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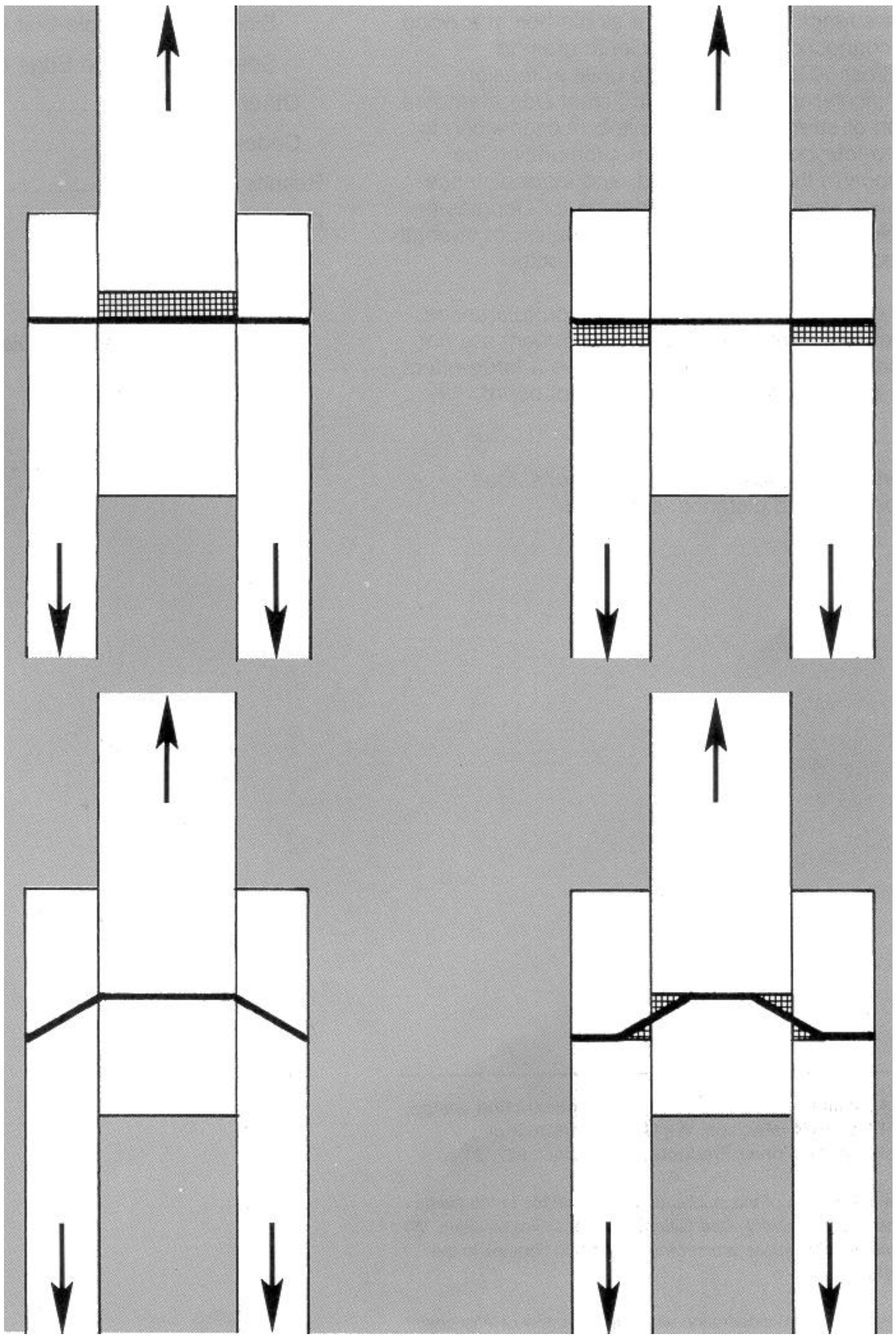
FPL-GTR-54

Bolted-Connection Design

Lawrence A. Soltis
Thomas Lee Wilkinson



Abstract

Recent failures of bolted connections have raised doubts about our knowledge of their design. Some of the design criteria are based on research conducted more than 50 years ago. This paper compares results found in the literature, using the European Yield Theory as a basis of comparison, to summarize what is known about bolted-connection design and what needs further research. By putting all this information in one place we hope to help engineers and architects design safer timber buildings and structures.

In general, the strength is known for a single bolt in a wood side member connection loaded parallel to grain in compression. Less is known for single bolts in tension, loaded perpendicular to grain or having steel side members. The distribution of strength in a multiple-bolt connection is known for up to four bolts in a row. In summarizing the literature we confirm the spacing, end, and edge distance requirements in current multiple-bolt design for Douglas-fir connections. No information exists for distribution of strength or spacing requirements for multiple rows of bolts.

The effects of other factors such as fabrication tolerances, duration of load, and preservative or fire treatment are not known. Fabrication tolerances appear to have a large effect on connection strength, but this effect has not been quantified.

Keywords: Connections, fasteners, timber, bolts, load distribution, spacing, end distance, structures

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Contents

Errata

February 1990

Bolted-Connection Design

Lawrence A. Soltis
Thomas Lee Wilkinson

United States Department of Agriculture

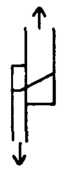
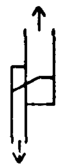
Forest Service

Forest Products Laboratory

General Technical Report FPL-GTR-54

Page 4, Table 2

The yield theory equations for joints 3 and 3A should be transposed as follows:

3		$F_y = \frac{DS_2L_2}{(1+2\beta)}\left[\sqrt{2(1+\beta) + \frac{2S_y(1+2\beta)}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$	$P_n = \frac{S_2}{f_{c_2}(1+2\beta)}\left[\sqrt{2(1+\beta) + \frac{2S_y(1+2\beta)}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$
3A		$F_y = \frac{DS_2L_2}{\alpha(2+\beta)}\left[\sqrt{\frac{2(1+\beta)}{\beta} + \frac{2S_y(2+\beta)\alpha^2}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$	$P_n = \frac{S_2\frac{L_1}{D}}{f_{c_2}(2+\beta)\left(\frac{L_2}{D}\right)}\left[\sqrt{\frac{2(1+\beta)}{\beta} + \frac{2S_y(2+\beta)}{3S_2\left(\frac{L_1}{D}\right)^2}} - 1\right]$

Bolted-Connection Design

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Introduction

Current U.S. design for the strength of a single-bolt connection is based mainly on the research conducted by Trayer in 1932. The strength of a multiple-bolt connection is calculated by summing the single-bolt strength values after multiplying by a modifying factor that depends on how the load is distributed to each bolt. The modifying factors derive from research in the 1960's (Cramer 1966, Lantos 1969). Strength in both single- and multiple-bolt connections is achieved by having sufficient spacing, end, and edge distances, as recommended by Trayer.

Recent failures have raised doubts about our basic understanding of bolted connections. The objective of this paper is to combine and compare data in the literature on single- and multiple-bolt connections to see what is known and what needs further research.

The strength and stiffness of a single-bolt connection depends on the physical and geometrical properties of the members and the bolt. Member properties include number and thickness of members, type of side members (steel or wood), wood species, moisture content, and direction of loading to the wood grain. Bolt properties include diameter and yield stress. Geometric properties include bolthole fabrication tolerances, spacing, and end and edge distances. In addition, multiple-bolt connection strength depends on the number of bolts in a row, the spacing and end and edge distance of bolt rows, the distance between bolt rows, and whether the rows are staggered or symmetric. Other factors that affect both single- and multiple-bolt connections are duration and rate of load and preservative or fire treatment.

During the past 60 years, several studies on bolted connections have been conducted. Each study has investigated one or more of the properties that affect connection behavior. Direct comparison is difficult because the studies usually differ in more than one connection property. Therefore, in this report we use the European Yield Theory (Johansen 1949) to form a common basis of comparison.

Significant conclusions appear to have been drawn from various results based on small samples having narrow ranges of connection properties such as the ratio of main member thickness to bolt diameter, L_2/D. The results indicate that current design values for the proportional limit of single-bolt connections are generally correct but that information on the load-slip behavior and the distribution of load among bolts is inadequate if the data are to be used for limit-states design or multiple-bolt connections.

Review of the Literature

Analytical

Single-Bolt Connections

The only model describing the strength of a single-bolt connection is the European Yield Theory originated by Johansen (1949).

McLain and Thangjitham (1983) examined this theory for bolted wood connections loaded parallel to grain and found good agreement between predicted and observed values. Soltis and others (1986) found agreement between predicted and observed values for both parallel- and perpendicular-to-grain loading.

The yield theory assumes that the bearing capacity of a bolted connection is attained when either (a) the compressive strength of the wood beneath the bolt is exceeded (Mode I failure) or (b) one or more plastic hinge develops in the bolt (Mode II or Ill failure). These assumptions provide for several modes of failure depending on connection member dimensions, member strength, and bolt strength. Failure modes are displayed in table 1 for three-member connections and in table 2 for two-member connections together with the formulas for the yield strength, F_y, corresponding to each failure mode.

A common way of presenting bolted-connection test results is to plot the normalized bolt-bearing stress versus the L_2/D ratio. The normalized bolt-bearing strength is

$$P_n = \frac{F_p}{L_2 D f_c} \tag{1}$$

where

P_n = normalized bolt-bearing strength
F_p = proportional limit strength, lb
L_2 = main member thickness, in
D = bolt diameter, in
f_c = main member compressive strength, lb/in^2.

If the yield strengths, F_y, are normalized, the formulas in tables 1 and 2 result.

There is a difference between proportional limit strength and yield strength (fig. 1). The proportional limit is defined as the point where the load-deformation curve becomes nonlinear. For this paper, the yield strength is defined as the load at the intersection of the tangents to the linear and nonlinear portions of the curve. It has been defined differently by some researchers. Thus the yield theory may be expected to give higher normalized bolt-bearing strengths than reported in the literature.

Multiple-Bolt Connections

Three types of analysis have been used in research on multiple-bolt connections: basic mechanics, finite element, and fracture mechanics analysis. Sometimes finite element and fracture mechanics analyses are combined.

The methods of basic mechanics suggested the use of modifying factors to account for the distributions of load among bolts in a row. lsyumov (1967) modeled connected members and bolted timber connectors as a series of linear or nonlinear springs and then used a flexibility matrix method of analysis. lsyumov verified his analysis by testing bolts in combination with connectors and shear plates. Cramer (1968) developed an analysis of mechanics based on the extensional elastic stiffness of the connected members, the nonuniform stress distribution in the members, and a connection slip modulus. He included deflection caused by bolt bending in his analysis. Cramer's work was based on earlier work for steel construction. He verified his analysis with perfectly machined connections having small L_2/D ratios. Lantos (1969) developed a similar approach except that he assumed uniform stress distribution in the members. His work does not contain experimental verification.

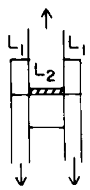
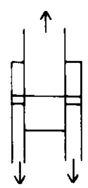
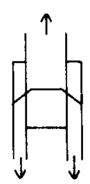
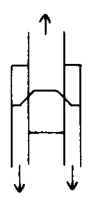
The modifying factors in current design practice (American Institute of Timber Construction (AITC) 1985, National Forest Products Association (NFPA) 1986) for load distribution between bolts is based on the Lantos (1969) and Cramer (1968) analyses. Wilkinson (1980) compared modifying factors calculated by the Cramer and Lantos methods and found the resulting values for proportional limit loads varied by less than 2 percent. He also found the two methods predicted the experimentally-found proportional limit strength but overestimated the failure strengths. This was to be expected because, by either method, the calculation assumes linear load-slip behavior in single bolts. Wilkinson (1986) extended Cramer's work by using a piecewise linear load-slip curve to predict failure loads. He took account of variability in single-bolt load-slip behavior and fabrication tolerances to reflect actual connections. He concluded that the load distribution in any row of bolts is unique and depends on the random fabrication effects on single-bolt load-slip curves.

Finite element analyses have been used to find the influence of the connection parameters on stresses and strength. Much of the finite element work was developed for orthotropic composite materials rather than wood.

Tsiang (1984) reviewed the literature on finite element analyses of composite laminates. Most of the studies cited dealt with two-dimensional analyses of a single bolt. References to multiple-bolt connections indicated that their failure mode was related to the ratio of the bolt load to the total applied connection load. As this ratio approached unity, the failure mode changed from a tensile or crack failure to a local bearing failure.

Wong and Matthews (1981) did a two-dimensional analysis of a two-hole bolt connection. They ignored the through-thickness stresses (tacitly assuming small L_2/D). Although their analysis showed some correlation with experimental data, they concluded a three-dimensional finite element method was needed.

Table 1.–Yield theory equations for three-member joints

Mode of failure number and failure geometry		Yield strength F_y	Normalized yield strength P_n
1a		$F_y = DL_2S_2$	$P_n = \frac{S_2}{f_c}$
1b		$F_y = 2DL_1S_1$	$P_n = \frac{2S_2}{f_c\alpha\beta}$
2		$F_y = \frac{2DL_2S_2}{\alpha(2+\beta)}\left[\sqrt{\frac{2(1+\beta)}{\beta}+\frac{2S_y(2+\beta)\alpha^2}{3S_2\left(\frac{L_2}{D}\right)^2}}-1\right]$	$P_n = \frac{2S_2\frac{L_1}{D}}{f_c\left(\frac{L_2}{D}\right)(2+\beta)}\left[\sqrt{\frac{2(1+\beta)}{\beta}+\frac{2S_y(2+\beta)}{3S_2\left(\frac{L_1}{D}\right)^2}}-1\right]$
3		$F_y = 2D^2S_2\sqrt{\frac{2S_y}{3S_2(1+\beta)}}$	$P_n = \frac{2}{f_c\left(\frac{L_2}{D}\right)}\sqrt{\frac{2S_2S_y}{3(1+\beta)}}$

D = bolt diameter.

S_y = bolt yield stress.

S_1 = embedment yield stress of side members.

S_2 = embedment yield stress of main member.

L_1 = side member thickness.

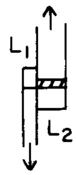
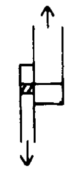
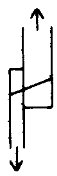
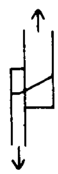
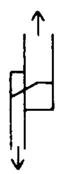
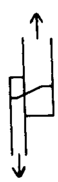
L_2 = main member thickness.

f_c = main member compressive strength.

$\beta = S_2/S_1$.

$\alpha = L_2/L_1$

Table 2.–Yield theory equations for two-member joints

Mode of failure number and failure geometry	Yield strength F_y	Normalized yield strength P_n
1 	$F_y = DL_2S_2$	$P_n = \frac{S_2}{f_{c_2}}$
1A 	$F_y = \frac{DL_2S_2}{\alpha\beta}$	$P_n = \frac{S_2^{\frac{L_1}{D}}}{\beta f_c\left(\frac{L_2}{D}\right)}$
2 	$F_y = \frac{DS_2L_2}{\alpha\beta} \; \frac{\sqrt{\beta + 2\beta^2(1 + \alpha + \alpha^2) + \alpha^2\beta^3} - \beta(1 + \alpha)}{(1 + \beta)}$	$P_n = \frac{S_2\frac{L_1}{D}}{f_{c_2}\left(\frac{\beta L_2}{D}\right)} \; \frac{\sqrt{\beta + 2\beta^2\left(1 + \frac{\frac{L_2}{D}}{\frac{L_1}{D}} + \frac{\left(\frac{L_2}{D}\right)^2}{\left(\frac{L_1}{D}\right)^2}\right) + \frac{\left(\frac{L_2}{D}\right)^2}{\left(\frac{L_1}{D}\right)^2}\beta^3} - \beta\left(1 + \frac{\frac{L_2}{D}}{\frac{L_1}{D}}\right)}{(1 + \beta)}$
3 	$F_y = \frac{DS_2L_2}{\alpha(2 + \beta)}\left[\sqrt{\frac{2(1 + \beta)}{\beta} + \frac{2S_y(2 + \beta)\alpha^2}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$	$P_n = \frac{S_2\frac{L_1}{D}}{f_{c_2}(2 + \beta)\left(\frac{L_2}{D}\right)}\left[\sqrt{\frac{2(1 + \beta)}{\beta} + \frac{2S_y(2 + \beta)}{3S_2\left(\frac{L_1}{D}\right)^2}} - 1\right]$
3A 	$F_y = \frac{DS_2L_2}{(1 + 2\beta)}\left[\sqrt{2(1 + \beta) + \frac{2S_y(1 + 2\beta)}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$	$P_n = \frac{S_2}{f_{c_2}(1 + 2\beta)}\left[\sqrt{2(1 + \beta) + \frac{2S_y(1 + 2\beta)}{3S_2\left(\frac{L_2}{D}\right)^2}} - 1\right]$
4 	$F_y = D^2\sqrt{\frac{2S_2S_y}{3(1 + \beta)}}$	$P_n = \frac{1}{f_{c_2}\left(\frac{L_2}{D}\right)}\sqrt{\frac{2S_2S_y}{3(1 + \beta)}}$

D = bolt diameter.
S_y = bolt yield stress.
S_1 = embedment yield stress of member 1.
S_2 = embedment yield stress of member 2.
L_1 = thickness of member 1.
L_2 = thickness of member 2.

Rowlands and others (1982) looked specifically at wood materials and did a finite element analysis of a two-bolt pin-loaded hole connection based on assumptions of rigid pins and linear elastic material properties. They found the strength of a two-bolt ,connection was constant for bolt spacings or end distances more than four times the bolt diameter.

Fracture mechanics analysis is concerned with determining the stress intensity factors at a cracked fastener hole. Often a finite element procedure is used to compute the stress intensity factors. Orringer and Stalk (1975) presented a method for Mode I and Mode II stress intensity factors at a cracked fastener hole in a single row or a double staggered row of fasteners in a panel under tension. Other literature is available on orthotropic composites, but little has been applied to lumber.

Experimental

Single- and Multiple-Bolt Connections

Over the past 60 years, several experimental studies have been conducted on bolted connections. Some of the more significant studies for single-bolt connections are summarized in table 3 and for multiple bolt connections in table 4. All the studies listed are for seasoned wood with moisture content ranging from 9 to 12 percent.

Not all geometric and material properties were given in these studies. Often the yield stress of the bolt and the hole fabrication procedure were not reported. (Goodell and Phillips (1944) specifically investigated the effect of hole roughness on single-bolt connection behavior. They drilled holes with different drill bits, feed rates, and revolutions per minute.) Although the wood species is usually given, the specific gravity often is not. This lack of information makes comparison of results difficult.

Spacing, End, and Edge Distances

Current design criteria for spacing, end, and edge distance of bolted connections are based on experimental observation. The 1986 National Design Specifications (NDS) (National Forest Products Association) for spacing, end, and edge distance are essentially those recommended by Trayer in 1932. Trayer had broad experience with bolted connections for aircraft components during the 1920's and based his recommendations on this experience. He also recommended, for connections with two rows of bolts, having the bolts opposite each other, not staggered, for parallel-to-grain loading.

Fahlbusch (1949) made recommendations for spacing, end, and edge distance based on maintaining critical tensile and shear cross sections. His recommendations permit closer spacing and less end distance than Trayer's. Fahlbusch also remarked that staggered bolt arrangements are to be avoided but did not present any experimental results to support this conclusion. in addition, he developed an empirical modifying factor to relate the strengths of multiple- and single-bolt connections.

New Zealand researchers, Harding and Fowkes (1984), studied the effect of end distance for parallel- and perpendicular-to-grain loading. Their results indicated that end distance had a marked effect on perpendicular-to-grain strength. As yet, however, New Zealand Standards have no requirements for perpendicular-to-grain end distance.

Other Factors

Other factors that affect connection strength are moisture content, tension or compression loading, fabrication tolerances, duration of load, and preservative or fire treatment.

Several researchers have studied the effect of the moisture content of timber on the strength of bolted connections (Doyle and Scholten 1963, Kunesh and Johnson 1968, Longworth and McMullin 1963). in general, the connections were fabricated at high moisture content and either tested at the high moisture content or seasoned to a lower moisture content and then tested. They compared results of both tests with the strength of connections fabricated and tested at the lower moisture content.

in assigning equal parts of a load to the bolts in a row, one tacitly assumes all boltholes have identical fabrication tolerance. Wilkinson (1980, 1986) identified variability in fabrication tolerances as having a large effect on how the load is distributed among bolts in a row. Dannenberg and Sexsmith (1976) also observed the significant effect of fabrication tolerances on load distribution for shear plate connectors.

No research has been reported on the effects on the strength of a connection of duration of load or treatment with preservative or fire retardant.

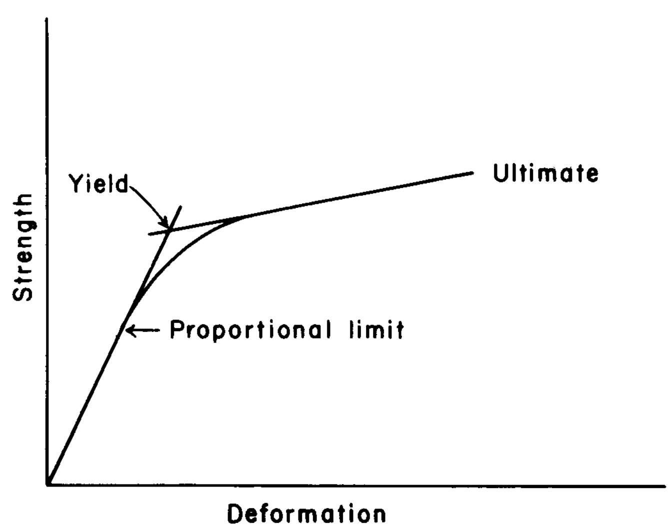


Figure 1–Schematic load-deformation curve for a bolted joint. (ML86 5420)

Table 3.–Geometric and material parameters for various studies of single-bolt connections

Source reference	Main member	Side members	Number of members in connection	Bolt and hole diameters	L_2/D	Load angle to grain	Number of replications
				In			
Grenoble (1925)	White ash, Sitka spruce	1/4-in steel plate and wood[1]	2, 3	0.18 to 1/2 0.18 to 1/2	1 to 16.5	Parallel	4
Trayer (1927)	Sitka spruce	1/4-in steel plate	2, 3	1/4 to 1/2 1/4 to 1/2	2 to 12	30° to 90°	3
Trayer (1928)[2]	—	—	—	—	—	—	—
Trayer (1932)	Douglas-fir, yellow pine, Sitka spruce, oak, and maple	1/4-in steel[3] and wood	3	1/4 to 1	0 to 12	Parallel	4 or 5
Do.	do.	do.	3	1/2	0 to 12	Perpendicular	4 or 5
Goodell and Phillips (1944)	Douglas-fir and Sitka spruce Plywood	Steel	3	1/4, 1/2 1/4, 1/2	3 to 4	Parallel	5, 13
Pitz (1952)	Douglas-fir	Steel	3	1/2 to 1 1/2 to 1	4	0 to 90 in 7-1/2° increments	3
Doyle and Scholten (1963)	do.	5/1 8-in steel plate or wood	3	1/2 to 1 (plus 1/16 in)	2.6 to 5.3	Parallel and perpendicular	3
Wilkinson (1978)	do.	Wood	2, 3, 4	3/8 to 3/4 (plus 1/32 in)	2 to 6	Parallel	6
Smith (1982)	Canadian and Polish spruce	Wood	3	5/8 (plus 1/16 in)	6	Parallel and perpendicular	50
Hirai and Sawada (1982a)	Spruce and fir	1/8-in steel plate	3	5/16 to 1/2	2 to 10	Parallel	3
Hirai and Sawada (1982b)	Spruce and fir	Wood	3	3/8	2 to 10	Parallel	3
Soltis et al. (1988)	Douglas-fir	Wood	3	1/2 to 1-1/2 (plus 1/16 in)	2 to 13.5	Parallel and perpendicular	15

[1] Wood side members are the same species as the main member.

[2]Summarized Trayer (1927) and Grenoble (1925).

[3]Except for connections with 1 -in bolts which used 5/8-in steel plate.

Codes

Current design criteria (AITC 1985, NFPA 1986) for single-bolt connections are based on Trayer's (1932) research. The codes present single-bolt design values in tabular form for various species, diameters, and L_2/D ratios.

The National Standard of Canada (Canadian Standards Association 1984) has presented single-bolt design values in a similar tabular form. The British Standard (Booth 1982) defines single-bolt design values by an empirical formula fitted to test data based on species and L_2/D parameters.

The European CIB-Structural Design Code (International Council for Building Research Studies and Documentation 1983) in contrast, bases the single-bolt design values on equations derived from the European Yield Theory.

Current design criteria for multiple-bolt connections generally have a load distribution factor and minimum spacing requirements.

Table 4.–Geometric and material parameters for various studies of multiple-bolt connections

Reference	Main member		Side member		Bolt diameter	Number of bolts	L_2/D	Bolt spacing	Bolt end distance
	Species	Specific gravity	Species	Specific gravity					
					In			*In*	In
Doyle and Schotten (1963)	Douglas-fir	0.43	5/16-in steel and Douglas-fir	0.43	1/2, 3/4, 1	[1]1,4	2.5 to 5	4	5.25
Longworth and McMullin (1963)	do.	0.49 to 0.52	Douglas-fir	0.49 to 0.52	3/4	4	3.5	3	3
Doyle (1964)	do.	0.47 to 0.52	1/2-in steel	—	1/2, 3/4,	[1]1, 8	4.3 to 6.5	3; 4.5	5.25
Kunesh and Johnson (1968)	do.	0.42 to 0.54	Douglas-fir	0.42 to 0.54	3/4	1 to 6	2.2	1.5; 3	—
Potter (1982)	European redwood	0.47	European redwood	0.47	3/8	1 to 40	5.3	1.5	1.5
Hirai and Sawada (1982b) and Hirai (1983)	spruce	0.42 to 0.45	steel	—	1/4 to 5/8	1	1.3 to 6.7	—	2.5 to 10.5 x diameter
Wilkinson (1986)	Douglas-fir	0.41 to 0.55	3/16-in steel	—	3/4	2 to 7	1.5	6	—

[1]Total number of bolts in two rows.

The load distribution factor, expressed as a modifying factor, K, modifies the total allowable load on a connection, F.

$$F = nKF_b \tag{2}$$

where n is the number of bolts in the connection and F_b, is the allowable strength for a single bolt. Modifying factors for steel and wood side plates differ in value.

The modifying factors used in the United States (AITC 1985, NFPA 1988) are based on the elastic analyses of Lantos (1989) and Cramer (1988). They are presented in tabular form in terms of cross-sectional areas of the main and side members. Spacing, end, and edge distance are presented as multiples of bolt diameter.

The National Standard of Canada (Canadian Standards Association 1984) similarly has presented modifying factors and spacing, end, and edge distance requirements.

The European CIB-Structural Design Code (International Council for Building Research Studies amd Documentation 1988) presented a modifying factor as an empirical equation for connections having more than four bolts in a row. The load-carrying capacity of the connection is reduced by one-third for each bolt above the four-bolt threshold. Spacing, end, and edge distance requirements are similar to those in the United States.

The British Standard (Booth 1982) presented a modifying factor as an empirical equation. The spacing, end, and edge distance requirements are the same as in the United States.

Results

The proportional limit load has been used as the basis of comparison of the effects of connection variables on single-bolt connections. Data on maximal loads or loads at a given deformation, although included by some researchers, are usually missing from the record. Actual load-deformation curves have not been published, but example curves are sometimes given.

In figures 2 to 17 we show the yield theory predictions for comparison with experimental results. In some cases, the compressive strength of the wood and/or yield strength of the bolt had to be assumed.

Figures 2 and 3 show results from Grenoble (1925) as plots of normalized proportional limit-bearing stress versus L_2/D ratio for two- and three-member connections with steel side plates and loading parallel-to-grain.

Figures 4 to 8 show results from Trayer (1932). Three-member connections with steel side plates, with softwood species (fig. 4) and hardwood species (fig. 5) were loaded parallel to grain and perpendicular to grain (fig. 6). Three-member connections with steel and wood side plates are compared in figures 7 and 8.

Figures 9 to 11 show results from Doyle and Scholten (1963). Three-member connections with steel side plates (fig. 9) and wood side plates (fig. 10) were loaded parallel to grain. Perpendicular-to-grain results are shown in figure 11 along with results from Trayer (1932) on effect of bolt diameter.

Results from Wilkinson (1978) are presented in figures 12 and 13 for connections loaded parallel to grain with various ratios of main- to side-member thickness (fig. 12). Two- and three-member connections are compared in figure 13.

Results from Soltis and others (1986) are presented in figures 14 and 15 for three-member wood connections of Douglas-fir loaded parallel to grain (fig. 14) and perpendicular to grain (fig. 15).

Geometric and material parameters published with experimental research on multiple-bolt connections are listed in table 4.

Experimental results of three research teams are given for a bolt pattern of 2 rows by 2 columns (table 5). The results are given as modifying factors at proportional limit and at ultimate strength. The current NDS design values (NFPA 1986) are given for comparison but recall that the NDS values are based on the elastic theories of Lantos (1969) and Cramer (1968) and applicable only below the proportional limit. The results for a composite product (Pyner and Matthews 1979) are included because of the orthotropic nature of the material, the geometry of bolt patterns, and the observed failure types in which composite resembles wood.

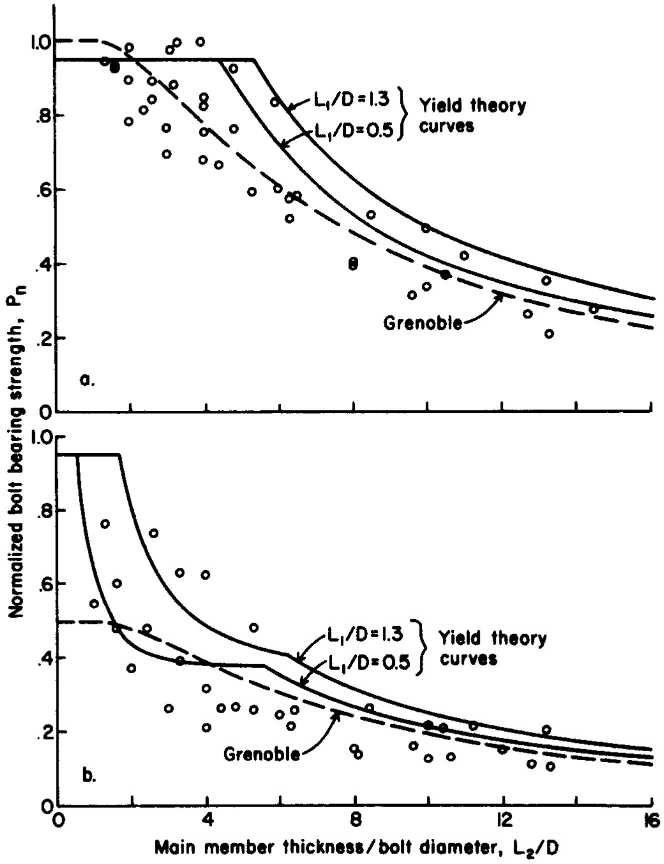


Figure 2–Results for proportional limit (Grenoble 1925) and yield load (European Yield Theory) for (a) three-member and (b) two-member connections of ash with steel side plates. Parallel-to-grain loading (S_y = 125,000 lb/in^2; f_c = 7,600 lb/n^2; β = 0.16; S_2 = 7.220 lb/in^2). (ML86 5428)

The results are useful for comparing load distribution among bolts. Longworth and McMullin (1983) also tested the 2 by 2 bolt pattern to determine the effects of moisture content, but we could not determine a modifying factor because they did not give a value for a single bolt to use as a basis.

Experimental results for load distribution of various bolt patterns are compared to the NDS-based analytic results in table 6. Note some modifying factors have experimental values greater than unity, which is theoretically impossible. These results reflect experimental variability.

Results of elastic theory analyses have been compared by Wilkinson (1980). The studies of finite element analysis were undertaken to determine the influence of diverse parameters on connection strength and thus are not comparable.

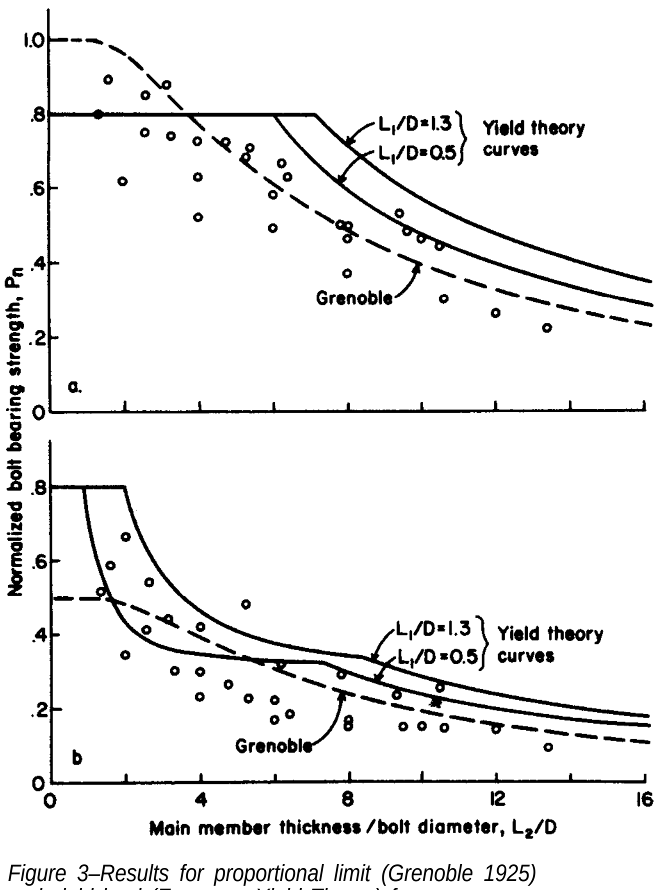


Figure 3–Results for proportional limit (Grenoble 1925) and yield load (European Yield Theory) for (a) three-member and (b) two-member connections of Sitka spruce with steel side plates. Parallel-to-grain loading (S_y = 125,000 lb/in^2; f_c = 5,400 lb/in^2; b = 0.10; S_2 = 4,320 lb/in^2). (ML54 5427)

The modifying factors used in the United States and Canada, Britain, and Europe are compared for connections having 1 to 10 bolts in a row with wood side members (fig. 16a) and steel side members (fig. 16b). In all the standards, bolt spacings are four times bolt diameter, while bolt end distance is four times bolt diameter for compression and seven times bolt diameter for tension loading. Edge distance vanes from 1.5 to 2.0 times bolt diameter in the various standards.

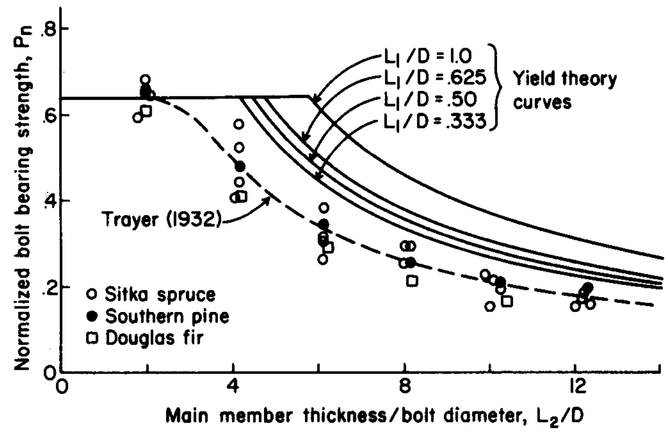


Figure 4–Results for proportional limit (Trayer 1932) (fig. 5) and yield load (European Yield Theory) for three-member connections of softwood species with steel side plates. Parallel-to-gain loading (S_y = 45,000 lb/in^2; f_c = 5,250 lb/in^2; β = 0.07; S_2 = 3,360 lb/in^2). (ML55 5421)

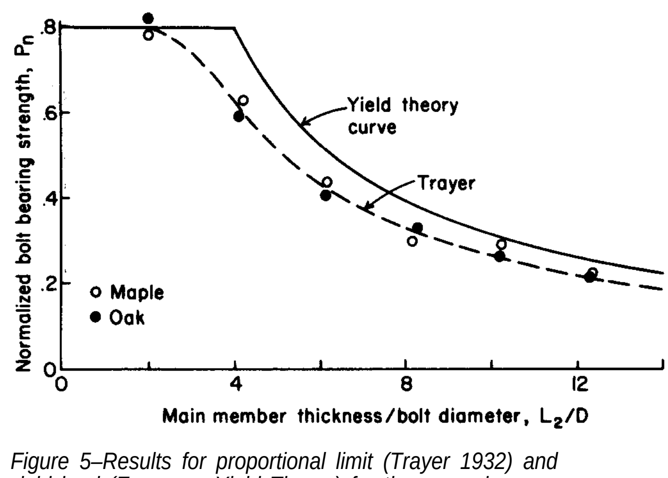


Figure 5–Results for proportional limit (Trayer 1932) and yield load (European Yield Theory) for three-member connections of hardwood species with steel side plates. Parallel-to-grain loading (S_y = 45,000 lb/in^2; f_c = 5,0245 lb/in^2; β = 0.09; S_2 = 4,020 lb/in^2; L_1/D = 0.5). (ML86 5422)

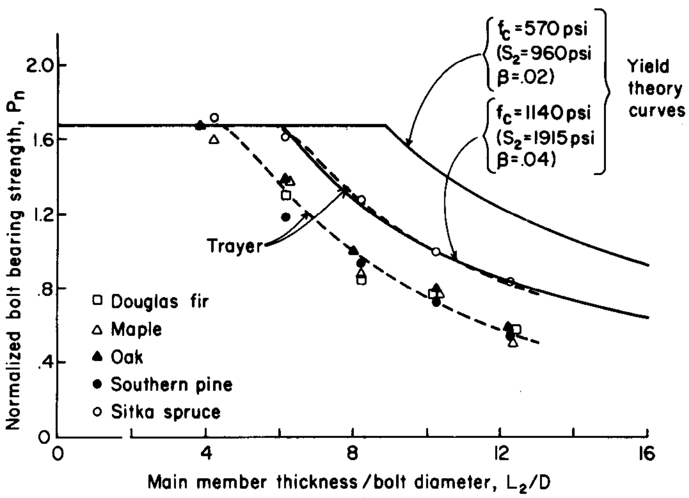


Figure 6–Results for proportional limit (Trayer 1932) and yield load (European Yield Theory) for three-member connections with steel side plates. Perpendicular-to-grain loading (S_y = 45,000 lb/in^2; L_1/D = 0.5). (ML86 5412)

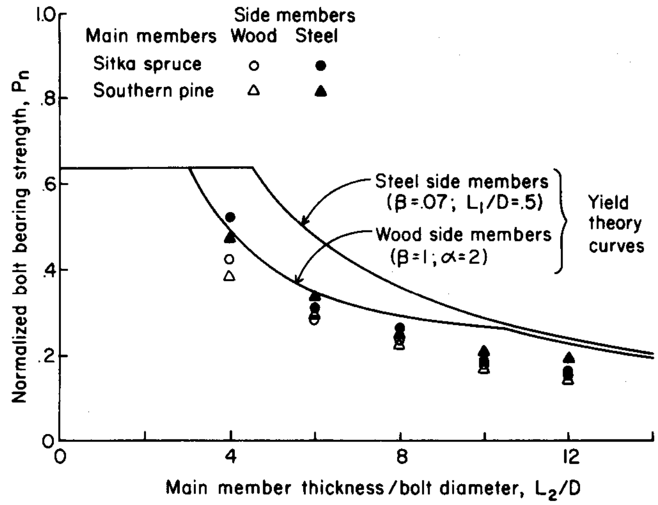


Figure B-Results for proportional limit (Trayer 1932) and yield load (European Yield Theory) for three-member connections of softwoods with steel and wood side plates. Parallel-to-grain loading (S_y= 45,000 lb/in^2; f_c = 5,130 lb/in^2; S_2 = 3,283 lb/in^2). (ML86 5413)

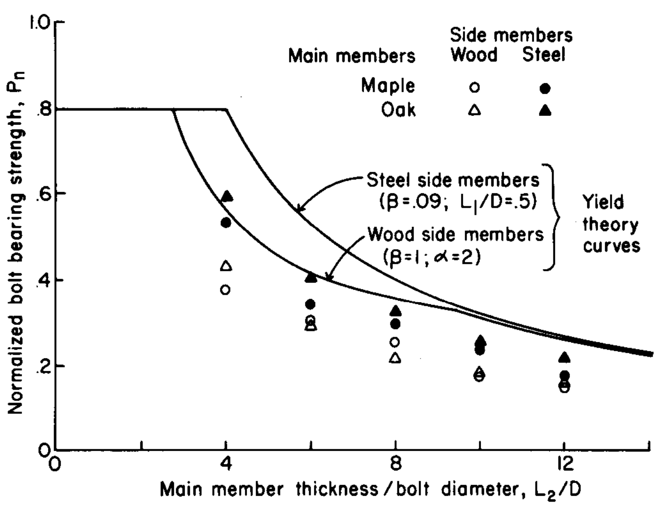


Figure 7–Results for proportional limit (Trayer 1932) and yield load (European Yield Theory) for three-member connections of hardwoods with steel and wood side plates. Parallel-to-grain loading (S_y = 45,000 lb/in^2. f_c = 5,024 lb/in^2; S_2 = 4,020 lb/in^2). (ML86 5424)

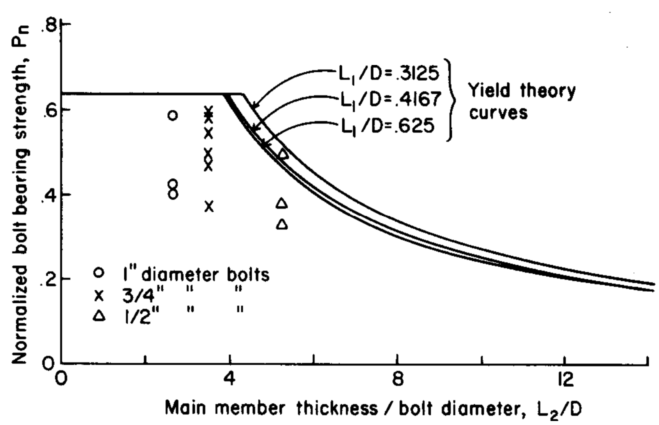


Figure 9–Results for proportional limit (Doyle and Scholten 1963) and yield load (European Yield Theory) for three-member Douglas-fir connections with steel side plates. Parallel-to-grain loading (S_y= 48,000 lb/in^2; f_c = 6,460 lb/in^2; β = 0.09; S_2 = 4,134 lb/in^2). (ML86 5414)

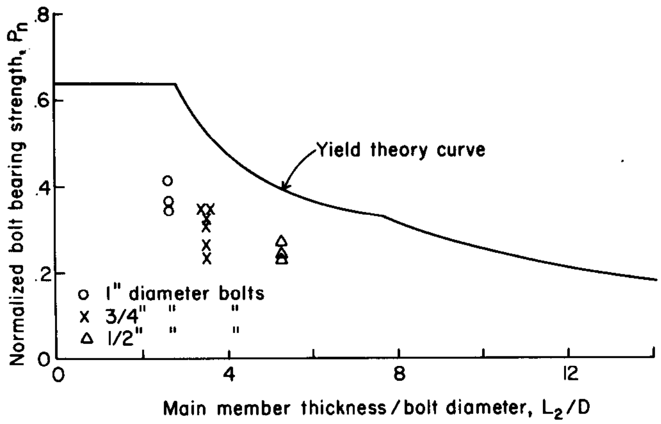


Figure 10–Results for proportional limit (Doyle and Scholten 1963) and yield load (European Yield Theory) for three-member Douglas-fir connections with wood **side** *plates. Parallel-to-grain loading (S_y = 48,000 lb/in^2; f_c = 6,460 lb/in2 S_2 = 4,194 lb/in^2; α = 1.615; β = 1). (ML86 5415)*

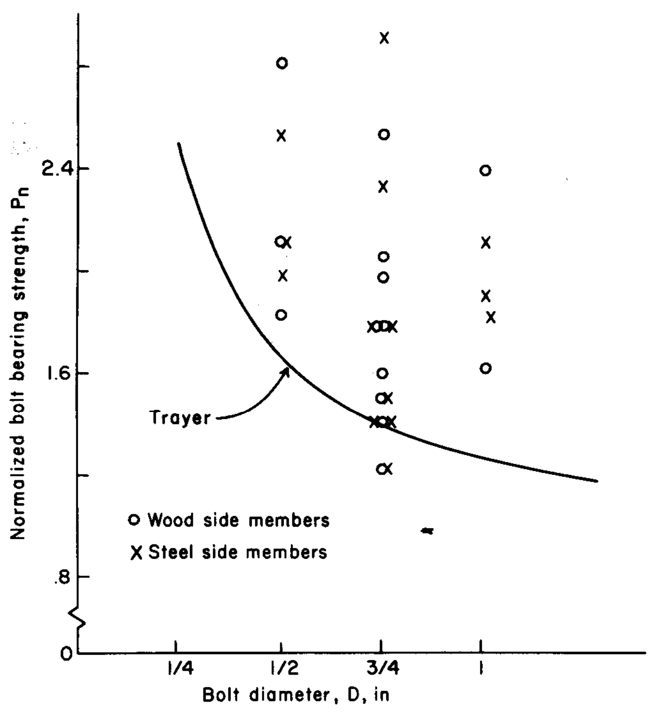


Figure 11–Effect of bolt diameter on normalized bearing stress at zero L_2/D. Perpendicular-to-grain loading. (ML86 5416)

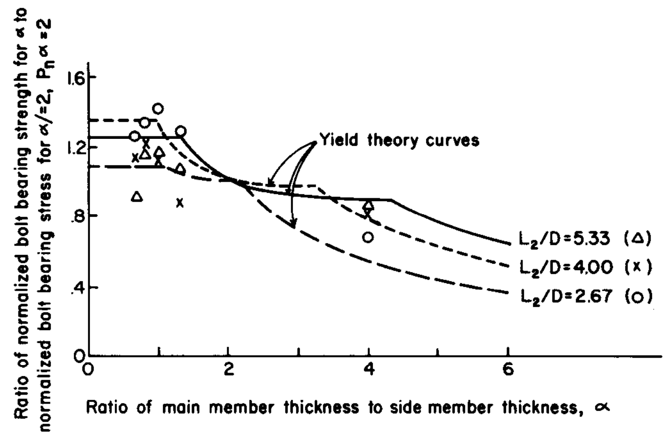


Figure 12–Results for proportional limit (Wilkinson 1978) and yield load (European Yield Theory) for three-member all-wood connections with various side member thicknesses. Parallel-to-grain loading (S_y = 45,000 lb/n^2; f_c = 7,670 lb/in^2; S_2 = 4,960 lb/in^2; β = 1). (ML86 5425)

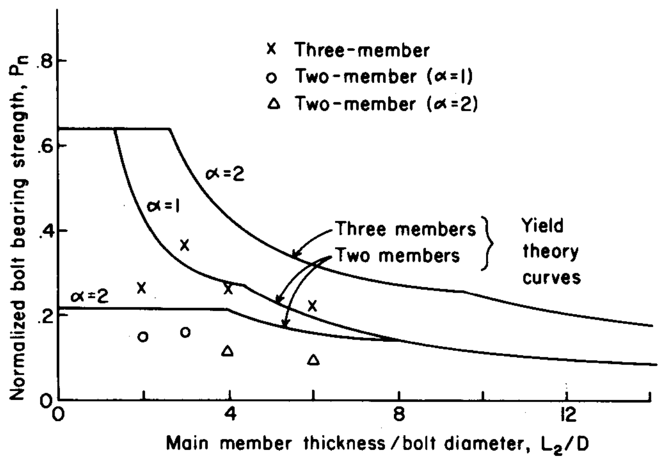


Figure 13–Results for proportional limit (Wilkinson 1978) and yield load (European Yield Theory) for two- and three-member all-wood connections. Parallel-to-grain loading (S_y = 45,000 lb/in^2; f_c = 7,000 lb/in^2; β = 1; S_2 = 4,480 lb/in^2). (ML86 5410)

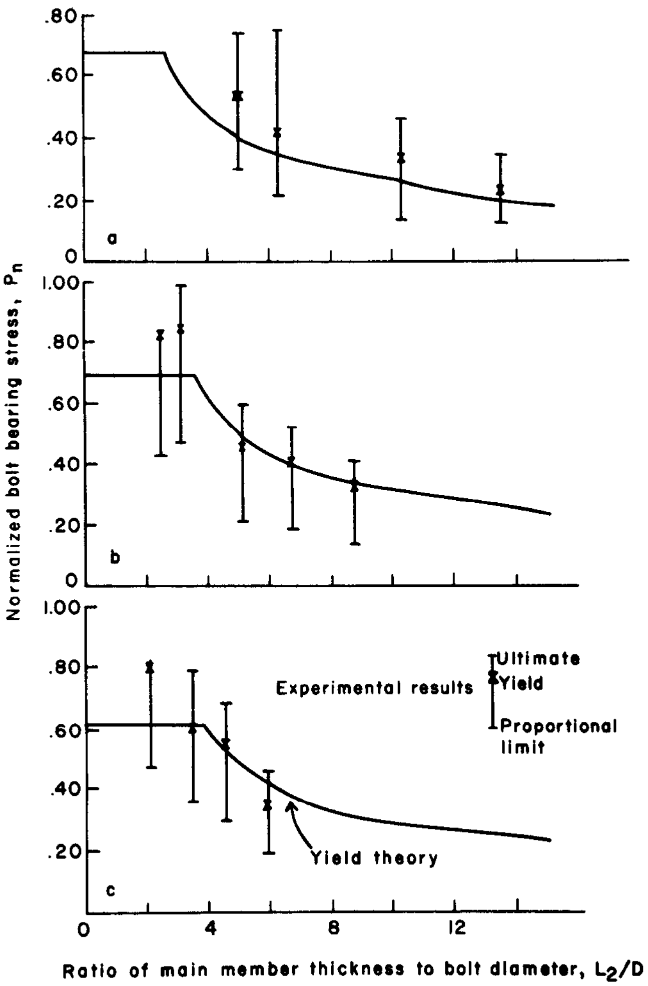


Figure 14–Results from Soltis, Hubbard, and Wilkinson (1986) comparing yield theory with experimental results for (a) 1/24-inch-, (b) 1-inch-, and (c) 1-1/2-inch-diameter bolts in connections loaded parallel to grain. (ML86 5417)

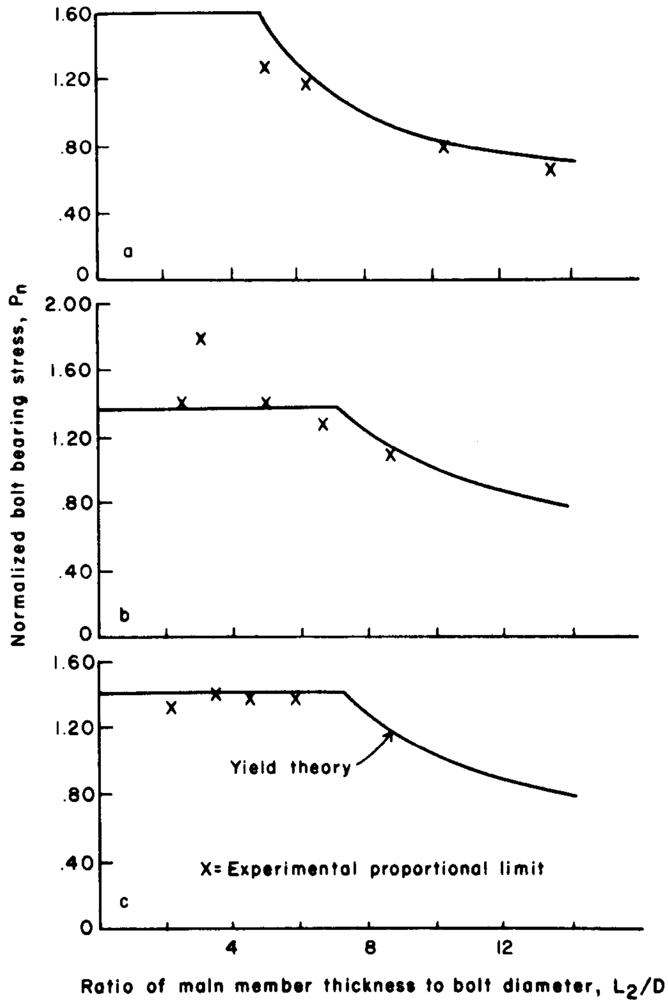


Figure 15–Results from Soltis, Hubbard, and Wilkinson (1985) comparing yield theory with experimental results for (a) 1/2-inch-, (b) 1-inch-, and (c) 1-1/2-inch-diameter bolts in connections loaded perpandicular to grain. (ML86 5409)

Table 5.–Values of modifying factor K for a two-row by tow-column bolt pattern.

Reference	Proportional limit	Ultimate	NDS
Doyle and Scholten (1963)			
Wood side plates, parallel-to-grain	[1]0.65-0.95	[2]0.85-0.98	0.97
steed side plates, parallel-to-grain	[1].76- .92	[2].84- 94	.88
Wood side plates, parpendicular-to-grain	[1].53- .82	[2].70- .80	.97
Steel side plates, perpendicular-to-grain	[1].68- .89	[2].63- .82	.83
Kunesh and Johnson (1988)			
Wood side plates, parallel-to-grain	0.91	0.99	.92
Pyner and Matthews (1979)			
Laminated glass fiber reinforced plastic		.81	—

[1]Based on 0.08-in slip; proportional limit values not given in the original paper.

[2]Dependant on bolt diameter.

Table 6.–Values of modifying factor K for various bolt patterns

Reference	Bolts		Modifying factor, K		
	Number	Pattern	Proportional limit	Ultimate	NDS
Doyle (1964)					
Steel side plates, parallel-to-grain	8	4 rows × 2 columns	[1]1.03 to 1.24	[1]0.61 to 0.87	0.87
Kunesh and Johnson (1988)					
wood side plates, parallel-to-grain	2	1 row × 2 columns	1.05	1.08	1.0
	2	2 rows × 1 column	.90	.86	1.0
	8	3 rows × 2 columns	.89	.71	.98
	8	2 rows × 3 columns	1.10	.79	1.0
Potter (1982)					
Wood side plates, parallel-to-grain	2	1 row × 2 columns	—	.96	1.0
	2	2 rows × 1 column	—	.92	1.0
	6	3 rows × 2 columns	—	.79	.95

[1]Dependant on bolt diameter and spacing.

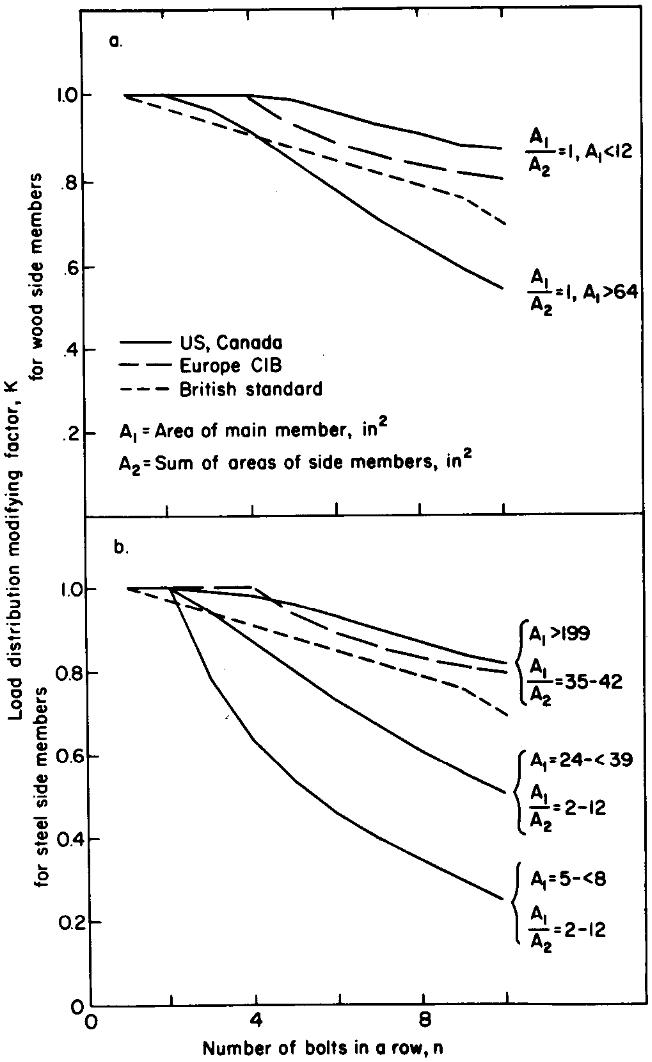


Figure 16–Load-distribution modifying factors defined by U.S. and Canadian, British, and European standards for (a) wood side members and (b) steel side members. (ML86 5426)

Single-Bolt Connections

The yield theory, giving the equations of tables 1 and 2, appears to predict the trend seen in the results of all researchers. In general, experimental values fall below the yield theory curves, as expected, because the proportional limit load is smaller than the yield load (fig. 1). Soltis and others (1988) give yield loads that agree closely with the yield theory curves (fig. 14).

Results for parallel-to-grain loading appear to agree more closely with the yield theory than results for perpendicular-to-grain loading (fig. 6). The ratio between proportional limit load and yield load is not known but is suspected to be larger for perpendicular-to-grain loading.

Hardwood species have a higher normalized bearing stress than softwoods at zero L_2/D Grenoble (1925) had P_n = 0.8 for Sitka spruce (fig. 3) and P_n = 0.95 for ash (fig. 2). Trayer (1932) had P_n = 0.65 for softwoods (fig. 4), and P_n = 0.80 for hardwoods (fig. 5). Differences in bolthole diameter are suspected of causing the differences between researchers.

Trayer (1932) concluded that three-member connections with steel side plates carried 20 percent more load at proportional limit than connections with wood side members when loaded parallel to grain. The yield theory indicates a difference over only a limited range of L_2/D values (figs. 7 and 8). Trayer did not compare connections that had small L_2/D values. The scatter in his data is greatest where the yield theory predicts the greatest difference between steel and wood side plates.

Trayer (1932) concluded there was no difference between steel and wood side plates in connections when the loading was perpendicular to grain. He gave no results for connections with wood side plates. The yield theory indicates no difference over a wide range of L_2/D values and a slight difference at L_2/D values greater than 9 (fig. 17).

Grenoble (1925) concluded that two-member connections carried half the load of three-member connections having the same L_2/D value. His data and the yield theory appear to support his conclusion over the range of L_2/D values evaluated (figs. 2 and 3). His conclusion does not appear valid for L_2/D values less than 2. Wilkinson (1978) obtained similar results for two- and three-member connections with wood side plates (fig. 13).

Most tests of three-member wood connections have been with a main member twice the thickness of the side member. Doyle and Scholten (1963) used a main-member thickness 1.6 times that of the side member (fig. 10). Wilkinson (1978) examined several ratios of main- to side-member thickness (fig. 12). Again, the yield theory predicts the general effect of various member thicknesses.

Tests of connections with steel side plates have generally been made with a constant steel thickness for all bolt diameters and lengths. The yield theory predicts differing results for various ratios of side-plate thickness to bolt diameter (figs. 2-4 and 9). This effect could account for some of the scatter in experimental results.

For connections loaded parallel to grain the NDS allows 75 percent more strength with steel than with wood side members for bolts of 1/2-inch diameter or less, 25 percent more for 1-1/2-inch-diameter bolts, and proportional values for intermediate diameters. The NDS recommendation is based in part on having equal connection deformation for wood and steel side members. The yield theory indicates that the increased strength for steel side members should be related to the L_2/D ratio and to the ratio of steel thickness to bolt diameter, L_1/D.

Researchers have used a variety of bolt yield stresses. Steel aircraft bolts with a yield stress of 125,000 lb/in_2 and low-carbon steel bolts with a yield stress of 45,000 lb/in_2 have both been used. Trayer (1932) indicated that different results might be expected for high-strength bolts, and this is borne out by the yield theory. Soltis and others (1986) found that the yield stress varied with the bolt diameter:

Bolt diameter In	Yield stress Lb/in^2
1/2	44,600
1	87,100
1-1/2	90,500

The size of boltholes has ranged from being equal to the bolt diameter to being 1/16 inch larger. It has already been pointed out that bolthole size probably affects the normalized bearing stress at zero L_2/D, and thus the embedment yield stresses S_1 and S_2. Hirai and Sawada (1982b, c) had a $P_n = 0.47$ for zero L_2/D, much lower than the values obtained by other researchers. Their hole diameter is unknown.

The smoothness of the bolthole can also affect the value of the proportional limit load, as indicated from results of Goodell and Phillips (1944) (fig. 18).

Trayer (1932) indicated an effect of bolt diameter on the normalized bearing stress at zero L_2/D when loading is perpendicular to grain (fig. 11). Results from Doyle and Scholten (1968) indicate the possibility of a similar effect although obscured by the scatter in their data.

Trayer (1927) and Pitz (1952) investigated directions of loading between 0 and 90° to the grain. In both studies, boltholes were equal to the bolt diameter, and steel side plates were used. The Hankinson formula is

$$N = \frac{PQ}{P\sin^2\theta + Q\cos^2\theta} \tag{3}$$

where

N = bearing stress at angle θ
P = bearing stress parallel-to-grain
Q = bearing stress perpendicular-to-grain

Both Trayer and Pitz found that this formula could be used to calculate the bearing stress at various angles to the grain. Pitz found that the Osgood formula fitted his data slightly better than the Hankinson formula.

The Osgood formula is

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

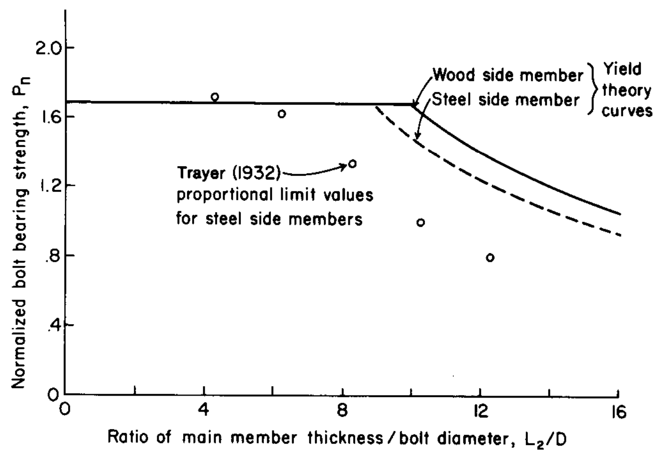


Figure 17–Comparison of three-member connections of Sitka spruce and steel side plates when loaded perpendicular to grain.

Wood side plates	*Steel side member*
$S_y = 45{,}000\ lb/in^2$	$S_y = 45{,}000\ lb/in^2$
$f_{c\perp} = 570\ lb/in^2$	$f_{c\perp} = 570\ lb/in^2$
$f_{c\parallel} = 5{,}610\ lb/in^2$	$S_2 = 960\ lb/in^2$
$S_2 = 960\ lb/in^2$	$L_1/D = 0.5$
$S_1 = 3{,}650\ lb/in^2$	$\beta = 0.02$
$\alpha = 2.0$	
$\beta = 0.26$	

(ML86 5423)

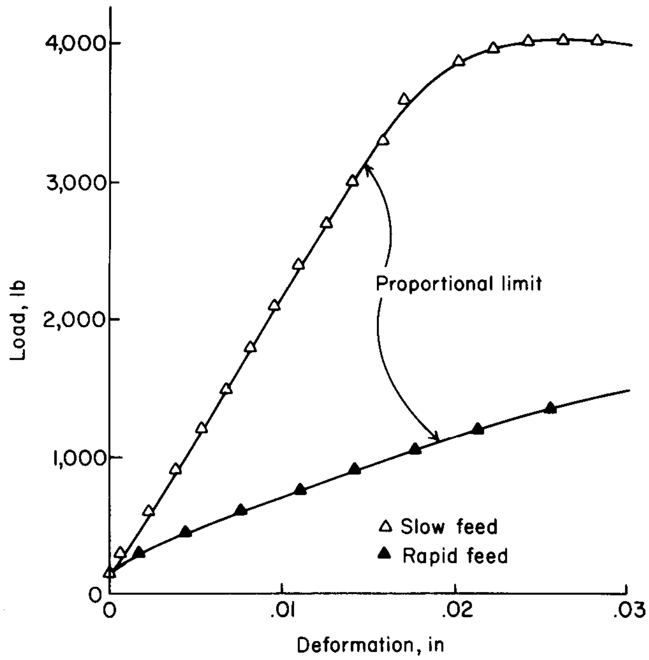


Figure 18–Load-deformation curves for 1/2-inch aircraft bolts in 2-inch Sitka spruce showing effect of rate of feed (Goodell and Phillips 1944). (ML86 5411)

Most researchers have used sample sizes of three to five specimens. Soltis and others (1986) used 16 replications. Smith (1982) concluded that sample size should be 20 or more for parallel-to-grain loading and 30 or more for perpendicular-to-grain loading in order to describe the population of connection behaviors adequately. Smith found that the three-parameter Weibull distribution best described the results, while the normal distribution gave good representation in most cases.

Trayer (1932) gave some data for bearing stress at a deformation of 0.1 inch that indicate that the ratio of bearing stress at 0.1 inch to bearing stress at proportional limit decreases with increased L_2/D ratio. Doyle and Scholten (1963) gave loads at several levels of deformation as well as at maximal loads. Their range of L_2/D values was too narrow to indicate any effect of the L_2/D ratio as compared to deformation results at proportional limit.

None of the researchers presented load-deformation curves nor any model for describing the load-deformation relationship.

Multiple-Bolt Connections

In general, modifying factors determining the load distribution among bolts in a row are determined by methods of basic mechanics analysis that assume fairly widely spaced bolts and elastic material properties. The calculations use values of single-bolt elastic load-slip modulus and areas and moduli of elasticity for the main and side members. The theory derivation assumes bolt elastic bending is included in the deformation. The experimental verification of the theory was based on small values of L_2/D, implying Mode I failure.

Experimental results are the basis for spacing, end, and edge distances. All experimental studies were three-member connections with a wood main member and either wood or steel side members (table 4). Most studies had small L_2/D ratios corresponding to Mode I failures; some studies had L_2/D ratios corresponding to Mode II failures, but no studies had large L_2/D ratios corresponding to Mode III failures. Most studies had connections fabricated with carefully aligned holes 1/16 inch larger than the bolt diameter, washers, and nuts tightened finger tight. Usually the yield strength of the bolts was not determined. Tests were conducted at slow rates of loading, and loads were considered static.

The analyses done by finite element and fracture mechanics usually determined the influence of some connection variable on its strength properties. All finite element analyses were two-dimensional analyses; by implication, they are valid only for small L_2/D ratios and Mode I failures. Most analyzed two bolts in a row at most, using elastic properties. Most assumed a load imparted by the bolt to the main member and thus cannot be used for determining modifying factors.

It has been known for many years that the load is distributed unequally among bolts in a row. In a butt-type joint, the outermost and innermost bolts in the row transmit a greater proportion of the load than intermediate bolts.

Comparing theoretical (NDS) modifying factors with experimental results for a bolt pattern consisting of two rows by two columns (table 5), we find relatively good agreement for wood side members parallel-to-grain and steel side members parallel- and perpendicular-to-grain. The NDS modifying factor for wood side members perpendicular-to-grain, however, is larger than that from experimental results. It appears that, when NDS values were derived, the same modulus of elasticity was used for both grain directions. Because the longitudinal modulus is much greater than the radial or tangential modulus of elasticity for wood, the resulting NDS values are too high. The same observation applies to values both at the proportional limit and at ultimate strength, even though the NDS values are theoretically correct only to the proportional limit. This problem does not occur in regard to steel side plates which are isotropic.

Comparing NDS modifying factors with experimental results for other bolt patterns (table 5) we see that four is the maximum number of bolts in a row considered. The modifying factor for four bolts in a row is nearly unity (figs. 16a, b). The modifying factor decreases between 4 and 10 bolts in a row. Except for Cramer's work (based on perfectly machined holes and small L_2/D ratios), there is little experimental verification of the distribution of a load among more than four bolts in a row.

Comparing United States, Canadian, and European recommendations (figs. 16a, b), we encountered the prevailing uncertainty regarding more than four bolts in a row. All recommend modifying factors of 0.9 to 1.0 for a row of less than four bolts, but their values for K diverge for a row of more than four bolts.

Few research results exist for less than four bolts in a row (tables 5 and 6). Comparisons for one, two, or three bolts in a row show modifying factors near unity at the proportional limit and slightly less at ultimate load.

No theory or experimental results are available to determine how load is distributed when there are multiple columns of bolts with either staggered or symmetric rows.

Spacing, End, and Edge Distances

Current spacing, end, and edge distance design requirements for bolts in a row are those recommended by Trayer (1932). Trayer recognized that the stress distribution beneath the bolt for various L_2/D ratios affects the spacing and end distance required if maximal capacity of the connection is to be developed. He concluded, however, that using spacing and end distance requirements based on small L_2/D ratios would be conservative for larger L_2/D ratios.

The effect of L_2/D on end distance has only recently been quantified by Hirai and Sawada (1982a) for spruce specimens. Their results are reproduced in figure 19.

Other researchers, referred to in the review of literature above, have studied spacing and end distances experimentally or analytically. Almost al&have confirmed that the current recommendation of an end distance four times the bolt diameter is satisfactory in parallel-to-grain compression loading. The experimental studies have noted a change in failure mechanism at this end distance. For end distances less than four diameters, a shear plug or tensile crack failure indicates insufficient end distance. For end distances greater than four diameters, there is a bolt-bearing failure. Figure 19 also confirms this; for end distances greater than 4.5 times bolt diameter, the bolt-bearing strength is relatively constant for all L_2/D ratios. For an end distance of 2.5 times bolt diameter, the bolt-bearing strength is notably less than for larger end distances whatever the L_2/D ratio.

The standard test procedure (ASTM D 1761) requires either compression or tension loading. However, to simplify testing most researchers have used compressive loading. The effects of tension loads on spacing, end, and edge distances are not known.

Most of the research results mentioned above were based on loading parallel-to-grain. Fewer perpendicular-to-grain results are available, but these limited data suggest that current design is adequate. In most of the research Douglas-fir species were used for the main member. For other species data are very limited.

No research is available to determine the spacing requirements between rows of bolts for either staggered or symmetric configurations.

Other Factors

Other factors that influence connection strength are moisture content, fabrication tolerances, duration of load, and preservative or fire treatment.

The limited research (Doyle and Scholten 1963, Kunesh and Johnson 1968, Longworth and McMullen 1963) on the effect of moisture content on the strength of multiple-bolt connections yields consistent results. A connection at 30 percent moisture content has about 60 percent of the proportional limit strength of a connection at 12 percent moisture content. The three studies were for relatively small L_2/D ratios (table 4). No results are available for larger L_2/D ratios. The yield theory suggests effects of moisture content may not be as large at the larger L_2/D ratios where bolt-bending Mode II and III failures occur.

Fabrication tolerances have been identified as the variable having perhaps the largest effect on connection strength (Wilkinson 1980, 1986). Earlier experimental research usually followed current design practice of making the bolt hole 1/16-inch oversize. This assumes 1/16 inch is an attainable fabrication tolerance. To approximate actual fabrication practices more closely, current research at the Forest Products Laboratory is considering the effects on connection strength of other oversize hole sizes as well as improper hole alignments.

The effect of the duration of load on the strength properties of wood has long been recognized. Current design assumes the duration-of-load factor applies to the stressed wood beneath a bolt regardless of L_2/D ratio. Intuitively, one expects the effect to be greater for Mode I failures (wood-crushing) than for Mode Ill failures (bolt-bending), but no research has been done to verify this.

The effect of preservative or fire treatment on single-bolt strength or on the distribution of load or spacing, end, or edge distances has not been researched.

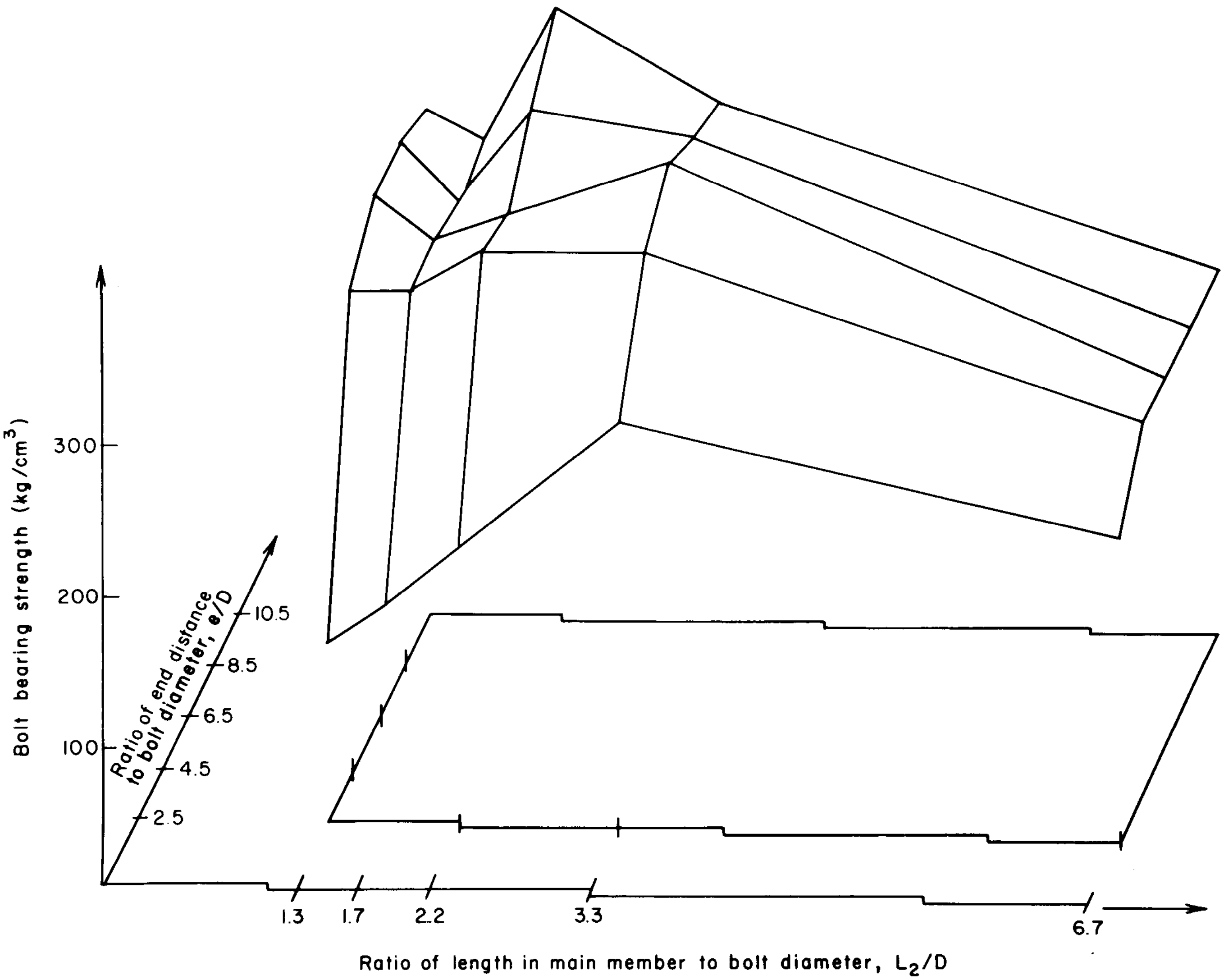


Figure 19–Connection bearing strength dependent on ratios of length in main member to bolt diameter, L_2/D, and end distance to bolt diameter, e/D. (ML86 5419)

Some other observations relate to method of loading, proportional limit versus ultimate strength comparisons, and strength of bolt. Most early research was based on the ASTM standard test of subjecting a three-member connection to parallel-to-grain compressive loads. Few data exist for tension, moment, or combined axial/moment loading or for other angle-to-grain loading. Also, no data exist for dynamic or cyclic loading of multiple-bolt connections.

The analytical methods are based on elastic theory, the results being valid only to the proportional limit. Experimental results seem to give more repeatable results at ultimate load, but proportional limit loads are used to compare to theoretical results. Modifying factors might be changed if ultimate strength were used as a basis. Additionally, all factors are based on strength; connection stiffness is not considered.

Most of the early work was done before the research related to the yield theory. Thus bolt yield strength was not deemed an important parameter. It is difficult to predict how research results would be affected by a consideration of higher strength bolts and larger L_2/D ratios.

Summary

The yield theory presents a means of looking at the results for single-bolt connections of a number of researchers together. It expresses the general trend of existent data. As expected, experimental results at the proportional limit usually fall below the yield theory curves. The ratio to be expected between yield and proportional limit load is unknown. However, the trend of the results indicates current design values based on the proportional limit are generally correct.

The yield theory indicates that the greater strength of a connection with steel rather than wood side plates should be related to the L_2/D ratio and to the ratio of steel thickness to bolt diameter, L_1/D.

Most research has been done with parallel-to-grain loading. The yield theory agrees more closely with the results of parallel-to-grain loading than of perpendicular-to-grain loading, for which fewer data exist.

Significant conclusions appear to have been drawn from results based on a small sample size and narrow range of connection properties such as the L_2/D ratio. Information on the load-slip behavior and on the distribution of properties is inadequate if bolt data are to be used for limit-states design or multiple-bolt connections.

The strength of a multiple-bolt connection is the sum of the single-bolt values, multiplied by a modifying factor. The modifying factor is based on an elastic theory of load distribution; it is valid only to the proportional limit. The theory is well verified by a number of experimental studies for two bolts in a row where the modifying factor is unity. Less experimental verification exists for two to four bolts in a row, but the results do indicate a factor near unity (although the NDS has substantially lower values for steel side plates and small main member). For more than four bolts in a row, data to substantiate the theory are very limited.

The NDS modifying factors for connections with wood side members loaded perpendicular to grain are higher than test results. This may result from using the longitudinal modulus of elasticity in the Cramer (1968) theory rather than the radial or tangential modulus.

The elastic theory does take elastic bending of bolts into account. Most of the experimental verification is for small L_2/D ratios corresponding to Mode I failures. Some verification exists for L_2/D ratios corresponding to Mode II failures. No verification exists for a row of bolts with larger L_2/D ratios.

Neither theory nor experimental data exist to determine load distribution to more than one row of bolts. No theory or data exist to recommend staggered or symmetric bolt patterns for multiple rows of bolts.

Spacing and end distance requirements of four times the bolt diameter have been theoretically and experimentally verified for Douglas-fir main members with parallel-to-grain compressive loading. Limited information is available for other species, loading, or angle to grain. No information is available for spacing between staggered or symmetric bolt rows.

United States, Canada, and Europe have similar code requirements for modifying factors and spacing requirements.

Moisture content affects connection strength at small L_2/D ratios. Its effect at larger L_2/D ratios is not known. The effects of other factors such as fabrication tolerances, duration of load, and preservative or fire treatments are not known. Fabrication tolerances are known to have a large impact on connection strength, but this impact has not been quantified.

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to ourselves and our Posterity,
do ordain and establish this Constitution
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