# VIII: BOLTS

## 8.1-GENERAL

### 8.1.1-Quality of Bolts

In the previous three editions, the bolt quality standard referenced in the Specification was ASTM Standard A307, Low Carbon Steel Externally and Internally Threaded Fasteners. The current standard of this designation, A307-88a, is now titled, Standard Specification for Carbon Steel Bolts and Studs, 60,000 psi Tensile Strength. The scope of the present ASTM A307 is too narrow for it to be used as a reference standard as bolt provisions in the 1991 edition of the Specification provide for the use of bolts of any bending yield strength (see Appendix I). ANSI/ASME Standard B18.2.1-1981, Square and Hex Bolts and Screws (Inch Series), which is being used as the bolt quality reference in the 1991 edition, is the referenced standard for dimensions in A307.

Bolt design values given in previous editions of the Specification were based on a bending yield strength, $F_{yb}$ , of 45,000 psi. This value is applicable to A36 steel having an ultimate tensile strength of 58,000 psi. Bolt design values tabulated in Tables 8.2A-D and 8.3A-D of the 1991 edition are based on bolts made of steel having these properties.

**Maximum bolt diameter.** Bolt design values for bolts up to 1-1/4 inches in diameter were tabulated in the 1968 and earlier editions of the Specification. Values for 1-1/2 inch bolts were added to the design load tables beginning with the 1971 edition. In the 1991 edition, bolt design provisions and tabulated bolt design loads apply only to bolts having diameters of 1 inch or less. This conservative change was made following several reported field problems with connections involving large diameter bolts in glued laminated timber members and the results of new research (40,173). The latter showed drying in service, workmanship variables and perpendicular to grain load components could interact to affect the capacity of connections made with multiple large diameter, relatively stiff bolts.

It is recognized that the experience record shows most large diameter bolted connections, particularly those involving only one or two bolts, have performed satisfactorily over many years. However, it is to be understood that use of the procedures in the 1991 edition of the Specification to establish allowable loads for 1-1/4 or 1-1/2 inch bolted connections is the sole responsibility of the designer.

### 8.1.2-Fabrication and Assembly

**8.1.2.1** The range of allowable bolt holes of 1/32-inch to 1/16-inch larger than the bolt diameter has been a provision of the Specification since 1948. Designation of these limits as minimum and maximum oversizes was added in the 1986 edition.

Generally, the smaller diameter bolts will use the smaller oversize holes and the larger bolts the larger oversize. The same target oversize is to be used for all holes in the same connection.

**8.1.2.2** Centering of holes and avoidance of forcible driving have been good practice provisions of the Specification since the 1944 edition.

**8.1.2.3** Use of washers or equivalent metal parts under the head and nut to prevent localized crushing of the wood at bolt holes has been a requirement of the Specification since the 1960 edition.

**8.1.2.4** Design values for bolted joints have been applied to connections having both tight and loose nuts since the 1944 edition. This provision is based on the original bolted joint tests used to establish design values in which the nuts were intentionally not tightened in order to simulate the additional shrinkage that can occur during service (183). It is to be noted that 8.1.2.4 addresses only the loosening of nuts that may occur from shrinkage and not the effects of moisture on bearing strength or the effects of checks and cracks that may occur from seasoning after fabrication. Reduction of bolt design values for these factors is required when connections are assembled with wet or partially seasoned wood (see 7.3.3).

**8.1.2.5** The requirement that the threaded portion of bolts bearing on wood members be kept to a practical minimum was added as a good practice recommendation in the 1986 edition. Allowance of some thread bearing on wood without modification of bolt design values is long standing practice supported by field experience. Note should be made that when threads occur in the shear plane, the effect of such threads on the design shear strength of the bolt itself is to be taken into account (see 7.2.3).

## 8.2-DESIGN VALUES FOR SINGLE SHEAR CONNECTIONS

### 8.2.1-Wood-to-Wood Connections

**Background**

Bolt design values tabulated in the Specification prior to the 1991 edition were based on early research (183) which related the bearing strength of wood under bolts to the compression parallel and perpendicular to grain properties of the wood members and the resistance of the bolt to bending as measured by the ratio of the length of bolt in the main member to its diameter. Except for adjustments made to reflect the change in lumber design values in 1970 resulting from the introduction of new clear wood strength properties and species grouping criteria in ASTM D2555, and the provisions of the new softwood lumber product standard PS 20, the general methodology for establishing bolt design values remained essentially unchanged from the 1944 through the 1986 editions. Under this methodology, allowable bolt design values were established in accordance with the equations shown below.

Bolt design values parallel to grain:

$$Z = c_1 F_1 L d r_1 \qquad \text{(C8.2-1)}$$

where:

$Z$ = nominal bolt design value, pounds
$c_1$ = adjustment for difference between proportional limit load of bolted joints and proportional limit under uniform compression load = 0.80
$F_1$ = unseasoned clear wood maximum compression design value parallel to grain 5 percent exclusion value for each species or species group (based on ASTM D2555), reduced 1.0/1.9 for load duration and factor of safety based on ASTM D245, and increased 1.20 for seasoning based on early research (183), psi
$L$ = length of bolt in main member, inches
$d$ = bolt diameter, inches
$r_1$ = adjustment depending on $L/d$ ratio of bolt and $F_1$ (ranging from 1.00 at $L/d$ of 2.00 to 0.310 at $L/d$ of 13)

Although the proportional limit joint load factor ($c_1$) for hardwoods was found to be 1.00 (183), and the ASTM D245 load duration/factor of safety adjustment for hardwoods is 2.1, the softwood values for these terms were used for all species for simplicity.

Bolt design values perpendicular to grain:

$$Z = c_2 F_2 L d r_2 \qquad \text{(C8.2-2)}$$

where:

$c_2$ = width of bearing increase based on bolt diameter (183) (ranging from 2.50 for 1/4-inch bolt to 1.27 for 1-inch bolt)
$F_2$ = clear wood unseasoned average proportional limit stress for species or species group based on ASTM D2555, reduced 1.0/1.5 for ring placement based on ASTM D245, increased 1.10 for normal loading, and increased 1.20 for seasoning based on early research (183)
$r_2$ = adjustment depending on $L/d$ ratio of bolt and $F_2$ (ranging from 1.00 at $L/d$ of 5.00 to 0.375 at $L/d$ of 13)

other symbols as previously defined

The bolt design values established by the foregoing equations were applicable to three member joints in which the side members were one-half the thickness of the main member. Allowable bolt design values for two member or four or more member joints were established as proportions of the three member bolt design values in accordance with specified rules.

**1991 Edition.** In the 1991 edition, lateral design values for bolts are based on a yield limit model which considers the different ways the bolted connection can deform under load (see Commentary for 7.2.1 and Appendix I). The capacity of a specific joint is determined for each yield mode and the lowest design value calculated for the different modes is selected as the nominal bolt design value, $Z$, for the joint. The yield mode equations are entered with the dowel bearing strengths and thicknesses of the wood members and the diameter and bending yield strength of the fastener.

Wood dowel bearing strengths used in the yield mode equations are based on a load representing a 5 percent diameter offset on the load-deformation curve obtained from a bolt embedment test. This load is intermediate between the proportional limit and ultimate loads obtained from such a test (see Commentary section 7.2.1).

Although the yield limit model represents significantly different methodology than that used previously to establish bolt design values, the relative effects of various joint variables shown by both procedures are generally similar (166). Short-term bolt design values obtained from application of the yield model equations have been reduced to the average bolt design value levels published in previous editions of the Specifications for connections made with the same species and member sizes (see Commentary for 7.2.1). As noted above, these previous tabulated bolt design values are indexed to nominal proportional limit bolt design values rather than 5 percent offset bolt design values.

**Previous Methodology for Single Shear Bolted Connections.** Bolt design values tabulated in the 1986 and earlier editions applied to three member joints in which the side members were each one-half the thickness of the main member. For two member joints, those with a single shear plane, a proportion of the bolt design value for the three member connection was used. Prior to the 1977 edition, the two-member proportion was taken as one-half the tabulated three member bolt design value for a piece twice the thickness of the thinner piece. This early practice for establishing single shear bolt design values, introduced in 1935 (127), was considered appropriate application of the results of the original bolt research (58). In 1977, based on new research and reevaluation of original test results (82,183,200), the basis for bolt design values for two member connections was changed to the smaller of (i) one-half the tabulated three member bolt design value for a piece the thickness of the thickest member, or (ii) one-half the tabulated three member bolt design value for a piece twice the thickness of the thinner member. This change provided more conservative design values for all two member wood-to-wood single shear bolted connections in which the thicker member was less than twice the thickness of the thinner member, with reductions of 50 percent occurring in some joints in which the two members were of equal thickness. The new practice for establishing design values for two member bolted connections was continued through the 1986 edition.

Results from application of the yield model confirm the general relationship between design values for two member and three member bolted connections established in the 1977 edition (202).

**Joint Members Loaded in Different Directions.** The change in single shear joint provisions introduced in the 1977 edition resulted in inconsistent treatment of joints in which one member was loaded parallel to grain and the other perpendicular to grain. The inconsistency occurred when the parallel loaded member was just slightly thicker than the perpendicular loaded member. In this case the joint was assigned a perpendicular bolt design value based on a piece twice the thickness of the perpendicular member rather than a piece the thickness of this member. The deficiency was corrected in the 1986 edition by assigning single shear joints in which one member was loaded parallel to grain and the other at an angle to grain a bolt design value equal to the lesser of (i) one-half the tabulated bolt design value for a piece the thickness of the parallel to grain loaded member, or (ii) the bolt design value obtained from the equation for allowable bearing at an angle to grain (Appendix J of the Specification) using one half the tabulated parallel to grain and perpendicular to grain bolt design values for a piece the thickness of the angle-to-grain member as P and Q, respectively. This procedure assured that allowable loads on joints in which one member was loaded at an angle to grain would converge with those for joints in which the angled member was at 90° to the parallel loaded member.

In the 1991 edition, the condition where members of a single shear bolted connection are loaded at different angles to the grain is provided for by the factor, $K_\theta$, in the denominator of the yield mode equations which accounts for the maximum angle of load to grain for any member in the connection; and by Equation 8.2-7 of the Specification which uses the bearing angle to grain equation (Appendix J) to adjust the dowel bearing strength of each member loaded at an angle to grain.

### Yield Mode Equations

The bolt design value equations (8.2-1 to 8.2-6) for single shear wood-to-wood connections were developed from European research (93,104) and have been confirmed by bolt tests on domestic species (116,163,166,167). The limiting yield modes covered by these equations are bearing in the main or side members (Mode I), bolt rotation without bending (Mode II), development of a plastic hinge in the bolt in main or side member (Mode III) and development of plastic hinges in the bolt in both main and side members (Mode IV) (see Appendix I of the Specification). The term $4K_\theta$, $3.6K_\theta$ or $3.2K_\theta$ in the denominator of equations 8.2-1 to 8.2-6 represents the average factor relating yield model design value for each mode based on 5 percent offset dowel bearing strength to the proportional limit based bolt design values tabulated in the 1986 edition (202). For bolts loaded parallel to grain, $K_\theta$ equals one. For perpendicular to grain loading, $K_\theta$ equals 1.25 for a connection with one member loaded parallel to grain and the other member loaded perpendicular to grain (202).

Dowel bearing strengths used in the yield mode equations are tabulated in Table 8A for all structurally graded lumber species. These values also apply to main members of glued laminated timber. The values in Table 8A represent 5 percent diameter offset values determined in accordance with the following equations (203):

Parallel to grain:

$$F_e = (11{,}200)\,G \qquad \text{(C8.2-3)}$$

Perpendicular to grain:

$$F_e = (6{,}100)\,G^{1.45}D^{-0.5} \qquad \text{(C8.2-4)}$$

where:

$F_e$ = dowel bearing strength, psi

$G$ = specific gravity based on oven dry weight and volume

$D$ = bolt diameter, inches

Effect of specific gravity on dowel bearing strength was established from 3/4-inch dowel embedment tests on Douglas-fir, southern pine, spruce-pine-fir. sitka spruce, red oak, yellow poplar and aspen. Diameter effects were evaluated from tests of 1/4-, 1/2-, 3/4-, 1-, and 1-1/2 inch dowels in southern pine using bolt holes 1/16-inch larger than the dowel diameter. Diameter was found to be a significant variable only in perpendicular to grain loading. Bearing specimens were 1/2-inch or thicker such that width and number of growth rings did not influence results (203).

The specific gravity values given in Table 8A for each species or species group are those used to establish the corresponding dowel bearing strength values, $F_e$. These specific gravity values represent average values from in-grade lumber test programs (see Commentary for 4.2.3.2) or are based on information from ASTM D2555. No separate specific gravity values are available for dense Douglas-fir and dense southern pine; therefore, no dowel bearing strength values nor bolt design values are tabulated for these dense species combinations in the 1991 edition.

The bending yield strength, $F_{yb}$, of the bolt used in the yield mode equations is taken as the average of the yield and ultimate tensile strengths of the metal (see Appendix I). For A36 and stronger steels, $F_{yb}$ equal to 45,000 psi is a conservative value and is equivalent to the bolt strength reported in the original bolt test research (183).

For each particular joint configuration, the nominal bolt design value for each yield mode must be calculated to determine the limiting value for the connection. Such yield mode bolt design values for a number of different joint designs are shown in Example C8.2-1 to illustrate the results of applying equations 8.2-1 to 8.2-6.

**Example C8.2-1**

Yield mode design values for wood-to-wood single shear bolted connections:

Hem-fir two member connections made with 1/2 and 1 inch bolts, side member thickness of 1-1/2 inches, main member thicknesses of 1-1/2, 3 and 5-1/2 inches, and loads applied parallel and perpendicular to the grain of main and side members

$F_{em}$, $F_{es}$ = 4800 psi parallel
= 2550 psi perpendicular, 1/2 inch bolt
= 1800 psi perpendicular, 1 inch bolt

$F_{yb}$ = 45,000 psi

| Thickness, in. & grain direction | | Bolt Diam. | Yield Mode Design Value, lbs | | | | | |
|---|---|---|---|---|---|---|---|---|
| Main | Side | in. | $ZI_m$ | $ZI_s$ | $ZII$ | $ZIII_m$ | $ZIII_s$ | $ZIV$ |
| 1-1/2 // | 1-1/2 // | 1/2 | 900 | 900 | 414 | 550 | 550 | 663 |
| 1-1/2 // | 1-1/2 ⊥ | 1/2 | 720 | 382 | 250 | 380 | 324 | 442 |
| 1-1/2 ⊥ | 1-1/2 // | 1/2 | 382 | 720 | 250 | 324 | 380 | 442 |
| 3 // | 1-1/2 // | 1/2 | 1800 | 900 | 674 | 845 | 550 | 663 |
| 3 // | 1-1/2 ⊥ | 1/2 | 1440 | 382 | 472 | 592 | 324 | 442 |
| 3 ⊥ | 1-1/2 // | 1/2 | 765 | 720 | 345 | 433 | 380 | 442 |
| 3 // | 1-1/2 // | 1 | 3600 | 1800 | 1359 | 2200 | 1884 | 2652 |
| 3 // | 1-1/2 ⊥ | 1 | 2880 | 540 | 874 | 1378 | 1143 | 1567 |
| 3 ⊥ | 1-1/2 // | 1 | 1080 | 1440 | 551 | 1117 | 1135 | 1567 |
| 5-1/2 // | 1-1/2 // | 1 | 6600 | 1800 | 2451 | 3161 | 1884 | 2652 |
| 5-1/2 // | 1-1/2 ⊥ | 1 | 5280 | 540 | 1678 | 2021 | 1143 | 1567 |
| 5-1/2 ⊥ | 1-1/2 // | 1 | 1980 | 1440 | 856 | 1311 | 1135 | 1567 |

**Member Loaded at Angle to Grain.** Equation 8.2-7 is used to calculate the dowel bearing strength for a main or side member loaded at an angle to grain. This equation, a form of the bearing angle to grain equation (Appendix J), has been used since the 1944 edition to determine allowable design values for bolts acting in a plane inclined to the direction of grain. In earlier editions, the equation was entered with allowable bolt design values parallel and perpendicular to grain. In the 1991 edition, the equation is entered with the parallel and perpendicular dowel bearing strengths for the member and the bolt design value is determined from the yield mode equations using $F_{e\theta}$ as the dowel bearing strength for the main or side member. The bolt design value obtained from this procedure is similar to that obtained from using parallel to grain and perpendicular to grain $Z$ values in the bearing interaction formula to obtain a $Z_\theta$ design value for the connection (202). Determining a $Z_\theta$ design value using this latter approach is an acceptable alternative to calculating $F_{c\theta}$ for use in each yield mode equation and allows the use of tabulated $Z$ values from the Specification.

**Tabulated Two Member Wood-to-Wood Bolt Design Values.** Bolt design values for lumber to lumber (Table 8.2A) and glued laminated timber to lumber (Table 8.2B) connections have not previously been tabulated in the Specification, being taken as a proportion of the tabulated three member bolt design value in earlier editions (see Commentary on Previous Methodology for Single Shear Bolted Connections). With the added refinement of the yield mode equations, two member bolt design values are not necessarily a fixed one-half the three member bolt design value. Separate two member bolted connection design values are therefore given to facilitate designer use. All tabular bolt design values are based on a bolt bending yield strength of 45,000 psi. Two bolt design values for perpendicular to grain loading are shown: one for a connection with the side member loaded perpendicular to grain and the main member loaded parallel to grain ($Z_{s\perp}$); and one for a connection with main member loaded perpendicular to grain and the side member loaded parallel to grain ($Z_{m\perp}$).

The soft conversion procedure used to translate short term yield mode bolt design values to bolt design values previously tabulated in the Specification involved use of average adjustment factors for each mode based on all bolt and member sizes, all species combinations, and both wood and steel side members. For two member connections, the adjustment factor was based on joints in which the side member was one-half the thickness of the main member. Under this indexing procedure, new bolt design values are both higher and lower than previous bolt design values depending upon the bolt diameter, the thicknesses of the main and side member, and the particular species combination involved.

**Comparison of 1991 and Earlier Edition Bolt Design Values.** Differences between 1991 and earlier edition wood-to-wood single shear bolt design values for two species combinations are shown in Table C8.2-1.

For the joint configurations compared, bolt design values based on the 1991 edition for the parallel and perpendicular to grain loading cases averaged 11 percent and 54 percent higher, respectively, than those based on the 1986 edition. It is to be noted that the short-term yield mode bolt design values on which the 1991 bolt design values are based were reduced to the average level of 1986 and earlier tabulated bolt design values using average factors for joints in which the side member was one-half the thickness of the main member. This was the configuration used for the early tests from which the bolt design values in the 1986 and earlier editions were derived. Only one configuration (3-1/2 main and 1-1/2 side) in the table above meets this calibration condition. The average ratios of 1991 to 1986 bolt design values for the joints in the table which have this calibration geometry are 0.97 and 1.18 for parallel and perpendicular to grain loading, respectively.

**Table C8.2-1 - Comparison of 1991 and 1986 NDS Wood-to-Wood Single Shear Bolt Design Values**

| Thickness, in. | | Bolt | Bolt Design Value, lbs | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | $Z_{//}$ | | | $Z_{m\perp}$ | | |
| Main, Wood | Side, Wood | Diam. in. | 1991 | 1986 | Ratio | 1991 | 1986 | Ratio |
| Southern pine: | | | | | | | | |
| 1-1/2 | 1-1/2 | 1/2 | 530 | 470 | 1.13 | 330 | 215 | 1.53 |
| | | 3/4 | 800 | 710 | 1.13 | 460 | 270 | 1.70 |
| | | 1 | 1060 | 945 | 1.12 | 580 | 325 | 1.78 |
| 3 | 1-1/2 | 1/2 | 660 | 635 | 1.04 | 470 | 430 | 1.09 |
| | | 3/4 | 1270 | 1315 | 0.97 | 620 | 540 | 1.15 |
| | | 1 | 1740 | 1875 | 0.93 | 750 | 650 | 1.15 |
| 3-1/2 | 3-1/2 | 1/2 | 750 | 635 | 1.18 | 520 | 490 | 1.06 |
| | | 3/4 | 1690 | 1400 | 1.21 | 960 | 630 | 1.52 |
| | | 1 | 2480 | 2135 | 1.16 | 1360 | 760 | 1.79 |
| 5-1/2 | 1-1/2 | 3/4 | 1270 | 1315 | 0.97 | 850 | 540 | 1.57 |
| | | 1 | 2150 | 1875 | 1.15 | 1190 | 650 | 1.83 |
| 5-1/2 | 3-1/2 | 3/4 | 1690 | 1430 | 1.18 | 1090 | 940 | 1.16 |
| | | 1 | 2870 | 2435 | 1.18 | 1550 | 1190 | 1.30 |
| Spruce-Pine-Fir: | | | | | | | | |
| 1-1/2 | 1-1/2 | 1/2 | 410 | 340 | 1.21 | 240 | 140 | 1.71 |
| | | 3/4 | 610 | 510 | 1.20 | 340 | 175 | 1.94 |
| | | 1 | 810 | 680 | 1.19 | 430 | 210 | 2.05 |
| 3 | 1-1/2 | 1/2 | 540 | 580 | 0.93 | 330 | 280 | 1.18 |
| | | 3/4 | 1000 | 1015 | 0.99 | 440 | 350 | 1.26 |
| | | 1 | 1330 | 1355 | 0.98 | 540 | 425 | 1.27 |
| 3-1/2 | 3-1/2 | 1/2 | 660 | 580 | 1.14 | 430 | 325 | 1.32 |
| | | 3/4 | 1420 | 1155 | 1.23 | 730 | 410 | 1.78 |
| | | 1 | 1890 | 1585 | 1.19 | 1000 | 495 | 2.02 |
| 5-1/2 | 1-1/2 | 3/4 | 1080 | 1015 | 1.06 | 690 | 350 | 1.97 |
| | | 1 | 1760 | 1355 | 1.30 | 830 | 425 | 1.95 |
| 5-1/2 | 3-1/2 | 3/4 | 1480 | 1310 | 1.13 | 900 | 640 | 1.41 |
| | | 1 | 2330 | 2275 | 1.02 | 1120 | 775 | 1.45 |

The yield mode equations on which the 1991 bolt design values are based provide a fully rationalized and consistent measure of the effects of main and side member thicknesses and bolt diameter. As such they are more reliable indicators of relative joint load-carrying capacity than the bolt design values given in earlier editions which represent a conservative application of test results for only one main to side member configuration.

**Mixed Species Connections.** Design values for bolted connections made with a side member of different species than the main member can be calculated using the yield mode equations (Eqs. 8.2-1 to 8.2-6) and the appropriate dowel bearing strength for each species. Mixed species connections were provided for in the 1982 and 1986 editions by assigning the joint the lesser of the bolt design value applicable to a comparable joint made with members of the main member species or a comparable joint made with members of the side member species. In lieu of using the yield mode equations, bolt design values for connections made with different main and side member species may be based on bolt design values in Table 8.2A and 8.2B for the species with the lower dowel bearing strength, $F_e$

## 8.2.2-Wood-to-Metal Connections

### Background

In the 1977 and earlier editions of the Specification, bolted connections made with steel side members were assigned design values that were 25 percent larger than tabulated bolt design values for wood main members loaded parallel to grain. No increase for steel side plates was recognized when wood main members were loaded perpendicular to grain. These provisions were based on early bolt tests (183) involving two softwood and two hardwood species which showed that proportional limit bolt design values for joints made with steel side members and the main member loaded parallel to grain averaged 25 percent higher than proportional limit bolt design values for the comparable wood side member joint.

In the 1981 edition, following additional research on two and three member joints made with steel and wood side plates, the adjustment of tabulated parallel to grain bolt design values for use of steel side plates was increased from 25 to 75 percent. The new research (113) was conducted as a result of the impact of the 1977 change in the procedure for establishing bolt design values for two member connections (see Commentary for 8.2.1 - Previous Methodology for Single Shear Bolted Connections). The 1977 change reduced allowable design values for single shear bolted connections, including those made with steel side members, as much as 50 percent below bolt design values previously used successfully for many years. Applications particularly affected included shear wall tie-downs where the change in the bolt design provisions of the Specification required the number of bolts used in such hardware to be doubled for the same code specified loads.

Results from the additional parallel to grain bolt tests (113), which involved joints made with 1/2-inch bolts, 1/4-inch thick steel side plates and southern pine main and side members, showed that the average proportional limit bolt design values for joints made with steel side plates were 20 to 63 percent higher than those for the matching wood side plate joints; that the lowest proportional limit joint design value (wood-to-wood three member joint) was 32 percent lower than the applicable tabulated bolt design value; and that the slip of the metal-to-wood joints was less than that of the wood-to-wood joints. The design values of the metal-to-wood joints associated with the proportional limit slip of the matching wood-to-wood joints were 75 percent larger on the average than the proportional limit design values for the wood-to-wood joints. These average equivalent test slip design values for the metal side plate joints in turn averaged from 20 to 70 percent lower than the maximum average test loads for these joints.

The information obtained from the new testing on the difference in bolt design values on joints made with steel and wood side members at the proportional limit slip of the latter was consistent with that reported in the early bolt research which formed the basis of the Specifications bolt design provisions (183). In the earlier work, a slip of about 0.025 inches was reported associated with the proportional limit design values of joints made with steel side plates and 1/2 inch diameter bolts, and these bolt design values averaged about 25 percent larger than those for comparable joints made with wood side plates. However, the slip associated with the proportional limit load of the wood side member joints was 0.035 inches, or about 40 percent greater than the proportional limit slip of the steel side member joints. As a first approximation, these results indicated the load of the steel side member joints associated with the proportional limit slip of the wood side member joints was $[(1.25P_w)/0.025] \times 0.035$, or about 75 percent larger than the proportional limit load of the wood-to-wood joint; the same as observed in the new tests.

On the basis of the 30 years of successful performance of two member bolted connections made with steel side plates at bolt design values up to twice those established in the 1977 edition, and recognizing that the structural serviceability of a joint is related to its stiffness or slip; a 75 percent adjustment factor for joints loaded parallel to grain and made with steel side plates, as shown by the slip equivalent loads, was introduced in the 1982 edition. This revision had the effect of increasing design values for steel side member bolted connections loaded parallel to grain approximately 40 percent over those provided by 1977 provisions.

In the 1986 edition, more conservative provisions for establishing design values for connections made with steel side members and large diameter bolts were introduced. This further revision followed unsatisfactory field experience with a long span truss design involving glued laminated timber tension chords and large diameter bolts designed in accordance with provisions of the 1977 edition. In response to questions raised about design procedures for all joints made with 1-inch and larger bolts, a special study (97) was conducted of double-shear connections made with glued laminated 4-1/2 by 4-1/2 inch main members, 1-1/4-inch bolts and 2-1/4-inch wood and 1/2-inch steel side members. Although the average ratio of ultimate test load to 1982 NDS bolt design value was 2.7 for the joints made with steel side plates, the ratio was lower than the comparable ratio of 3.9 obtained for connections made with 1/2-inch bolts and steel side members that were tested earlier (113). Wood-to-wood 1/2 inch and 1-1/4 inch bolted joints both had test to design load ratios exceeding 4.0.

Based on these results, it was considered appropriate to reduce the design values for connections made with steel side members and large diameter bolts such that the load ratio factor for the 1-1/4 inch bolt tests was comparable to that for the 1/2 inch bolt tests. This was accomplished in the 1986 edition by providing a variable adjustment for connections loaded parallel to grain and made with steel side members with limits of 75 percent for bolts 1/2 inch or less in diameter and 25 percent for bolts 1-1/2 inch in diameter, with proportionate adjustments for intermediate diameters.

**1991 Edition.** Bolt design provisions in the current edition are limited to bolts 1 inch or less in diameter. This new limitation reflects concern about the effects of workmanship variables and drying in service on the performance of large diameter bolted connections (see Commentary for 8.1.2). Further, all bolt design values are now based on the yield limit model which does not account for degree of deformation or slip (see Commentary for 8.2.1 - Background). Short-term bolt design values based on direct application of the yield model equations have been reduced to the nominal average bolt design value levels published in earlier editions for connections made with the same species and member sizes, with the exception that conversion factors for joints made with steel side members were referenced to 1977 edition bolt design values which utilized a 25 percent increase for joints loaded parallel to grain and made with metal side plates. This approach was used on the basis that bolt design values in the 1977 and earlier editions were related to proportional limit joint loads, yield model bolt design values are based on loads associated with an offset of 5 percent of fastener diameter, and both are independent of a specific slip level. In addition, the same conversion factors are applied to connections made with steel side members as to wood side members.

It is recognized that relating short-term yield model bolt design values for parallel to grain loaded connections made with steel side members to 1977 edition bolt design values for these connections results in a reduction in bolt design values from 1982 and 1986 edition levels. The advantages of consistency of treatment across all dowel fastener types, and the ability to determine by fully rationalized methodology the effects of member thickness, member strength, bolt size, bolt strength and number of members, both singly and in combination, were considered to outweigh the impact of the reduction in bolt design values for those connections employing steel side members. Differences between 1991 and 1986 design values for such bolted joints are illustrated in the Commentary for 8.2.2.1. In this regard, it is to be noted that design values for bolted connections involving proprietary tie-downs or similar hardware may be established by other procedures than those given in the 1991 edition (see 7.1.1.4 and 1.1.1.4 of the Specification). However, use of such alternate methodologies is the sole responsibility of the manufacturer and of the designer utilizing the design values so derived.

**8.2.2.1** The same yield mode equations used for single shear wood-to-wood bolted connections are used for wood-to-metal bolted connections except equation 8.2-2 for mode $I_s$ which is for uniform bearing in the metal side member. This condition is checked independently in accordance with 8.2.2.2 and 7.2.3 (see Commentary for this latter section).

The yield mode equations of 8.2.1 may be entered with a dowel bearing strength, $F_{es}$, of 58,000 psi for the side member when A36 or higher strength steel side plates are used. This value is equivalent to the ultimate tension strength of the steel (see Appendix I.2).

A nominal bolt design value, $Z$, is calculated for each of the five applicable yield mode equations and the lowest value is selected as the design value for the bolted connection. The effects of different main member thicknesses, bolt diameters and loading directions on values of $Z$ that can be obtained from each yield mode equation, and on the limiting load for the connection, are illustrated by the example joint design calculations shown in Example C8.2-2.

Tabulated bolt design values for lumber and glued laminated timber single shear connections with 1/4 in. steel side plates given in Tables 8.2C and 8.2D assume A36 steel having a dowel bearing strength of 58,000 psi, and a bolt bending yield strength of 45,000 psi.

**Comparison of 1991 and Earlier Edition Bolt Design Values.** Differences in bolt design values for lumber-to-steel single shear connections between the 1991 and the 1986 editions are illustrated in Table C8.2-2. For the species and joint configurations compared, the parallel to grain design values for single shear steel-to-wood bolted connections based on the provisions of the 1991 edition average 25 percent lower than those based on provisions in the 1986 edition. Perpendicular to grain bolt design values average 30 percent higher in the new edition than the perpendicular to grain bolt design values in previous editions which contained no increase for use of steel side plates.

The 1986 edition bolt design values in Table C8.2-2 represent one-half the bolt design value for a three member joint made with wood side members one-half the thickness of the main member and then, for the case of parallel to grain loading only, increased 75 percent, 62-1/2 percent and 50 percent for 1/2 inch, 3/4 inch and 1 inch bolts, respectively, when steel rather than wood side members were used. As previously discussed in the Background commentary to this section, the 1991 bolt design values are based on a conversion of short term yield mode bolt design values

### Example C8.2-2

Yield mode design values for wood-to-metal single shear bolted connections:

Hem-fir two member wood-to-metal connections made with 1/2 and 1 inch bolts, steel side member thickness of 1/4 inch, main member thicknesses of 1-1/2, 3, 3-1/2 and 5-1/2 inches, and loads applied parallel and perpendicular to the grain of main and side members

$F_{em}$ = 4800 psi parallel
= 2550 psi perpendicular, 1/2 inch bolt
= 1800 psi perpendicular, 1 inch bolt
$F_{es}$ = 58,000 psi
$F_{yb}$ = 45,000 psi

| Wood Thickness, in. | Load Direction | Bolt Diam. in. | Yield Mode Design Value, lbs $ZI_m$ | $ZII$ | $ZIII_m$ | $ZIII_s$ | $ZIV$ |
|---|---|---|---|---|---|---|---|
| 1-1/2 | // | 1/2 | 900 | 470 | 700 | 706 | 901 |
| | | 1 | 1800 | 940 | 2557 | 2577 | 3604 |
| | ⊥ | 1/2 | 382 | 238 | 384 | 428 | 535 |
| | | 1 | 540 | 381 | 1367 | 1310 | 1809 |
| 3 | // | 1/2 | 1800 | 829 | 1040 | 706 | 901 |
| | | 1 | 3600 | 1658 | 2798 | 2577 | 3604 |
| | ⊥ | 1/2 | 765 | 375 | 495 | 428 | 535 |
| | | 1 | 1080 | 555 | 1280 | 1310 | 1809 |
| 3-1/2 | // | 1/2 | 2100 | 957 | 1174 | 706 | 901 |
| | | 1 | 4200 | 1915 | 2979 | 2577 | 3604 |
| | ⊥ | 1/2 | 893 | 427 | 546 | 428 | 535 |
| | | 1 | 1260 | 626 | 1294 | 1310 | 1809 |
| 5-1/2 | // | 1/2 | 3300 | 1486 | 1741 | 706 | 901 |
| | | 1 | 6600 | 2971 | 3902 | 2577 | 3604 |
| | ⊥ | 1/2 | 1402 | 646 | 775 | 428 | 535 |
| | | 1 | 1980 | 929 | 1461 | 1310 | 1809 |

**Table C8.2-2 - Comparison of 1991 and 1986 NDS Wood-to-Metal Single Shear Bolt Design Values**

| Thickness, in. Main, Wood | Side, Steel | Bolt Diam. in. | Bolt Design Value, lbs $Z_{//}$ 1991 | $Z_{//}$ 1986 | $Z_{//}$ Ratio | $Z_{\perp}$ 1991 | $Z_{\perp}$ 1986 | $Z_{\perp}$ Ratio |
|---|---|---|---|---|---|---|---|---|
| Southern pine: | | | | | | | | |
| 1-1/2 | 1/4 | 1/2 | 570 | 822 | 0.69 | 310 | 215 | 1.44 |
| | | 3/4 | 860 | 1154 | 0.75 | 390 | 270 | 1.44 |
| | | 1 | 1140 | 1418 | 0.80 | 480 | 325 | 1.48 |
| 3 | 1/4 | 1/2 | 780 | 1111 | 0.70 | 500 | 360 | 1.39 |
| | | 3/4 | 1560 | 2136 | 0.73 | 640 | 540 | 1.19 |
| | | 1 | 2090 | 2812 | 0.74 | 750 | 650 | 1.15 |
| 3-1/2 | 1/4 | 1/2 | 780 | 1111 | 0.70 | 500 | 490 | 1.02 |
| | | 3/4 | 1650 | 2275 | 0.73 | 730 | 630 | 1.16 |
| | | 1 | 2420 | 3202 | 0.76 | 850 | 760 | 1.12 |
| 5-1/2 | 1/4 | 3/4 | 1650 | 2324 | 0.71 | 950 | 940 | 1.01 |
| | | 1 | 2880 | 3802 | 0.76 | 1290 | 1190 | 1.08 |
| Spruce-Pine-Fir: | | | | | | | | |
| 1-1/2 | 1/4 | 1/2 | 460 | 595 | 0.77 | 230 | 140 | 1.64 |
| | | 3/4 | 690 | 829 | 0.83 | 300 | 175 | 1.71 |
| | | 1 | 920 | 1020 | 0.90 | 370 | 210 | 1.76 |
| 3 | 1/4 | 1/2 | 700 | 1015 | 0.69 | 360 | 280 | 1.29 |
| | | 3/4 | 1220 | 1649 | 0.74 | 460 | 350 | 1.31 |
| | | 1 | 1630 | 2032 | 0.80 | 540 | 425 | 1.27 |
| 3-1/2 | 1/4 | 1/2 | 700 | 1015 | 0.69 | 410 | 325 | 1.26 |
| | | 3/4 | 1410 | 1877 | 0.75 | 520 | 410 | 1.27 |
| | | 1 | 1880 | 2378 | 0.79 | 610 | 495 | 1.23 |
| 5-1/2 | 1/4 | 3/4 | 1470 | 2129 | 0.69 | 770 | 640 | 1.20 |
| | | 1 | 2550 | 3412 | 0.75 | 900 | 775 | 1.16 |

to the average levels of previously tabulated bolt design values increased only 25 percent for steel side members. The fact that the 1991 bolt design values for the parallel to grain load cases in Table C8.2-2 average 25 percent lower than the 1986 bolt design values reflects the average effect of this difference (1-1.25/1.625 or 23 percent) in steel side member adjustment. (For additional discussion see Commentary for 8.2.1 - Comparison of 1991 and Earlier Edition Bolt Design Values).

**8.2.2.2** (See Commentary for 7.2.3)

### 8.2.3-Wood-to-Concrete Connections

A specific provision for establishing design values for a single shear connection involving a wood member attached to concrete or masonry through an embedded bolt in the latter was introduced in the 1982 edition. Such connections were assigned a bolt design value equal to one-half the tabulated bolt design value for a piece twice the thickness of the wood member. This procedure was based on the conservative assumption that the concrete or masonry was providing bearing support and fixity at least equivalent to that provided by a wood member twice the thickness of the attached wood member. The 1982 provision for connections involving concrete and masonry was continued in the 1986 edition.

The 1991 edition continues the approach established in earlier editions of considering a wood-to-concrete bolted connection equivalent to a single shear wood-to-wood bolted connection made with a main member twice the thickness of the side member. To determine the allowable bolt design value for such a connection, the yield mode equations of 8.2.1 can be entered with $t_m$ equal to twice $t_s$ and $F_{em}$ equal to $F_{es}$; or the design value from Table 8.2A or 8.2B for the applicable main and side member relative thicknesses and bolt diameter can be used.

It is the designer's responsibility to assure that the concrete or masonry has sufficient embedment and dowel bearing strength to resist loads imposed through the embedded fastener.

### 8.2.4-Load at Angle to Bolt Axis

Two member connections in which the load acts at an angle to the axis of the bolt are checked using the component of the load acting at 90° to the axis and member thicknesses equal to the length of the bolt in each member measured at the centerline of the bolt (see Specification Figure 8B). This methodology has been a provision of the Specification since the 1944 edition. Prior to the 1977 edition, the allowable bolt design values for such joints were taken as one-half the tabulated bolt design value for a member whose thickness was twice the bolt length in the thinner piece. This was changed in the 1977 and subsequent editions to the lesser of one-half the tabulated bolt design value of the thicker member or one-half the tabulated bolt design value for a piece twice the thickness of the thinner member (see Commentary for 8.2.1 - Previous Methodology for Single Shear Bolted Connections), where the length of the bolt in each member was used as the thickness of that member. The centerline of the bolt was made the reference for measuring the bolt length in each member in the 1986 edition.

In the 1991 edition, allowable bolt design values for connections in which the load acts at an angle to the bolt axis are based on the yield mode equations of 8.2.1. The lowest value of $Z$ obtained, using $t_m$ and $t_s$ equal to the length of bolt in each member, divided by the cosine of the angle of intersection of the two members is the maximum nominal design value for the bolted connection.

The adequacy of the bearing area under washers and plates to resist the component of force acting parallel to the bolt axis can be checked using tabulated compression design values perpendicular to grain, $F_{c\perp}$, adjusted as appropriate by the bearing area factor, $C_b$, (see 2.3.10); and bearing on the angled member should be evaluated using tabulated and allowable bearing design values parallel to grain, $F_g$ and $C_b F_{c\perp}$, in the interaction equation of 3.10.3.

## 8.3-DESIGN VALUES FOR DOUBLE SHEAR CONNECTIONS

### 8.3.1-Wood-to-Wood Connections

**Background** (See Commentary for 8.2.1)

**Yield Mode Equations**

The yield mode equations for three member, double shear bolted connections parallel those for two member, single shear bolted connections in 8.2.1 except that two of the modes for the latter configuration are not applicable: bolt rotation without bending, Mode II; and development of a plastic hinge in one of the side members, Mode $III_m$. The equations for the remaining modes ($I_m$, $I_s$, $III_s$ and IV) are the same as those for the single shear configuration except for three of the conversion factors, $n K_\theta$, used to relate short-term yield model bolt design values to average nominal proportional limit bolt design values tabulated in previous editions of the Specification, as shown below.

| Mode | $nK_\theta$ Single Shear | Double Shear |
|---|---|---|
| $I_m$ | $4\,K_\theta$ | $4\,K_\theta$ |
| $I_s$ | $4\,K_\theta$ | $2\,K_\theta$ |
| $III_s$ | $3.2\,K_\theta$ | $1.6\,K_\theta$ |
| IV | $3.2\,K_\theta$ | $1.6\,K_\theta$ |

The angle factor $K_\theta$ has the same values for the double shear as for the single shear case: 1.00 for parallel to the grain loading and 1.25 for perpendicular to grain loading of either the main or side member.

The four yield mode equations for double shear bolted connections are solved for $Z$ and the lowest value obtained is the allowable design value for the joint. The effects of member thickness, bolt diameter and direction of loading on the $Z$ values for each mode and the limiting value are illustrated in Example C8.3-1.

### Example C8.3-1

Yield mode design values for wood-to-wood double shear bolted connections:

Hem-fir three member connections made with 1/2 and 1 inch bolts, side member thickness of 1-1/2 inches, main member thicknesses of 1-1/2, 3 and 5-1/2 inches, and loads applied parallel and perpendicular to the grain of main and side members

$F_{em}, F_{es}$ = 4800 psi parallel
= 2550 psi perpendicular, 1/2 inch bolt
= 1800 psi perpendicular, 1 inch bolt
$F_{yb}$ = 45,000 psi

| Thickness, in. & grain direction: Main | Side | Bolt Diam. in. | Yield Mode Design Value, lbs: $ZI_m$ | $ZI_s$ | $ZIII_s$ | $ZIV$ |
|---|---|---|---|---|---|---|
| 1-1/2 // | 1-1/2 // | 1/2 | 900 | 1800 | 1100 | 1326 |
| 1-1/2 // | 1-1/2 ⊥ | 1/2 | 720 | 765 | 649 | 884 |
| 1-1/2 ⊥ | 1-1/2 // | 1/2 | 382 | 1440 | 760 | 884 |
| 3 // | 1-1/2 // | 1/2 | 1800 | 1800 | 1100 | 1326 |
| 3 // | 1-1/2 ⊥ | 1/2 | 1440 | 765 | 649 | 884 |
| 3 ⊥ | 1-1/2 // | 1/2 | 765 | 1440 | 760 | 884 |
| 3 // | 1-1/2 // | 1 | 3600 | 3600 | 3768 | 5303 |
| 3 // | 1-1/2 ⊥ | 1 | 2880 | 1080 | 2286 | 3133 |
| 3 ⊥ | 1-1/2 // | 1 | 1080 | 2880 | 2269 | 3133 |
| 5-1/2 // | 1-1/2 // | 1 | 6600 | 3600 | 3768 | 5303 |
| 5-1/2 // | 1-1/2 ⊥ | 1 | 5280 | 1080 | 2286 | 3133 |
| 5-1/2 ⊥ | 1-1/2 // | 1 | 1980 | 2880 | 2269 | 3133 |

Allowable bolt design values, $Z$, for sawn lumber and glued laminated timber double shear connections are tabulated in Tables 8.3A and 8.3B. The latter table is applicable to glued laminated timber main members and sawn lumber side members.

**Comparison of 1991 and Earlier Edition Bolt Design Values.** Differences in design values for double shear bolted connections in the 1991 and 1986 editions are illustrated for two species in Table C8.3-1. For the joint configurations compared in this table, bolt design values in the 1991 edition average 16 percent and 35 percent higher for parallel and perpendicular to grain loading, respectively, than bolt design values based on 1986 provisions. Average differences for the one configuration in which the thickness of the

**Table C8.3-1 - Comparison of 1991 and 1986 NDS Wood-to-Wood Double Shear Bolt Design Values**

| Thickness, in. Main, Wood | Side, Wood | Bolt Diam. in. | $Z_{//}$ 1991 | $Z_{//}$ 1986 | $Z_{//}$ Ratio | $Z_{m\perp}$ 1991 | $Z_{m\perp}$ 1986 | $Z_{m\perp}$ Ratio |
|---|---|---|---|---|---|---|---|---|
| Southern pine: | | | | | | | | |
| 1-1/2 | 1-1/2 | 1/2 | 1150 | 940 | 1.22 | 550 | 430 | 1.28 |
| | | 3/4 | 1730 | 1420 | 1.22 | 660 | 540 | 1.22 |
| | | 1 | 2310 | 1890 | 1.22 | 770 | 650 | 1.18 |
| 3 | 1-1/2 | 1/2 | 1320 | 1270 | 1.04 | 940 | 860 | 1.09 |
| | | 3/4 | 2550 | 2630 | 0.97 | 1330 | 1080 | 1.23 |
| | | 1 | 4310 | 3750 | 1.15 | 1530 | 1300 | 1.18 |
| 3-1/2 | 3-1/2 | 1/2 | 1500 | 1270 | 1.18 | 1040 | 980 | 1.06 |
| | | 3/4 | 3380 | 2800 | 1.21 | 1550 | 1260 | 1.23 |
| | | 1 | 5380 | 4270 | 1.26 | 1790 | 1520 | 1.18 |
| 5-1/2 | 1-1/2 | 3/4 | 2550 | 2630 | 0.97 | 1690 | 1080 | 1.56 |
| | | 1 | 4310 | 3750 | 1.15 | 2700 | 1300 | 2.08 |
| 7-1/2 | 3-1/2 | 3/4 | 3380 | 2860 | 1.18 | 2180 | 1870 | 1.17 |
| | | 1 | 5740 | 5090 | 1.13 | 3680 | 2950 | 1.25 |
| Spruce-Pine-Fir: | | | | | | | | |
| 1-1/2 | 1-1/2 | 1/2 | 880 | 680 | 1.29 | 370 | 280 | 1.32 |
| | | 3/4 | 1320 | 1020 | 1.29 | 450 | 350 | 1.29 |
| | | 1 | 1760 | 1360 | 1.29 | 530 | 420 | 1.26 |
| 3 | 1-1/2 | 1/2 | 1080 | 1160 | 0.93 | 740 | 560 | 1.32 |
| | | 3/4 | 2160 | 2030 | 1.06 | 900 | 700 | 1.29 |
| | | 1 | 3530 | 2710 | 1.30 | 1050 | 850 | 1.24 |
| 3-1/2 | 3-1/2 | 1/2 | 1310 | 1160 | 1.13 | 860 | 650 | 1.32 |
| | | 3/4 | 2950 | 2310 | 1.28 | 1050 | 820 | 1.28 |
| | | 1 | 4110 | 3170 | 1.30 | 1230 | 990 | 1.24 |
| 5-1/2 | 1-1/2 | 3/4 | 2160 | 2030 | 1.06 | 1380 | 700 | 1.97 |
| | | 1 | 3530 | 2710 | 1.30 | 1930 | 850 | 2.27 |
| 7-1/2 | 3-1/2 | 3/4 | 2950 | 2550 | 1.16 | 1820 | 1360 | 1.34 |
| | | 1 | 4660 | 4650 | 1.00 | 2630 | 1960 | 1.34 |

side members is one-half the thickness of the main members (3 inch main and 1-1/2 inch side) are 8 percent and 22 percent for parallel and perpendicular to grain loading, respectively. This main to side member thickness ratio was used to establish the conversion factors between short term yield mode bolt design values and bolt design values tabulated in previous editions of the Specification (see Commentary for 8.2.1 - Comparison of 1991 and Earlier Edition Bolt Design Values).

**Mixed Species Connections.** Where the side member species differs from the main member species, bolt design values in Tables 8.3A and 8.3B for the species with the lowest dowel bearing strength may be used (see Commentary for 8.2.1 - Mixed Species Connections).

**Loads at Angle to Grain.** When the main or side members are loaded at an angle to grain, the yield mode equations of 8.3.1 may be entered with dowel bearing strengths, $F_{e\theta}$, determined in accordance with Equation 8.2.7.

### 8.3.2-Wood-to-Metal Connections

**Background** (See Commentary for 8.2.2 - Background)

**8.3.2.1** Yield mode equations used for double shear wood-to-wood bolted connections are used for double shear bolted connections made with wood main members and metal side plates except equation 8.3-2 for mode $I_s$, which is for uniform bearing in the metal side members, is not applied. This condition is checked independently in accordance with 8.3.2.3 and 7.2.3.

A nominal bolt design value, $Z$, is calculated for each of the three applicable yield mode equations and the lowest value is selected as the design value for the bolted connection. Values of $Z$ and the limiting mode bolt design value for example joint configurations are illustrated in Example C8.3-2.

Where the wood member of a double shear wood-to-metal bolted connection is loaded at an angle to grain, a dowel bearing strength, $F_{e\theta}$, based on Equation 8.2.7 may be used.

Tabulated bolt design values for lumber and glued laminated timber double shear bolted connections made with 1/4 in. steel side plates given in Tables 8.3C and 8.3D are based on A36 steel having a dowel bearing strength of 58,000 psi, and a bolt bending yield strength of 45,000 psi.

**Example C8.3-2**

Yield mode design values for wood-to-metal double shear bolted connections:

Hem-fir double shear bolted connections made with metal side plates with 1/2 and 1 inch bolts, steel side member thickness of 1/4 inch, main member thicknesses of 1-1/2, 3 and 5-1/2 inches, and loads applied parallel and perpendicular to the grain of main and side members

$F_{em}$ = 4800 psi parallel
= 2550 psi perpendicular, 1/2 inch bolt
= 1800 psi perpendicular, 1 inch bolt
$F_{es}$ = 58,000 psi
$F_{yb}$ = 45,000 psi

| Wood Thickness, in. | Load Direction | Bolt Diam. in. | Yield Mode Design Value, lbs $ZI_m$ | $ZIII_s$ | $ZIV$ |
|---|---|---|---|---|---|
| 1-1/2 | // | 1/2 | 900 | 1412 | 1802 |
| | | 1 | 1800 | 5154 | 7208 |
| | ⊥ | 1/2 | 383 | 857 | 1070 |
| | | 1 | 540 | 2621 | 3619 |
| 3 | // | 1/2 | 1800 | 1412 | 1802 |
| | | 1 | 3600 | 5154 | 7208 |
| | ⊥ | 1/2 | 765 | 857 | 1070 |
| | | 1 | 1080 | 2621 | 3619 |
| 3-1/2 | // | 1/2 | 2100 | 1412 | 1802 |
| | | 1 | 4200 | 5154 | 7208 |
| | ⊥ | 1/2 | 892 | 857 | 1070 |
| | | 1 | 1260 | 2621 | 3619 |
| 5-1/2 | // | 1/2 | 3300 | 1412 | 1802 |
| | | 1 | 6600 | 5154 | 7208 |
| | ⊥ | 1/2 | 1402 | 857 | 1070 |
| | | 1 | 1980 | 2621 | 3619 |

**Comparison of 1991 and Earlier Edition Bolt Design Values.** Differences in bolt design values for lumber-to-steel double shear connections between the 1991 and the 1986 editions are illustrated in Table C8.3-2. For the joint configurations compared, the 1991 parallel to grain bolt design values for double shear joints made with steel side plates average 23 percent lower than those based on provisions in the 1986 edition. This difference is a result of the procedure used to convert short-term yield mode bolt design values to the level of previous edition bolt design values wherein a metal side plate adjustment of 1.25 rather than from 1.75 to 1.50 as provided in the 1986 edition (see Commentary for 8.2.2.1 - Comparison of 1991 and Earlier Edition Bolt Design Values). Also, for the joint configurations compared, the 1991 perpen-

**Table C8.3-2 - Comparison of 1991 and 1986 NDS Wood-to-Metal Double Shear Bolt Design Values**

| Thickness, in. Main, Wood | Thickness, in. Side, Steel | Bolt Diam. in. | $Z_{\parallel}$ 1991 | $Z_{\parallel}$ 1986 | $Z_{\parallel}$ Ratio | $Z_{\perp}$ 1991 | $Z_{\perp}$ 1986 | $Z_{\perp}$ Ratio |
|---|---|---|---|---|---|---|---|---|
| Southern pine: | | | | | | | | |
| 1-1/2 | 1/4 | 1/2 | 1150 | 1645 | 0.70 | 550 | 430 | 1.28 |
| | | 3/4 | 1730 | 2308 | 0.75 | 660 | 540 | 1.22 |
| | | 1 | 2310 | 2835 | 0.81 | 770 | 650 | 1.18 |
| 3 | 1/4 | 1/2 | 1570 | 2222 | 0.71 | 1000 | 860 | 1.16 |
| | | 3/4 | 3300 | 4274 | 0.77 | 1330 | 1080 | 1.23 |
| | | 1 | 4610 | 5625 | 0.82 | 1530 | 1300 | 1.18 |
| 3-1/2 | 1/4 | 1/2 | 1570 | 2222 | 0.71 | 1000 | 980 | 1.02 |
| | | 3/4 | 3300 | 4550 | 0.73 | 1550 | 1260 | 1.23 |
| | | 1 | 5380 | 6405 | 0.84 | 1790 | 1520 | 1.18 |
| 5-1/2 | 1/4 | 3/4 | 3300 | 4648 | 0.71 | 1910 | 1880 | 1.02 |
| | | 1 | 5750 | 7605 | 0.76 | 2810 | 2380 | 1.18 |
| Spruce-Pine-Fir: | | | | | | | | |
| 1-1/2 | 1/4 | 1/2 | 880 | 1190 | 0.74 | 370 | 280 | 1.32 |
| | | 3/4 | 1320 | 1658 | 0.80 | 450 | 350 | 1.29 |
| | | 1 | 1760 | 2040 | 0.86 | 530 | 420 | 1.26 |
| 3 | 1/4 | 1/2 | 1400 | 2030 | 0.69 | 740 | 560 | 1.32 |
| | | 3/4 | 2640 | 3299 | 0.80 | 900 | 700 | 1.29 |
| | | 1 | 3530 | 4065 | 0.87 | 1050 | 850 | 1.24 |
| 3-1/2 | 1/4 | 1/2 | 1400 | 2030 | 0.69 | 840 | 650 | 1.29 |
| | | 3/4 | 2940 | 3754 | 0.78 | 1050 | 820 | 1.28 |
| | | 1 | 4110 | 4766 | 0.86 | 1230 | 990 | 1.24 |
| 5-1/2 | 1/4 | 3/4 | 2940 | 4258 | 0.69 | 1590 | 1280 | 1.24 |
| | | 1 | 5110 | 6825 | 0.75 | 1930 | 1550 | 1.25 |

dicular to grain bolt design values average 30 percent higher than similar bolt design values based on the 1986 edition. In previous editions, no increase in perpendicular to grain bolt design values was made for use of steel side plates.

**8.3.2.2** In the 1982 and 1986 editions, double shear bolted connections made with metal main members and wood side members were assigned bolt design values equal to the tabulated parallel to grain bolt design value for a piece twice the thickness of one of the side members increased by the applicable steel plate adjustment factor. In the 1991 edition, bolt design values for such joints are determined from yield mode equations as for joints with metal side members (see 8.3.2.1) except that Equation 8.3-2 for bearing in side members replaces Equation 8.3-1 for bearing in main member. Bearing of the bolt in the metal main member is covered in the design check for metal parts (see 8.3.2.3 and 7.2.3).

Bolt design values determined from the applicable yield mode equations for several different configurations of steel main member joints are shown in Example C8.3-3.

**Example C8.3-3**

Yield mode design values for steel main member double shear bolted connections:

Hem-fir three member connections made with 1/2 and 1 inch bolts, 1/4 inch steel main member, wood side member thicknesses of 1-1/2, 3 and 5-1/2 inches, and loads applied parallel to the grain of the side members

$F_{em}$ = 58,000 psi
$F_{es}$ = 4800 psi
$F_{yb}$ = 45,000 psi

| Thickness, in. Main, Steel | Thickness, in. Side, Wood | Bolt Diam. in. | Yield Mode Design Value, lbs $ZI_s$ | $ZIII_s$ | $ZIV$ |
|---|---|---|---|---|---|
| 1/4 | 1-1/2 | 1/2 | 1800 | 1399 | 1802 |
| | | 1 | 3600 | 5114 | 7208 |
| 1/4 | 3 | 1/2 | 3600 | 2080 | 1802 |
| | | 1 | 7200 | 5597 | 7208 |
| 1/4 | 5-1/2 | 1/2 | 6600 | 3482 | 1802 |
| | | 1 | 13200 | 7804 | 7208 |

**8.3.2.3** (see Commentary for 7.2.3)

## 8.4-DESIGN VALUES FOR MULTIPLE SHEAR CONNECTIONS

### Background

Evaluating bolted connections made with four or more members of equal thickness on the basis of the sum of the allowable bolt design value for each shear plane has been a provision of the Specification since 1944. In the 1977 edition, provisions were added to cover allowable bolt design values for joints of four or more members that were not of equal thickness. Such joints were resolved into the maximum number of contiguous three-member joints and one-half the bolt design value applicable to each such joint was assigned to each shear plane in the connection. For shear planes assigned two different bolt design values, the lowest value was assigned the plane. Where the loads on each member were known, the bolt design value for any member in the joint was the sum of the bolt design values for each shear plane acting on that member. Where members in the joint carried equal loads or the loads carried by each were unknown, the design value for the bolted connection was taken as the lowest bolt design value for any shear plane times the number of shear planes. These new provisions were carried forward unchanged to the 1982 and 1986 editions.

**1991 edition.** The procedures of the current edition require evaluation of each individual shear plane using the yield mode equations of 8.2.1 or 8.2.2 and then assigning the connection a bolt design value equal to the lowest value for any single plane times the number of planes in the joint. This methodology, which encourages use of symmetrical member thicknesses, presumes that the connection load is shared in proportion to member thickness and that members are loaded in no more than two different angles to grain. Where more complex connection configurations occur, evaluation of the adequacy of the bolt design value for each individual shear plane may be required (179).

Where a multiple member connection consists of members loaded at three or more different angles to grain, the following procedures may be used to determine allowable bolt design values on individual shear planes:

(1) Determine the loads in each member or pair of members entering the connection;

(2) Number consecutively the "i" members in the connection from outside toward the center as 1, 2, 3 ...i;

(3) Enter the yield mode equations of 8.2.1 or 8.2.2 with $F_{e\theta}$ based on the angular difference ($\theta$) between the direction of the resultant force in the shear plane and the grain direction of the member, and with $\theta_{max}$ equal to the largest $\theta$ for the two members adjacent to the shear plane being considered. Calculate the applicable $\theta$ for each member adjacent to each shear plane and the allowable bolt design value for that plane by the steps below:

(i) Determine the allowable bolt design value for the plane between members 1 and 2 based on a load ($P_1$) acting in the direction of the stress in member 1, where $\theta_1$ = 0° or 90° and $\theta_2$ is the difference between the grain orientations of members 1 and 2;

(ii) Determine the resultant ($P_{1,2}$) of the forces in members 1 and 2 and the direction ($\theta_{1\text{-}2}$) this resultant is acting. Determine the allowable bolt design value for the plane between members 2 and 3 based on a load acting at $\theta_{1\text{-}2}$, where $\theta_2$ is the difference between $\theta_{1\text{-}2}$ and the grain orientation of member 2 and $\theta_3$ is the difference between $\theta_{1\text{-}2}$ and the grain orientation of member 3;

(iii) Determine the resultant ($P_{1,2,3}$) of the forces in members 1, 2 and 3 and the direction ($\theta_{1\text{-}2\text{-}3}$) this resultant is acting. Determine the allowable bolt design value for the plane between members 3 and 4 based on a load acting at $\theta_{1\text{-}2\text{-}3}$, where $\theta_3$ is the difference between $\theta_{1\text{-}2\text{-}3}$ and the grain orientation of member 3 and $\theta_4$ is the difference between $\theta_{1\text{-}2\text{-}3}$ and the grain direction of member 4;

(iv) Continue as in (iii) until the allowable bolt design value in the plane between members i-1 and i has been determined;

(v) For symmetrical joints in which members on either side of the center member are oriented in the same direction and carry equal load to the connection, only bolt design values for the shear planes between the outer member and the center member on one side of the connection need to be evaluated.

(3) Determine the adequacy of the connection design by checking the resultant loads in each shear plane ($P_1$, $P_{1,2}$, $P_{1,2,3}$ ... $P_{1,2,3,i}$) against the allowable bolt design value determined for that plane.

The foregoing procedure is illustrated in Example C8.4-1, which is adapted from an earlier reference (179).

## 8.5-PLACEMENT OF BOLTS

### 8.5.1-Terminology

**8.5.1.2** For a joint in which one member is loaded at an angle to the bolt axis (see Specification Figure 8B), end distance requirements are expressed in terms of shear area. Shear area for such a joint is defined as the triangular area in the thickness plane of the member which is enclosed between the tip of the member and the centerline of the bolt (see Figure 8B). This shear area for the angled member is compared to the shear area of a joint in which both members are loaded perpendicular to the bolt axis (members parallel to each other) and which meet end distance requirements. The equivalent shear area for the parallel member joint is the product of the required end distance and the length of the bolt in the member.

The use of equivalent shear areas to check end distance requirements in members loaded at an angle to the bolt axis was introduced in the 1986 edition. The methodology is used to check end distances for joints loaded parallel to grain in both tension (bolt bearing toward member end) and compression (bolt bearing away from member end) (see Specification and Commentary for 8.5.4.3 and Example C8.5-1).

### 8.5.2-Geometry Factor, $C_\Delta$

The geometry factor expresses the provisions in the previous edition for proportionate reduction of bolt design value for less than full end distance or less than full spacing distance given in the equation format of the 1991 edition. It should be noted that the lowest geometry factor for any bolt in a joint applies to all other bolts in that same connection, not just to the end bolt or a pair of bolts in a row. This is a continuation of the provision in the 1982 and 1986 editions which required reduction of the full bolt design value on a joint when less than full end or spacing distances were used.

The requirement of 8.5.2 that bolt design values for multiple shear plane connections or asymmetric three member connections be based on the application of the lowest geometry factor for any shear plane to all bolts in the joint presumes that members are loaded in only one or two angles to grain and that total joint capacity is proportional to the number of shear planes. For

### Example C8.4-1

Procedures to determine allowable design values for a multiple shear bolted connection:

Assume a seven member connection consisting of two compression diagonal web members (members 1 and 7), two main bottom chord members (2 and 6), two tension diagonal web members (3 and 5), and a single vertical web member (4) with total loads and orientations as shown in the member force and configuration diagram of Figure A. The magnitude and direction of the resultant total shear force on each pair of shear planes (1-2 and 6-7, 2-3 and 5-6, and 3-4 and 4-5) are shown in the load vector analysis of Figure B.

Assume all members are Southern pine 1-1/2 inch thick and that a 3/4 inch diameter bolt is being used. The angle the resultant load in each shear plane is acting relative to the grain direction of the members (from Figure B), the dowel bearing strengths applicable to each member for each shear plane (Table 8A and Equation 8.2-7 of the Specification), the allowable bolt design value for each shear plane (Equations 8.2-1 to 8.2-6) and the applied load on each plane (one-half the resultant vector loads from Figure B) are tabulated below.

| Shear Plane | Angle of Load to Member | Dowel Bearing Strength ($F_{e\theta}$), psi | $\theta_{max}$ | $K_\theta$ | Bolt Design Value, lbs | Applied Load, lbs |
|---|---|---|---|---|---|---|
| 1-2 | $\theta_1 = 0°$<br>$\theta_2 = 39°$ | 6150<br>4302 | 39° | 1.108 | 608 | 510 |
| 2-3 | $\theta_2 = 29°$<br>$\theta_3 = 7°$ | 4900<br>6052 | 29° | 1.081 | 656 | 658 |
| 3-4 | $\theta_3 = 54°$<br>$\theta_4 = 0°$ | 3597<br>6150 | 54° | 1.150 | 544 | 96 |

**Assuming a $C_D$ of 1.15 applies, a 3/4 inch bolt is adequate for all shear planes in the connection.**

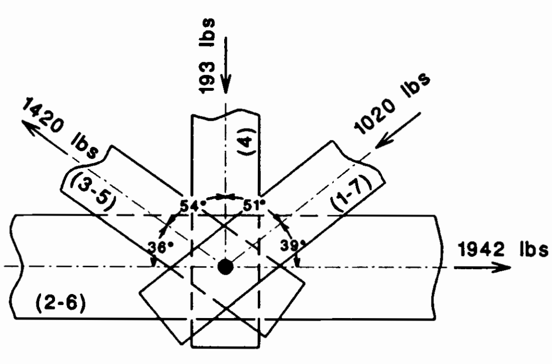



*Figure A Member Forces and Configuration*

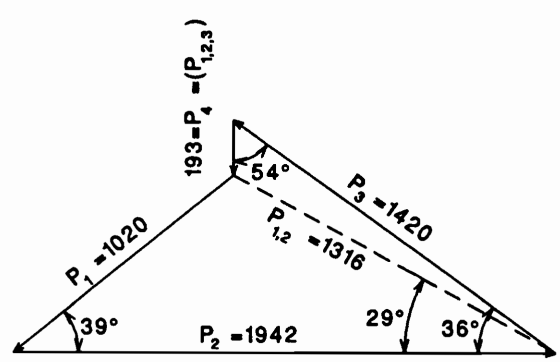



*Figure B Load Vector Analysis*

those connections in which members are loaded in three or more different angles to grain and allowable bolt design values for each shear plane are evaluated (see Commentary for 8.4), the geometry factor for each member adjoining a shear plane may be used to determine the allowable design value on that plane for an individual bolt. Although shear planes may be assigned different geometry factors, all bolts intersecting the same planes are assigned the lowest factor applicable to any bolt in that plane.

### 8.5.3-Edge Distance

**8.5.3.1** Minimum edge distance requirements in Table 8.5.3 for parallel to grain loading of $1.5D$ or the greater of $1.5D$ or 1/2 the spacing between rows for $\ell/D$ greater than 6, and for loaded edge - perpendicular to grain loading of $4D$ are based on early research (183) and have been provisions of the Specification since the 1944 edition. The unloaded edge - perpendicular to grain minimum of $1.5D$ was introduced in the 1971 edition as a good practice recommendation.

Section 8.5.3.1 does not provide specific guidance on edge distance requirements for loads applied at angles other than 0° and 90°, and on edge distance requirements for loaded edge - perpendicular to grain under less than full allowable bolt design value. Where these conditions are encountered, the following procedures, based on long standing split ring and shear plate connector design provisions (see Part 10 of the Specification), may be used.

(1) For angles of loading between 0° and 45°, minimum edge distances (loaded edge) shall be based on linear interpolation between $1.5D$ and $4D$, or

$$ED_{\perp_L} = 1.5D + (A)(2.5D/45) \qquad \text{(C8.5-1)}$$

where:

$ED_{\perp_L}$ = loaded edge - minimum edge distance
$A$ = angle of load
$D$ = bolt diameter

For angles of loading greater than 45°, a loaded edge - minimum edge distance of $4D$ shall apply.

(2) For angles of loading greater than 15°, a reduced loaded edge - minimum edge distance not less than $2D$ may be used if the bolt design value is reduced proportionately to the reduction in edge distance.

For perpendicular to grain (90°) loading, (2) provides for a loaded edge - minimum edge distance of not less than $2D$ when the full bolt design value is reduced 50 percent or more. For a load acting 30° to the grain, the required edge distance for full bolt design value from (1) is $3.2D$. This edge distance may be reduced to not less than $2D$ if the full bolt design value is reduced 37-1/2 percent or more.

**8.5.3.2** The $\ell/D$ equations for determining minimum edge distance requirements for parallel to grain loading have been added to the Specification to clarify the specific ratio being referenced. It is to be noted that the ratio of the length of bolt in side member material to bolt diameter, $\ell_s/D$, is based on the total thickness of both wood side members when connections of three or more wood members are involved. For connections involving metal main or side members, only the $\ell/D$ ratio for the wood members are considered for determination of edge distance requirements.

**8.5.3.3** Avoidance of heavy or medium suspended loads below the neutral axis of a beam was introduced as a good practice recommendation in the 1982 edition. This recommendation was added to the Specification as a result of several reported field problems involving glued laminated timber beams subject to a line of concentrated loads applied through bolted hangers or ledger strips attached in the tension zone or at the bottom edge of the beam. Concentrated loads less than 100 pounds and spaced more than 24 inches apart may be considered a light load condition.

It should be noted that any bolted connection which transmits a transverse load to a bending member is required to be checked for shear in accordance with 3.4.5 of the Specification using a reduced depth, $d_e$, equivalent to the beam depth ($d$) less the distance from the unloaded edge of the beam to the center of the nearest bolt. When a connection is within $5d$ from the end of the member, the actual shear stress based on $d_e$ is further increased by the ratio $d/d_e$.

### 8.5.4-End Distance

**8.5.4.1** End distance requirements in Table 8.5.4 for full bolt design value parallel to grain are based on early recommendations (183) and have been provisions of the Specification since the 1944 edition. For tension loads (bolts bearing toward member end), the minimum end distances for full bolt design value of $7D$ for softwoods and $5D$ for hardwoods were established by test. For compression loads (bolt bearing away from member end), the minimum end distance for full bolt design value of $4D$ was based on the minimum spacing for full design value for bolts in a row (183). The limit for full bolt design value for perpendicular to grain loading of $4D$ was introduced in 1962. Earlier editions depended on the requirements for checking shear at connections (3.4.5) to provide for appropriate joint designs for perpendicular loading. Minimum end distances for reduced bolt design values, limited to one-half those for full bolt design values, were introduced in the 1982 edition to provide for design flexibility.

Special note should be made of the requirements of 3.4.5 when bolted connections produce perpendicular to grain loading in bending members.

End distances for angle to grain tension loadings may be linearly interpolated from those for perpendicular to grain and tension parallel to grain tabulated bolt design values.

**8.5.4.2** The provisions introduced in 1982 for use of reduced end distances for bolted connections when proportionate reductions (geometry factors) are made in design values are supported by early research (57,62,-183) which showed a linear relationship between end distance and joint proportional limit strength. A subsequent study showed that a minimum end distance of only $5D$ was sufficient to develop the full proportional limit load of Douglas-fir joints made with metal side plates and loaded in tension parallel to grain (162). Other recent research further substantiates the adequacy of the end distance requirements for bolted joints loaded in both compression and tension parallel to grain (144,151).

Reduced end distances less than 50 percent of those required for full bolt design value (geometry factors less than 0.50) are not allowed. The reduced end distance provisions for bolted connections are similar to those that have been used with timber connectors since 1944.

**8.5.4.3** For members loaded at an angle to the bolt axis, end distance requirements are expressed in terms of equivalent shear areas (see Commentary for 8.5.1.2). As with end distance requirements for parallel member connections, reduced shear areas less than 50 percent of those required for full bolt design value are not allowed. It is recommended as good practice that the distance between the bolt axis and the inside juncture of the angled side member and the main member (see Figure 8B of the Specification) be at least $1.5D$. Example C8.5-1 illustrates the use of end and edge distance requirements.

### 8.5.5-Spacing Between Bolts in a Row

**8.5.5.1** The minimum spacing requirement of $4D$ for bolts in a row loaded at full design value parallel to grain has been a requirement of the Specification since 1944. This requirement is sufficient to cover the effects of nonuniform distribution of shear stresses through the thickness of the member (concentrated at the edges) that occur as the bolt bends (183). The practice of basing spacing requirements for joint members loaded perpendicular to grain on the requirements for the other attached members also has been a provision since 1944.

Use of reduced spacing between bolts in a row in proportion to the ratio of the applied load to the bolt design value for the attached members has been recognized since 1944. The lower spacing limit for reduced design value of $3D$ for both parallel and perpendicular to grain loading was introduced in 1982.

**8.5.5.2** Reduced spacings less than 75 percent of those required for the full design value (geometry factors less than 0.75) are not permitted.

### 8.5.6-Spacing Between Rows of Bolts

**8.5.6.1** Minimum distances between rows of bolts for perpendicular to grain loading in Table 8.5.6 are based on early research (183) and have been provisions of the Specification since 1944. These requirements relate the tendency of the bolt to bend and cause nonuniform bearing stresses and the resistance of the wood between rows to resist splitting. It is for this reason that staggering of bolts loaded perpendicular to grain is desirable (see 8.5.7.2 of Specification).

The minimum spacing between bolt rows for parallel to grain loading of $1.5D$ was added to the Specification in 1971 as a good practice recommendation. Prior to the revision, this distance was considered sufficiently controlled by net section requirements.

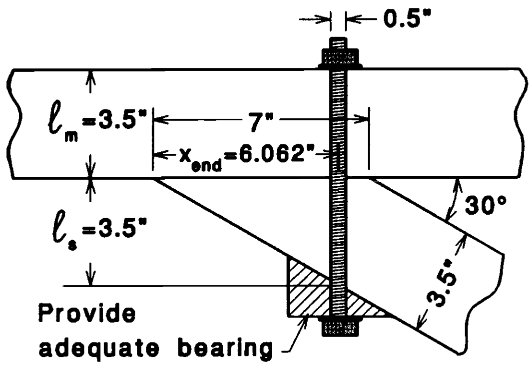
**8.5.6.2** In computing the $\ell/D$ ratio for determining the appropriate minimum spacing between rows for perpendicular to grain loading, the ratio for side members is based on the combined thickness ($\ell$) of both side members where three or more wood member joints are involved (see Commentary for 8.5.3.2).

**8.5.6.3** In the 1960 edition, a maximum limit of 5 inches between rows of bolts paralleling the member was introduced. The provision was made more restrictive in the 1991 edition by limiting the maximum distance between outer rows of bolts on the same splice plate to 5 inches. Although connections with greater distances between the outer rows of bolts have been used successfully in the past, the new criterion has been added as a good practice recommendation to avoid splitting that could occur in members at connections as a result of restraint of shrinkage associated with drying in service.

The limitation on row spacing applies to metal as well as wood side plates, to members loaded perpendic-

**Example C8.5-1**

A No. 3 Hem-Fir 2x4 tension web intersects a No. 2 Hem-Fir 2x4 chord at a 30° angle, connected by a single 1/2-in. bolt as shown. Check edge and end distance requirements in the web and check the adequacy of the web to resist the resultant tension force based on the allowable bolt design value. Assume $C_D$ = 1.0. Also assume that the bolt is centered on the members 1.5" dimension.




**Check Edge Distance Requirements** (8.5.3)

For parallel to grain loading

minimum edge distance: $= 1.5D = 1.5(0.5)$

$= 0.75$ in. $= 0.75$ in. **ok**

**Check End Distance Requirements** (8.5.4)

For the loaded end of the web use an equivalent shear area for a parallel member with $\ell_s$ = 3.5 in.

minimum end distance (tension):

for full design value $= 7D$

minimum shear area:

for full design value $= (7D)(\ell_s)$

$= (7)(0.5)(3.5) = 12.25$ in$^2$

for reduced design value $= (1/2)(12.25) = 6.125$ in$^2$

actual shear area: $= (1/2)(x_{end})(\ell_s)$

where $x_{end} = \ell_s / \tan \alpha$

$= (1/2)(3.5/\tan 30°)(3.5)$

$= 10.61$ in$^2$ < 12.25 in$^2$ **ng**

> 6.125 in$^2$ **ok**

Since the actual shear area is between the minimums for reduced and full design values, the geometry factor, $C_\Delta$, must be calculated:

$$C_\Delta = \frac{\textit{actual shear area}}{\textit{minimum shear area for full design}} \qquad (8.5.4.3)$$

$= 10.61/12.25 = 0.866$

**Allowable Bolt Design Value**

For a 1/2-in. bolt in single shear with $t_m = t_s$ = 3.5 in. and Hem-Fir lumber:

$Z_{//}$ = 660 lb/bolt (Table 8.2A)

$Z' = Z_{//}C_DC_gC_\Delta = (660)(1.0)(1.0)(0.866)$ (7.3.1)

= 572 lb (load acts perpendicular to bolt)

**Tension in Web Based on Allowable Bolt Design Value** (3.8.1)

resultant tension force $= Z'/\cos \alpha$

$= (572)/(\cos 30°) = 660$ lb

Check net section at bolt (critical)

$A_{net} = (3.5)(1.5 - (1/2 + 1/16)) = 3.28$ in$^2$

For No. 3 Hem-Fir 2x4:

$F_t$ = 300 psi $C_F$ = 1.5

$F_t' = F_tC_DC_F = (300)(1.0)(1.5) = 450$ psi (2.3.1)

$f_t = P/A_{net} = 660/3.28 = 201$ psi < $F_t'$ = 450 psi **ok**

**Bolted web connection satisfies NDS edge and end distance and strength criteria**

ular as well as parallel to grain, and to three or more member connections occurring at truss panel points.

### 8.5.7-Multiple Bolts

8.5.7.1 It is to be noted that whenever a connection involves two or more bolts in a row, the group action factor, $C_g$, is applied to the design value for a single bolt in accordance with 7.2.2 and 7.3.6 when establishing the total allowable design value on the bolt group.

8.5.7.2 (See Commentary for 8.5.6.1.)

8.5.7.3 The difficulty of prescribing general rules for the placement and spacing of bolts that would cover all directions of bolt loading and number of members in the connection was specifically noted in the 1986 and all earlier editions. For such connections, the Specification imposed the requirement that the gravity axis of all members pass through the center of resistance of the bolts in the connection if uniform stress in main members and uniform distribution of load to all bolts is to be assumed. This criterion is continued in the 1991 edition. If it is not possible to achieve intersection of member gravity axes with the center of resistance of the bolt group, the designer has the responsibility to fully evaluate and account for the effects of the resultant eccentric loading on both the load carrying capacity of the members and the capacity of the connection (see Specification and Commentary for 3.1.3).