PART VII: MECHANICAL CONNECTIONS

7.1-GENERAL

7.1.1-Scope

7.1.1.1 The individual fasteners in a connection should generally be of the same size to assure comparable load-slip or stiffness characteristics. Such equivalency is required to obtain appropriate distribution of load among fasteners in the connection and is a condition for use of the group action factor, C_g, of 7.3.6.

It is recognized that some designers have used different fastener types in the same connection where the addition of one more fasteners of the type being used is precluded by area restrictions or is considered uneconomical. Such mixed-type connections, for example the use of a single 1/2-inch bolt with three split-ring connectors or the use of a 16*d* nail with two 1/2-inch bolts, are not covered by the design provisions of the Specification. Because of the different load-slip behavior of different fastener types, the allowable load on such connections can not be assumed to be the sum of the allowable loads for each fastener type, even when the different types are in different rows.

Allowable loads for connections employing more than one type or size of fastener shall be based on analyses that account for different connection stiffnesses, on test results or on field experience (see Commentary for 1.1.1.4). It is the designer's responsibility to assure that load capacities assigned to such connections contain adequate margins of safety and are achievable under field conditions.

7.1.1.3 (See Commentary for 3.1.3, 3.1.4 and 3.1.5.)

7.1.1.4 The adequacy of alternate methods or procedures for designing and verifying the strength of connections which provide allowable loads that differ from those in the Specification is the responsibility of the designer. This responsibility includes providing for appropriate margins of safety; assuring the applicability of load duration, wet service and other adjustment factors in the Specification; and confirming the applicability of test results to field fabrication and service conditions (see Commentary for 1.1.1.4).

7.1.2-Stresses in Members at Connections

All connection designs should be checked for conformance of structural members to the net section area requirements of 3.1.2 and the shear design provisions of 3.4.5 (see Commentary for these sections). All single shear or lapped joints also should be checked to determine the adequacy of the member to resist the additional stresses induced by the eccentric transfer of load at the joint (see 3.1.3 provision). This often will involve bending and compression or bending and tension interaction where the bending moment induced by the eccentric load at the joint results in bending about the weak axis of the member. Example C7.1-1 illustrates consideration of these provisions.

Example C7.1-1

Design a bolted connection to join two 2×4's to carry an axial tension live load of 2000 lb.

1. **Try a lapped joint using No. 2 Southern Pine and 1/2 in. bolts**

F_b = 1500 psi E = 1,600,000 psi (Table 4B)
F_t = 825 psi C_D = 1.0 C_F = 1.0 C_{fu} = 1.1

Bolt Design

For 1/2-in. bolts in single shear,

$Z_{//}$ = 530 lb/bolt (Table 8.2A)
Number of bolts = P/Z = 2000/530 = 3.8 bolts

Try four 1/2-in. bolts in a single row at 2 in. on-center

With n = 4, $E_m = E_s$ = 1,600,000 psi, $A_m = A_s$ = 5.25 in^2 and s = 2 in.

C_g = 0.985 (Eq. 7.3-1)
$Z' = Z_{//} C_D C_g C_{\blacktriangle}$ = (530)(1.0)(0.985)(1.0) (7.3.1)
= 522 lb
P = 2000 lb < nZ' = (4)(522) = 2088 lb **ok**

Net Section (3.1.2)

Allow for an additional 1/16 in. per bolt hole (8.1.2.1)
A_{net} = (1.5)(3.5 – (1/2 + 1/16)) = 4.41 in^2
S_{net} = (3.5 – (1/2 + 1/16))(1.5)^2/6 = 1.10 in^3

Tension (3.8.1)

$F_t' = F_t C_D C_F$ = (825)(1.0)(1.0) = 825 psi (2.3.1)
$f_t = P/A_{net}$ = 2000/4.41 = 454 psi < F_t' = 825 psi **ok**

(cont.)

Example C7.1-1 (cont.)

Bending (3.3)

Bending is induced about the weak axis due to the eccentricity of the lapped connection of 1.5 in.

$F_b^* = F_bC_DC_FC_{fu} = (1500)(1.0)(1.0)(1.1) = 1650$ psi(3.9.1)

Since $d < b$ (1.5 < 3.5 in.), $C_L = 1.0$ (3.3.3.1)

$F_b' = F_b^{**} = F_bC_DC_LC_FC_{fu}$ (3.9.1)

$= (1500)(1.0)(1.0)(1.0)(1.1) = 1650$ psi

$f_b = M/S_{net} = Pe/S_{net} = (2000)(1.5)/(1.10)$

$= 2727$ psi $> F_b' = 1650$ psi ng

Connection is not adequate for bending, try another design

2. **Try a lapped joint using Select Structural Southern Pine and 1/2 in. bolts**

$F_b = 2850$ psi $E = 1{,}800{,}000$ psi (Table 4B)

$F_t = 1600$ psi $C_D = 1.0$ $C_F = 1.0$ $C_{fu} = 1.1$

Bolt Design

As before, $Z_{//} = 530$ lb/bolt (Table 8.2A)

Try four 1/2-in. bolts in a single row at 2 in. on-center

With $n = 4$, $E_m = E_s = 1{,}800{,}000$ psi, $A_m = A_s = 5.25$ in^2 and $s = 2$ in.

$C_g = 0.987$ (Eq. 7.3-1)

$Z' = Z_{//}C_DC_gC_\Delta = (530)(1.0)(0.987)(1.0)$ (7.3.1)

$= 523$ lb

$P = 2000$ lb $< nZ' = (4)(523) = 2092$ lb ok

Tension (3.8.1)

$F_t' = F_tC_DC_F = (1600)(1.0)(1.0) = 1600$ psi (2.3.1)

$f_t = P/A_{net} = 2000/4.41 = 454$ psi $< F_t' = 1600$ psi ok

Bending (3.3)

Eccentricity = 1.5 in.

$F_b^* = F_bC_DC_FC_{fu} = (2850)(1.0)(1.0)(1.1) = 3135$ psi(3.9.1)

$F_b' = F_b^{**} = F_bC_DC_LC_FC_{fu}$ (3.9.1)

$= (2850)(1.0)(1.0)(1.0)(1.1) = 3135$ psi

$f_b = M/S_{net} = Pe/S_{net} = (2000)(1.5)/(1.10)$

$= 2727$ psi $< F_b' = 3135$ psi ok

Combined Bending and Axial Tension (3.9.1)

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} = \frac{454}{1600} + \frac{2727}{3135} = 1.15 > 1.0 \quad \text{ng}$$

$$\frac{f_b - f_t}{F_b^{**}} = \frac{2727-454}{3135} = 0.73 < 1.0 \quad \text{ok}$$

Connection is not adequate for combined bending and tension, try another design

3. **To reduce the eccentricity of the connection, try a spliced joint using Select Structural Southern Pine with a single 5/16-in. steel plate and 1/2 in. bolts**

Bolt Design

For 1/2-in. bolts in single shear with 5/16-in. side plate,

$Z_{//} = 613$ lb/bolt (Eq. 8.2-3)

Number of bolts = $P/Z = 2000/613 = 3.3$ bolts

Try four 1/2-in. bolts in a row on each side of splice at 2 in. on-center

With $n = 4$, $E_m = 1{,}800{,}000$ psi, $E_s = 30{,}000{,}000$ psi, $A_m = 5.25$ in^2, $A_s = 1.094$ in^2 and $s = 2$ in.

$C_g = 0.949$ (Eq. 7.3-1)

$Z' = Z_{//}C_DC_gC_\Delta = (613)(1.0)(0.949)(1.0)$ (7.3.1)

$= 582$ lb

$P = 2000$ lb $< nZ' = (4)(582) = 2328$ lb ok

Tension (3.8.1)

$F_t' = F_tC_DC_F = (1600)(1.0)(1.0) = 1600$ psi (2.3.1)

$f_t = P/A_{net} = 2000/4.41 = 454$ psi $< F_t' = 1600$ psi ok

Bending (3.3)

There is still bending induced about the weak axis; however the eccentricity is reduced to 1/2(1.5)+1/2(5/16) = 0.906in.

$F_b^* = F_bC_DC_FC_{fu} = (2850)(1.0)(1.0)(1.1) = 3135$ psi(3.9.1)

$F_b' = F_b^{**} = F_bC_DC_LC_FC_{fu}$ (3.9.1)

$= (2850)(1.0)(1.0)(1.0)(1.1) = 3135$ psi

$f_b = M/S_{net} = Pe/S_{net} = (2000)(0.906)/(1.10)$

$= 1648$ psi $< F_b' = 3135$ psi ok

Combined Bending and Axial Tension (3.9.1)

$$\frac{f_t}{F_t'} + \frac{f_b}{F_b^*} = \frac{454}{1600} + \frac{1648}{3135} = 0.81 < 1.0 \quad \text{ok}$$

$$\frac{f_b - f_t}{F_b^{**}} = \frac{1648-454}{3135} = 0.38 < 1.0 \quad \text{ok}$$

This connection is adequate, but requires 8 bolts. Try a double plate/double shear spliced connection to eliminate the eccentricity, reduce the number of bolts and allow for the use of a lower grade of lumber.

4. **Try a spliced joint using No. 3 Southern Pine with two 1/8-in. steel plates and 1/2 in. bolts**

$F_b = 850$ psi $E = 1{,}400{,}000$ psi (Table 4B)

$F_t = 475$ psi $C_D = 1.0$ $C_F = 1.0$ $C_{fu} = 1.1$

(cont.)

Example C7.1-1 (cont.)

Bolt Design

For 1/2-in. bolts in double shear with 1/8-in. side plates,

$Z_{//} = 1153$ lb/bolt (Eq. 8.3-1)

Number of bolts = $P/Z = 2000/1153 = 1.73$ bolts

Try two 1/2-in. bolts in a row on each side of splice

With $n = 2$, $E_m = 1{,}400{,}000$ psi, $E_s = 30{,}000{,}000$ psi

$A_m = 5.25$ in^2, $A_s = 0.875$ in^2 and $s = 2$ in.

$C_g = 0.991$ (Eq. 7.3-1)

$Z' = Z_{//}C_DC_gC_{\Delta} = (1153)(1.0)(0.991)(1.0)$ (7.3.1)

$= 1143$ lb

$P = 2000$ lb $< nZ' = (2)(1143) = 2286$ psi **ok**

Tension (3.8.1)

$F_t' = F_tC_DC_F = (475)(1.0)(1.0) = 475$ psi (2.3.1)

$f_t = P/A_{net} = 2000/4.41 = 454$ psi $< F_t' = 475$ psi **ok**

Since the eccentricity of the connection has been eliminated there is no bending. The connection design satisfies NDS provisions.

5. **Try a spliced joint using No. 3 Southern Pine with two Industrial 45 (No. 2 stresses) Southern Pine 1x4 stress rated boards as side plates and 1/2 in. bolts**

No. 3: $F_t = 475$ psi $E = 1{,}400{,}000$ psi (Table 4B)

$C_D = 1.0$ $C_F = 1.0$

Ind. 45 (No. 2): $F_t = 825$ psi $E = 1{,}600{,}000$ psi

(SPIB Standard Grading Rules for Southern Pine Lumber)

Bolt Design

For 1/2-in. bolts in double shear with two 1x4 side plates,

$Z_{//} = 1077$ lb/bolt (Eq. 8.3-3)

Number of bolts = $P/Z = 2000/1077 = 1.86$ bolts

Try two 1/2-in. bolts in a row on each side of splice

With $n = 2$, $E_m = 1{,}400{,}000$ psi, $E_s = 1{,}600{,}000$ psi

$A_m = 5.25$ in^2, $A_s = 5.25$ in^2 and $s = 2$ in.

$C_g = 0.999$ (Eq. 7.3-1)

$Z' = Z_{//}C_DC_gC_{\Delta} = (1077)(1.0)(0.999)(1.0)$ (7.3.1)

$= 1076$ lb

$P = 2000$ lb $< nZ' = (2)(1076) = 2152$ lb **ok**

Tension (3.8.1)

1x4's

$A_{net} = 2(0.75)(3.5 - (1/2 + 1/16)) = 4.41$ in^2

$F_t' = F_tC_DC_F = (825)(1.0)(1.0) = 825$ psi (2.3.1)

$f_t = P/A_{net} = 2000/4.41 = 454$ psi $< F_t' = 825$ psi **ok**

2x4's

$F_t' = F_tC_DC_F = (475)(1.0)(1.0) = 475$ psi (2.3.1)

$f_t = P/A_{net} = 2000/4.41 = 454$ psi $< F_t' = 475$ psi **ok**

No eccentricity and therefore, no bending. The connection design satisfies NDS provisions.

Connection designs 3, 4 and 5 all satisfy NDS provisions, with designs 4 or 5 probably being the most practical/economical.

7.1.3-Eccentric Connections

Avoidance of fastener eccentricity that induces tension perpendicular to grain stresses in the main wood member at the connection was first introduced as a cautionary note in the 1944 edition of the Specification. Where multiple fasteners occurred with eccentricity, fasteners were to be placed, insofar as possible, such that the wood between them was placed in compression rather than in tension (load coming into the joint through the right hand member and leaving the joint through the left hand member in Figure 7A of the Specification).

The cautionary provisions on tension perpendicular to grain stresses at eccentric connections were dropped from the Specification in 1948 when new provisions for shear design of bending members at connections were introduced. The present provision that eccentric connections that induce tension perpendicular to grain stresses are not to be used unless it has been shown by analysis or test that such joints can safely carry all applied loads has been a part of the Specification since the 1982 edition.

Because of building code requirements calling for design checks for uplift or other load reversals, avoidance of tension perpendicular to grain stresses in configurations such as that shown in Figure 7A often is not possible. An alternative to this detail is to lap both web members on the same fastener axis or, where monoplane webs are required, to use steel straps attached to the ends of the webs to carry the loads to and from the chord member through the same bolt or pin.

It is to be emphasized that no tension design values perpendicular to grain are given in the Specification (see Commentary for 3.8). This is because undetectable ring shake and checking and splitting that may occur as a result of drying in service make it impractical to establish reliable, generally applicable design values for the property.

The determination of the type and extent of the analysis and/or testing required to demonstrate the adequacy of eccentric connections that induce tension perpendicular to grain stresses in the wood members is the responsibility of the designer. Use of stitch bolts or plates to resist such stresses when they can not be avoided is a common practice.

7.2-DESIGN VALUES

7.2.1-Single Fastener Connections

Previous Basis. Design values for connections in the 1986 and earlier editions of the Specification were based on generalized relationships established from tests of the various types of fasteners. These relationships used compression parallel or perpendicular to the grain strength, or specific gravity, which is relatively closely correlated with clear wood compression properties, as the measure of the influence of wood quality on connection load-carrying capacity. Adjustment of these basic wood properties for fastener diameter, length and placement was based on the results of joint tests.

1991 Edition. Lateral load design values for dowel type fasteners (bolts, lag screws, wood screws, nails and spikes) are based on a yield limit model which specifically accounts for the different ways these connections can behave under load. These behavior patterns or modes (see Appendix I of the Specification) are uniform bearing in the wood under the fastener, rotation of the fastener in the joint without bending, and development of one or more plastic hinges in the fastener (93,167). Equations have been developed for each mode relating the joint load to the maximum stresses in the wood members and in the fastener (93,166). The capacity of the connection under each yield mode is keyed to the bearing strength of the wood under the fastener and the bending strength of the fastener, with the lowest capacity calculated for the various modes being taken as the design load for the connection.

The yield limit model provides a consistent basis for establishing the relative effects of side and main member thickness and bearing strength, and fastener bending strength on the load-carrying capacity of connections involving dowel type fasteners. Because the yield strength of a wood connection is not well defined on the load-deformation curve for a connection, the limiting wood stresses used in the yield model are based on the load at which the load-deformation curve from a fastener embedment test intersects a line represented by the initial tangent modulus offset 5 percent of the fastener diameter (163). This nominal yield point is intermediate between the proportional limit and maximum loads for the material and for the connection. Figure C7.2-1 graphically illustrates a typical load-deformation curve from a fastener embedment test.

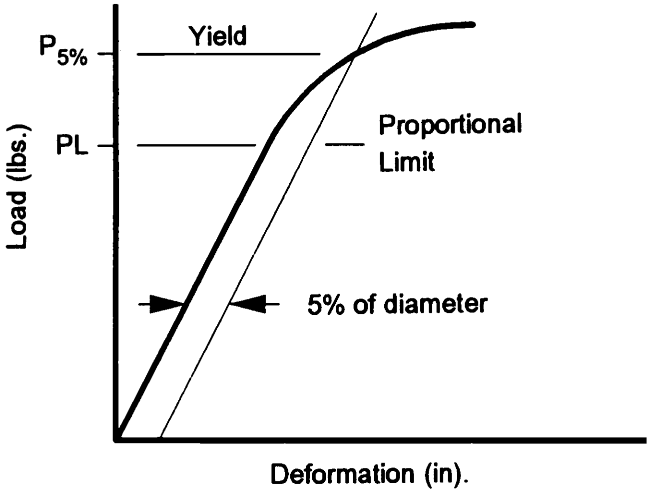

Figure C7.2-1 Typical load-deformation curve to determine bending yield design value for dowel type fasteners.

Lateral design loads for connections in previous editions of the Specification represented nominal proportional limit values. For purposes of transition and to build on the long record of satisfactory performance obtained with these previous values, short-term loads based on direct application of the yield limit equations have been reduced to the nominal average design load levels published in previous editions for connections made with equivalent species and member sizes. This was done by establishing average ratios of previous Specification loads to yield model loads for each mode of failure and direction of loading (parallel and perpendicular to grain). As noted under the commentaries for specific fastener types, this soft conversion procedure based on average design load levels results in some new design loads for each fastener type being higher and some lower than previous values depending upon the fastener diameter and the thickness of main and side member.

7.2.2-Multiple Fastener Connections

The allowable design value for a connection containing two or more fasteners is obtained by summing the allowable loads for each individual fastener. It is to be understood that this provision requires application of the group action factor of 7.3.6 to the individual fastener design value wherever a row of two or more split ring connectors, shear plate connectors, bolts or lag screws are involved.

Summation of individual fastener design values to obtain a total design value for a connection containing two or more fasteners is limited to designs involving the same type and the same size of fastener (see Commentary for 7.1.1.1). Fasteners of the same type, diameter and length joining the same members and resisting load in the same shear plane may be assumed to exhibit the same yield mode.

7.2.3-Design of Metal Parts

Metal parts, including fasteners, are to be designed in accordance with national standards of practice and specifications applicable to the material. Tension stresses in fasteners as a result of withdrawal loads, shear in cross-sections of fasteners, bearing of fasteners on metal side plates, tension and shear of plates, and buckling of plates and rods are included under this provision.

Standard metal design practices are not to be used to account for bending stresses occurring in dowel type fasteners in wood connections subject to lateral loads. These stresses are accounted for in this Specification under the provisions for the particular fastener type involved. In all cases where the design value for a connection involving metal fasteners is based on the provisions of the Specification, the adjustment factors of 7.3 are to be applied.

Where the capacity of the connection is controlled by the strength of the metal fastener or part, the adjustment factors of 7.3 are not to be applied. In these cases, the design for such metal fasteners and parts are not to be increased 1/3 for wind or earthquake loadings if the design load on the connection or part has been reduced for load combinations in accordance with the applicable building code or national standard (10). Load combinations in which the probability of simultaneous occurrence is reflected in a reduced total design load generally will include dead load plus live load plus wind load or dead load plus live load plus earthquake load (see Commentary 2.3.2.3).

7.3-ADJUSTMENT OF DESIGN VALUES

7.3.1-Applicability of Adjustment Factors

Table 7.3.1 indicates what adjustment factors apply to connector design values based on the type of load on the connection: Z, P and Q refer to lateral loads; W refers to withdrawal loads.

Design values for all fastener types are adjusted for load duration, wet service and temperature except values for toe-nails loaded in withdrawal are not modified for wet service. Adjustments for multiple fasteners in a row, C_g, apply only to laterally loaded bolts, lag screws, shear plates, split rings, drift pins and drift bolts. The geometry factors, C_Δ, refer to end and edge distance and spacing requirements for these same fasteners and for laterally loaded spike grids. Diaphragm and toe-nail adjustment factors are limited to nail and spikes only.

Specific design provisions for drift bolts, drift pins and spike grids are not given in the Specification. Other authoritative sources for the design of connections with these fastener types should be consulted (see Part XIV of the Specification).

The metal side plate adjustment factor, C_{st}, cited in the footnote of Table 7.3.1 refers to the modification of design values for shear plate connectors when metal rather than wood side plates are used (see 10.2.4 of Specification).

7.3.2-Load Duration Factor, C_D

The impact load duration factor of 2.0 is not to be applied to design loads for connections. This new limitation is a result of the use of yield model equations to establish the capacities of laterally loaded connections made with dowel type fasteners. These equations take into account the bending yield strength of the metal fasteners, a property which influences the design load of the connection in many configurations. As load duration adjustments for wood properties are not applicable to metal properties, increases in connector lateral design loads where impact loading conditions occur is not appropriate. Lateral connector loads based on the procedures of the Specification may be increased for other load durations, including the 1.6 modification for wind and earthquake loads, because the reduction factors used to adjust yield model values (10 minute duration) to normal load design levels include a 1/1.6 component.

Extension of the 1.6 maximum duration of load adjustment limit to connections made with non-dowel type fasteners and to those where the fastener is subject to withdrawal loads has been made for purposes of uniformity and in recognition that design loads for these other connections also are derived from the results of standard short-term tests (5-10 minute duration) rather than impact tests.

7.3.3-Wet Service Factor, C_M

Applications representing dry conditions of use are discussed in the Commentary for 2.3.3.

The wet service factors in Table 7.3.3 for bolts and lag screws, split ring and shear plate connectors, wood screws and common nails have been provisions of the Specification since the 1944 edition. The factor for threaded, hardened nails was added in 1962. These adjustments were recommended as part of the early research on wood connections (57,62).

The factors for metal connector plates were added in 1968. The 0.80 factor for plates installed in partially seasoned or wet lumber is based on the results of both truss and tension in-line joint tests (1,150,195). The factors for drift pins were added in 1977.

The factor of 0.40 in the footnote of Table 7.3.3 for multiple rows of bolts or lag screws installed in partially seasoned wood used in dry conditions of service has been a provision of the Specification since 1948. In earlier editions, this factor was 1/3. The adjustment is based on limited tests of connections fabricated with unseasoned members joined at right angles to each other and tested after drying (62).

7.3.4-Temperature Factor, C_t

The temperature adjustment factors for connections in Table 7.3.4 are equivalent to those for bending, compression and shear design values in 2.3.4 (see Commentary for this section). Bearing under metal fasteners is closely correlated with compression parallel to grain or compression perpendicular to grain properties.

7.3.5-Fire Retardant Treatment

(See Commentary for 2.3.6.)

7.3.6-Group Action Factor, C_g

Background

Modification factors for two or more split ring connectors, shear plate connectors, bolts or lag screws in a row were added to the Specification in the 1973 edition. Earlier tests of bolted and shear plate connector joints had shown that the load capacity of connections containing multiple fasteners in a row was not directly proportional to the number of fasteners, with those located near the ends of the row carrying a greater proportion of the applied load than those located in the interior of the row (46,48,50,92,100).

The tables of factors included in the 1973 edition to account for the nonuniform loads on a row of fasteners was based on a linear analysis wherein the direct stresses in the main and side members of the connection were assumed to be uniformly distributed across their cross section, and the relationship between fastener slip and fastener load was assumed to be linear (103). This analytical procedure showed that the transfer of load from side to main members and the proportion of the total load carried by each fastener were determined by the modulii of elasticity (E) and cross sectional areas of the side and main members, the number of fasteners in a row, the spacing between fasteners, and the joint load/slip modulus.

Two tables of modification factors for joints containing two or more fasteners in a row were developed using the linear analysis: one for connections with wood side plates and one for connections with metal side plates. For purposes of simplicity, factors were tabulated only in terms of the number of fasteners in the row and the cross sectional areas of the members being joined. Other variables were assumed to have the following values (201):

Wood to wood connections:

E of side and main members	1,800,000 psi
Load/slip fastener modulus	220,000 lb/in.
Spacing between fasteners	6.5 inches

Wood to metal connections:

E of main member	1,400,000 psi
Load/slip fastener modulus	330,000 lb/in.
Spacing between fasteners	5.75 inches

With the foregoing constant values, the analytical procedure was used to calculate modification factors for 3 to 8 fasteners in a row and then results were extrapolated up to 12 fasteners and down to 2 fasteners in a row (201). The resulting tables of factors, ranging from 1.00 for two fasteners in a row to as low as 0.34 and 0.15 for 12 fasteners in a row in joints made with wood and metal side plates respectively, were continued essentially unchanged through the 1986 edition.

1991 Edition. The group action factor equation given in 7.3.6 is a newly developed consolidated expression for the analytical procedure used to establish the modification factors given in previous editions (234). Concurrent with the development of the compact single equation for accounting for group action, more recent load-slip data for bolted joints and split ring and shear plate connectors have been used to establish new representative load-slip modulii for different types of connections (234). These new joint stiffness parameters are:

4-inch split ring or shear plate connectors	500,000 lb/in.
2.5-inch split ring or 2.625-inch shear plate connectors	400,000 lb/in.
Bolts or lag screws:	
wood to wood connection	180,000 ($D^{1.5}$)
wood to metal connections	270,000 ($D^{1.5}$)

where: D = diameter, inches

The foregoing modulii for 4-inch connectors and 1-inch diameter bolts or lag screws were used to develop the group action factors given in Tables 7.3.6A-7.3.6D. Factors for connections involving wood side plates (Tables A and B) were developed assuming an E of 1,400,000 psi for both main and side members and a spacing of 4 inches for bolts or lag screws (Table A) and 9 inches for connectors (Table B). The effects of assuming different properties and spacings than those used previously to develop tabulated group action factors for connections with wood side plates is illustrated by the selected comparisons shown in Table C7.3-1.

Table C7.3-1 - Comparison of Group Action Factors for Connections made with Wood Side Plates

Basis		Previous editions	1991 edition
E_s, E_m		1,800,000	1,400,000
Load/slip modulus:			
1-inch bolts or lag screws		220,000	180,000
4-inch connectors		220,000	500,000
Spacing:			
1-inch bolts or lag screws		6.5	4.0
4-inch connectors		6.5	9.0
Group Action Factor			
1-inch bolts or lag screws:			
$A_s = 5$, $A_m = 10$:	$n = 4$	0.78	0.84
	$n = 12$	0.32	0.38
$A_s = 64$, $A_m = 64$:	$n = 4$	0.99	0.99
	$n = 12$	0.82	0.88
4-inch connectors:			
$A_s = 5$, $A_m = 10$:	$n = 4$	0.78	0.59
	$n = 12$	0.32	0.20
$A_s = 64$, $A_m = 64$:	$n = 4$	0.99	0.95
	$n = 12$	0.82	0.58

The foregoing comparisons show that the group action factors for wood side plate connections tabulated in the 1991 edition for 1-inch bolts and lag screws (Table 7.3.6A) are slightly larger than those applicable to these connections in previous editions. For smaller diameter bolts and smaller spacings, the differences would be larger. Alternatively, group action factors tabulated in the 1991 edition for 4-inch split ring or shear plate connectors (Table 7.3.6B) are significantly lower than those applicable to these connections in previous editions. This is primarily a result of the larger load/slip modulus assigned to this size connector relative to the average modulus assigned all connectors in previous editions. Differences between group action factors tabulated in the 1991 edition and previous editions would be less for connections made with smaller split ring or shear plate connectors and smaller spacings.

Differences between tabulated group action factors in the 1991 edition and previous editions for connections made with metal side plates are similar to those for connections made with wood side plates. This is illustrated by the comparisons shown in Table C7.3-2.

Effect of Joint Properties. As indicated in the footnotes to Tables 7.3.6A and 7.3.6B, group action

Table C7.3-2 - Comparison of Group Action Factors for Connections made with Steel Side Plates

Basis		Previous editions	1991 edition
E_m		1,400,000	1,400,000
E_s		30,000,000	30,000,000
Load/slip modulus:			
1-inch bolts or lag screws		330,000	270,000
4-inch connectors		330,000	500,000
Spacing:			
1-inch bolts or lag screws		5.75	4.0
4-inch connectors		5.75	9.0
Group Action Factor			
1-inch bolts or lag screws:			
$A_s = 2$, $A_m = 24$:	$n = 4$	0.90	0.94
	$n = 12$	0.49	0.59
$A_s = 5$, $A_m = 120$:	$n = 4$	0.98	0.99
	$n = 12$	0.81	0.88
4-inch connectors:			
$A_s = 2$, $A_m = 24$:	$n = 4$	0.90	0.82
	$n = 12$	0.49	0.35
$A_s = 5$, $A_m = 120$:	$n = 4$	0.98	0.96
	$n = 12$	0.81	0.66

factors for connections made with members having E values greater than 1,400,000 psi, spacings less than 4 inches, fasteners less than 1-inch in diameter and connectors less than 4-inch in diameter will be higher than the tabulated factors. The sensitivity of the group action factor to changes in these variables is shown in Table C7.3-3.

As shown in the Table C7.3-3, changes in E of the joint members, spacing and connector size result in changes to group action factors of less than 20 percent. These tables may be used to help determine when the general equation of 7.3.6.1 should be used rather than Tables 7.3.6A - 7.3.6D to assign group action factors for specific designs.

Table C7.3-3 - Effect of Joint Properties on Group Action Factors

Wood Side Plates: $A_m = 10$, $A_s = 5$

			Group Action Factor			
No. of bolts	$E \times 10^6$		1-inch bolt load/slip = 180,000		1/2-inch bolt load/slip = 63,640	
	main	side	$s = 4$	$s = 6$	$s = 4$	$s = 2$
5	1.8	1.8	0.79	0.73	0.90	0.95
	1.8	1.4	0.73	0.67	0.87	0.93
	1.4	1.4	0.75	0.69	0.88	0.94
	1.0	1.0	0.70	0.64	0.85	0.91
10	1.8	1.8	0.49	0.43	0.68	0.80
	1.8	1.4	0.43	0.37	0.62	0.74
	1.4	1.4	0.45	0.39	0.64	0.76
	1.0	1.0	0.40	0.35	0.57	0.70

Steel Side Plates: $A_m = 24$, $A_s = 2$

			Group Action Factor			
No. of bolts	$E \times 10^6$		1-inch bolt load/slip = 270,000		1/2-inch bolt load/slip = 95,459	
	main	side	$s = 4$	$s = 6$	$s = 4$	$s = 2$
5	1.8	30.0	0.93	0.90	0.97	0.99
	1.4	30.0	0.90	0.86	0.96	0.98
	1.0	30.0	0.85	0.80	0.94	0.97
10	1.8	30.0	0.74	0.67	0.87	0.93
	1.4	30.0	0.67	0.60	0.83	0.91
	1.0	30.0	0.58	0.51	0.77	0.86

It is to be noted that the variable A_s in the group action equation (7.3-1) represents the sum of the cross-sectional area of the side members. Thus the equation accounts for single shear as well as double shear connections. For a connection with four or more members, each shear plane is evaluated as a single shear connection (see 8.4). Where such a connection contains two or more fasteners in a row, a group action factor is calculated for each shear plane using an A_s based on the thinnest member adjacent to the plane being considered.

The modulus of elasticity values to be used with equation 7.3-1, and which are the basis for the factors in Tables 7.3.6A - 7.3.6D, are the design E values given in Tables 4 and 5 of the Supplement and Table 6A.

Perpendicular to Grain Loading. Connections involving perpendicular to grain loading, such as joints between web and chord members, generally do not involve rows of bolts containing large numbers of fasteners in a row perpendicular to grain. Similarly, large beams are generally supported on hangers rather than on stacked fasteners aligned perpendicular to grain in order to avoid splitting that can occur as a result of drying in low relative humidity service conditions.

Group action factors are limited by the maximum loads on the end fasteners in a row without any adjustment for the redistribution of load to other more lightly loaded fasteners in the row that is known to occur as a result of yielding under load. Such redistribution is considered to be significant where fasteners load the member perpendicular to the grain because of the relatively low stiffness of wood in this direction.

Based on the infrequent use of more than two bolts or other fasteners in a row perpendicular to grain, and the redistribution of load that occurs between fasteners in such connections, it is standard practice to use the same group action factor for rows of fasteners aligned perpendicular to grain as that for fasteners aligned parallel to grain. This procedure, which has been satisfactorily used since 1973 when the group action factor was first introduced, is continued in the current Specification.

7.3.6.2 The criteria for determining when staggered fasteners are considered to represent a single row have been part of the Specification since 1977.

7.3.6.3 The use of gross section areas and the definition of cross-sectional area for fastener groups loaded perpendicular to the grain were introduced in the 1973 edition at the time group action modifications were added to the Specification.