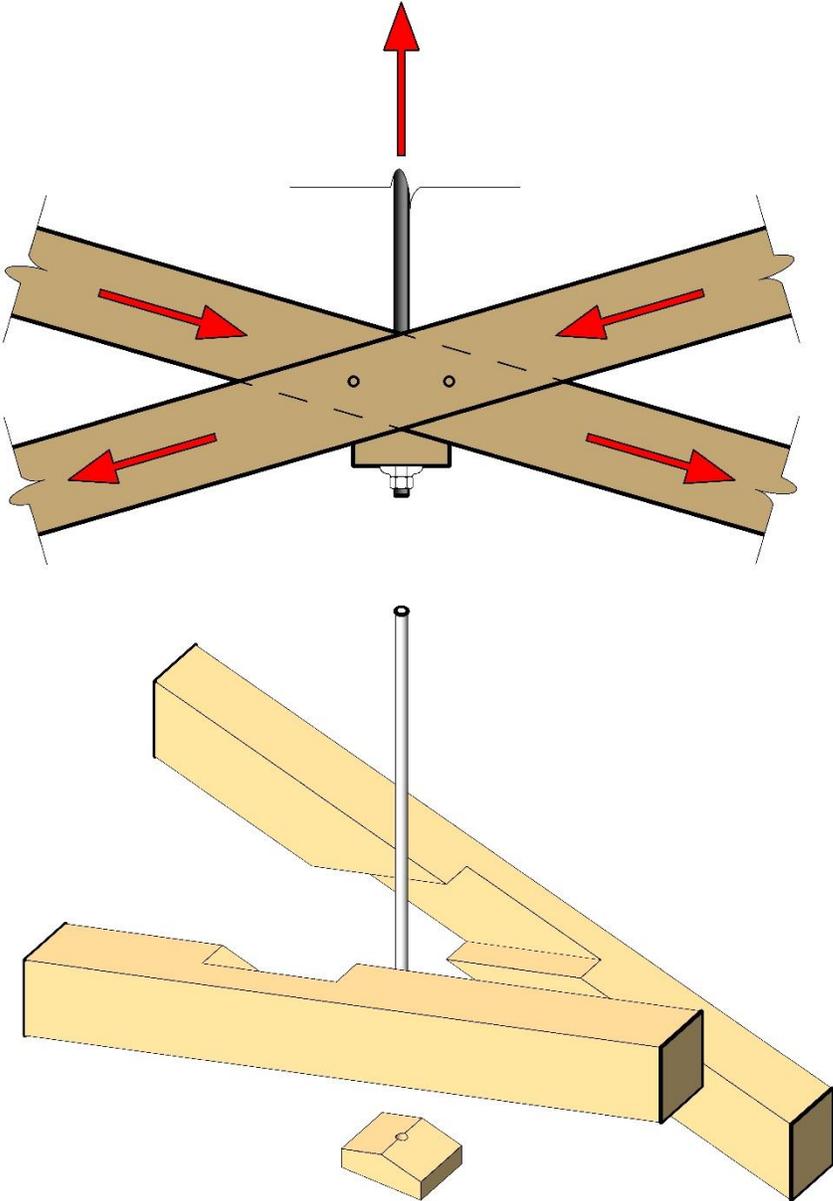


The joint where the king post meets the bottom chord is the same idea. The diagonal compression struts bear against an inclined shoulder on the side of the king post, called a joggle, converting their compression forces into tension in the king post. The member forces take a short cut and bypass the bottom chord altogether. The bottom chord isn't even aware of all of the action happening just inches above it. While in theory, the king post does not even need to connect to the bottom chord, the connection provides lateral bracing to the king post and resists any incidental gravity loads applied to the bottom chord.

Scissor Truss Example

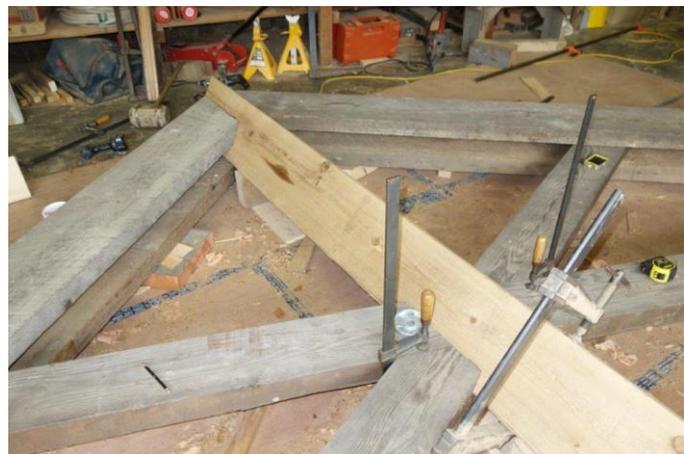
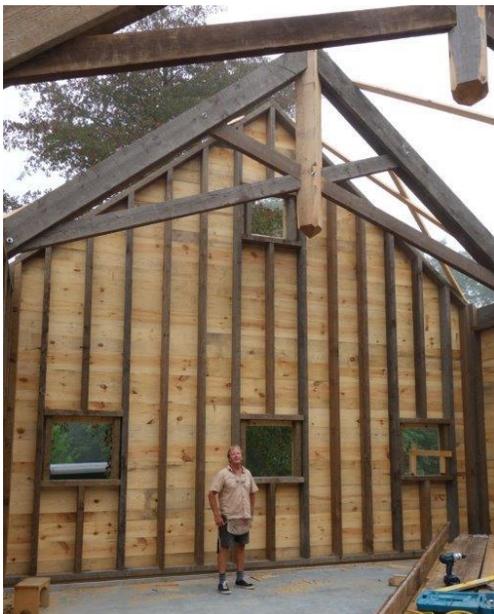


For a scissor truss, the crux joint where the bottom chords and king rod join is a challenging joint. Five truss members meet at the joint and three of them have large tension forces. By half lapping the bottom chords, all of the member forces are transferred in bearing.



Scissor Truss with a Clasping King

If the diagonals really must be flush-faced, then the only real option is to dado them into one another (laying up thin glued veneers that alternate continuity past the crux is an interesting and pricey option). These members now have lost half their bending and axial capacities, as well as adopting an eccentric axial load path that induces yet another bending stress. The challenge is to do no more damage to them at this point; specifically, by not having to rout in channels for a centered rod. One way is to split the king “post” and sandwich it around the dadoed diagonals and the top chords. The tension force is converted into compression bearing on the underside of the diagonals at the crux, and without further damage to the diagonals. This method also avoids the flush joints at peak and crux that can prove so unforgivingly unflush with any distortion or shrinking among the in-situ members.



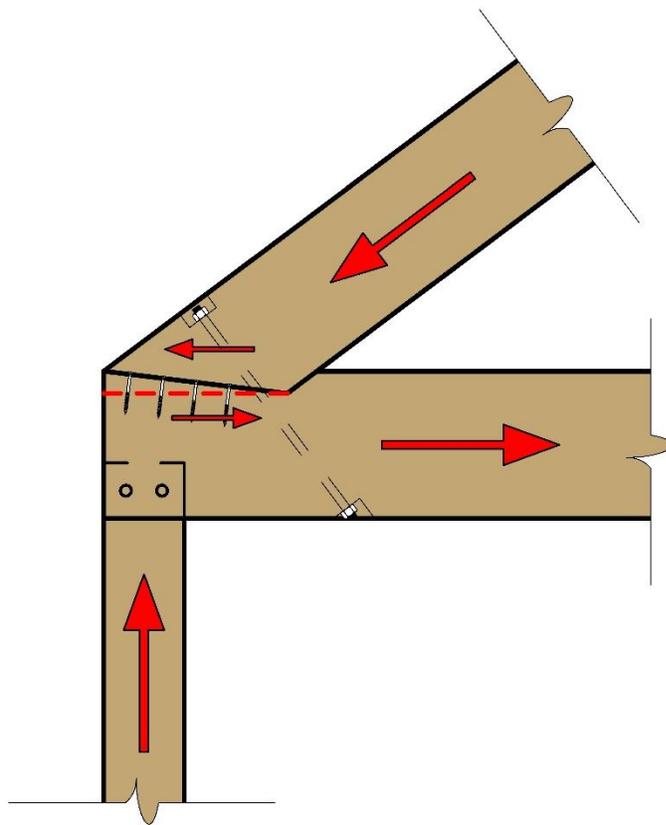
The photos are courtesy of South Mountain Company, using recycled heavy planks to craft the trusses. They took the extra complicating step of spinning the king 45° about its axis, “because they could.”

Block Shear

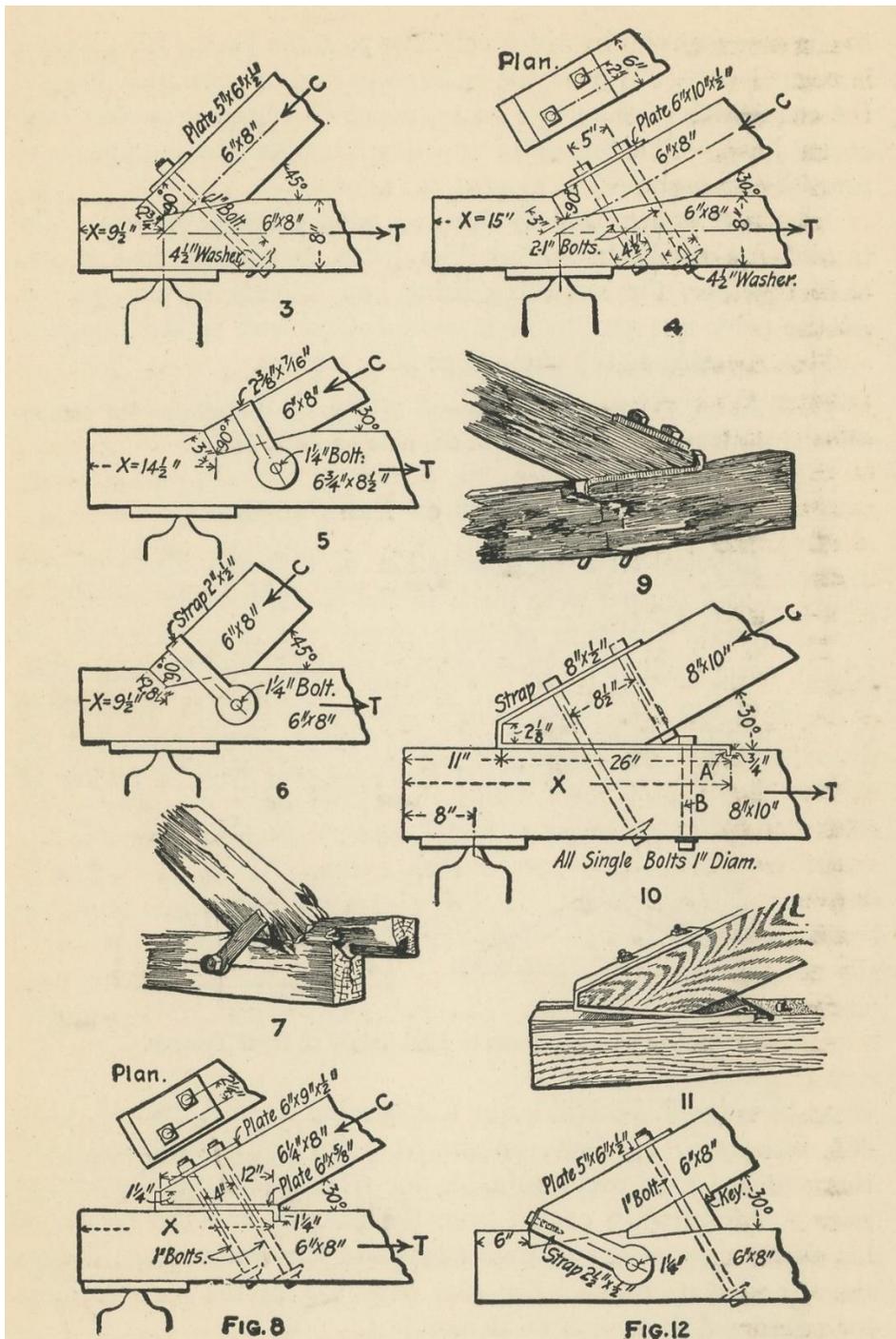
Block shear strength often controls the design of truss joinery. Block shear acts parallel to the grain at a bearing notch. The shear stress distribution is assumed to be triangular. Consequently, the calculated block shear capacity is one half the allowable shear strength multiplied by the shear area (section 3.6 of TFEC 1-2019).

The referenced design values for shear strength contained in the NDS are intended for beam shear, not block shear. There is an extra factor of safety of 2.0 built into the referenced design values to compensate for the potential of a split or check at the end of a beam. Consequently, using the values from the NDS to calculate block shear capacity is conservative, unless a seasoning check happens to coincide with the potential shear plane, in which case it is unconservative.

A block shear failure of a connection is a sudden, non-ductile failure mode. This was first observed in truss heel joint tests performed at MIT in 1897.



It is reasonable to assume that reinforcing a timber with screws can make a block shear failure more ductile and can help to prevent a seasoning check from forming at the shear plane. The block shear capacity should be calculated based on the shear strength of the timber alone and the screws should be viewed as improving the performance of the connection. It is unwise to expect load to be shared between the timber and the screws. The reinforcing screws should be just long enough to cross the potential shear plane with 2 inches of thread engagement beyond the shear plane. Using screws that are too long can restrain shrinkage and induce splitting.



Results of full-scale load tests of timber truss heel joints performed at the engineering laboratory of the Massachusetts Institute of Technology (MIT) in 1897.

Source – *Structural Details or Elements of Design in Timber Framing* by Henry S. Jacoby 1914

Friction in Joinery

Any bearing face that is at anything other than perpendicular to the axis of the force being transmitted through bearing is inevitably relying on friction – historically an unpredictable and unreliable phenomenon. As RJ Brungraber, Ph.D., P.E. says, in his authoritative *Overview of Floor Slip-Resistance Research*: “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.” Relying on such an unpredictable phenomenon for structural capacity seems innately to be avoided by reasonable designers and builders. However, some traditional joinery details rely on some friction and some modern practitioners seem dedicated to those details, so it is worth our time to be as thorough as is possible.

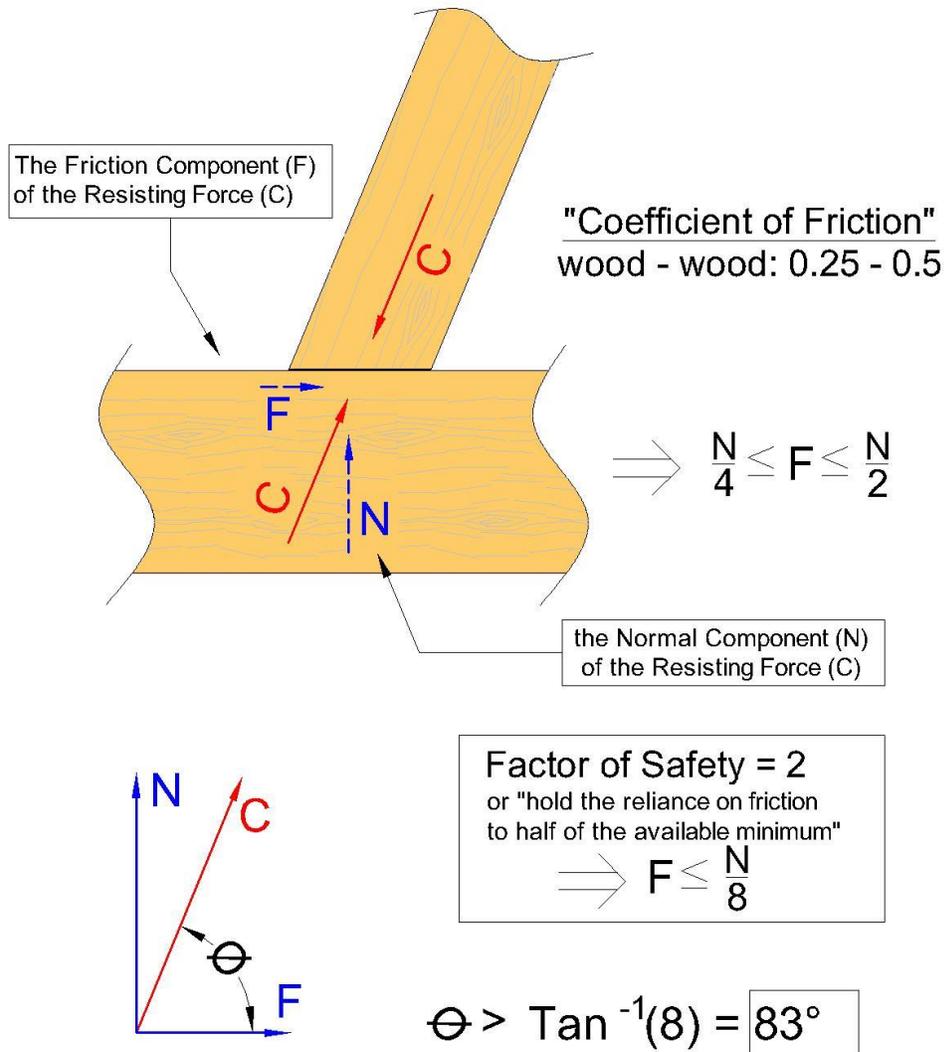
About the best information we have on the nature of friction between pieces of wood is from the slightly less-known researchers E. H. Messiter and R.C. Hanson, in their only slightly more contemporary 1894 work on the topic. They found coefficients of friction that ranged from 0.215 for sanded pieces, to 0.365 in rough-sawn timbers. They found roughly the same values for sanded pieces that were trying to slide alongside grain and ones that were bearing end grain to end grain. Among researchers who knew what they were doing, it is rare to find a coefficient of friction much above 0.4 between timbers.

Relying on friction in a structure seems a bad idea. We actually do it in almost all modern high strength steel-bolted connections. The highly tensioned bolts induce such reliable clamping that they do not bear on the hole edges, but the friction between the connected pieces does the job. What if they do slip? Well, the bolts can still work in shear, when the hole slop runs out as the surfaces slide under the applied load. Similarly, I suppose, many of the sloped bearing faces I have seen are associated with tenons in mortises that could “catch” the sliding members. An entirely different connection scheme to check and, frankly, a bit desperate – better to use an angled clamping bolt.

What is this measure of stickiness we call the *Coefficient of Friction*? Thankfully, we will deal solely with the static coefficient of friction. Those of you who had a good high school physics class, or even pursued higher education in such arcane topics, might recall the distinction between static and dynamic (often termed “sliding”) coefficients. Scientists now understand that, while static friction is still valid for objects not moving relative to one another, dynamic coefficients are a function of the speed of their relative motion – friction is not a simple dichotomy of still/moving phenomenon. Not to worry, not another word on non-static states of being.

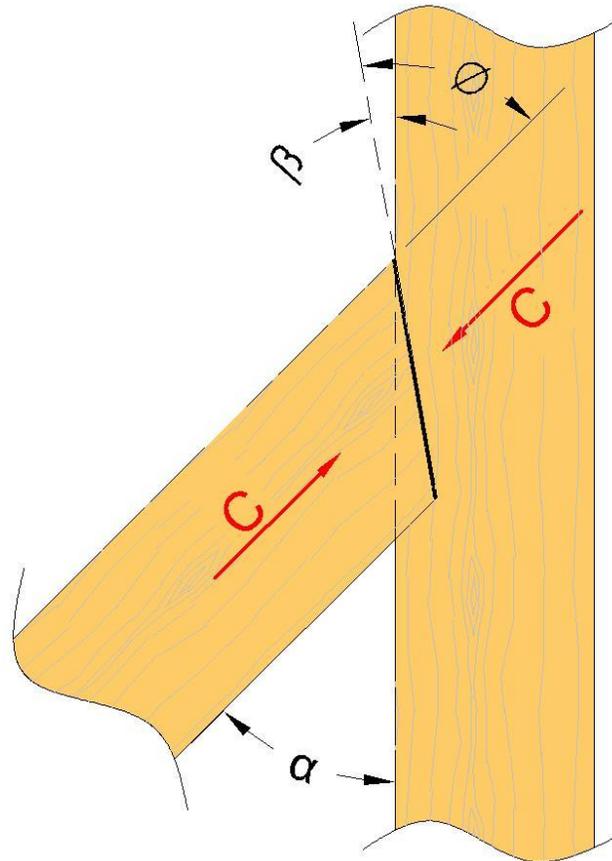
At its simplest, the coefficient of static friction is expressed as a simple ratio of the largest available force (given the nature of the body interactions at the bearing surface) to resisting an induced slide-inducing force, and the normal (pressing) force acting between the two bodies. One aspect 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. Doubling the bearing area between two timbers does not double the available friction resistance to sliding; using a smaller timber does not increase the available friction force, either, until the compressed timber actually penetrates the other surface, and causes some inadvertent mortise and tenon shear action. This ratio of forces can also be viewed as an angle – the angle at which an axially loaded member will slide along the surface of another. How flatly can you tip that ladder, on a smooth surface, before you are utterly reckless even to put a foot on it.

Friction In Joinery



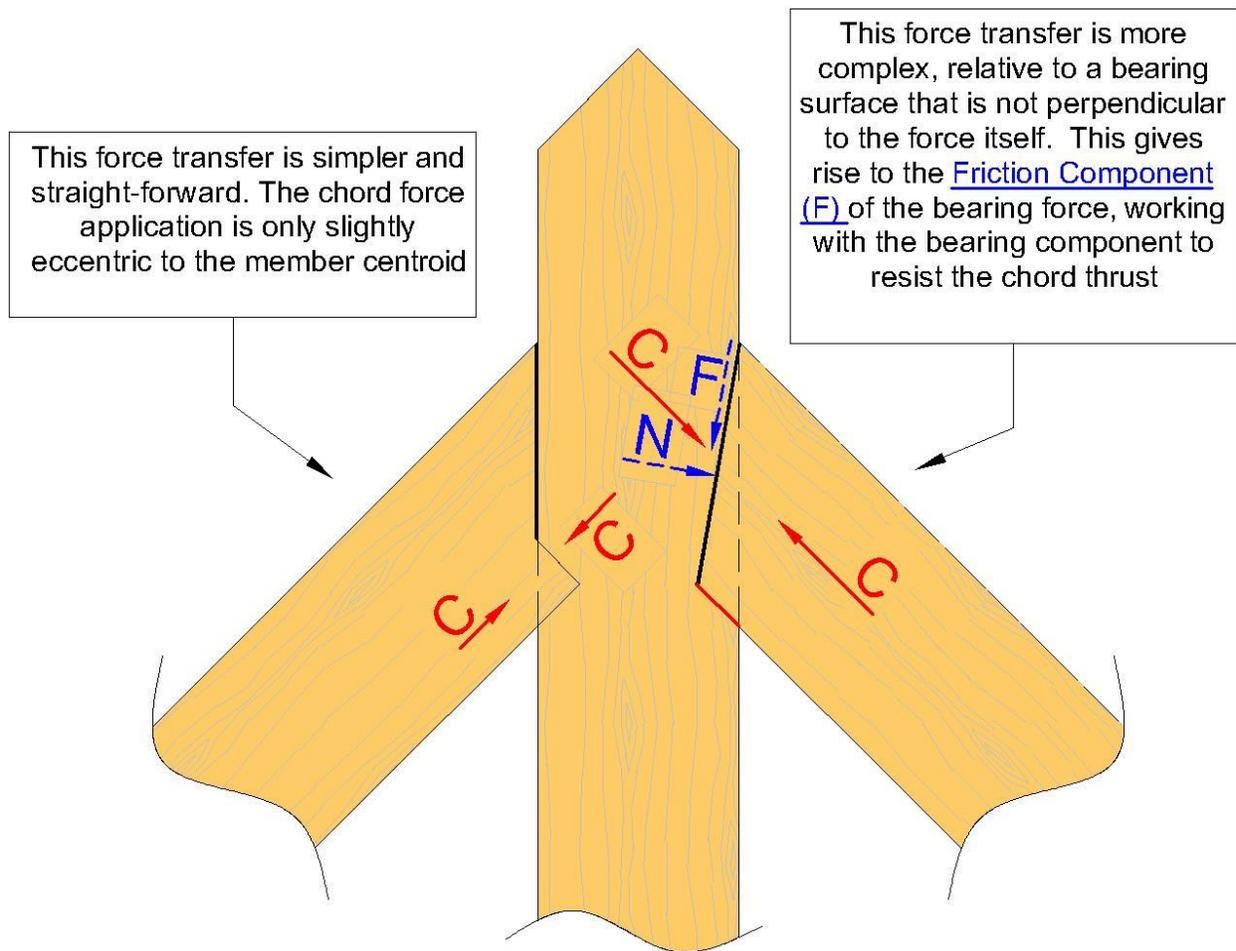
A bare angle joint is, clearly, ill advised. Perpendicular is reasonable. So, any beveled bearing face is, at its root, a move from reckless toward reasonable. Much like positioning collar ties – smart/minimum tension is available down at eave. Foolish / infinite tension hits collars placed at the peak. So, the question becomes how close do you care to get, to foolish. 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.

Friction Angle in Notch

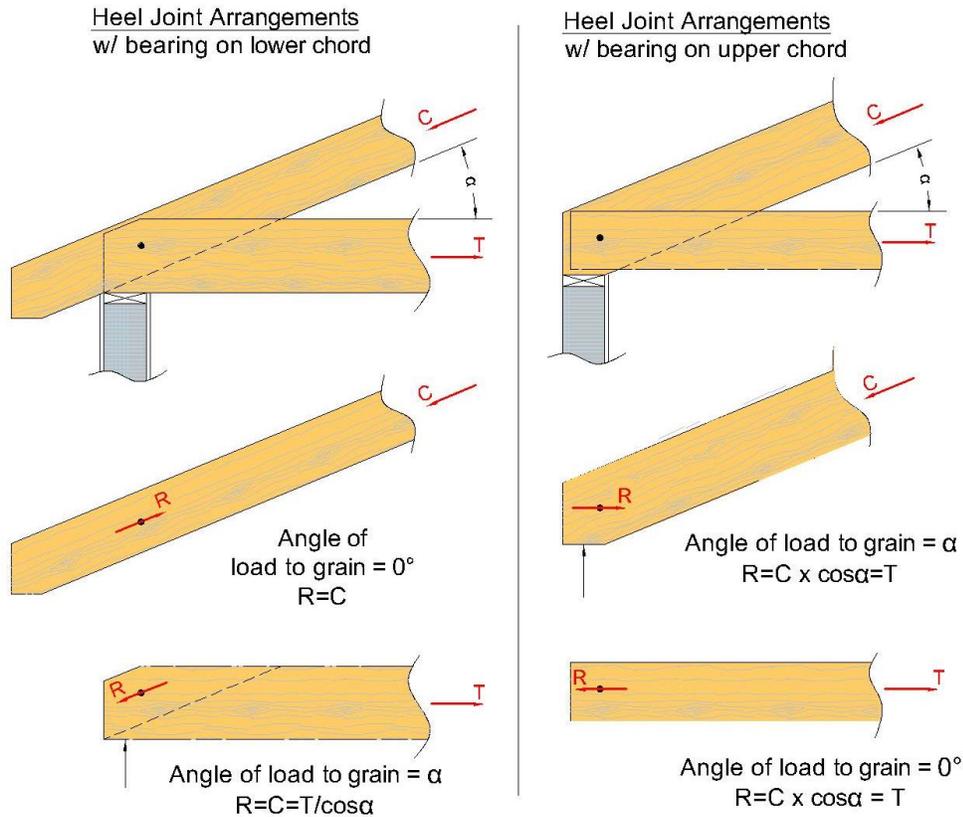


$$\theta = \alpha + \beta$$
$$(\geq 83^\circ)$$

Friction vs. Bearing



Free-Body Diagrams



(Inspired by an illustration in Hoyle's: *Wood Technology in the Design of Structures*)

The Hoyle illustration is a pure example of powerful, basic, and too-rarely applied analytical tool known as the *Free Body Diagram*. An assembly of pieces can be "disassembled" into components and the forces applied to and among those "bodies" can be evaluated in terms of the body's equilibrium. The free body is sitting still SOLELY 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. We are, even, able to use a "virtual free body", cut with imaginary scissors free of the surrounding material. We will be sticking to the more readily imagined, "exploded connection" versions of these potent analytics.

Steel Side Plates

Timber trusses started as all-wood, from maybe 1400 to our Civil War. The Industrial Revolution generated great demand for more and longer trusses. This inspired the market to provide wrought iron rods and bars for tension members and cast-iron fittings for compression joinery strengthening and simplification. These techniques developed to the point where one could find catalog offerings for this hardware. Around the same time, early steel mills were only producing small and simple cross sections that could be riveted into larger members – which could be, in turn, connected to each other to build trusses with either lovely pins and eye bars, or riveted gusset/cover plates at the connections. These brutish, big cover plates only lasted until a better method came along. As soon as site welding became prevalent, steel trusses went to simpler welds – and for some of the same reasons that make big steel gusset plates such sorry truss connections in timber.

The problems with mixing large steel plates with timbers in trusses starts with fabrication. 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 bolts to fit through the far hole – thereby reducing the bearing capacity of that bolt significantly. Even with perfectly aligned holes, load sharing among multiple heavy bolts in a row can be so uneven that we have reduction factors to account for this - and the reduction can be severe enough that adding another bolt in a row can weaken the connection. Timbers that dry after truss fabrication can wreak havoc; as the timber shrinks and the holes try to close up across the grain, the steel prevents this and can induce splitting in the timber. The reduction in capacity for some joints can be as high as 60%. In certain instances, the capacity of a joint can be increased by removing bolts. The differing thermal properties of timber and steel can also raise hell, especially in un-tempered environments. Even when protected from rain, timbers in open mill buildings will decay under the plates and around the heavy bolts, and the steel will rust, as moisture condenses on the chilled steel each morning.

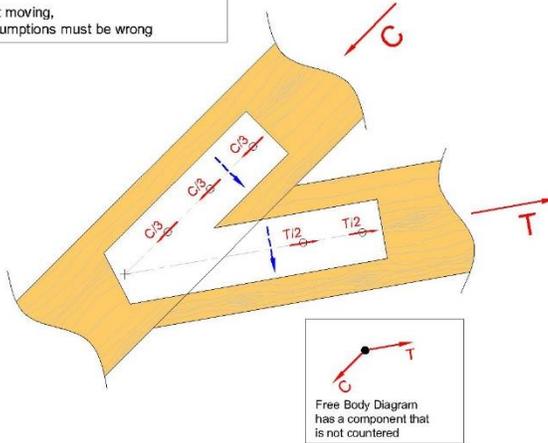
But even more insidious than these fabrication realities and interaction concerns are the likely prying tendencies caused by the eccentric load path through many bolted gusset plates. One basic problem seems to be the fading familiarity with Free Body Diagrams to analyze and assess the flow of forces between and among objects. The axial force (and possible shear) in one truss member is transferred by bolts into the gusset plates, and then the forces are transferred back out through bolts into the other connected member. We can look at the side plates as separate bodies and, recognizing that the plates are just as unmoving (neither shifting up or down, nor spinning about any point) as are the connected timbers, we can analyze the forces being transferred to the gusset plates by the bolts in such a way as to allow for this lack of motion – or equilibrium among the applied forces and moments.

The bolt loading that many designers seem to wish for

The plate simply cannot be in equilibrium if the bolts are loaded in the orientation shown.

There is a net force that is not countered and would mean the plate must move.

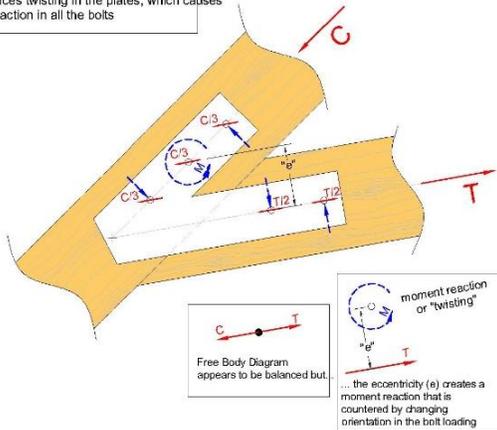
Since it is not moving, our force assumptions must be wrong



Factors that affect the orientation of bolt loading

The line of action of the T bolts is offset by (e) from the centroid of the compression bolt pattern.

This induces twisting in the plates, which causes a prying action in all the bolts



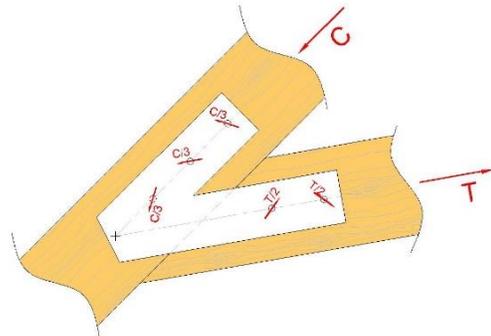
The "actual" bolt loading that may be more realistic

The bolt forces that the plates actually feel are quite complex and are combinations of the shared (we assume equally, and certainly incorrectly) basic axial loads.

The bolt force components that are induced by resisting eccentric load paths are:

- Perpendicular to the line of action of the bolts in a member and
- Proportional to the bolt's distance from the centroid of the "bolt group"

There are closed form solutions for analysts to use in determining the final combined forces on any bolt; but they are not trivial and only rarely used, in this author's experience.





Straps on repaired scissor truss – basically, the aspect ratio of this plate and its connectors makes it behave more like a single axial strap (or, even, akin to a fully-threaded screw)



Pinned straps on scissor truss

Hardwood Pegs

All timber connections require some type of fastener to hold the joint together, and sometimes to transfer structural forces through the connection. For traditional timber joinery, the fastener of choice is a hardwood peg, traditionally called a tree-nail or trunnel. They are usually made from straight grain, seasoned hardwood, and are most commonly one-inch diameter with a tapered end.



White Oak is an extremely common species to use for pegs, but locust, ash, maple or hickory are also options. Pegs can either be turned on lathe, or riven.

Ideally, pegs should be used as a positive connection in compression joinery. In such cases, the axial load is transferred along bearing surfaces of the joining members. Shear capacities of pegs play little role in the overall success of the joint, unless the bearing surfaces are not exactly square to the axial load being transferred.

It is not always practical to have compression joinery at every truss joint and sometimes a tension joint is unavoidable. A pegged mortise and tenon joint does have a modest amount of tension capacity. The

capacity of the joint is typically controlled by the shear strength of the pegs and the splitting strength perpendicular to the grain of the mortised timber.

The shear strength of a hardwood peg is covered in ***Standard for Design of Timber Frame Structures and Commentary (TFEC 1-2019)*** and explained further in ***TFEC Technical Bulletin No. 01***. Finally, ***TFEC Technical Bulletin No. 2016-07***, discusses evaluation of splitting strength of timber perpendicular to grain and its effect on the ideal edge spacing for pegs. Depending on the species of timber and peg, the shear capacity of a 1-inch diameter peg is approximately 1 kip. Consequently, if the tension force on a joint exceeds 5 kips, it is usually impractical to develop the required connection capacity with hardwood pegs loaded in shear. Steel bolted connections are substantially stronger than pegged joints. As a structural fastener, a hardwood peg is a poor substitute for a steel bolt. The peg does hold some advantages over a bolt: driven into tight holes, less take-up and better load sharing; equivalent thermal and moisture content behavior are a few.



Pegged mortise and tenon king-post connection to a bottom chord

Nuts and Bolts

As a structural fastener, a steel bolt is a bit of a brute that lacks the grace, elegance, or romance of a hardwood peg. A bolt is, without a doubt, substantially stronger than a hardwood peg when loaded in shear. When the fasteners are used to transfer joint forces, a hardwood peg can't compete with a steel bolt.



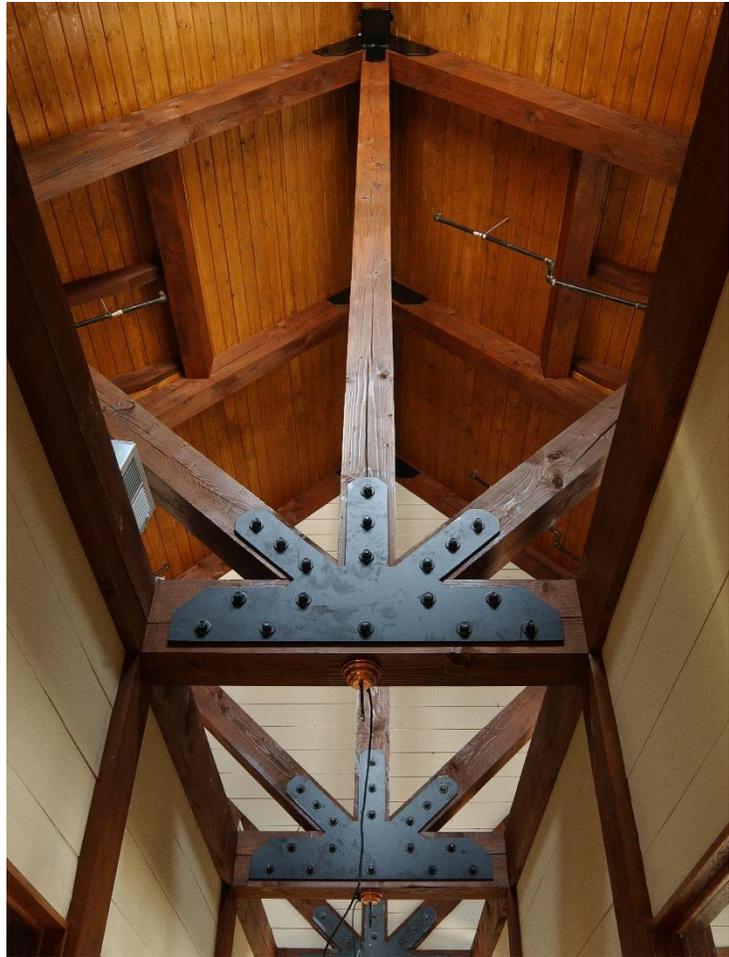
A hardwood peg is a poor substitute for a steel bolt as a structural fastener

Bolted connections have long been popular with engineers, particularly engineers who are more comfortable engineering a structural steel frame than a timber truss. The engineering of a bolted connection is covered in depth in the *AWC National Design Specification for Wood Construction* (NDS). Determining bolt capacity is a tedious process described in cookbook fashion in the NDS and facilitated by a free on-line connection calculator.

It is not uncommon for bolted timber connections to emulate structural steel connections, with rows of bolts and steel side plates, also called gusset plates. It is also not uncommon for an engineer to specify high-strength bolts for a timber connection for no other reason than that is what is common practice in structural steel construction. Since the strength of a bolted timber connection is seldom governed by bolt strength, specifying high-strength bolts is foolish.

Bolted gusset plate truss connections emulate structural steel construction.

Photo courtesy of Carolina Timberworks



While timber structures generally perform well in a fire due to the insulating value of the external char layer that develops, timber structures with exposed gusset plates and bolts often fail prematurely and suddenly during a fire. It is better to use internal steel kerf plates that are protected from the heat of a fire by the surrounding wood.

Tight fitting bolt holes are crucial with bolted connections. If the bolt holes are oversized or reamed to facilitate fit up, unequal load sharing between bolts can result, as well as excessive joint distortion and truss deflection.

When engineering a timber truss connection, it is far more efficient to configure the joint so that the bolts are loaded in tension rather than shear (see previous discussion).

Ogee Washers

Cast iron ogee washers perform better than flat washers where a bolt head or nut is bearing on the face of a timber. They also look cool, particularly when paired up with a square nut.



Split Rings and Shear Plates

Split rings have been used for timber truss connections since 1934 when the *Timber Engineering Company (TECO)* purchased the rights to the split ring connector from a European manufacturer. The engineering of split ring connections is covered in detail in the NDS and these provisions have changed little since the 1944 edition.

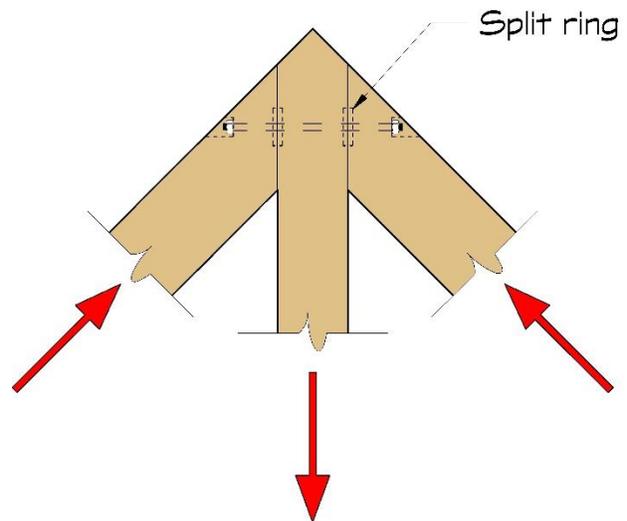


A special tool cuts a groove in the face of each timber to be joined. The diameter of the groove is slightly larger than the diameter of the split ring. When the split ring is seated in the groove, it has to expand and is said to have been *sprung*, which explains the purpose of the split in the ring. This allows the split ring to accommodate

seasoning shrinkage of the timbers. A center bolt clamps the joint together, but all of the shear is transferred through the split ring, not the bolt.

A split ring has substantially higher shear capacity than a single bolt. It is a practical connector when you wish to minimize the number of bolts in a connection. Since the spacing requirements between split rings is substantially greater than those for bolts, it is seldom practical to put more than two split rings in a particular truss connection.

Shear plates are made of malleable cast iron. They behave similar to split ring connections, but unlike split rings, they can be used to connect a timber to a steel gusset plate. Shear plates can be used in place of split rings, and can be much easier to fit in groups, because of the reduced tolerances involved with shear plates. But split rings are immediately snug, remove less wood, and are cheaper – high value connectors, if done well.



Top chord connection to king-post

Tension Joinery

Resisting tension is not something that timber joinery does well. Through tenons are commonly used when the tension force is modest. The strength of the joint is usually controlled by the shear strength of the pegs.

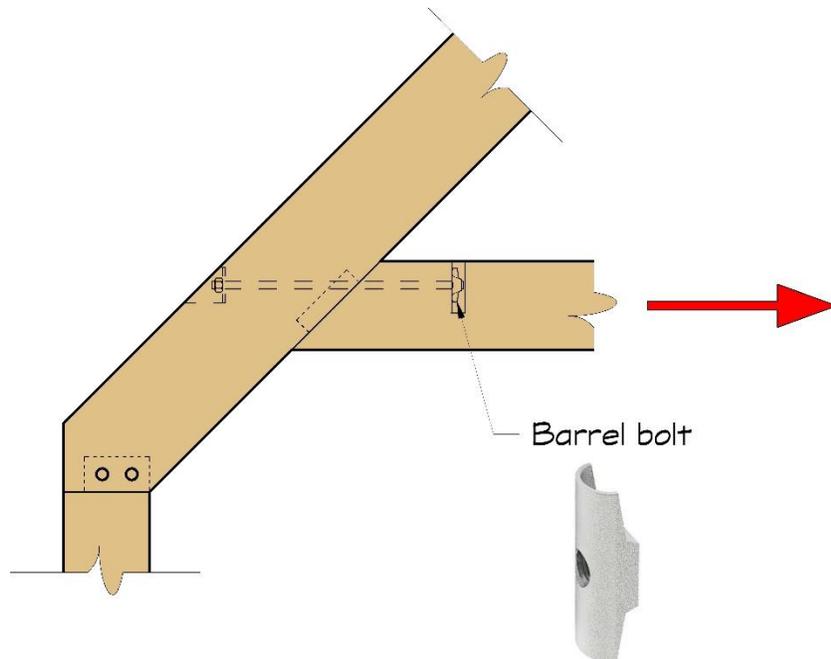


Variations on the through tenon joint include the keyed through tenon joint (see TFEC Technical Bulletins 2016-08 and 2018-08A) and the wedged dovetail through tenon joint (see TFEC Technical Bulletin 2018-13).

If the tension forces that must be resisted are substantial, some form of steel fasteners and connection hardware is usually required. Bolted gusset plates are always an option, but seldom a good option.

A double through tenon joint with the pegs loaded in quad shear, effectively doubling the peg capacity.

Connections with bolts loaded in tension rather than shear are usually a smarter choice. Barrel bolt connections are very efficient at resisting tension forces and the steel hardware is out of sight. If you are going to use steel fasteners, it is nice to be able to hide them from view. Bear in mind that it is seldom possible to tighten the bolt after the timbers have seasoned.



Special Considerations

Truss Bracing

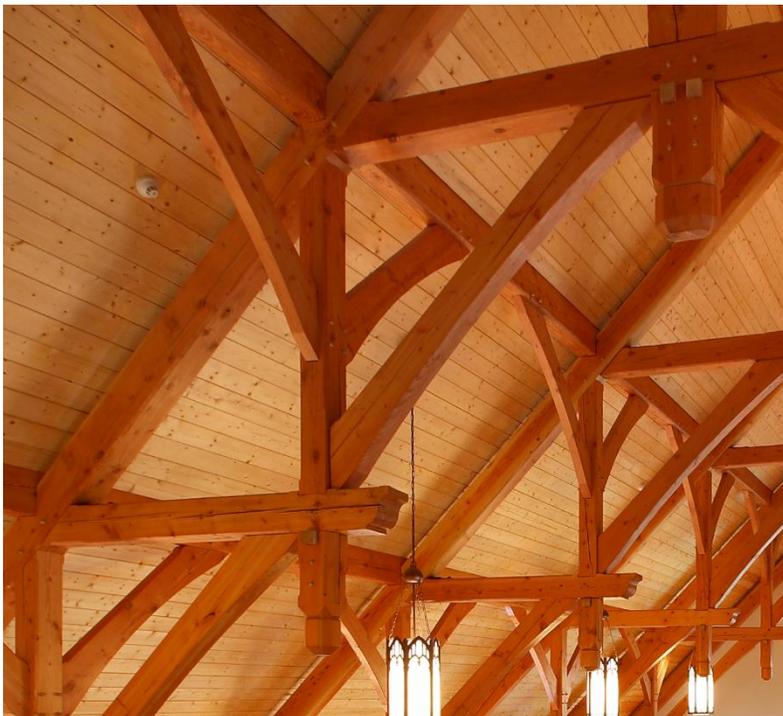
Most timber trusses of modest span require little if any supplemental bracing. In most cases, the roof construction effectively braces the top chords. Struts connected to the bottom chord are effectively braced since the tension in the chord exerts a restoring force should the strut attempt to buckle out of plane. Hammer-beam structures are a notable exception – you can never have too much bracing on a hammer-beam.

For long span trusses, or structures in high wind regions, failure to provide adequate bracing can have dire consequences.

Trusses in exterior walls are subject to transverse wind loads. Unless the truss bottom chord is capable of resisting the wind loads about its weak axis, some bracing is usually required. Knee braces from roof purlins are often the most practical method of providing out of plane bracing.

In high wind regions, wind uplift pressures can sometimes exceed the dead load of the structure. This can result in stress reversal in the truss members. If the bottom chord goes into compression, it may be too slender to resist buckling. In such cases, bracing of the bottom chord may be required.

When designing stability bracing to restrain buckling of compression members, the brace should be capable of resisting an axial compression load of not less than 5% of the axial load in the member being braced. Braces between trusses need to be stabilized by cross bracing, otherwise, all of the connected trusses could buckle sympathetically.

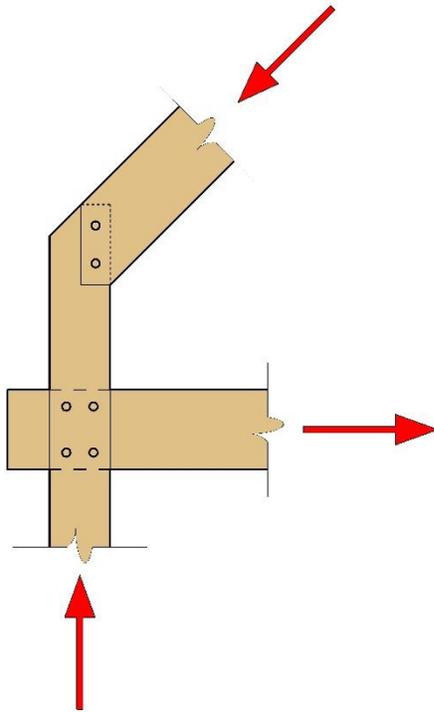
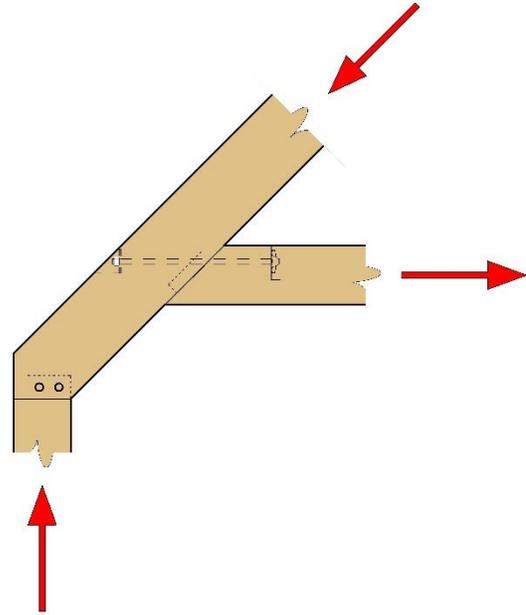


Hammer-beam trusses braced to roof purlins

Raised and Dropped Bottom Chords

It is sometimes necessary to position a truss bottom chord above or below its ideal location for architectural reasons.

When the bottom chord is raised above its ideal location, it usually connects to the face of the top chord as shown in the drawing to the right. This condition introduces very substantial bending moments and shear into the segment of the top chord between the support post and the bottom chord. The higher the bottom chord is raised, the greater the magnitude of the bending and shear stresses in the top chord and the tension force in the bottom chord. Bear in mind that any mortise cut in the top chord will serve to weaken the member at its point of maximum stress.



When the bottom chord is dropped below its ideal location, it usually connects to the side of the post as shown in the drawing to the left. This condition introduces very substantial bending moments and shear into the segment of the post above the bottom chord, sometimes causing the post to split. The lower the bottom chord is dropped, the greater the magnitude of the bending and shear stresses.

Raised and dropped bottom chords should be avoided whenever possible.



Howe truss with a dropped bottom chord



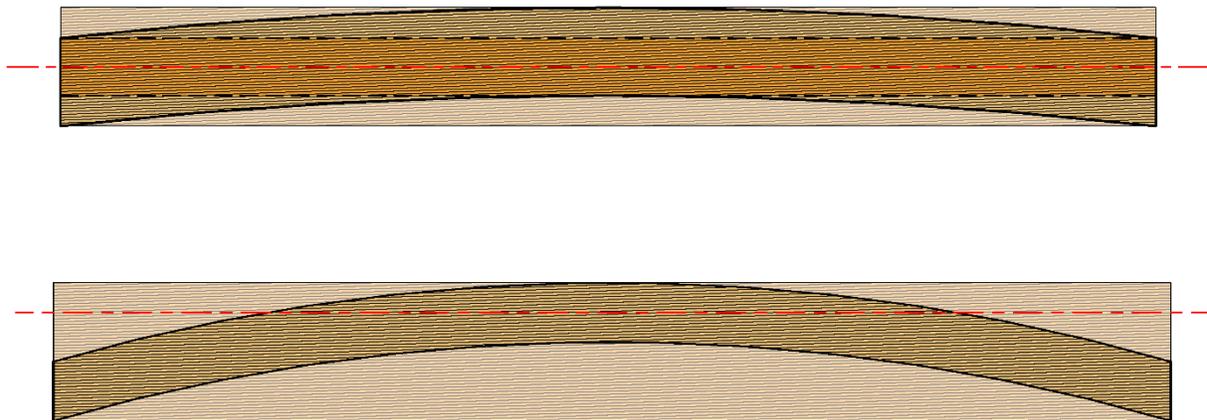
Raised bottom chord aka "the rafter buster"

Curved Members

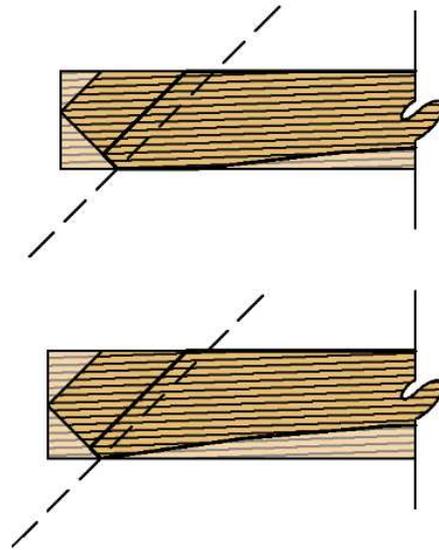
When building structural frames and trusses, straight milled timbers will always provide the most efficient use of the material. While naturally appealing in this form, the client or architect will often seek to soften the appearance of these exposed structures by adding curved members. A favorite explanation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

Curved members are sometimes created using glued-laminated (glulam) timbers, but this section is mainly focused on curved members cut from a solid timber. When a relatively straight tree 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 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 waste 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. The remaining wood, outside of this cross-section, simply creates the desired shape. For tension or compression only members, only the continuous straight cross section should be considered for calculations. 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.



At member ends, it is acceptable use the full depth cross section to transfer shear loads, but it is important that the entry/exit cuts for the curve do not create short grain "run out" corners at the ends of the member. 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. This condition can often be alleviated by using milder curves and/or moving the entry/exit point of the curve cut a few inches away from the intended bearing surface.



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



Another common curved member condition in trusses is a full bay arch extending between two vertical posts (i.e. queen posts) while also connecting to the bottom of a horizontal tie beam. In this case, multiple 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 mid-span 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.

We cannot really discuss curved members without at least touching on glulam curved beams. These products are constructed using multiple layers of dimensional lumber, bent to the specified radius, and then glued together. They are available in a variety of surface finishes to closely match all other solid-sawn timbers in a project. Grain matched curved laminated timbers, which are produced using much thinner laminations cut from a single timber, are nearly indistinguishable from a solid-sawn timber, and in some cases, can even appear too perfect by comparison.





Glulam trusses with curved bottom chords

Glulam beams are typically stronger and more dimensionally stable than solid-sawn timbers; however, when they are fabricated with a curve, unique forces will develop that sometimes demand increased cross sections or special reinforcement. Curved glulams subject to bending stress are also subject to radial tensile stress. Radial tension acts perpendicular to the radius of the curve and attempts to pull the laminations apart as the beam tries to straighten out. To account for this, the members cross section can be increased, or the laminations can be reinforced with structural screws installed perpendicular to the glue line.



Seasoning checks indicate the grain orientation.



Curved glulam bottom chord for Norm Abram's great room

Grain-Matched Laminated Timber



Shipbuilders from days gone by would steam bend timber planks to compound curves. For the ship's ribs, they would hunt for bent trees to cut their curved ribs from. Today, if you want a curve in a timber, it is best to use a laminated timber. Forming curved shapes is like falling off a log for a glulam manufacturer. But sometimes you



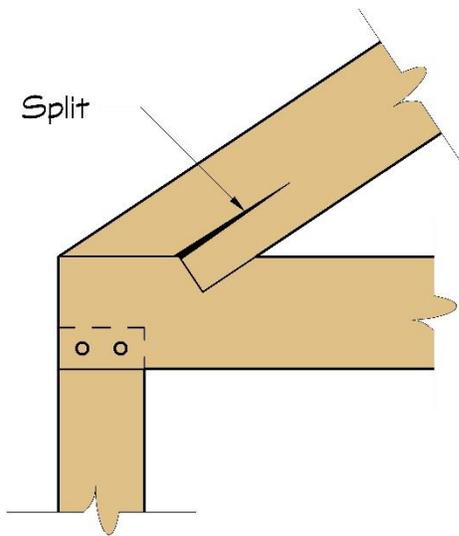
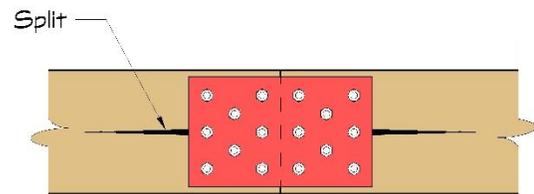
don't want to see the lamination stripes that are characteristic of a glulam. If you want it to look like it was always one piece of wood, then it calls for a "grain-matched" laminated timber. You start with a full-size sawn timber and then you saw it into a stack of thin slicers that are glued back together in the desired curved shape.

Seasoning Shrinkage Considerations

It is common to fabricate timber trusses from unseasoned timbers with a Moisture Content (MC) above the Fiber Saturation Point (FSP). The timbers will season and dry in service until the MC has reached its Equilibrium Moisture Content (EMC). As the timbers season and dry, they experience cross grain shrinkage. The magnitude of the anticipated cross grain shrinkage is approximately 4.5% depending on the wood species and EMC. Failure to consider the effects of seasoning shrinkage on truss connections can result in disappointing performance.

When bolts and steel rods are used to resist tension forces, they can become loose when the truss timbers shrink. It is advisable to have accessible nuts on bolts and rods to allow them to be tightened as the timbers shrink. A maintenance program of tightening should be established until the timbers have fully seasoned and reached their EMC.

Connections with steel gusset plates and multiple bolts can restrain seasoning shrinkage and induce splitting. These types of connections should be avoided if possible.



Bird's mouth cuts on heel joints can induce splitting of the top chord. Intentionally leaving a gap between the top chord and the bottom chord is seldom a practical solution.

Naturally, using Radio Frequency kiln dried timber or glulam timber will eliminate or minimize shrinkage considerations.

Aside from these considerations, when a timber end is cut at an angle, the cross-grain shrinkage will cause a change in the angle. As mentioned above, this must be accounted for if the connection relies on bearing at the connection.



The photo above shows a roof truss from a late nineteenth century building in New Haven, Connecticut. When the timbers seasoned, a compression strut fell out of the truss. When the plaster ceiling was removed for renovations, the timber strut was found lying on top of the attic joists where it had been resting for over a century. The moral to the story is that struts should be secured with lags or screws, don't rely on the clamping action of the tension ties to hold them in place.

Epilogue – Topped Out



Well there you have it – the collective wisdom from the current grand masters of timber engineering on the design and engineering of timber trusses.

Some have said that the golden age of timber truss construction ended a century ago, but that is not true – we are living in the golden age. While many of the surviving timber trusses built between 1880 and 1920 are fine trusses to be admired, and there is much to be learned from them, they were utilitarian structures that were only intended to support a roof or bridge. The trusses that we are building today can 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 is often an architectural masterpiece. Conversely, when an architect or designer attempts to design a timber structure without collaborating with an engineer, and designs a unique truss that is unlike anything that anybody has ever seen before, it seldom ends well.

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 and elegant unless it is also structurally efficient.



80-foot span queen-post trusses