CHAPTER 4

BOLTS, DRIFT BOLTS, AND PINS

4.1 INTRODUCTION

4.1.1 Purpose and Scope

Bolts, drift bolts, and pins are mostly used for lateral connections in glue-laminated or heavy timber construction. They transmit forces through single shear (two members) or double shear (three members) connections. They develop moderately high lateral strength through an interaction of wood bearing and bolt bending. Beam-to-beam, beam-to-column, beam-to-wall, and column-to-foundation are typical bolted connections in building construction. Beam-to-cap or sill beam, and pile cap-to-pile are typical drift bolt or pin connections in bridge or cribbing construction.

This chapter first summarizes design requirements as specified by the National Design Specification for Wood Construction (NDS) (AF&PA 1991), Timber Construction Manual (TCM) (AITC 1994), and some foreign specifications. It then discusses fabrication and construction details. Finally, it gives the research basis for current design practice to give the designer insight into safety factors and connection details not covered by design specification.

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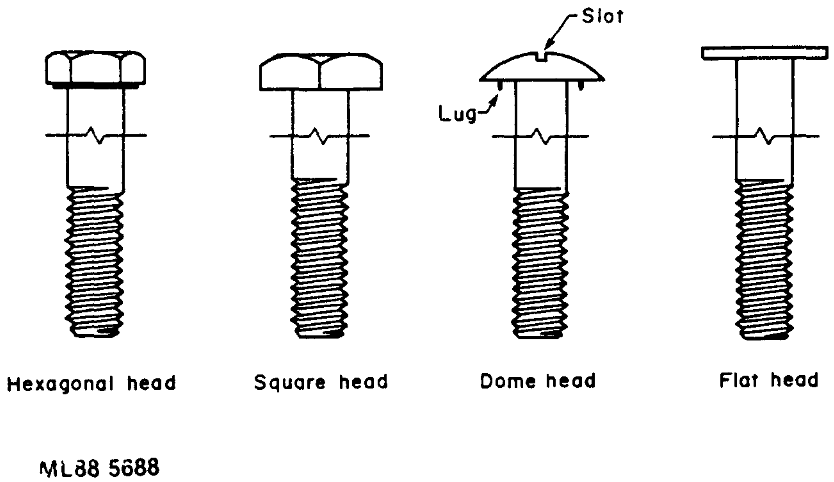

Figure 4.1—Typical Bolt Types.

4.1.2 Types

Bolts are manufactured in a variety of types based on the configuration of the bolt head. The most common types are the hexagonal head, square head, dome head, and flat head (fig. 4.1). The standard hex or square heads are used when the bolt head is in contact with wood or steel. More specialized bolts such as the dome head and flat head provide an increased head diameter and are used when the bolt head is in wood contact. Bolts with dome heads are also referred to as economy bolts or mushroom bolts and maybe slotted or provided with lugs to facilitate installation and tightening.

Drift bolts and pins are long unthreaded steel rods that are driven in prebored holes for lateral connections in large timber members. Drift bolts have a head for use with steel side plates and convenience in driving, while drift pins have no head. Manufactured fasteners generally conform to ASTM A307, but pins of concrete reinforcing steel are also used. Because they have poor resistance in withdrawal, drift bolts and pins are not recommended for connections subjected to significant withdrawal forces.

4.2 MATERIALS

Bolts most commonly used for timber construction conform to specification J429 (SAE, 1983); drift bolts and pins conform to ASTM standard

A307 (ASTM, 1984). Specification J429 tabulates bolt tensile yield and ultimate strengths. The NDS requires a fastener bending yield strength for the European Yield Theory. The bending yield strength is defined by the intersection of an offset line at .05 times the fastener diameter and the load-displacement curve found from a fastener bending test. An alternate procedure is to assume the bending yield strength is the average of the tensile yield and ultimate strengths. ASTM A307 does not specify tensile yield strength but does specify a minimum tensile ultimate strength of 60 ksi. In the past, the tensile yield strength has been assumed to be 33 ksi. Using the alternate procedure would result in a bending yield strength of 46.5 ksi which approximates the 45 ksi referenced in past NDS.

Bolts are generally available in diameters of 1/4 to 2 inches and lengths up to 24 inches or more in 1/2 inch increments; however, the designer should verify availability prior to specifying diameters over 1-1/4 inch or lengths over 16 inches. The NDS and the TCM limit maximum bolt diameter to 1 inch. Past editions of the NDS have allowed 1-1/2 inch diameter bolts.

4.3 BOLT DESIGN

The design of bolted connections for lateral loads underwent significant revision with the 1991 NDS. Prior to 1991, laterally loaded single bolt design was based on an empirical approach. After 1991, the design was based on a theoretical approach. Both approaches will be described to provide historic reference and to better understand current design methods.

4.3.1 Laterally Loaded Single Bolt Design-Prior to 1991

The capacity of a single bolt depends on the bearing strength of the wood, the bending strength, and the slenderness ratio of the bolt. The slenderness ratio, l/d, is defined as the length of bolt in the main member, l, divided by the bolt diameter, d. For bolted connections with low slenderness ratios, the bolt is relatively stiff and the full bearing strength of the wood is developed. As the slenderness ratio increases bolt stiffness is reduced and bending may occur before full bearing strength is achieved, reducing the capacity of the connection (fig. 4.2).

Past design criteria (AITC 1977; 1985; 1986; 1987; AFPA 1986) presented single-bolt design in tabular form for various species, diameters, and slenderness, l/d, ratios. Single-bolt values were given for both parallel and perpendicular-to-grain directions.

Single-bolt design values for parallel-to-grain loading were based *on the five* percent exclusion value of the ultimate green compressive strength for a given species adjusted for seasoning, duration of load, factor of safety, and l/d ratio. The exclusion value is multiplied by 1.20 to adjust to dry strength; it is divided by 1.90 to adjust to normal duration of load and provide a factor of safety. These adjustments result in a basic bolt-bearing strength for a zero l/d ratio for steel side plate connections. A further adjustment of 0.8 was necessary for wood side plate connections. Figure 4.3 adjusts the basis strength for other l/d rations. This l/d adjustment is based on research conducted by Trayer (1932) in which he fit an empirical adjustment curve to his test data. Thus, this approach, prior to 1991, is referred to as the empirical approach.

Single-bolt design values for perpendicular-to-grain loading were based on adjusted proportional limit green compressive strength for a given species. (Compressive strength values determined by procedures in ASTM D 2555.)

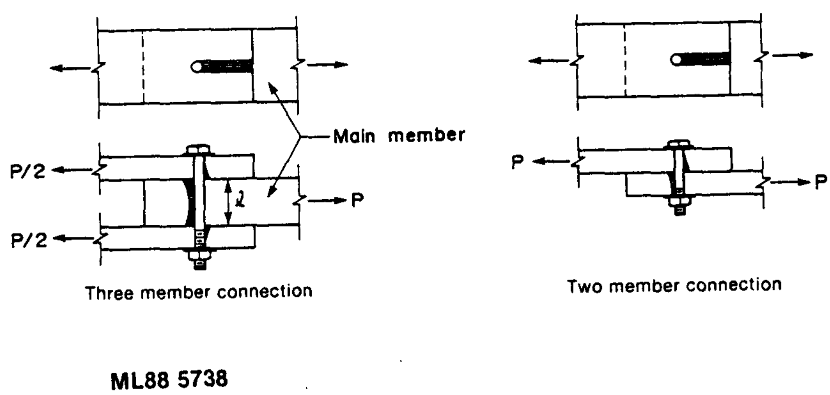

Figure 4.2—Configuration and Stress Distribution for a Laterally Loaded Bolted Connection.

The proportional limit strength is multiplied by 1.20 to adjust to dry strength and by 1.10 to adjust to normal duration of load; it is divided by 1.5 for a factor of safety. These adjustments result in a basic bolt-bearing strength for a zero l/d ratio; figure 4.4 is used to adjust the basic strength for other l/d ratios. The basic bolt-bearing strength perpendicular-to-grain is also affected by bolt diameter. Figure 4.5 is used to adjust for the diameter effect.

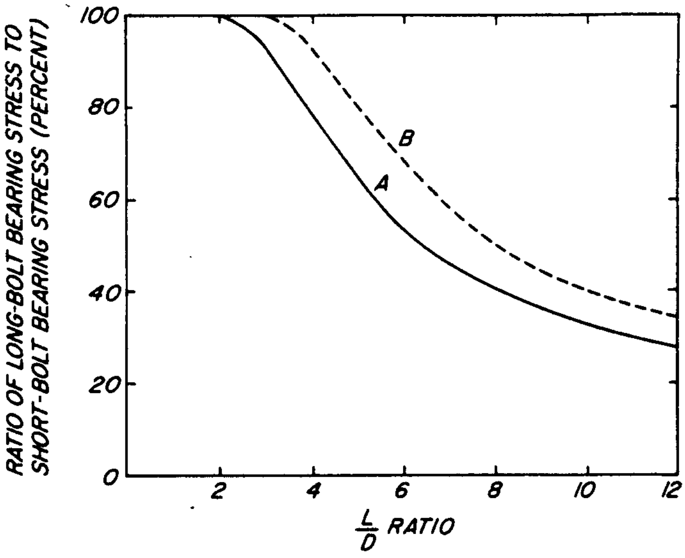

Figure 4.3— Variation in Bolt Bearing Stress at Proportional Limit Parallel-to-Grain vs. Slenderness Ratio. Curve A is Obtained from Experimental Evaluation; Curve B Modifies Curve A to Maintain a Constant Ratio Between Maximum and Design Strengths.

It is worth noting that the parallel-to-grain values are based on ultimate strength, while the perpendicular-to-grain values are based on proportional limit strength. Thus, different factors of safety apply to each.

There are different parallel-to-grain values for steel and wood side plates, whereas the wood main member always governs for perpendiculat-to-grain values regardless of side plate material.

The allowable load for one bolt is equal to the tabulated design load adjusted by all applicable modifiers and loading at an angle to the grain, when required. When more than one bolt is used, the allowable connection load is the sum of the design values of the individual bolts, adjusted by the Group Action Modification Factor, C_g

For laterally loaded bolts the applicable modification factors for loading parallel-to-grain (P) and perpendicular-to-grain (Q) are summarized below.

$$P' = P\, C_D\, C_M\, C_t\, C_r\, C_n\, C_s\, C_g\, C_{st}, \tag{4.1}$$
$$Q' = Q\, C_D\, C_M\, C_t\, C_r\, C_n\, C_s\, C_g. \tag{4.2}$$

The modifiers, C, are defined in Chapter 1.

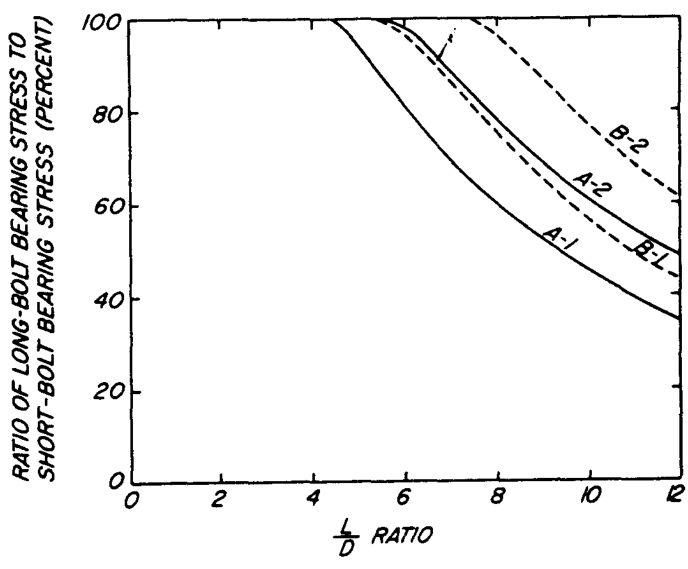

Figure 4.4—Variation in Bolt Bearing Stress at Proportional Limit Perpendicular-to-Grain vs. Slenderness Ratio. Curves A-1 and A-2 are for Experimental Average Compression Perpendicular Stresses of 1140 and 570 Psi, Respectively. Curves B-1 and B-2 Modify Curve A to Establish Design Strengths.

Although tabulated loads in NDS are for a balanced three member connection, the table is also used for other member thicknesses and two member connections. For three member connections where side members are less than one-half the thickness of the main member, the tabulated load is determined by assuming a main member twice the thickness of the thinnest side member. When steel side plates are used, the length of the bolt is based on the thickness of the wood member. For a bolted connection consisting of two members of equal thickness (single shear), the tabulated load is one-half that given for a main member the thickness of one of the members. When the two members are of unequal thickness, the tabulated load is the lesser of one-half the tabulated value for the thicker member, or one-half the tabulated value for a piece twice the thickness of the thinner member. For a two member comection consisting on one wood member connected to a steel plate, the design load is one-half of the tabulated value for the thickness of the wood member.

The tabulated values may be increased if there are steel rather than wood side members (for members loaded parallel to grain). There is a difference between the NDS-86 and TCM as to the amount of increase

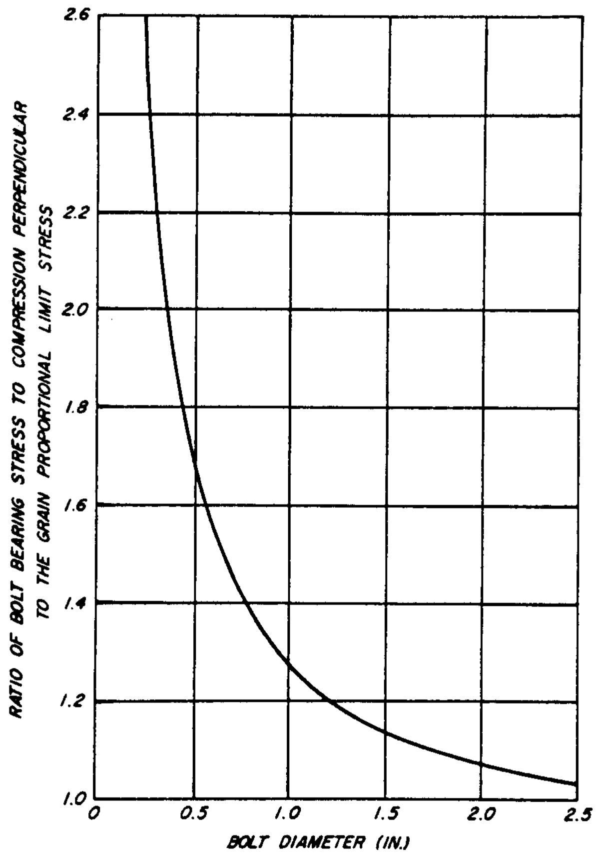

Figure 4.5—Bearing Stress Perpendicular-to-Grain as Affected by Bolt Diameter.

(steel side plate factor, C_{st}). The NDS allowed C_{st} equal to 1.75 for bolts equal to or less than 1/2-inch diameter; C_{st} equal to 1.25 for bolts equal to 1-1/2 inch diameter; and a linear interpolation to compute C_{st} for diameters between 1/2 and 1-1/2 inches. The TCM allows C_{st} to equal 1.25 for all diameters. The authors agree with the more conservative approach taken by the TCM based on our research findings (discussed later).

Single bolt design values tabulated in the specification must be further adjusted for loads at an angle to the grain by the commonly used Hankinson formula (defined in Chapter 1).

In addition to the bolted connection design, the capacity of the connected members must be checked because of the reduction in area due to the bolt holes. The net area at a bolted connection is equal to the gross area of the connected member minus the projected area of the bolt holes at the

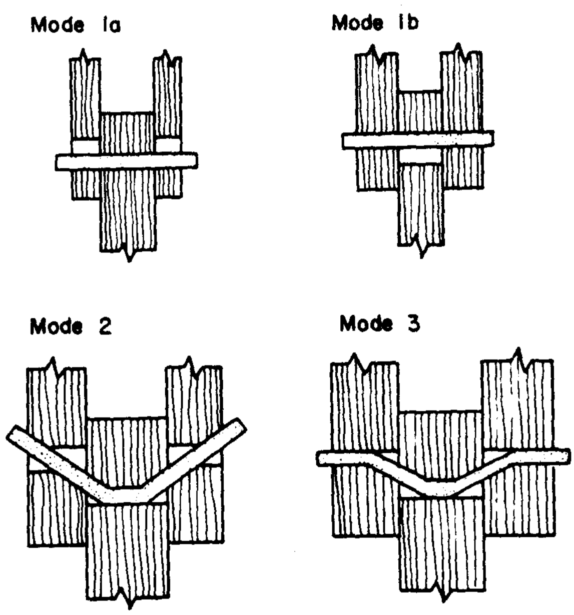

Figure 4.6—Failure modes assumed in the European Yield Model.

section (bolt holes are typically 1/32 to 1/16 inch greater than the bolt diameter). For parallel to grain loading with staggered bolts, the nearest bolt in the adjacent row is considered to occur at the same critical section unless the parallel-to-grain spacing of bolts in each row is a minumum of 8 times the bolt diameter.

4.3.2 Laterally Loaded Single Bolt Design-After 1991

The empirical design approach used prior to 1991 was based on a tabular value for a single bolt in a wood to wood, three member connection where the side members are each a minimum of one-half the thickness of the main member. The single bolt value must then be modified for any variation from these reference conditions. The theoretical approach, after 1991, is much more general and is not limited to these reference conditions.

The theoretical approach is based on work done in Europe (Johansen 1949) and is referred to as the European Yield Model (EYM). The EYM describes a number of possible failure modes that can occur in a dowel-type connection (fig 4.6). The yield strength of these different failure modes is determined from a static analysis that assumes the wood and the bolt both behave perfectly plastic. The failure mode that results in the lowest yield load for a given geometry is the theoretical connection yield load.

Equations corresponding to the failure modes for a three member joint are given in table 4.2. The nominal single bolt value is dependent on the joint geometry (thickness of main and side members), bolt diameter and bending yield strength, dowel bearing strength, and direction of load to the grain. The equations are equally valid for wood or steel side members which is taken into account by thickness and dowel bearing strength parameters. The equations are also valid for various load to grain directions which are taken into account by the K parameter.

The dowel bearing strength is a material property not generally familiar to structural designers. The dowel bearing strength is determined from tests that relate species specific gravity and dowel diameter to bearing strength. Empirical equations for these relationships are:

$$F_e = 11{,}200G \text{ (parallel-to-grain)}, \tag{4.3}$$
$$F_e = 6{,}100G^{1.45}\, D^{-0.5} \text{ (perpendicular-to-grain)}, \tag{4.4}$$

where F_e dowel bearing strength (psi), G = specific gravity based on oven-dry volume, and D = bolt diameter (inches).

4.3.3 Tension Loaded Single Bolt Design

In tension connections the bolt is loaded in axial tension parallel to its axis. The strength of a tension connection depends on the bearing strength of the wood and the tensile strength of the bolt (20,000 lb/in^2 for A307 bolts). The bearing stress under the washer must not exceed the allowable stresses for compression perpendicular to grain. To compute bearing stress, the bolt load is divided by the total washer area minus the area of the bolt hole. Distance and spacing requirements for bolts loaded in tension only are not specified in NDS and should be based on designer judgment.

4.3.4 Multiple Bolt Design

The strength of a multiple-bolt connection is calculated by summing the single-bolt strength values and multiplying by a adjustment factor that depends on how the load is distributed to each bolt. The adjustment factor reduces connection strength when there are three or more bolts in a row.

TABLE 4.1 Yield Theory Equations for Three-Member Joints.

$$Z = \frac{Dt_m F_{em}}{4K_\theta} \qquad \text{Mode } I_m$$

$$Z = \frac{Dt_s F_{es}}{2K_\theta} \qquad \text{Mode } I_S$$

$$Z = \frac{k_3 Dt_s F_{em}}{1.6(2+R_e)K_\theta} \qquad \text{Mode } III_S$$

$$\text{where } k = -1 + \sqrt{\frac{2(1+R_e)}{R_e} + \frac{2F_y(2+R_e)D^2}{3F_{em}t_s^2}},$$

$$Z = \frac{D^2}{1.6K_\theta}\sqrt{\frac{2F_{em}F_y}{3(1+R_e)}} \qquad \text{Mode IV}$$

F_y = bending yield strength of bolt, psi,
D = nominal bolt diameter, inches,
K_θ = $1 + \theta/360$,
Z = nominal single bolt design value,
t_m = thickness of main (center) member, inches,
t_s = thickness of side member, inches,
F_{em} = dowel bearing strength of main (center) member, psi,
F_{es} = dowel bearing strength of side members, psi,
θ = angle of load to grain,
R_e = F_{em}/F_{es}.

The adjustment factor is dependent on whether the side plates are wood or steel, the ratio of the main member cross sectional area to that of the side member, and on the area of the side main. The adjustment factor table from the 1991 NDS is reproduced here (table 4.2). The 1991 NDS presents this table in equation format. Bolts in staggered rows shall be considered as one row of bolts if the distance between staggered bolts is less than 1/4 of the spacing between adjacent staggered bolts.

The center of gravity of the bolt group should coincide with the line of action of the forces being transmitted to avoid eccentricity in the joint. Eccentricity in connections induces tension perpendicular-to-grain stresses that can severely reduce the capacity of the connected members. This capacity reduction has not been quantified and connections of this type are to be avoided. Eccentricity can also occur if one or more of the bolt holes is misfabricated resulting in a transfer of forces among the bolts in a group.

TABLE 4.2 Multiple-Bolt Adjustment Factor, C_g, for Laterally Loaded Bolts.

A_s/A^1_m	A^1_s in²	Number of fasteners in a row										
		2	3	4	5	6	7	8	9	10	11	12
	5	0.98	0.92	0.84	0.75	0.68	0.61	0.55	0.50	0.45	0.41	0.38
	12	0.99	0.96	0.92	0.87	0.81	0.76	0.70	0.65	0.61	0.57	0.53
0.5	20	0.99	0.98	0.95	0.91	0.87	0.83	0.78	0.74	0.70	0.66	0.62
	28	1.00	0.98	0.96	0.93	0.90	0.87	0.83	0.79	0.76	0.72	0.69
	40	1.00	0.99	0.97	0.95	0.93	0.90	0.87	0.84	0.81	0.78	0.75
	64	1.00	0.99	0.98	0.97	0.95	0.93	0.91	0.89	0.87	0.84	0.82
	5	1.00	0.97	0.91	0.85	0.78	0.71	0.64	0.59	0.54	0.49	0.45
	12	1.00	0.99	0.96	0.93	0.88	0.84	0.79	0.74	0.70	0.65	0.61
1	20	1.00	0.99	0.98	0.95	0.92	0.89	0.86	0.82	0.78	0.75	0.71
	28	1.00	0.99	0.98	0.97	0.94	0.92	0.89	0.86	0.83	0.80	0.77
	40	1.00	1.00	0.99	0.98	0.96	0.94	0.92	0.90	0.87	0.85	0.82
	64	1.00	1.00	0.99	0.98	0.97	0.96	0.95	0.93	0.91	0.90	0.88

1. When $A_s/A_m > 1.0$, use A_m/A_s and use A_m instead of A_s.
2. Tabulated group action factors (C_g) are conservative for D < 1", s < 4" or E > 1,400.000 psi.

4.3.5 Spacing End, and Edge Distances

Single and multiple-bolt values are based on having sufficient spacing, end, and edge distances to develop the full capacity of the connection. These requirements vary for parallel and perpendicular-to-grain loading and are summarized in figure 4.7. If the spacing is less than the minimum specified (for parallel-to-grain only), the bolt values must be reduced by the ratio of the actual bolt spacing to the minimum bolt spacing.

In addition to minimum spacing requirement, there is also a maximum requirement of five inch spacing between rows of bolts paralleling the connected member. This applies to steel side plates only since the purpose of this requirement is to eliminate cracks from the wood shrinkage being restrained by the steel side plates.

4.3.6 Drift Bolts

There is little design information available on drift bolts or pins and requirements for net area, end distance, edge distance and spacing are taken as the same as a bolt of the same diameter. NDS specifies that lateral design loads in the side grain of wood not exceed 75 percent of the design value for a comparable bolt of the same diameter and length in main member. Fastener penetration is left to the judgement of the designer. Drift bolts and drift pins are driven in prebored poles that are 1/8-inch to 1/16-inch smaller in diameter than the fastener diameter.

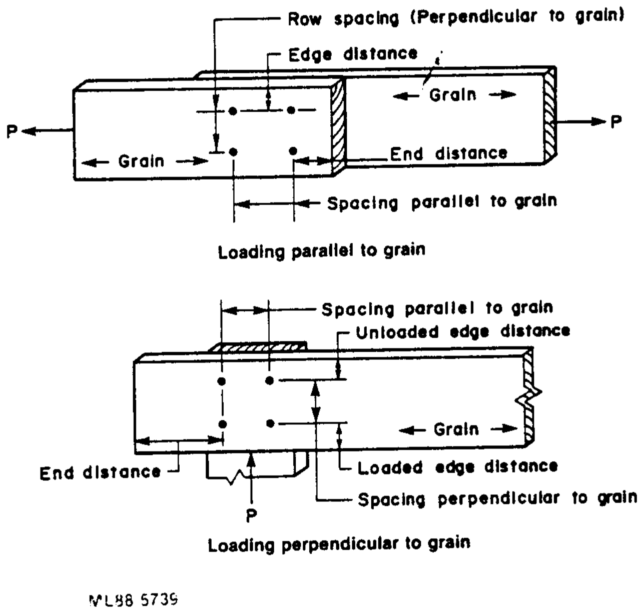

Figure 4.7—Edge and End Distance and Spacing Requirements for Bolted Connections.

4.4 DETAILS AND FABRICATION PRACTICES

4.4.1 Connection Details

Problems and failures in construction details have been observed and are discussed in Chapter 1. The problems are usually associated with shrink/swell and decay of lumber. The TCM is an excellent reference for standard details as well as do's and don't examples.

4.4.2 Fabrication Considerations

The strength of a bolted connection depends on fabrication tolerances. The strength of a laterally loaded bolted connection can be significantly affected by the diameter of the hole and the manner in which it is bored. When holes are too large, bearing is nonuniform and the capacity of the connection is reduced. If holes are too small the bolt cannot be inserted without driving, which may split wood members. NDS specifies that bolt holes be a minimum of 1/32 inch to a maximum of 1/16 inch larger than the bolt diameter. In some cases it may be necessary to slightly increase the hole to compensate for galvanized coatings on large fasteners.

Fabrication tolerances have been identified (Wilkinson 1986; 1989) as the variable having the largest effect on the strength of a row of bolts. If one or more of the bolt holes are misfabricated, there will be a redistribution of forces among the other bolts in the row. This redistribution may overstress one or more of the other bolts. There are no published fabrication tolerances for multiple bolted joints. Designers should exercise engineering judgment in their specifications and inspection practices to ensure all bolts participate in resisting applied forces.

When bolts are installed in wood members, washers of the proper size or a steel plate or strap is required under all nuts and under square or hexagonal bolt heads. Nuts must be tightened so that member surfaces are brought into close contact without crushing the wood.

Tabulated design values for bolts include an allowance for the loosening of nuts due to member shrinkage; however, when bolts are installed in unseasoned wood it is advisable to retighten connections at least every six months until the wood reaches equilibrium moisture content.

4.5 FOREIGN SPECIFICATIONS

Strength of single bolt connections are generally presented in either tabular or equation format. The tabular format (such as NDS, TCM) is based on either European Yield Method (EYM) or test data with species and l/d parameters. This format is used in Canada (Canadian Standards Association 1984) and Great Britain. The equation format is based on a theoretical model (discussed later), the European Yield Theory Model originated by Johansen (1949). The European CIB-Structural Design Code (1983) uses this approach.

Multiple-bolt connection generally have a load distribution factor and minimum spacing requirements. The modification factor used in the NDS and TCM is based on an elastic analysis (Cramer 1968; Lantos 1969). The National Standard of Canada uses these same modification factors. The British Standard and the European CIB Structural Design Code use an empirical equation for the modification factor when there are more than four bolts in a row. Spacing, end, and edge distances are the same for the foreign codes as the NDS and TCM.

4.6 RESEARCH BASIS FOR DESIGN PRACTICE

4.6.1 Single Bolt Connection

The U.S. and Canadian specifications for single bolt strength is based on calibrating the EYM to the experimental work of Trayer (1932). The European specification for single bolt strength is based on the analytic work of Johansen (1949) called the European Yield Method.

Trayer empirically fit experimental data to define the relationship of the bolt's strenglh to its l/d ratio. The European Yield Theory has been compared to Trayer's result by several researchers. 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 (Soltis et al, 1986; Soltis and Wilkinson 1987) found agreement between predicted and observed values for 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 hinges develop in the bolt (Mode III or IV 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 4.2 for three-member connections together with the formulas for the yield strength, Z, corresponding to each failure mode.

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

$$P_n = \frac{F_p}{t_m D f_c}, \tag{4.5}$$

where

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

If the yield strengths, Z, are normalized, the formulas in tables 4.2 result.

Trayer (1932) concluded that three-member comections with steel side plates carried 20 percent more load at the 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 t_m/D values (fig. 4.8). Trayer did not compare connections that had small t_m/D values. The scatter in his data is greatest where the yield theory predicts the greatest difference between steel and wood side plates.

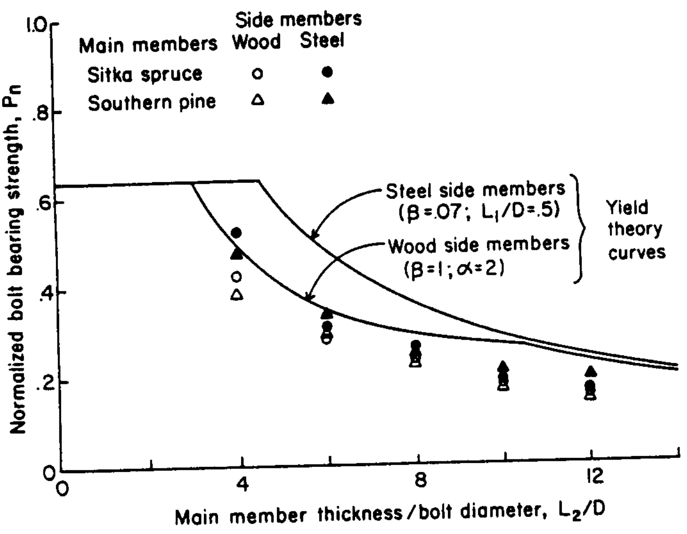

Figure 4.8—Results for Proportional Limit (Trayer 1932) and Yield Load (European Yield Theory) for Three-Member Connections with Steel and Wood Side Plates and Parallel-to-Grain Loading (F_y = 45000 Psi, f_c =5130 Psi, F_{em} = 3280 Psi).

Trayer 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 t_m/D values and a slight difference at t_m/D values greater than nine.

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.

For connections loaded parallel-to-grain the NDS-86 allowed 75 percent more strength with steel than with wood side members for bolts 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 was 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 t_m/D ratio and to the ratio of steel thickness to bolt diameter, t_s/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 indicated that different results might be expected for high-strength bolts, and this is borne out by the yield theory.

4.6.2 Multiple Bolt Connection

The adjustment factors for multiple bolt connections in current design practice for load distribution between bolts is based on the Lantos (1969) and Cramer (1968) analyses. 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 t_m/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.

Wilkinson (1989) compared adjustment 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.

4.6.3 Spacing, End, and Edge Distance

Current design criteria for spacing, end, and edge distance of bolted connections are based on experimental observation. The NDS specifications 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. 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.

4.6.4 Fabrication Considerations

In assigning equal parts of a load to the bolts in a row, one tacitly assumes all bolt holes have identical fabrication tolerance. Wilkinson (1986; 1989) 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.

4.7 SUMMARY RECOMMENDATIONS

Allowable lateral and tensile strength values of a single bolt connection for a three member joint loaded parallel to grain are well documented. The tabulated values for lateral strengths are based on ASTM A307 bolts and wood side members. The yield theory presents a means to take into account other variables such as higher strength bolts or different thicknesses of steel or wood side members.

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.

The strength of a multiple-bolt connection is the sum of the singie-bolt values, multiplied by an adjustment factor. The adjustment 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 adjustment 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. 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.

The United States, Canada, and Europe have similar code requirements for adjustment factors and spacing requirements. The effects of other factors such as fabrication tolerances, eccentricity, duration of load, and preservative or fire treatments are not known. Fabrication tolerances and eccentricity are known to have a large impact on connection strength, but this impact has not been quantified. Specifications and inspection practices should pay particular attention to fabrication tolerances and eccentricity.

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ASCE Manuals and Reports on Engineering Practice No. 84

Mechanical Connections in Wood Structures

Prepared by the Task Committee on
Fasteners of the Committee on Wood of the
Structural Division of the
American Society of Civil Engineers

Published by the
American Society of Civil Engineers
345 East 47th Street
New York, New York 10017-2398