Chapter J
Design of Connections

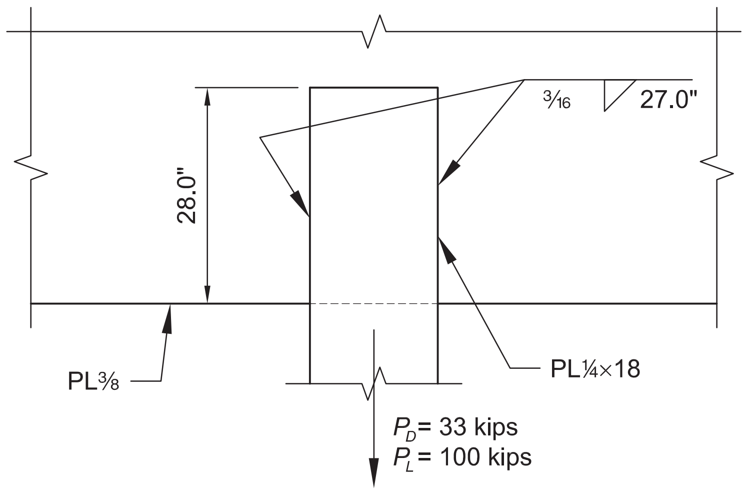
Chapter J of the AISC *Specification* addresses the design and review of connections. The chapter's primary focus is the design of welded and bolted connections. Design requirements for fillers, splices, column bases, concentrated forces, anchors rods and other threaded parts are also covered. Special requirements for connections subject to fatigue are not covered in this chapter.

EXAMPLE J.1 FILLET WELD IN LONGITUDINAL SHEAR

Given:

A ¼-in. × 18-in. wide plate is fillet welded to a ⅜-in. plate. The plates are ASTM A572 Grade 50 and have been properly sized. Use 70-ksi electrodes. Note that the plates would normally be specified as ASTM A36, but F_y = 50 ksi plate has been used here to demonstrate the requirements for long welds.

Verify the welds for the loads shown.



Solution:

From Chapter 2 of ASCE/SEI 7, the required strength is:

LRFD	ASD
P_u = 1.2(33.0 kips) + 1.6(100 kips) = 200 kips	P_a = 33.0 kips + 100 kips = 133 kips

Maximum and Minimum Weld Size

Because the thickness of the overlapping plate is ¼ in., the maximum fillet weld size that can be used without special notation per AISC *Specification* Section J2.2b, is a 3⁄16-in. fillet weld. A 3⁄16-in. fillet weld can be deposited in the flat or horizontal position in a single pass (true up to 5⁄16-in.).

From AISC *Specification* Table J2.4, the minimum size of fillet weld, based on a material thickness of ¼ in. is ⅛ in.

Length of Weld Required

The nominal weld strength per inch of 3⁄16-in. weld, determined from AISC *Specification* Section J2.4(a) is:

$$\begin{aligned} R_n &= F_{nw}A_{we} \qquad (Spec.\ \text{Eq. J2-4}) \\ &= (0.60\,F_{EXX})(A_{we}) \\ &= 0.60(70\text{ ksi})\left(\tfrac{3}{16}\text{ in.}/\sqrt{2}\right) \\ &= 5.57\text{ kips/in.} \end{aligned}$$

LRFD	ASD
$\dfrac{P_u}{\phi R_n}=\dfrac{200\text{ kips}}{0.75\left(5.57\text{ kips/in.}\right)}$ $= 47.9$ in. or 24 in. of weld on each side	$\dfrac{P_a\Omega}{R_n}=\dfrac{133\text{ kips}\left(2.00\right)}{5.57\text{ kips/in.}}$ $= 47.8$ in. or 24 in. of weld on each side

From AISC *Specification* Section J2.2b, for longitudinal fillet welds used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them.

$24\text{ in.} \geq 18\text{ in.}$ **o.k.**

From AISC *Specification* Section J2.2b, check the weld length to weld size ratio, because this is an end loaded fillet weld.

$$\frac{L}{w}=\frac{24\text{ in.}}{\tfrac{3}{16}\text{ in.}}$$

$= 128 > 100$. therefore, AISC *Specification* Equation J2-1 must be applied, and the length of weld increased, because the resulting β will reduce the available strength below the required strength.

Try a weld length of 27 in.

The new length to weld size ratio is:

$$\frac{27.0\text{ in.}}{\tfrac{3}{16}\text{ in.}}=144$$

For this ratio:

$$\beta = 1.2 - 0.002(l/w) \leq 1.0 \qquad (\textit{Spec}.\text{ Eq. J2-1})$$
$$= 1.2 - 0.002(144)$$
$$= 0.912$$

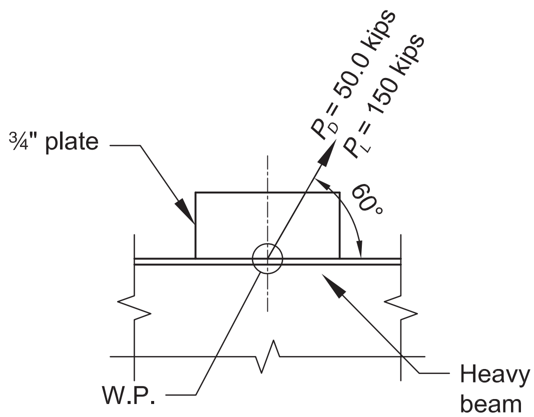
Recheck the weld at its reduced strength.

LRFD	ASD
$\phi R_n = \left(0.912\right)\left(0.75\right)\left(5.57\text{ kips/in.}\right)\left(54.0\text{ in.}\right)$ $= 206\text{ kips} > P_u = 200\text{ kips}$ **o.k.** Therefore, use 27 in. of weld on each side.	$\dfrac{R_n}{\Omega}=\dfrac{\left(0.912\right)\left(5.57\text{ kips/in.}\right)\left(54.0\text{ in.}\right)}{2.00}$ $= 137\text{ kips} > P_a = 133\text{ kips}$ **o.k.** Therefore, use 27 in. of weld on each side.

EXAMPLE J.2 FILLET WELD LOADED AT AN ANGLE

Given:

Design a fillet weld at the edge of a gusset plate to carry a force of 50.0 kips due to dead load and 150 kips due to live load, at an angle of 60° relative to the weld. Assume the beam and the gusset plate thickness and length have been properly sized. Use a 70-ksi electrode.



Solution:

From Chapter 2 of ASCE/SEI 7, the required tensile strength is:

LRFD	ASD
$P_u = 1.2(50.0\text{ kips}) + 1.6(150\text{ kips})$ $= 300\text{ kips}$	$P_a = 50.0\text{ kips} + 150\text{ kips}$ $= 200\text{ kips}$

Assume a 5⁄16-in. fillet weld is used on each side of the plate.

Note that from AISC *Specification* Table J2.4, the minimum size of fillet weld, based on a material thickness of ¾ in. is ¼ in.

Available Shear Strength of the Fillet Weld Per Inch of Length

From AISC *Specification* Section J2.4(a), the nominal strength of the fillet weld is determined as follows:

$$A_{we} = \frac{5/16\text{ in.}}{\sqrt{2}}$$
$$= 0.221\text{ in.}$$

$$F_{nw} = 0.60F_{EXX}\left(1.0 + 0.5\sin^{1.5}\theta\right) \qquad \text{(Spec. Eq. J2-5)}$$
$$= 0.60(70\text{ ksi})\left(1.0 + 0.5\sin^{1.5}60^\circ\right)$$
$$= 58.9\text{ ksi}$$

$$R_n = F_{nw}A_{we} \qquad \text{(Spec. Eq. J2-4)}$$
$$= 58.9\text{ ksi}(0.221\text{ in.})$$
$$= 13.0\text{ kip/in.}$$

From AISC *Specification* Section J2.4(a), the available shear strength per inch of weld length is:

LRFD	ASD
$\phi = 0.75$	$\Omega = 2.00$
$\phi R_n = 0.75(13.0 \text{ kip/in.}) = 9.75 \text{ kip/in.}$	$\dfrac{R_n}{\Omega} = \dfrac{13.0 \text{ kip/in.}}{2.00} = 6.50 \text{ kip/in.}$
For 2 sides:	For 2 sides:
$\phi R_n = 2(0.75)(13.0 \text{ kip/in.}) = 19.5 \text{ kip/in.}$	$\dfrac{R_n}{\Omega} = \dfrac{2(13.0 \text{ kip/in.})}{2.00} = 13.0 \text{ kip/in.}$

Required Length of Weld

LRFD	ASD
$l = \dfrac{300 \text{ kips}}{19.5 \text{ kip/in.}} = 15.4 \text{ in.}$	$l = \dfrac{200 \text{ kips}}{13.0 \text{ kip/in.}} = 15.4 \text{ in.}$
Use 16 in. on each side of the plate.	Use 16 in. on each side of the plate.

EXAMPLE J.3 COMBINED TENSION AND SHEAR IN BEARING TYPE CONNECTIONS

Given:

A ¾-in.-diameter ASTM A325-N bolt is subjected to a tension force of 3.5 kips due to dead load and 12 kips due to live load, and a shear force of 1.33 kips due to dead load and 4 kips due to live load. Check the combined stresses according to AISC *Specification* Equations J3-3a and J3-3b.

Solution:

From Chapter 2 of ASCE/SEI 7, the required tensile and shear strengths are:

LRFD	ASD
Tension: $T_u = 1.2(3.50\text{ kips}) + 1.6(12.0\text{ kips})$ $= 23.4\text{ kips}$ Shear: $V_u = 1.2(1.33\text{ kips}) + 1.6(4.00\text{ kips})$ $= 8.00\text{ kips}$	Tension: $T_a = 3.50\text{ kips} + 12.0\text{ kips}$ $= 15.5\text{ kips}$ Shear: $V_a = 1.33\text{ kips} + 4.00\text{ kips}$ $= 5.33\text{ kips}$

Available Tensile Strength

When a bolt is subject to combined tension and shear, the available tensile strength is determined according to the limit states of tension and shear rupture, from AISC *Specification* Section J3.7 as follows.

From AISC *Specification* Table J3.2,

$$F_{nt} = 90\text{ ksi},\ F_{nv} = 54\text{ ksi}$$

From AISC *Manual* Table 7-1, for a ¾-in.-diameter bolt,

$$A_b = 0.442\text{ in.}^2$$

The available shear stress is determined as follows and must equal or exceed the required shear stress.

LRFD	ASD
$\phi = 0.75$ $\phi F_{nv} = 0.75(54\text{ ksi})$ $= 40.5\text{ ksi}$ $f_{rv} = \dfrac{V_u}{A_b}$ $= \dfrac{8.00\text{ kips}}{0.442\text{ in.}^2}$ $= 18.1\text{ ksi} \le 40.5\text{ ksi}$	$\Omega = 2.00$ $\dfrac{F_{nv}}{\Omega} = \dfrac{54\text{ ksi}}{2.00}$ $= 27.0$ $f_{rv} = \dfrac{V_a}{A_b}$ $= \dfrac{5.33\text{ kips}}{0.442\text{ in.}^2}$ $= 12.1\text{ ksi} \le 27.0\text{ ksi}$ **o.k.**

The available tensile strength of a bolt subject to combined tension and shear is as follows:

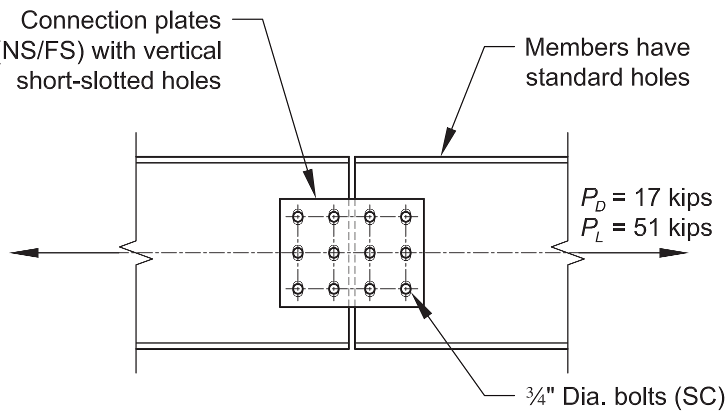
LRFD	ASD
$F'_{nt} = 1.3F_{nt} - \dfrac{F_{nt}}{\phi F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3.3a)	$F'_{nt} = 1.3F_{nt} - \dfrac{\Omega F_{nt}}{F_{nv}} f_{rv} \le F_{nt}$ (*Spec*. Eq. J3.3b)
$= 1.3(90 \text{ ksi}) - \dfrac{90 \text{ ksi}}{0.75(54 \text{ ksi})}(18.1 \text{ ksi})$	$= 1.3(90 \text{ ksi}) - \dfrac{2.00(90 \text{ ksi})}{54 \text{ ksi}}(12.1 \text{ ksi})$
$= 76.8 \text{ ksi} \le 90 \text{ ksi}$	$= 76.7 \text{ ksi} \le 90 \text{ ksi}$
$R_n = F'_{nt} A_b$ (*Spec*. Eq. J3-2)	$R_n = F'_{nt} A_b$ (*Spec*. Eq. J3-2)
$= 76.8 \text{ ksi}(0.442 \text{ in.}^2)$	$= 76.7 \text{ ksi}(0.442 \text{ in.}^2)$
$= 33.9 \text{ kips}$	$= 33.9 \text{ kips}$
For combined tension and shear, $\phi = 0.75$ from AISC *Specification* Section J3.7	For combined tension and shear, $\Omega = 2.00$ from AISC *Specification* Section J3.7
Design tensile strength:	Allowable tensile strength:
$\phi R_n = 0.75(33.9 \text{ kips})$	$\dfrac{R_n}{\Omega} = \dfrac{33.9 \text{ kips}}{2.00}$
$= 25.4 \text{ kips} > 23.4 \text{ kips}$ **o.k.**	$= 17.0 \text{ kips} > 15.5 \text{ kips}$ **o.k.**

EXAMPLE J.4A SLIP-CRITICAL CONNECTION WITH SHORT-SLOTTED HOLES

Slip-critical connections shall be designed to prevent slip and for the limit states of bearing-type connections.

Given:

Select the number of ¾-in.-diameter ASTM A325 slip-critical bolts with a Class A faying surface that are required to support the loads shown when the connection plates have short slots transverse to the load and no fillers are provided. Select the number of bolts required for slip resistance only.



Solution:

From Chapter 2 of ASCE/SEI 7, the required strength is:

LRFD	ASD
$P_u = 1.2(17.0 \text{ kips}) + 1.6(51.0 \text{ kips})$ $= 102 \text{ kips}$	$P_a = 17.0 \text{ kips} + 51.0 \text{ kips}$ $= 68.0 \text{ kips}$

From AISC *Specification* Section J3.8(a), the available slip resistance for the limit state of slip for standard size and short-slotted holes perpendicular to the direction of the load is determined as follows:

$\phi = 1.00 \qquad \Omega = 1.50$

μ = 0.30 for Class A surface
D_u = 1.13
h_f = 1.0, factor for fillers, assuming no more than one filler
T_b = 28 kips, from AISC *Specification* Table J3.1
n_s = 2, number of slip planes

$$R_n = \mu D_u h_f T_b n_s \qquad \text{(}\textit{Spec}\text{. Eq. J3-4)}$$
$$= 0.30(1.13)(1.0)(28 \text{ kips})(2)$$
$$= 19.0 \text{ kips/bolt}$$

The available slip resistance is:

LRFD	ASD
$\phi R_n = 1.00(19.0 \text{ kips/bolt})$ $= 19.0 \text{ kips/bolt}$	$\dfrac{R_n}{\Omega} = \dfrac{19.0 \text{ kips/bolt}}{1.50}$ $= 12.7 \text{ kips/bolt}$

Required Number of Bolts

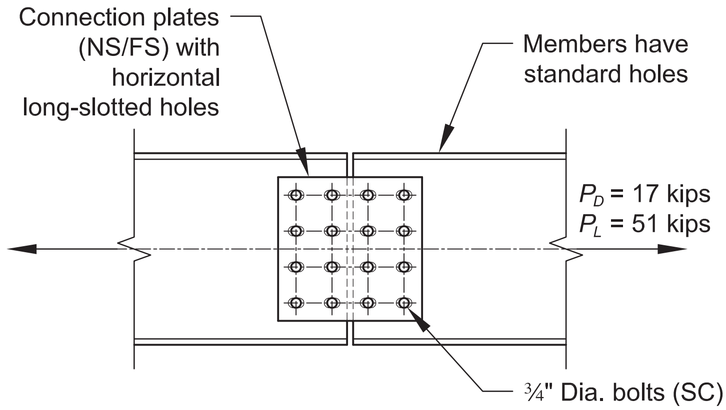
LRFD	ASD
$n_b = \dfrac{P_u}{\phi R_n}$ $= \dfrac{102 \text{ kips}}{19.0 \text{ kips/bolt}}$ $= 5.37$ bolts Use 6 bolts	$n_b = \dfrac{P_a}{\left(\dfrac{R_n}{\Omega}\right)}$ $= \dfrac{68.0 \text{ kips}}{(12.7 \text{ kips/bolt})}$ $= 5.37$ bolts Use 6 bolts

Note: To complete the design of this connection, the limit states of bolt shear, bolt bearing, tensile yielding, tensile rupture, and block shear rupture must be determined.

EXAMPLE J.4B SLIP-CRITICAL CONNECTION WITH LONG-SLOTTED HOLES

Given:

Repeat Example J.4A with the same loads, but assuming that the connected pieces have long-slotted holes in the direction of the load.



Solution:

The required strength from Example J.4A is:

LRFD	ASD
P_u = 102 kips	P_a = 68.0 kips

From AISC *Specification* Section J3.8(c), the available slip resistance for the limit state of slip for long-slotted holes is determined as follows:

$\phi = 0.70 \qquad \Omega = 2.14$

μ = 0.30 for Class A surface
D_u = 1.13
h_f = 1.0, factor for fillers, assuming no more than one filler
T_b = 28 kips, from AISC *Specification* Table J3.1
n_s = 2, number of slip planes

$$R_n = \mu D_u h_f T_b n_s \qquad \text{(Spec. Eq. J3-4)}$$
$$= 0.30(1.13)(1.0)(28 \text{ kips})(2)$$
$$= 19.0 \text{ kips/bolt}$$

The available slip resistance is:

LRFD	ASD
$\phi R_n = 0.70(19.0 \text{ kips/bolt})$ $= 13.3 \text{ kips/bolt}$	$\dfrac{R_n}{\Omega} = \dfrac{19.0 \text{ kips/bolt}}{2.14}$ $= 8.88 \text{ kips/bolt}$

Required Number of Bolts

LRFD	ASD
$n_b = \dfrac{P_u}{\phi R_n}$ $= \dfrac{102 \text{ kips}}{13.3 \text{ kips/bolt}}$ $= 7.67 \text{ bolts}$ Use 8 bolts	$n_b = \dfrac{P_a}{\left(\dfrac{R_n}{\Omega}\right)}$ $= \dfrac{68.0 \text{ kips}}{8.88 \text{ kips/bolt}}$ $= 7.66 \text{ bolts}$ Use 8 bolts

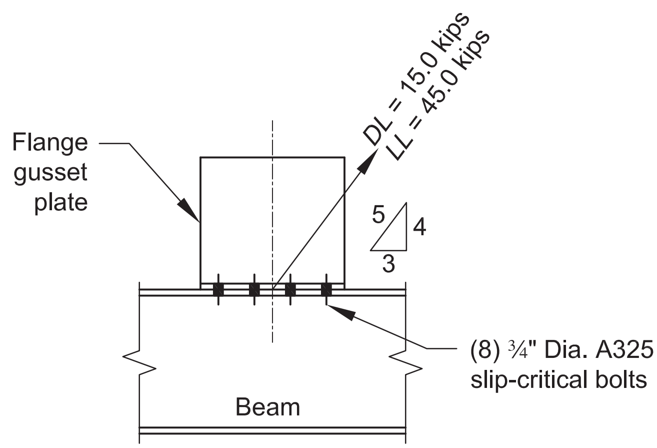
Note: To complete the design of this connection, the limit states of bolt shear, bolt bearing, tensile yielding, tensile rupture, and block shear rupture must be determined.

EXAMPLE J.5 COMBINED TENSION AND SHEAR IN A SLIP-CRITICAL CONNECTION

Because the pretension of a bolt in a slip-critical connection is used to create the clamping force that produces the shear strength of the connection, the available shear strength must be reduced for any load that produces tension in the connection.

Given:

The slip-critical bolt group shown as follows is subjected to tension and shear. Use ¾-in.-diameter ASTM A325 slip-critical Class A bolts in standard holes. This example shows the design for bolt slip resistance only, and assumes that the beams and plates are adequate to transmit the loads. Determine if the bolts are adequate.



Solution:

μ = 0.30 for Class A surface
D_u = 1.13
n_b = 8, number of bolts carrying the applied tension
h_f = 1.0, factor for fillers, assuming no more than one filler
T_b = 28 kips, from AISC *Specification* Table J3.1
n_s = 1, number of slip planes

From Chapter 2 of ASCE/SEI 7, the required strength is:

LRFD	ASD
P_u = 1.2(15.0 kips)+1.6(45.0 kips) = 90.0 kips By geometry, $T_u = \frac{4}{5}(90.0 \text{ kips}) = 72.0 \text{ kips}$ $V_u = \frac{3}{5}(90.0 \text{ kips}) = 54.0 \text{ kips}$	P_a = 15.0 kips + 45.0 kips = 60.0 kips By geometry, $T_a = \frac{4}{5}(60.0 \text{ kips}) = 48.0 \text{ kips}$ $V_a = \frac{3}{5}(60.0 \text{ kips}) = 36.0 \text{ kips}$

Available Bolt Tensile Strength

The available tensile strength is determined from AISC *Specification* Section J3.6.

From AISC *Specification* Table J3.2 for Group A bolts, the nominal tensile strength in ksi is, $F_{nt} = 90\text{ ksi}$. From AISC *Manual* Table 7-1, A_b = 0.442 in.^{2}

$$A_b = \frac{\pi(\frac{3}{4}\text{ in.})^2}{4}$$
$$= 0.442\text{ in.}^2$$

The nominal tensile strength in kips is,

$$R_n = F_{nt}A_b \qquad \text{(from } \textit{Spec}\text{. Eq. J3-1)}$$
$$= 90\text{ ksi}(0.442\text{ in.}^2)$$
$$= 39.8\text{ kips}$$

The available tensile strength is,

LRFD	ASD
$\phi R_n = 0.75\left(\frac{39.8\text{ kips}}{\text{bolt}}\right) > \frac{72.0\text{ kips}}{8\text{ bolts}}$ $= 29.9\text{ kips/bolt} > 9.00\text{ kips/bolt}$ **o.k.**	$\frac{R_n}{\Omega} = \left(\frac{39.8\text{ kips/bolt}}{2.00}\right) > \frac{48.0\text{ kips}}{8\text{ bolts}}$ $= 19.9\text{ kips/bolt} > 6.00\text{ kips/bolt}$ **o.k.**

Available Slip Resistance Per Bolt

The available slip resistance of one bolt is determined using AISC *Specification* Equation J3-4 and Section J3.8.

LRFD	ASD
Determine the available slip resistance (T_u = 0) of a bolt. $\phi = 1.00$ $\phi R_n = \phi\mu D_u h_f T_b n_s$ $= 1.00(0.30)(1.13)(1.0)(28\text{ kips})(1)$ $= 9.49\text{ kips/bolt}$	Determine the available slip resistance (T_a = 0) of a bolt. $\Omega = 1.50$ $\frac{R_n}{\Omega} = \frac{\mu D_u h_f T_b n_s}{\Omega}$ $= \frac{0.30(1.13)(1.0)(28\text{ kips})(1)}{1.50}$ $= 6.33\text{ kips/bolt}$

Available Slip Resistance of the Connection

Because the clip-critical connection is subject to combined tension and shear, the available slip resistance is multiplied by a reduction factor provided in AISC *Specification* Section J3.9.

LRFD	ASD
Slip-critical combined tension and shear coefficient: $k_{sc} = 1 - \frac{T_u}{D_u T_b n_b}$ (*Spec*. Eq. J3-5a) $= 1 - \frac{72.0\text{ kips}}{1.13(28\text{ kips})(8)}$ $= 0.716$ $\phi = 1.00$	Slip-critical combined tension and shear coefficient: $k_{sc} = 1 - \frac{1.5T_a}{D_u T_b n_b}$ (*Spec*. Eq. J3-5b) $= 1 - \frac{1.5(48.0\text{ kips})}{1.13(28\text{ kips})(8)}$ $= 0.716$ $\Omega = 1.50$

$\phi R_n = \phi R_n k_s n_b$ $= 9.49 \text{ kips/bolt}(0.716)(8 \text{ bolts})$ $= 54.4 \text{ kips} > 54.0 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = \dfrac{R_n}{\Omega} k_s n_b$ $= 6.33 \text{ kips/bolt}(0.716)(8 \text{ bolts})$ $= 36.3 \text{ kips} > 36.0 \text{ kips}$ **o.k.**

Note: The bolt group must still be checked for all applicable strength limit states for a bearing-type connection.

Chapter IIA
Simple Shear Connections

The design of simple shear connections is covered in Part 10 of the AISC *Steel Construction Manual*.

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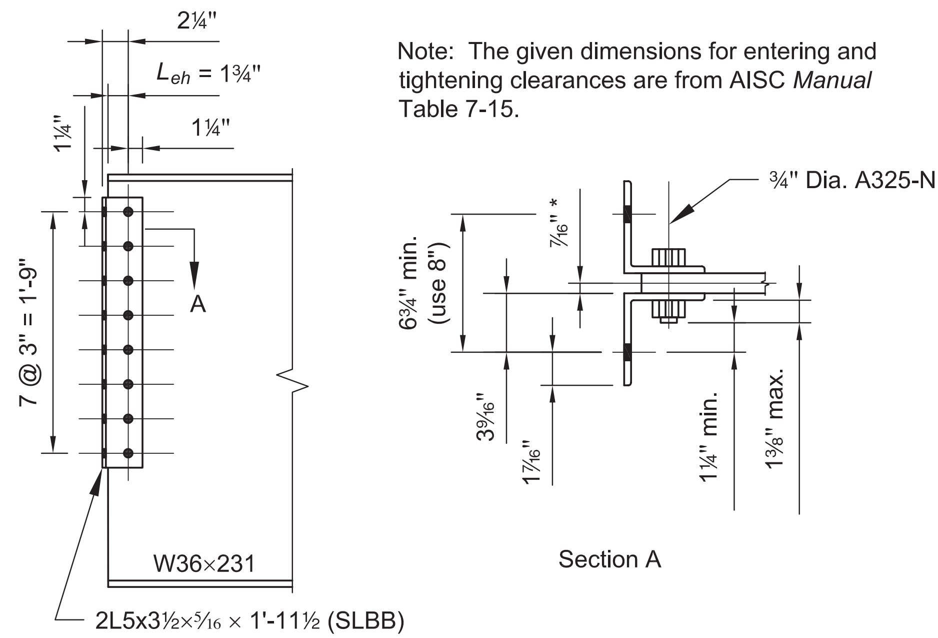
EXAMPLE II.A-1 ALL-BOLTED DOUBLE-ANGLE CONNECTION

Given:

Select an all-bolted double-angle connection between an ASTM A992 W36×231 beam and an ASTM A992 W14×90 column flange to support the following beam end reactions:

R_D = 37.5 kips
R_L = 113 kips

Use ¾-in.-diameter ASTM A325-N or F1852-N bolts in standard holes and ASTM A36 angles.



* This dimension (see sketch, Section A) is determined as one-half of the decimal web thickness rounded to the next higher 1⁄16 in. Example: 0.760"/2 = 0.380"; use 7⁄16 in. This will produce spacing of holes in the supporting beam slightly larger than detailed in the angles to permit spreading of angles (angles can be spread but not closed) at time of erection to supporting member. Alternatively, consider using horizontal short slots in the support legs of the angles.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles

ASTM A36
$F_y = 36$ ksi
$F_u = 58$ ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W36×231
$t_w = 0.760$ in.

Column
W14×90
$t_f = 0.710$ in.

From Chapter 2 of ASCE/SEI 7, the required strength is:

LRFD	ASD
$R_u = 1.2(37.5 \text{ kips}) + 1.6(113 \text{ kips})$ $= 226$ kips	$R_a = 37.5 \text{ kips} + 113 \text{ kips}$ $= 151$ kips

Connection Design

AISC *Manual* Table 10-1 includes checks for the limit states of bearing, shear yielding, shear rupture, and block shear rupture on the angles, and shear on the bolts.

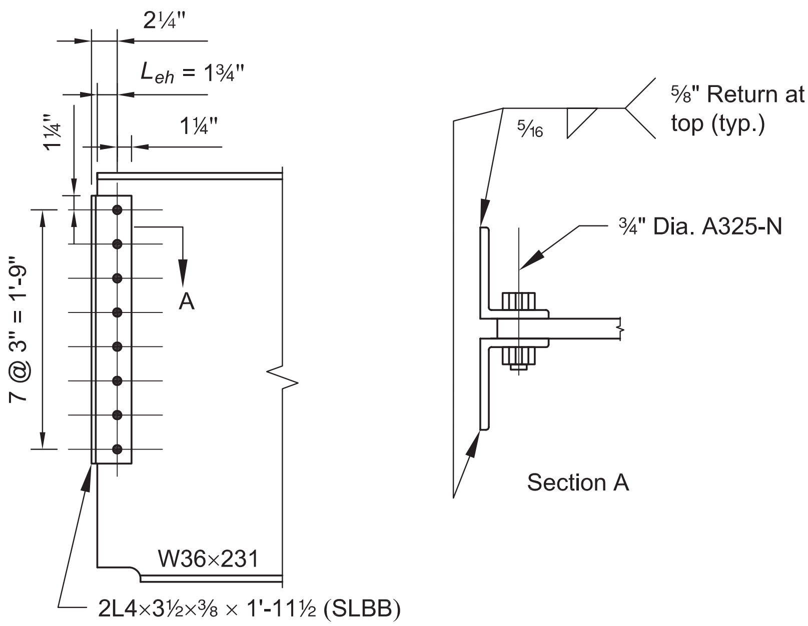
Try 8 rows of bolts and 2L5×3½×5⁄16 (SLBB).

LRFD	ASD
$\phi R_n = 247 \text{ kips} > 226 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 165 \text{ kips} > 151 \text{ kips}$ **o.k.**
Beam web strength from AISC *Manual* Table 10-1:	Beam web strength from AISC *Manual* Table 10-1:
Uncoped, L_{eh} = 1¾ in.	Uncoped, L_{eh} = 1¾ in.
$\phi R_n = 702 \text{ kips/in.}(0.760 \text{ in.})$ $= 534 \text{ kips} > 226 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 468 \text{ kips/in.}(0.760 \text{ in.})$ $= 356 \text{ kips} > 151 \text{ kips}$ **o.k.**
Bolt bearing on column flange from AISC *Manual* Table 10-1:	Bolt bearing on column flange from AISC *Manual* Table 10-1:
$\phi R_n = 1{,}400 \text{ kips/in.}(0.710 \text{ in.})$ $= 994 \text{ kips} > 226 \text{ kips}$ **o.k.**	$\dfrac{R_n}{\Omega} = 936 \text{ kips/in.}(0.710 \text{ in.})$ $= 665 \text{ kips} > 151 \text{ kips}$ **o.k.**

EXAMPLE II.A-2 BOLTED/WELDED DOUBLE-ANGLE CONNECTION

Given:

Repeat Example II.A-1 using AISC *Manual* Table 10-2 to substitute welds for bolts in the support legs of the double-angle connection (welds B). Use 70-ksi electrodes.



Note: Bottom flange coped for erection.

Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W36×231
t_w = 0.760 in.

Column
W14×90
t_f = 0.710 in.

From Chapter 2 of ASCE/SEI 7, the required strength is:

LRFD	ASD
R_u = 1.2(37.5 kips) + 1.6(113 kips) = 226 kips	R_a = 37.5 kips + 113 kips = 151 kips

Weld Design using AISC Manual Table 10-2 (welds B)

Try 5⁄16-in. weld size, L = 23 ½ in.

$t_{f\,min}$ = 0.238 in. < 0.710 in. **o.k.**

LRFD	ASD
ϕR_n = 279 kips > 226 kips **o.k.**	$\dfrac{R_n}{\Omega}$ = 186 kips > 151 kips **o.k.**

Angle Thickness

The minimum angle thickness for a fillet weld from AISC *Specification* Section J2.2b is:

t_{min} = w + 1⁄16 in.
= 5⁄16 in. + 1⁄16 in.
= 3⁄8 in.

Try 2L4×3½×3⁄8 (SLBB).

Angle and Bolt Design

AISC *Manual* Table 10-1 includes checks for the limit states of bearing, shear yielding, shear rupture, and block shear rupture on the angles, and shear on the bolts.

Check 8 rows of bolts and 3⁄8-in. angle thickness.

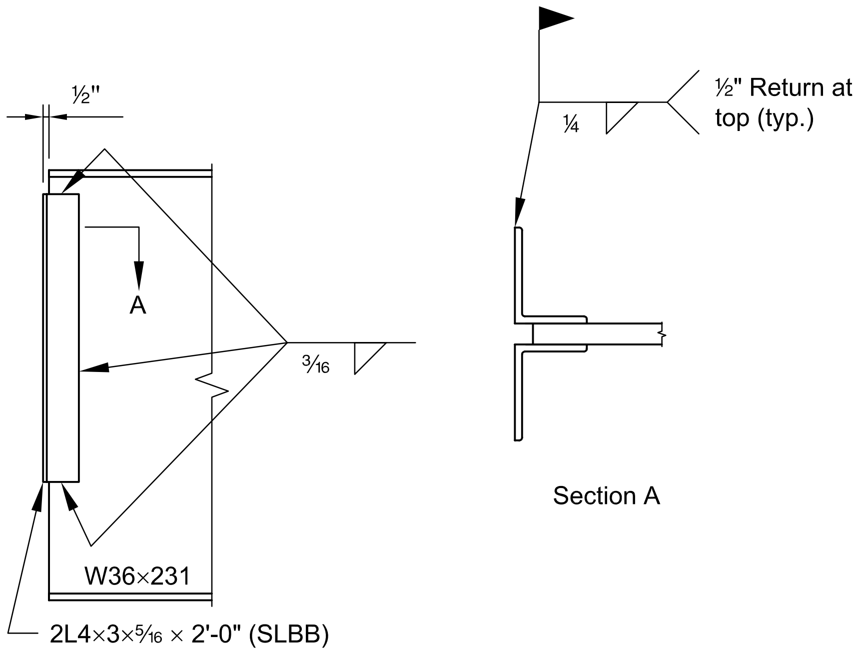
LRFD	ASD
ϕR_n = 286 kips > 226 kips **o.k.** Beam web strength: Uncoped, L_{eh} = 1¾ in. ϕR_n = 702 kips/in.(0.760 in.) = 534 kips > 226 kips **o.k.**	$\dfrac{R_n}{\Omega}$ = 191kips > 151 kips **o.k.** Beam web strength: Uncoped, L_{eh} = 1¾ in. $\dfrac{R_n}{\Omega}$ = 468 kips/in.(0.760 in.) = 356 kips > 151 kips **o.k.**

Note: In this example, because of the relative size of the cope to the overall beam size, the coped section does not control. When this cannot be determined by inspection, see AISC *Manual* Part 9 for the design of the coped section.

EXAMPLE II.A-3 ALL-WELDED DOUBLE-ANGLE CONNECTION

Given:

Repeat Example II.A-1 using AISC *Manual* Table 10-3 to design an all-welded double-angle connection between an ASTM A992 W36×231 beam and an ASTM A992 W14×90 column flange. Use 70-ksi electrodes and ASTM A36 angles.



Solution:

From AISC *Manual* Table 2-4, the material properties are as follows:

Beam
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Column
ASTM A992
F_y = 50 ksi
F_u = 65 ksi

Angles
ASTM A36
F_y = 36 ksi
F_u = 58 ksi

From AISC *Manual* Table 1-1, the geometric properties are as follows:

Beam
W36×231
t_w = 0.760 in.

Column
W14×90
t_f = 0.710 in.

From Chapter 2 of ASCE/SEI 7, the required strength is:

LRFD	ASD
R_u = 1.2(37.5 kips) + 1.6(113 kips) = 226 kips	R_a = 37.5 kips + 113 kips = 151 kips

Design of Weld Between Beam Web and Angle (welds A)

Try 3⁄16-in. weld size, L = 24 in.

$t_{w\,min}$ = 0.286 in. < 0.760 in. **o.k.**

From AISC *Manual* Table 10-3:

LRFD	ASD
ϕR_n = 257 kips > 226 kips **o.k.**	$\dfrac{R_n}{\Omega}$ = 171 kips > 151 kips **o.k.**

Design of Weld Between Column Flange and Angle (welds B)

Try ¼-in. weld size, L = 24 in.

$t_{f\,min}$ = 0.190 in. < 0.710 in. **o.k.**

From AISC *Manual* Table 10-3:

LRFD	ASD
ϕR_n = 229 kips > 226 kips **o.k.**	$\dfrac{R_n}{\Omega}$ = 153 kips > 151 kips **o.k.**

Angle Thickness

Minimum angle thickness for weld from AISC *Specification* Section J2.2b:

$$
\begin{aligned}
t_{min} &= w + \tfrac{1}{16}\text{ in.}\\
&= \tfrac{1}{4}\text{ in.} + \tfrac{1}{16}\text{ in.}\\
&= \tfrac{5}{16}\text{ in.}
\end{aligned}
$$

Try 2L4×3×5⁄16 (SLBB).

Shear Yielding of Angles (AISC Specification Section J4.2)

$$
\begin{aligned}
A_{gv} &= 2(24.0\text{ in.})(\tfrac{5}{16}\text{ in.})\\
&= 15.0\text{ in.}^2
\end{aligned}
$$

$$
\begin{aligned}
R_n &= 0.60F_y A_{gv} \qquad \text{(}\textit{Spec}\text{. Eq. J4-3)}\\
&= 0.60(36\text{ ksi})(15.0\text{ in.}^2)\\
&= 324\text{ kips}
\end{aligned}
$$

LRFD	ASD
$\phi = 1.00$ $\phi R_n = 1.00(324 \text{ kips})$ $= 324 \text{ kips} > 226 \text{ kips}$ **o.k.**	$\Omega = 1.50$ $\dfrac{R_n}{\Omega} = \dfrac{324 \text{ kips}}{1.50}$ $= 216 \text{ kips} > 151 \text{ kips}$ **o.k.**