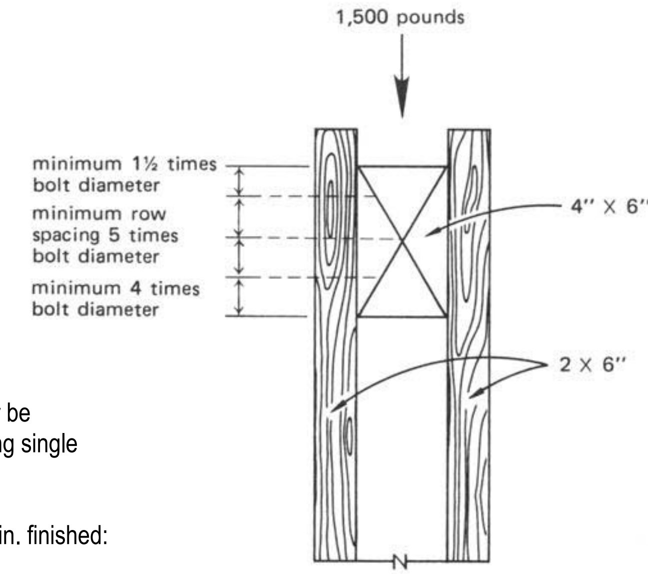
Examples:
Connections and Tension Members

Example 1

A nominal 4 x 6 in. redwood beam is to be supported by two 2 x 6 in. members acting as a spaced column. The minimum spacing and edge distances for the ½ inch bolts are shown. How many ½ in. bolts will be required to safely carry a load of 1500 lb? Use the chart provided.



SOLUTION:
The table requires that the length of the bolt in the main wood member be known, along with the diameter of bolt in inches, and if the bolt is seeing single shear or double shear and what direction it is bearing on the grain.

The main member is the beam. The 4 in. nominal size is actually 3 ½ in. finished:

The bolt is ½ inches in diameter, and sees **two** planes of shear at the interfaces with the 2 x 6's. This means double shear.

The vertical force is pushing the beam down onto the bolt, so the bolt is in contact with the grain running horizontally. That means the bolt is bearing **perpendicular** to the grain, and we should look up *q*.

The allowable load per bolt multiplied by the number of bolts will determine the capacity, which we need to be at least 1500 lb:

$q \times n \geq P$

knowing q & P, the equation for n becomes:

$n \geq \frac{P}{q} = \frac{1500lb}{980\,{}^{lb}\!/_{bolt}} = 1.5$ bolt *rounded up* = 2 bolts required

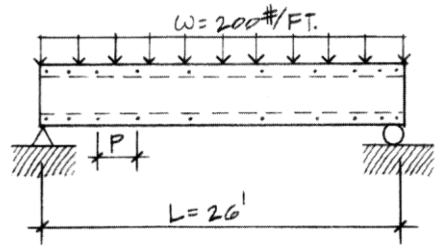
Table: Holding Power of Bolts

p = Safe loads parallel to grain in pounds
q = Safe loads perpendicular to grain in pounds

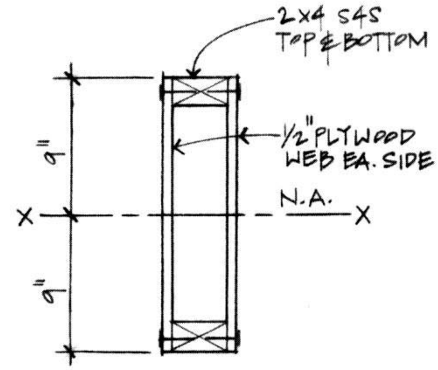
Length of Bolt in Main Wood Member[3] (in inches)		DIAMETER OF BOLT (IN INCHES) 3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 1/2
1½	Single *p*	325	470	590	710	830	945			
	Shear *q*	185	215	245	270	300	325			
	Double *p*	650	940	1180	1420	1660	1890			
	Shear *q*	370	430	490	540	600	650			
2½	Single *p*		630	910	1155	1370	1575			
	Shear *q*		360	405	450	495	540			
	Double *p*	710	1260	1820	2310	2740	3150			
	Shear *q*	620	720	810	900	990	1080			
3½	Single *p*			990	1400	1790	2135	2455	2740	3305
	Shear *q*			565	630	695	760	825	895	1020
	Double *p*	710	1270	1980	2800	3580	4270	4910	5480	6610
	Shear *q*	640	980	1130	1260	1390	1520	1650	1780	2040

[1]Tabulated values are on a normal load-duration basis and apply to joints made of seasoned lumber used in dry locations. See U.B.C. Standard No. 25-17 for other service conditions.
[2]Double shear values are for joints consisting of three wood members in which the side members are one half the thickness of the main member. Single shear values are for joints consisting of two wood members having a minimum thickness not less than that specified.
[3]The length specified is the length of the bolt in the main member of double shear joints or the length of the bolt in the thinner member of single shear joints.
[4]See U.B.C. Standard No. 25-17 for wood-to-metal bolted joints.

Example 2



8.11 A built-up plywood box beam with 2 × 4 S4S top and bottom flanges is held together by nails. Determine the pitch (spacing) of the nails if the beam supports a uniform load of 200 #/ft. along the 26-foot span. Assume the nails have a shear capacity of 80# each.



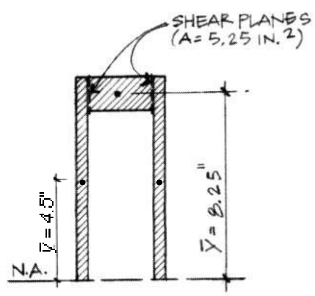
Solution:

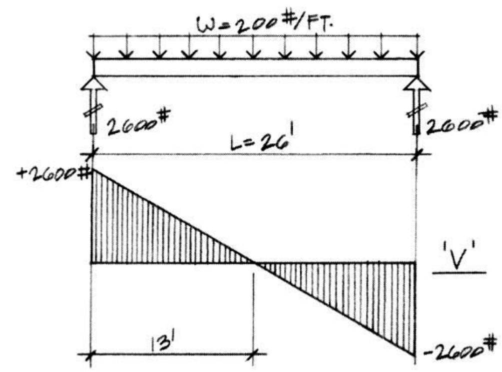
Construct the shear (V) diagram to obtain the critical shear condition and its location

Note that the condition of shear is critical at the supports, and the shear intensity decreases as you approach the center line of the beam. This would indicate that the nail spacing P varies from the support to midspan. Nails are closely spaced at the support, but increasing spacing occurs toward midspan, following the shear diagram.

$$f_v = \frac{VQ}{Ib}$$

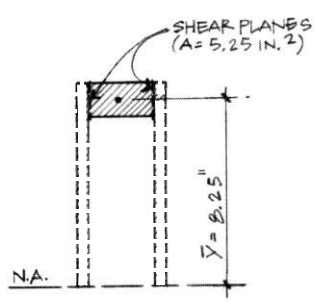
$$I_x = \frac{(4.5'')(18'')^3}{12} - \frac{(3.5'')(15'')^3}{12} = 1{,}202.6 \text{ in.}^4$$





$Q = \Sigma A\bar{y}$ = (9")(½")(4.5")+(9")(½")(4.5")+(1.5")(3.5")(8.25") = 83.8 in^3

$$f_{v-\max} = \frac{(2{,}600\#)(83.3in.^3)}{(1{,}202.6in.^4)(\frac{1}{2}"+\frac{1}{2}")} = 180.2\,psi$$



$Q = A\bar{y} = (5.25 \text{ in.}^2)(8.25'') = 43.3 \text{ in.}^3$

Shear force $= f_v \times A_v$

where:

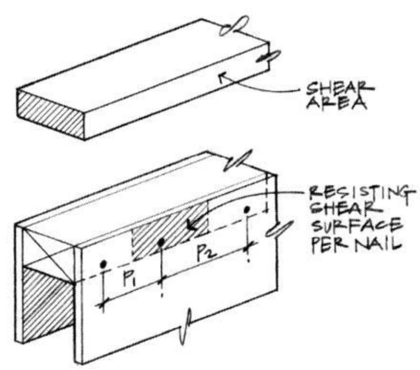
A_v = shear area

Assume:

F = Capacity of two nails (one each side) at the flange; representing two shear surfaces

$$(n)F \geq f_v \times b \times p = \frac{VQ}{Ib} \times bp$$

$$\therefore\ (n)F \geq p \times \frac{VQ}{I}; \qquad p \leq \frac{(n)FI}{VQ}$$

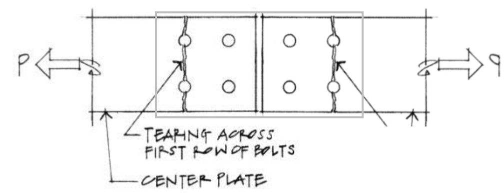


At the maximum shear location (support) where V = 2,600#

$$p = \frac{(2 \text{ nails} \times 80 \text{ \#/nail})(1{,}202.6 \text{ in.}^4)}{(2{,}600\#)(43.3 \text{ in.}^3)} = 1.71''$$

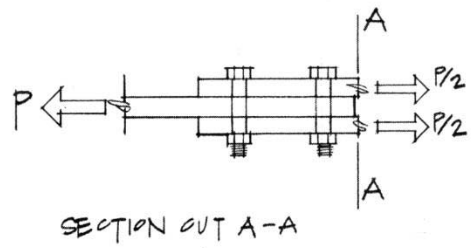
Example 3

10.2 The butt splice shown in Figure 10.22 uses two 8 × ⅜" plates to "sandwich" in the 8 × ½" plates being joined. Four ⅞"ϕ A325-SC bolts are used on both sides of the splice. Assuming A36 steel and standard round holes, determine the allowable capacity of the connection.



SOLUTION:

Shear, bearing and net tension will be checked to determine the critical conditions that governs the capacity of the connection. (The edge distance to the holes is presumed to be adequate.)



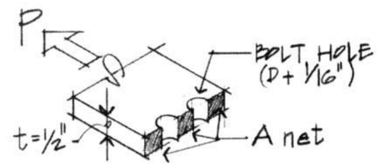
Shear: Using the AISC available shear in Table 7-3 (Group A):

ϕR_n = 26.4 k/bolt x 4 bolts = 105.6 k

Bearing: Using the AISC available bearing in Table 7-4:

There are 4 bolts bearing on the center (1/2") plate, while there are 4 bolts bearing on a total width of two sandwich plates (3/4" total). The thinner bearing width will govern. Assume 3 in. spacing (center to center) of bolts. For A36 steel, F_u = 58 ksi.

ϕR_n = 91.4 k/bolt/in. x 0.5 in. x 4 bolts = 182.8 k



Tension: The center plate is critical, again, because its thickness is less than the combined thicknesses of the two outer plates. We must consider tension yielding and tension rupture:

$\phi R_n = \phi F_y A_g$ *and* $\phi R_n = \phi F_u A_e$ where $A_e = A_{net} U$

A_g = 8 in. x ½ in. = 4 in^2

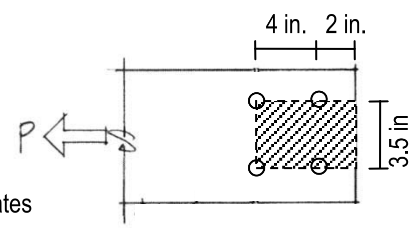
The holes are considered 1/8 in. larger than the bolt hole diameter = (7/8 + 1/8) = 1.0 in.

A_n = (8 in. – 2 holes x 1.0 in.) x ½ in. = 3.0 in^2

The whole cross section sees tension, so the shear lag factor U = 1

$\phi F_y A_g$ = 0.9 x 36 ksi x 4 in^2 = 129.6 k

$\phi F_u A_e$ = 0.75 x 58 ksi x (1) x 3.0 in^2 = 130.5 k



Block Shear Rupture: It is possible for the center plate to rip away from the sandwich plates leaving the block (shown hatched) behind:

$\phi R_n = \phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt}) \leq \phi(0.6F_y A_{gv} + U_{bs} F_u A_{nt})$

where A_{nv} is the area resisting shear, A_{nt} is the area resisting tension, A_{gv} is the gross area resisting shear, and U_{bs} = 1 when the tensile stress is uniform.

A_{gv} = (4 + 2 in.) x ½ in. = 3 in^2

$A_{nv} = A_{gv}$ – 1 ½ holes area = 3 in^2 – 1.5 x 1 in. x ½ in. = 2.25 in^2

A_{nt} = 3.5 in. x t – 1 holes = 3.5 in. x ½ in – 1 x 1 in. x ½ in. = 1.25 in^2

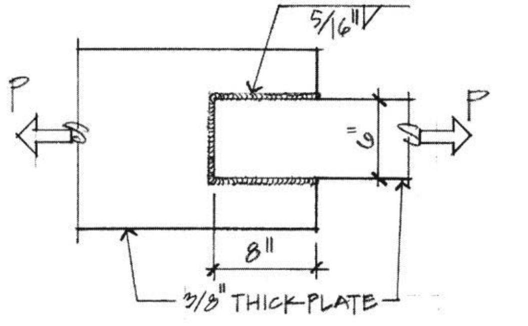
$\phi(0.6F_u A_{nv} + U_{bs} F_u A_{nt})$ = 0.75 x (0.6 x 58 ksi x 2.25 in^2 + 1 x 58 ksi x 1.25 in^2) = 113.1 k

$\phi(0.6F_y A_{gv} + U_{bs} F_u A_{nt})$ = 0.75 x (0.6 x 36 ksi x 3 in^2 + 1 x 58 ksi x 1.25 in^2) = 103.0 k

The maximum connection capacity (*smallest value*) is governed by block shear rupture.

ϕR_n = 103.0 k

Example 4



10.7 Determine the capacity of the connection in Figure 10.44 assuming A36 steel with E70XX electrodes.

Solution:

Capacity of weld:

For a 5⁄16″ fillet weld, $\phi S = 6.96$ k/in

Weld length = 8 in + 6 in + 8 in = 22 in.

Weld capacity = 22″ × 6.96 k/in = 153.1 k

Capacity of plate: 0.9 x 36 k/in² x 3/8″ x 6″= 72.9 k

$\phi P_n = \phi F_y A_g \quad \phi = 0.9$

Plate capacity = 0.9 x 36 k/in² x 3/8″ x 6″= 72.9 k

∴ Plate capacity governs, $P_u = 72.9$ k

The weld size used is obviously too strong. What size, then, can the weld be reduced to so that the weld strength is more compatible to the plate capacity? To make the weld capacity ≈ plate capacity:

22″ × (weld capacity per in.) = 72.9 k

$$\text{Weld capacity per inch} = \frac{72.9\text{ k}}{22\text{ in.}} = 3.31\text{ k/in.}$$

From Available Strength table, use 3/16″ weld ($\phi S = 4.18$ k/in.)

Minimum size fillet = 3⁄16″ based on a 3⁄8″ thick plate.

Available Strength of Fillet Welds per inch of weld (ϕS)

Weld Size (in.)	E60XX (k/in.)	E70XX (k/in.)
3⁄16	3.58	4.18
¼	4.77	5.57
5⁄16	5.97	6.96
3⁄8	7.16	8.35
7⁄16	8.35	9.74
½	9.55	11.14
5⁄8	11.93	13.92
¾	14.32	16.70

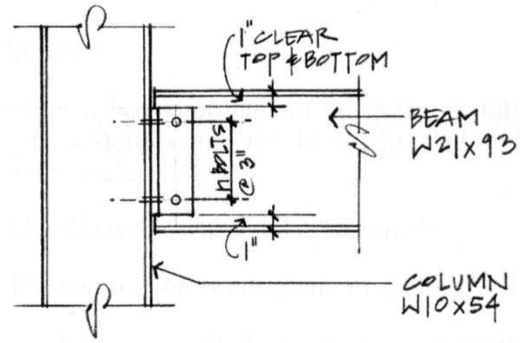
(not considering increase in throat with submerged arc weld process)

Table 7-1
Available Shear Strength of Bolts, kips

Nominal Bolt Diameter, *d*, in.					5/8		3/4		7/8		1	
Nominal Bolt Area, in.²					0.307		0.442		0.601		0.785	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	8.29	12.4	11.9	17.9	16.2	24.3	21.2	31.8
				D	16.6	24.9	23.9	35.8	32.5	48.7	42.4	63.6
	X	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
Group B	N	34.0	51.0	S	10.4	15.7	15.0	22.5	20.4	30.7	26.7	40.0
				D	20.9	31.3	30.1	45.1	40.9	61.3	53.4	80.1
	X	42.0	63.0	S	12.9	19.3	18.6	27.8	25.2	37.9	33.0	49.5
				D	25.8	38.7	37.1	55.7	50.5	75.7	65.9	98.9
A307	–	13.5	20.3	S	4.14	6.23	5.97	8.97	8.11	12.2	10.6	15.9
				D	8.29	12.5	11.9	17.9	16.2	24.4	21.2	31.9

Nominal Bolt Diameter, *d*, in.					1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.²					0.994		1.23		1.48		1.77	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Group A	N	27.0	40.5	S	26.8	40.3	33.2	49.8	40.0	59.9	47.8	71.7
				D	53.7	80.5	66.4	99.6	79.9	120	95.6	143
	X	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
Group B	N	34.0	51.0	S	33.8	50.7	41.8	62.7	50.3	75.5	60.2	90.3
				D	67.6	101	83.6	125	101	151	120	181
	X	42.0	63.0	S	41.7	62.6	51.7	77.5	62.2	93.2	74.3	112
				D	83.5	125	103	155	124	186	149	223
A307	–	13.5	20.3	S	13.4	20.2	16.6	25.0	20.0	30.0	23.9	35.9
				D	26.8	40.4	33.2	49.9	40.0	60.1	47.8	71.9
ASD	LRFD	For end loaded connections greater than 38 in., see AISC *Specification* Table J3.2 footnote b.										
Ω = 2.00	φ = 0.75											

Example 5



The steel used in the connection and beams is A992 with F_y = 50 ksi, and F_u = 65 ksi. Using A490-N bolt material, determine the maximum capacity of the connection based on shear in the bolts, bearing in all materials and pick the number of bolts and angle length (not staggered). Use A36 steel for the angles.

W21x93: d = 21.62 in, t_w = 0.58 in, t_f = 0.93 in
W10x54: t_f = 0.615 in

SOLUTION:

The maximum length the angles can be depends on how it fits between the top and bottom flange with some clearance allowed for the fillet to the flange, and getting an air wrench in to tighten the bolts. This example uses 1" of clearance:

Available length = beam depth – both flange thicknesses – 1" clearance at top & 1" at bottom

= 21.62 in – 2(0.93 in) – 2(1 in) = 17.76 in.

With the spaced at 3 in. and 1 ¼ in. end lengths (each end), the maximum number of bolts can be determined:

Available length ≥ 1.25 in. + 1.25 in. + 3 in. x (number of bolts – 1)

number of bolts ≤ (17.76 in – 2.5 in. - (-3 in.))/3 in. = 6.1, so 6 bolts.

It is helpful to have the All-bolted Double-Angle Connection Tables 10-1. They are available for ¾", 7/8", and 1" bolt diameters and list angle thicknesses of ¼", 5/16", 3/8", and ½". Increasing the angle thickness is likely to increase the angle strength, although the limit states include shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles.

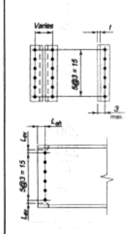
For these diameters the available **shear** (double) from Table 7-1 for 6 bolts is (6)41.5 k/bolt = 270.6 kips, (6)61.3 k/bolt = 367.8 kips, and (6)80.1 k/bolt = 480.6 kips.

Tables 10-1 (not all provided here) list a bolt and angle available strength of 271 kips for the ¾" bolts, 296 kips for the 7/8" bolts, and 281 kips for the 1" bolts. It appears that increasing the bolt diameter to 1" will not gain additional load. Use 7/8" bolts.

Beam F_y = 50 ksi, F_u = 65 ksi
Angle F_y = 36 ksi, F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections
7/8-in. Bolts

Bolt and Angle Available Strength, kips

6 Rows W40, 36, 33, 30, 27, 24, 21	Bolt Group	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	Group A	N	STD	98.6	148	123	185	148	222	195	292
		X	STD	98.6	148	123	185	148	222	197	296
		SC Class A	STD	98.6	148	106	159	106	159	106	159
			OVS	90.1	135	90.1	135	90.1	135	90.1	135
			SSLT	97.3	146	106	159	106	159	106	159
		SC Class B	STD	98.6	148	123	185	148	222	176	264
			OVS	93.5	140	117	175	140	210	150	225
			SSLT	97.3	146	122	182	146	219	176	264
	Group B	N	STD	98.6	148	123	185	148	222	197	296
		X	STD	98.6	148	123	185	148	222	197	296
		SC Class A	STD	98.6	148	123	185	133	199	133	199
			OVS	93.5	140	113	169	113	169	113	169
			SSLT	97.3	146	122	182	133	199	133	199
		SC Class B	STD	98.6	148	123	185	148	222	197	296
			OVS	93.5	140	117	175	140	210	187	281
			SSLT	97.3	146	122	182	146	219	195	292

ϕR_n = 368.7 kips for double shear of 7/8" bolts ϕR_n = 296 kips for limit state in angles

We also need to evaluate **bearing** of bolts on the beam web, and column flange where there are bolt holes. Table 7-5 provides available bearing strength for the material type, bolt diameter, hole type, and spacing per inch of material thicknesses.

a) Bearing for beam web: There are *6* bolt holes through the beam web. This is typically the critical bearing limit value because there are two angle legs that resist bolt bearing and twice as many bolt holes to the column. The material is A992 (F_u = 65 ksi), 0.58" thick, with 7/8" bolt diameters at 3 in. spacing.

$$\phi R_n = 6 \text{ bolts}\cdot(102 \text{ k/bolt/inch})\cdot(0.58 \text{ in}) = 355.0 \text{ kips}$$

b) Bearing for column flange: There are 12 bolt holes through the column. The material is A992 (F_u = 65 ksi), 0.615" thick, with 1" bolt diameters.

$$\phi R_n = 12 \text{ bolts}\cdot(102 \text{ k/bolt/inch})\cdot(0.615 \text{ in}) = 752.8 \text{ kips}$$

Although, the bearing in the beam web is the smallest at 355 kips, with the shear on the bolts even smaller at 324.6 kips, $\boxed{\textit{the maximum capacity for the simple-shear connector is 296 kips}}$ limited by the critical capacity of the angles.

Group A Bolts

A325, A325M
F1858
A354 Grade BC
A449

Table 7-3
Slip-Critical Connections
Available Shear Strength, kips
(Class A Faying Surface, μ = 0.30)

Group A Bolts

Hole Type	Loading	Nominal Bolt Diameter, d, in.							
		5/8		3/4		7/8		1	
		Minimum Group A Bolt Pretension, kips							
		19		28		39		51	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	4.29	6.44	6.33	9.49	8.81	13.2	11.5	17.3
	D	8.59	12.9	12.7	19.0	17.6	26.4	23.1	34.6
OVS/SSLP	S	3.66	5.47	5.39	8.07	7.51	11.2	9.82	14.7
	D	7.32	10.9	10.8	16.1	15.0	22.5	19.6	29.4
LSL	S	3.01	4.51	4.44	6.64	6.18	9.25	8.08	12.1
	D	6.02	9.02	8.87	13.3	12.4	18.5	16.2	24.2

Hole Type	Loading	Nominal Bolt Diameter, d, in.							
		1 1/8		1 1/4		1 3/8		1 1/2	
		Minimum Group A Bolt Pretension, kips							
		56		71		85		103	
		r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD/SSLT	S	12.7	19.0	16.0	24.1	19.2	28.8	23.3	34.9
	D	25.3	38.0	32.1	48.1	38.4	57.6	46.6	69.8
OVS/SSLP	S	10.8	16.1	13.7	20.5	16.4	24.5	19.8	29.7
	D	21.6	32.3	27.4	40.9	32.7	49.0	39.7	59.4
LSL	S	8.87	13.3	11.2	16.8	13.5	20.2	16.3	24.4
	D	17.7	26.6	22.5	33.7	26.9	40.3	32.6	48.9

STD = standard hole
OVS = oversized hole
SSLT = short-slotted hole transverse to the line of force
SSLP = short-slotted hole parallel to the line of force
LSL = long-slotted hole transverse or parallel to the line of force

S = single shear
D = double shear

Hole Type	ASD	LRFD
STD and SSLT	Ω = 1.50	ϕ = 1.00
OVS and SSLP	Ω = 1.76	ϕ = 0.85
LSL	Ω = 2.14	ϕ = 0.70

Note: Slip-critical bolt values assume no more than one filler has been provided or bolts have been added to distribute loads in the fillers.
See AISC *Specification* Sections J3.8 and J5 for provisions when fillers are present.
For Class B faying surfaces, multiply the tabulated available strength by 1.67.

Table 7-4
Available Bearing Strength at Bolt Holes Based on Bolt Spacing
kips/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	Nominal Bolt Diameter, d, in.							
			5/8		3/4		7/8		1	
			r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	2 2/3 d_b	58	34.1	51.1	41.3	62.0	48.6	72.9	55.8	83.7
		65	38.2	57.3	46.3	69.5	54.4	81.7	62.6	93.8
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	67.4	101
		65	48.8	73.1	58.5	87.8	68.3	102	75.6	113
SSLP	2 2/3 d_b	58	27.6	41.3	34.8	52.2	42.1	63.1	47.1	70.7
		65	30.9	46.3	39.0	58.5	47.1	70.7	52.8	79.2
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	58.7	88.1
		65	48.8	73.1	58.5	87.8	68.3	102	65.8	98.7
OVS	2 2/3 d_b	58	29.7	44.6	37.0	55.5	44.2	66.3	49.3	74.0
		65	33.3	50.0	41.4	62.2	49.6	74.3	55.3	82.9
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	60.9	91.4
		65	48.8	73.1	58.5	87.8	68.3	102	68.3	102
LSLP	2 2/3 d_b	58	3.62	5.44	4.35	6.53	5.08	7.61	5.80	8.70
		65	4.06	6.09	4.88	7.31	5.69	8.53	6.50	9.75
	3 in.	58	43.5	65.3	39.2	58.7	28.3	42.4	17.4	26.1
		65	48.8	73.1	43.9	65.8	31.7	47.5	19.5	29.3
LSLT	2 2/3 d_b	58	28.4	42.6	34.4	51.7	40.5	60.7	46.5	69.8
		65	31.8	47.7	38.6	57.9	45.4	68.0	52.1	78.2
	3 in.	58	36.3	54.4	43.5	65.3	50.8	76.1	56.2	84.3
		65	40.6	60.9	48.8	73.1	56.9	85.3	63.0	94.5
STD, SSLT, SSLP, OVS, LSLP	$s \geq s_{full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$s \geq s_{full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Spacing for full bearing strength s_{full}[a], in.		STD, SSLT, LSLT	1 15/16		2 5/16		2 11/16		3 1/16	
		OVS	2 1/16		2 7/16		2 13/16		3 1/4	
		SSLP	2 1/8		2 1/2		2 7/8		3 5/16	
		LSLP	2 13/16		3 3/8		3 15/16		4 1/2	
Minimum Spacing[a] = 2 2/3d, in.			1 11/16		2		2 5/16		2 11/16	

STD = standard hole
SSLT = short-slotted hole oriented transverse to the line of force
SSLP = short-slotted hole oriented parallel to the line of force
OVS = oversized hole
LSLP = long-slotted hole oriented parallel to the line of force
LSLT = long-slotted hole oriented transverse to the line of force

ASD	LRFD
Ω = 2.00	ϕ = 0.75

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC *Specification* Section J3.10.
[a] Decimal value has been rounded to the nearest sixteenth of an inch.