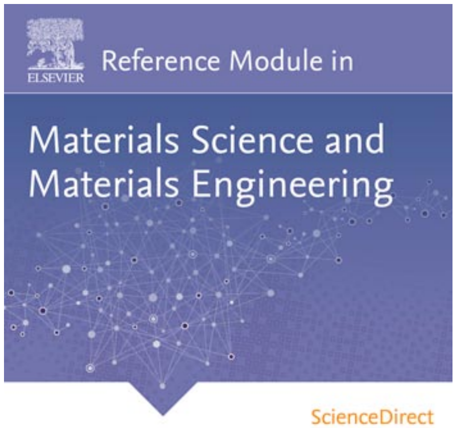

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Wood: Mechanical Fasteners☆

D Robert Rammer, USDA Forest Service Forest Products Laboratory, Madison, WI, USA

Introduction

The strength and stability of any structure depends heavily on the fasteners that hold its members together. One prime advantage of wood as a structural material is the ease with which wood structural parts can be joined together using a wide variety of fasteners: nails, staples, screws, lag screws, bolts, and various types of metal connectors. For the utmost rigidity, strength, and service each fastener type requires careful design. General requirements for moisture content, location, spacing, and fabrication in the design of joints should be checked with design manuals (American Wood Council, 2015; Beyer *et al.*, 2015; Porteous and Kermani, 2007). Each country has slightly different design approaches, capacity equations given within this text are largely based the United States experience.

Dowel Type Fasteners

Dowel type fasteners include nails, screws, and bolts. All dowel type fasteners are assumed to resist laterally applied loads by the same general engineering principles but withdrawal resistance of dowel type fasteners vary by type and will be addressed in the following section based on type. Bolts may be loaded in tension along there axis, but this is not considered withdrawal.

Withdrawal Strength

For dowel type fasteners the withdrawal capacity depends on the fastener type and is largely a function of the surface characteristic of the fasteners. Withdrawal strength expression for traditional nails, annularly threaded nails, screws, and lag screws will be presented.

Nails

The nail is the most common mechanical fastener used in wood construction. There are many types, sizes, and forms of nails, including common, box, annularly threaded, helically threaded, cement-coated, and galvanized (**Figure 1**).

Withdrawal resistance of a nail shank from a piece of wood depends on material density, nail diameter, penetration depth, and surface condition of the nail. For bright, common wire nails driven into the side grain of dry wood or green wood that remains wet, many test results have shown that the maximum withdrawal load is given by the empirical equation

$$p_w = 54.12G^{5/2}DL \qquad [1]$$

where p_w is the maximum load (N), L is the depth (mm) of penetration of the nail in the member holding the nail point, G is the specific gravity of the wood based on oven-dry weight and volume at 12% moisture content, and D is the nail diameter (mm) (Forest Products Laboratory, 2010).

Nail shanks may be deformed to increase the withdrawal strength. The form and magnitude of the deformations along the shank influence the performance of the nails in various wood species. In wood at uniform moisture content, the withdrawal resistance of these nails is generally somewhat greater than that of common wire nails of the same diameter. From tests in which nails were driven in the side grain of seasoned wood, bright annularly threaded nails, with shank-to-thread-crest diameter difference greater than 0.2 mm and thread spacing between 1.27 mm and 1.96 mm, the immediate maximum withdrawal load is given by the empirical equation

$$p_w = 73.11G^2DL \qquad [2]$$

where p_w is the maximum load (N), L is minimum of the depth (mm) of penetration of the nail in the member holding the nail point or length of thread, and the remaining variables were define previously. If a galvanized coating is applied to an annularly threaded nail the expression above will over predict mean withdrawal capacity by approximately 20%. For helically threaded nails, maximum withdrawal load is between value for the eqns [1] and [2]. (Rammer *et al.*, 2001; Rammer and Mendez, 2008).

☆*Change History*: March 2015. D. Rammer updated the text regarding three significant findings that have been made over the last few years related to the cyclic performance of nails, failure behavior, and deformed shank nails. Reference section has been updated with latest references.

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Figure 1 Various type of nails: (left to right) bright smooth wire nail, cement coated, zinc coated, annularly threaded, helically threaded, helically threaded, and barbed.

Staples

Pneumatically driven staples have been developed with various modifications in points, shank treatment and coatings, gage, crown width, and length. Most factors that affect the withdrawal and lateral loads of nails similarly affect the load on staples. Thus, eqn [1] is also used to determine maximum staple withdrawal load, but to date extensive verification tests have not been conducted.

Screws

Both wood screws and tapping screws are used in wood construction and are available in a wide range of materials and head types. The maximum withdrawal load p_w (N) for wood screws inserted in the side grain of seasoned wood may be expressed as

$$p_w = 108G^2DL \qquad [3]$$

where p_w is the maximum load (N), D is the screw shank diameter (mm) and L is the penetration length (mm) of the threaded part of the screw. This equation is applicable for screw lead holes with a diameter about 70% of the root diameter of the threads in softwoods, and about 90% in hardwoods. Withdrawal resistance of tapping screws is generally 10% greater when compared with wood screws of similar diameter and threaded length.

Recently, proprietary self-drilling screws have developed for cross laminated and heavy timber construction. Traditional wood or lag screws require pre-drilled holes, to reduce the wood splitting during insertion. Self-drilling screws thread designs typically incorporate a proprietary cutting feature at screw tip to eliminate pre-drilling and reduce wood splitting. Due to the proprietary thread design withdrawal capacity is not assigned by building codes in the United States. Self-tapping screws come in diameters that range from 4 to 12 mm and are available in lengths up to 600 mm.

Lag Screws

Lag screws are commonly used because of their convenience, particularly where it would be difficult to fasten a bolt or where a nut on the surface would be objectionable. Lag screws have larger diameter, when compared to a wood screws and a hexagonal-shaped head is used to tighten with a wrench (as opposed to wood screws, which have a slotted head and are tightened by a screw driver). The maximum withdrawal load p_w (N) for lag screws from seasoned wood is given by:

$$p_w = 125.4G^{3/2}D^{3/4}L \qquad [4]$$

Lag screws require lead holes that vary from about 40% to 85% of the root diameter, depending on the wood density.

Lateral Strength

The yield theory approach is used to determine the lateral strength of a dowel-type connection, assuming sufficient edge and end distances (American Wood Council, 2015; EuroCode 5). The yield theory describes a number of possible yield modes that can occur in a dowel-type connection (see **Figure 2**). Mode I is a wood-bearing failure in either the main or side member; mode II is a

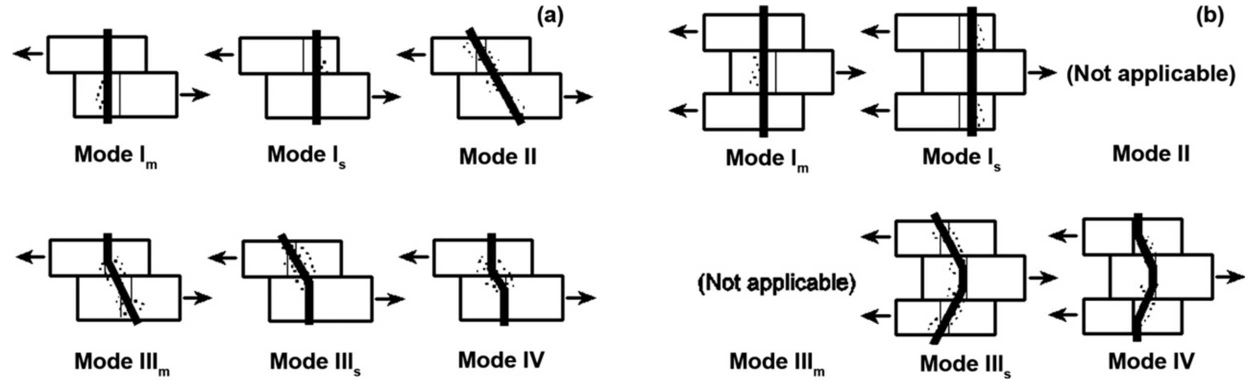

Figure 2 Various combinations of wood-bearing and fastener-bending yields for (a) two-member connections and (b) three-member connections.

rotation of the fastener in the joint without bending; modes III and IV are combinations of wood-bearing failure and one or more plastic hinge yield formations in the fastener. The yield strength of these different modes is determined from a static analysis that assumes the wood and the dowel are both perfectly plastic. It further assumes that no axial force is transmitted along the length of the dowel or friction between members. Axial force transmission can significantly increase the connection capacity, especially for threaded nails.

The yield mode that results in the lowest load for a given geometry is the theoretical connection yield load. Typically for small diameter nails, such as nails, the single shear yield expression applies; whereas, for larger diameter fasteners like bolts, the double shear yield expression applies. Modes I_m and II are not considered in nail and spike connections. The single shear yield mode equations (**Table 1**) are entered with the dowel- bearing strength, the dimensions of the wood members, the bending yield strength, and fastener diameter. Bending yield strength for each type of dowel is determined experimentally by conducting a 3 point bending until the plastic hinge has developed. The dowel bearing strength of the wood is determined experimentally by compressing a dowel into a wood member. In the United States, for dowels with a diameter less than 6.4 mm, bearing strength is assumed to be independent of grain orientation and dowel diameter. For dowel with diameters greater than 6.4 mm, the bearing strength is a function of grain orientation and is dependent on the dowel diameter when loading are applied perpendicular to grain.

Table 1 Yield theory expressions for single shear and double shear connections

Mode		*Lateral strength (Z) for single shear connection*	*Lateral strength for double shear connection*
I_m		Dl_mF_{em}	Dl_mF_{em}
I_s		Dl_sF_{es}	$2Dl_sF_{es}$
II		$k_1Dl_sF_{es}$	–
III_m		$\frac{k_2Dl_mF_{em}}{(1+2R_e)}$	–
III_s		$\frac{k_3Dl_sF_{em}}{(2+R_e)}$	$\frac{2k_3Dl_sF_{em}}{(2+R_e)}$
IV		$D^2\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$	$2D^2\sqrt{\frac{2F_{em}F_{yb}}{3(1+R_e)}}$
Definitions			
	D	Dowel diameter (mm)	
	F_{em}	Dowel bearing strength of main member (MPa)	
	F_{es}	Dowel bearing strength of side member (MPa)	
	F_{yb}	Dowel bending yield strength (MPa)	
	l_s	dowel bearing length in side member (mm)	
	l_m	dowel bearing length in main member (mm)	
	Z	lateral yield strength of connection (N)	
	R_e	$=F_{em}/F_{es}$	
	k_1	$=\frac{\sqrt{R_e+2R_e^2\left(1+R_t+R_t^2\right)+R_t^2R_e^3}-R_e(1+R_e)}{(1+R_e)}$	
	k_2	$=-1+\sqrt{2(1+R_e)+\frac{2F_{yb}(1+2R_e)D^2}{3F_{em}l_m^2}}$	
	k_3	$=-1+\sqrt{\frac{2(1+R_e)}{R_e}+\frac{2F_{yb}(2+R_e)D^2}{3F_{em}l_s^2}}$	

For angles of loading other than parallel or perpendicular to the grain, the bearing strength or connection load may be computed from the parallel and perpendicular values by using the Hankinson formula. For determining the bearing strength of wood at various angles to the grain,

$$N = \frac{PQ}{P\sin^2\theta + Q\cos^2\theta}$$

where P is load or stress parallel to the grain, Q load or stress perpendicular to the grain, and N load or stress at an inclination θ with the direction of the grain.

Additional factors that affect the withdrawal and lateral load carried include type of shanks, length of time the dowel remains in the wood, surface coatings, and moisture content changes in the wood.

Deformation Models

With the advancement of computing and structural analysis, the understanding of wood building system behavior has increased. A key parameter to understanding light frame construction system performance is the fastener deformation behavior. Models to described fastener behavior are presented for service, ultimate and cyclic loading conditions.

For most situations, the performance of the structure is only a concern while experiencing service loads, such that the fastener behavior remains linear elastic. While the US codes do not published connection slip modulus values, EuroCode5 (EN 1995–1–1–1, 2004) publishes a connection slip modulus values for dowel type connections that approximates the fastener deformation behavior to a load level that is approximately 40% of the maximum load. A load level assumed to be near the upper limit of the serviceability region. For a fastener inserted without drilling or splitting of the wood, the following expression, based on many tests, was developed assign a connection slip modulus

$$K_{service} = \rho_m^{1.5}\frac{d^{0.8}}{30} \qquad [5]$$

where ρ_m is the mean density, based on volume at 12% moisture content, of the wood based material and d is the dowel diameter. Wilkinson (1971) also developed the initial connection stiffness for nailed connection using a beam on elastic foundation approach to describe the lateral nail connection response.

EuroCode5 also published slip modulus of dowel type connections that required a pre-drilled hole to assist insertion into the wood, such as bolts and screws. Based on tests the following expression was developed

$$K_{service} = \rho_m^{1.5}\frac{d}{23} \qquad [6]$$

where ρ_m is the mean density, based on 12% moisture content volume, of the wood based material and d is the dowel diameter. For connection with oversized pre-drill holes, the hole tolerance should be included in the calculated total joint deformation, **Figure 3**.

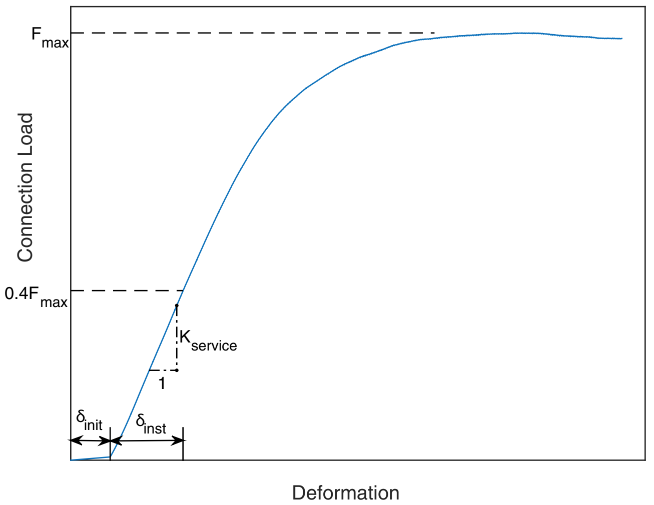

Figure 3 Typical relation between lateral load and joint deformation.

For wood system behavior at large deformations or near failure the nonlinear behavior of the connection needs to be considered. Researchers (Foschi, 1977; McLain, 1975) have advocated different expression to characterize the nonlinear behavior, but the model develop by Foschi (**Figure 4**) has gained acceptance for a monotonic loading and has been incorporated into cyclic connection behavior models.

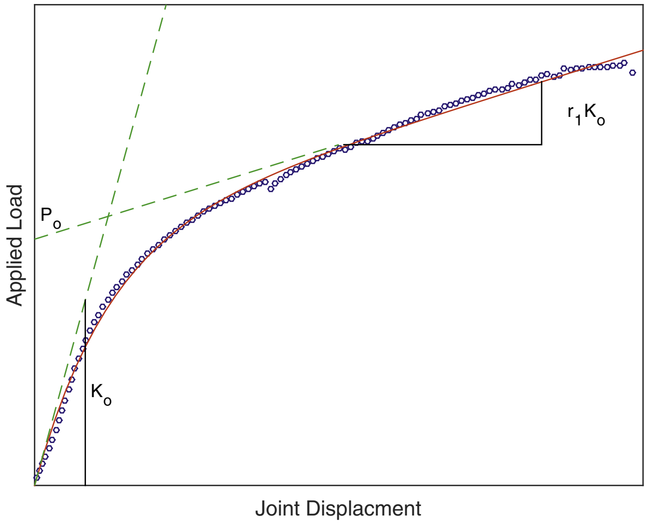

Figure 4 Typical dowel fastener load deformation curve showing the Foschi model parameters.

Foschi's model takes the following form:

$$P = (P_O + r_1 K_O \delta)\left(1 - e^{-K_O\delta/P_O}\right) \quad [7]$$

$$P = (P_u + r_2 K_O \delta)(\delta - \delta_u) \quad \delta_u \le \delta \le \delta_F \quad [8]$$

where K_O is the initial connection stiffness, r_1 is relates the asymptotic stiffness to the initial stiffness, and P_O is the intersection of the asymptotic stiffness with the y axis and r_2K_o is the post ultimate load slope of the load–deformation curve, δ_u is the deformation at ultimate load, and δ_F, is the deformation at failure. These parameters are determined experimentally for the given connection of interest. If post maximum load connection behavior needed, the linear reducing expression (eqn [8]) is applied after the critical displacement, δ_u, is achieved.

Recently, the load–deformation behavior of dowel type fasteners subjected to repeated loading has become of interest. The behavior of wood structures to dynamic or repeated loading condition from high wind or earthquakes is strongly linked to fastener models that consider the reversal of loading (Folz and Filiatrault, 2001). One hysteric model for nailed connection, called the modified stewart, is an extension of the previous monotonic fastener load–deformation model to a specific cyclic loading protocol as shown in **Figure 5**, where load–displacement paths OA and CD follow the monotonic envelope curve as expressed by eqn [8]. All other paths are assumed to exhibit a linear relationship between force and deformation. Unloading off the envelope curve follows a path such as AB with stiffness r_3K_0. Here, both the connector and wood are unloading elastically. Under continued unloading, the response moves onto path BC, which has reduced stiffness r_4K_0. Along this path, the connector loses partial contact with the surrounding wood because of permanent deformation that was produced by previous loading, along path OA in this case. The slack response along this path characterizes the pinched hysteresis displayed by dowel connections under cyclic loading. Loading in the opposite direction for the first time forces the response onto the envelope curve CD. Unloading off this curve is assumed elastic along path DE, followed by a pinched response along path EF, which passes through the zero-displacement intercept F_I, with slope r_4K_0. Continued reloading follows path FG with degrading stiffness K_p. Hysteretic fastener models are not single analytical expressions and are typically implemented in computer models.

Multiple Fasteners

For connections constructed with more than two dowel type fasteners or wood connectors in a row, the total joint capacity is not simply the sum of the individual fastener maximum capacity of the fasteners in that row.

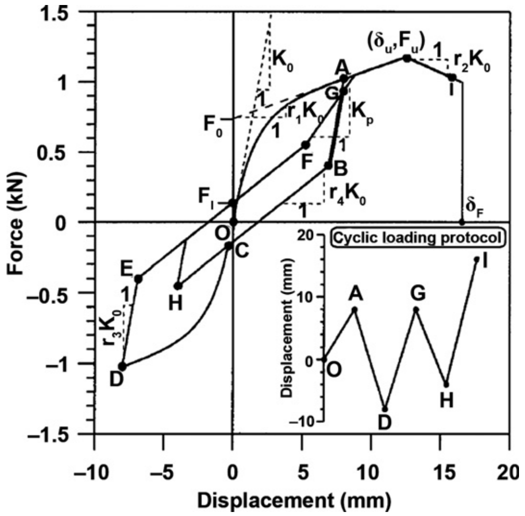

Figure 5 Modified Stewart hysteretic model for the behavior of laterally loaded wood connection for a specific cyclic loading protocol.

When fasteners are used in rows parallel to the direction of loading, total joint load is unequally distributed among fasteners in the row, and theoretically the two end fasteners carry a majority of the load. Simplified elastic methods of analysis have been developed to predict the load distribution among the fasteners in a row. These analyses indicate that the load distribution is a function of the extensional stiffness of the joint members, the fastener spacing, the number of fasteners, and the single-fastener load–deformation characteristics.

Theoretically, the two end fasteners carry a majority of the load. Adding bolts to a row tend to reduce the load on the less heavily loaded interior bolts. The most even distribution of bolt loads occurs in a joint where the extensional stiffness of the main member is equal to that of both splice plates. Increasing the fastener spacing tends to put more of the joint load on the end fasteners. Load distribution tends to be worse for stiffer fasteners.

Design modification factors used to reduce the capacity of a row of bolts, lag screws, or timber connectors have been developed based from theoretical analyses. The group action factor, C_g, was developed based on an algebraic simplification of the Lantos analysis for use in the US (Lantos, 1969; Zahn, 1991). The group action factor equation is given by:

$$C_g = \left[\frac{m(1-m^{2n})}{n(1+R_{EA}m^n)(1+m)-1+m^{2n}}\right]\left[\frac{1+R_{EA}}{1-m}\right]$$

where n is the number of fasteners is a row, R_{EA} is the smaller ratio of member stiffness, $m = u - \sqrt{(u^2-1)}, u = 1 + \gamma\left(\frac{s}{2}\right)\left[\frac{1}{E_m A_m} + \frac{1}{E_s A_s}\right]$, E_m is the modulus of elasticity of main member, E_s is the modulus of elasticity of side member, A_s and A_m are the area of the side and main member respectively, s is the spacing between fasteners in row, and is the load/slip modulus of for the specific type of connection. Actual load distribution in field-fabricated joints is difficult to predict because of fastener misalignment, spacing variations, and variability of the single-fastener load–deformation characteristics.

In 2001, experiments showed that closely spaced multiple fastener connections can fail before reaching the yield limit predicted connection capacities with application of the group action factor. For multiple fastener connections loaded in tension three possible failure modes must be consider in design: net section tension, row tear out, and group tear out. The net tension mode considers the affect the reduced cross section, as a result of the fastener holes, has on the joint. Row tear out considers the failure of the wood along the perimeter of each row of fasteners. Group tear out considers the shearing failure along the perimeter rows of fasteners and net section tension failure between outer rows of fasteners (**Figure 6**). Several approaches have been developed using classical mechanics or fracture mechanics based approaches for tension parallel to grain loading but further development is needed for case where the multiple fastener connection applies a force perpendicular to wood grain.

Timber Rivets

Timber rivets were developed for use in heavy timber structures in Canada during the 1970s to create an indeterminate structures and by so doing reduce the force demands on the wood structural elements (Madsen, 2000). Typical applications for timber rivets include tension splices, beam hangers, and moment resisting splices. The rivet itself has a flattened-oval shank with a tapered head. A timber rivet connection consists of a pre-drilled steel plate through which the rivets are driven into the underlying wood member. When driven though the pre-drilled steel plate the tapered timber rivet head is fixed against rotation and movement and

Figure 6 Failure of the multiple fastener connection due to group tear out.

the resulting head fixity adds to performance of the connection. By aligning long oval axis of the rivet with the grain direction it does not cut the wood fibers and densifies the wood within the perimeter of the rivet connection, especially when the rivets are first driven around the perimeter.

For each load direction, parallel or perpendicular to grain, the ultimate carrying capacity failure modes are either rivet bending, localized crushing or the strength of the wood at the perimeter (see **Figure 7**). For the rivet bending or localized crushing, the end rivets will initially carry a larger percentage of the load in comparison to the inner rivets. As load is applied, the end rivets yield so that load is redistributed to the inner rivets. At maximum capacity all the individual rivets have reached their maximum bearing capacity, as long as the wood capacity around the group of rivet has not been exceeded. When the wood capacity governs, for parallel to grain loading, the wood tension and shear strength are the controlling factor on the perimeter of the rivet group, whereas, for perpendicular to grain loading, the tension perpendicular to grain is the controlling wood strength. For timber rivet connection, design loads are based on the lower of maximum bearing or bending capacity of the rivet or the maximum load based on the wood strength. When load are applied at an angle to grain, the Hankinson formula presented for dowel connection is applied.

Split Rings and Shear Plates

Several types of connectors have been devised that increase joint bearing and shear areas by utilizing split rings as shown **Figure 8** or shear plates around bolts holding joint members together, see **Figure 9**. These connectors require closely fitting machined grooves in the wood members. The primary load-carrying portions of these joints are the connectors; the bolts usually serve to prevent transverse separation of the members but do contribute some load-carrying capacity. While the split ring can only be used for a wood to wood connection, shear plates be used for both wood to wood and wood to steel connections, but for wood to wood connection a shear plate is required in each wood member. The capacity of these connector when loaded parallel to grain is the smaller of the bearing failure of the wood around the connector or a shear plug failure of the wood at the end of the loaded member.

The strength of the split rings and shear plates are higher than laterally loading dowel connections and are used uses more in glued laminated arches and heavy timber connection. Joint strength depends on the type and size of the connector, the species of

Figure 7 Timber rivet failure modes: (clockwise from top left) rivet bending, rivet bearing, and wood tear out.

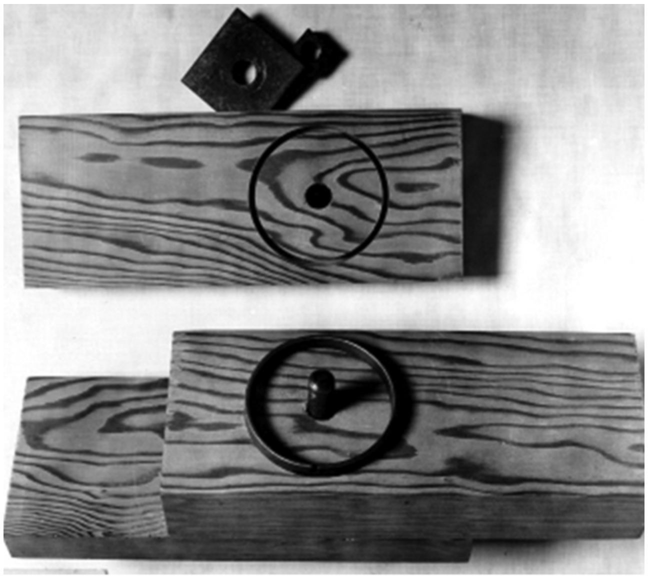

Figure 8 Joint with split ring connector showing connector, precut groove, bolt, washer, and nut.

wood, the thickness and width of the member, the distance of the connector from the end of the member, the spacing of the connectors, the direction of application of the load with respect to the direction of the grain of the wood, and other factors.

Metal Plate Connectors

Metal plate connectors, commonly called truss plates, have become a popular means of joining, especially in trussed rafters and joists. These connectors transmit loads by means of teeth, plugs, or nails, which vary from manufacturer to manufacturer. Examples of such plates are shown in Figure 10. Plates are usually made of light-gage galvanized steel and have an area and shape necessary to transmit the forces on the joint. Installation of plates usually requires a hydraulic press or other heavy equipment, although some plates can be installed by hand.

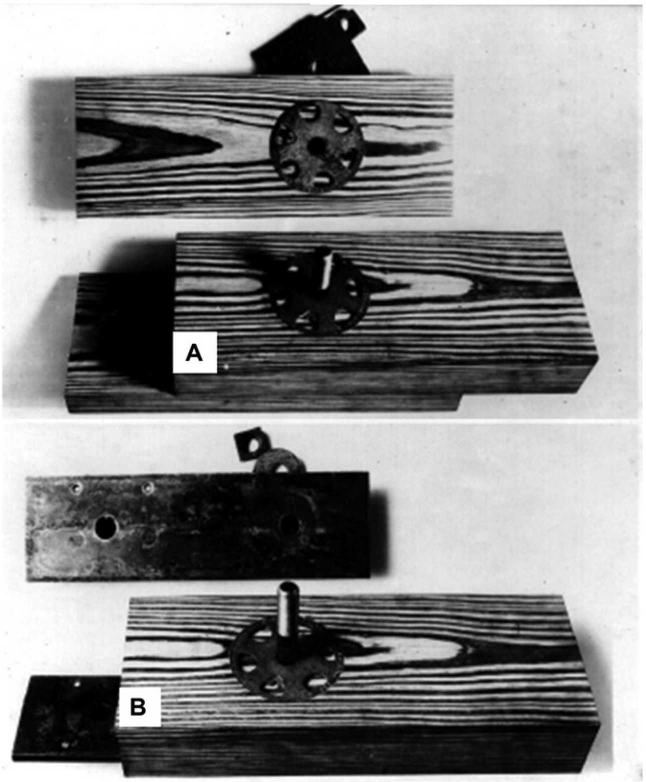

Figure 9 Joints with shear plate connectors with (A) wood side members and (B) steel side plate.

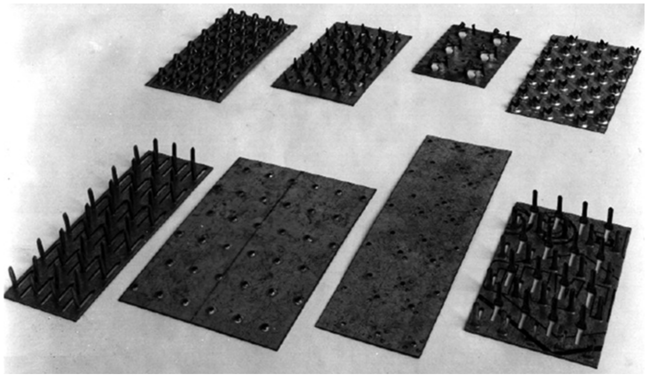

Figure 10 Examples of metal plate connectors.

Basic strength values for plate connectors are determined from load–slip curves from a series of tension tests of two butted wood members joined with two plates. Design load for normal duration of load in the United States is based the lowest of the following: load at 0.38-mm slip from plate to wood member (0.76-mm for slip measured from member to member) is divided by 1.6; or the average ultimate load divided by 3.0. Design values are expressed as load per tooth, nail, plug, or unit area of plate. The strength of a metal plate joint may also be controlled by the tensile or shear strength of the plate.

Hangers

Joist hangers have become a popular means of joining wood based joists to header beams or columns. Hangers are usually made of light-gage steel or welded from plate steel with shape and configuration necessary to transmit forces through the joint. Loads are transmitted from the joist to the hanger primarily through direct bearing of the joist, but for the uplift forces, load transfer is due to lateral loading of fasteners. How loads are transferred from the hanger to the header differs depending on whether the joist hanger is a face mount or top mount. Face-mount hangers transmit loads through lateral loading of dowel-type fasteners; top-mount hangers transmit loads by bearing on the top of the header and lateral loading of the dowel type fasteners. Design of the joist hanger varies from manufacturer to manufacturer. Examples of such plates are shown in **Figure 11**.

Figure 11 Typical joist hanger connectors.

Design loads are limited to the lowest values determined by experiment or by calculations in the United States. By experiment, design loads for joist hangers are determined from tests in which a joist is loaded at mid-span and supported by two joist hangers attached to headers, following ASTM D 7147 (2014) procedures. The smallest value as determined by two different means is the test design load for normal duration of load: the average load at 3.2-mm deformation between the joist and header or the average ultimate load divided by 3.0. Design loads for calculations are also highlighted in ASTM D 7147 (2014).

Concluding Remarks

The design and applications of Wood Fasteners in the US context have been presented and discussed in reasonable details in this article. It is evident that there are a wider range of materials and design considerations which come into play for different applications.

More detailed discussion on the behavior of mechanical connection in wood construction can be found in the *Wood Handbook* (Forest Products Laboratory, 2010), Madsen (2000), and American Society of Civil Engineers (1996).

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