SDS2 Steel Connection Design: Connection Cube Report

Cube: Ex. II.B-1
Revision: 0
Project: LRFD16ValidationExamples
Engineer:
Fabricator: ASD16ValidationExamples

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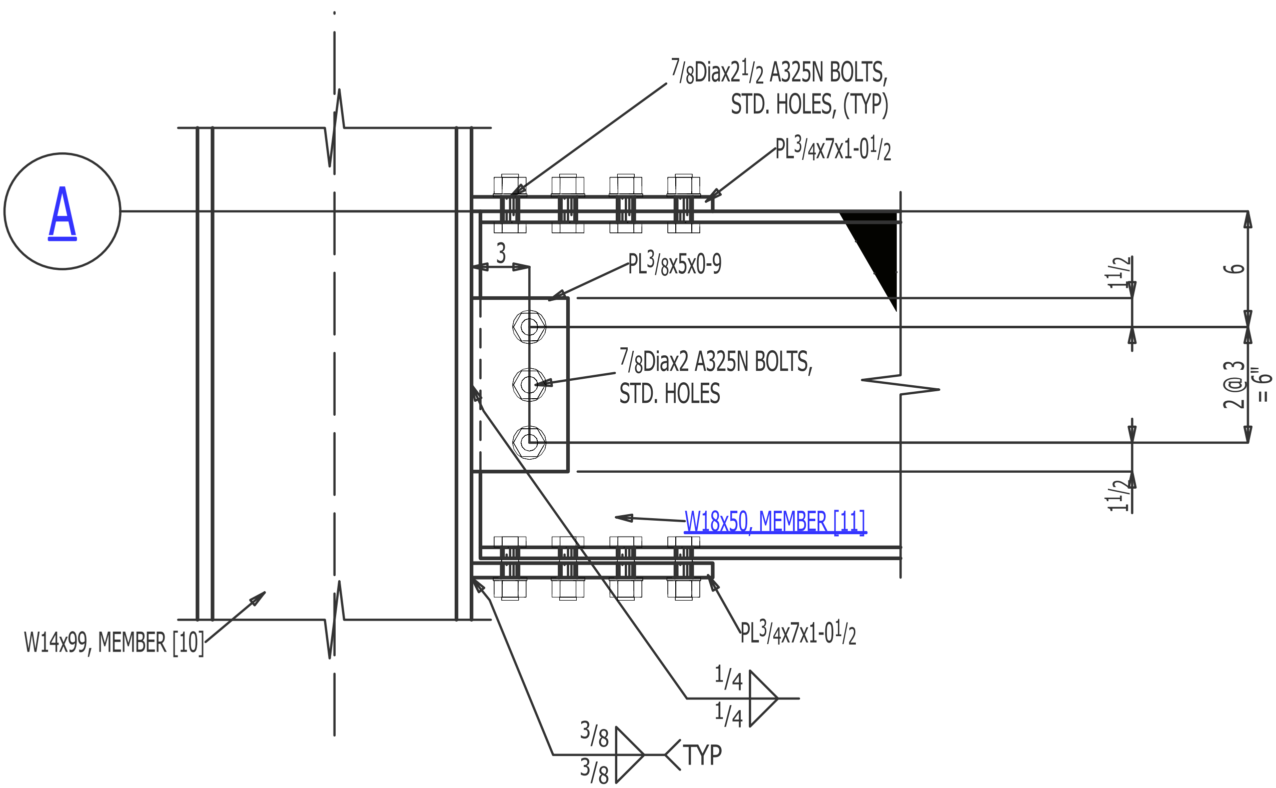
Ex. II.B-1 [5] at X=175-0, Y=100-0 Elev=-9



ELEVATION VIEW

Beam B_11 [11]

Design method

- AISC Steel Construction Manual, Sixteenth Edition (LRFD)
- AISC 360-22

Overview

Section size:	W18x50
Sequence:	1
ABM:	N/Assign
Plan length:	25-0
Camber:	0.00 in
Span length:	25-0
Slope:	0.00 °
Material length:	24-4 7/16
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	18 in
Web thickness, t_w:	0.355 in
Flange width, b_f:	7.5 in
Flange thickness, t_f:	0.57 in
Design k distance, k_{des}:	0.972 in
Detail k distance, k_{det}:	1.25 in
Distance between web toes of fillets, T:	15.5 in
Moment of inertia about the major axis, I_x:	800 in^4
Plastic section modulus about the major axis, Z_x:	101 in^3

Design summary

Left end

Connection:	Shear tab
	Plate, Size as required
	No Stiffener Opposite
	Shear plate on NS, Skew holes in beam
	Combine shear plates: No
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
	Plates on left end, Minimum Setup: No
	Bolted moment, Plate
	Design for column flange stiffener
	Design for column web doubler
Elevation:	0
Minus Dim:	7.5625 in
Mtrl Setback:	7.5625 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	42.0 kips
Story shear:	0.0 kips
Moment:	252.0 kip·ft
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_11 [11] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Column:	C_10 [10]
Section size:	W14x99
End 0 elevation:	-10-0
End 1 elevation:	10-0
Framing condition:	Flange of Column
Material grade:	A992
Detail k distance, k_{det}:	2.0625 in
Design k distance, k_{des}:	1.38 in
Supporting member thickness, t_{sup}:	0.78 in

Factored loads

***Shear*:**	42.0 kips
***Moment*:**	252.0 kip·ft

Design load notes

- Non-composite design
- Reaction has been input
- Moment has been input
- Design reaction is 34.7 % of the allowable uniform steel beam load.
- Design moment is 66.5 % of PHI*Mp.

Connection summary

- SHEAR PLATE WEB CONN. WITH FLANGE PLATE MOMENT CONN.

Connection details

Web plate:	**Grade:**	A572-50
	Tensile strength, F_u:	65 ksi
	Yield stress, F_y:	50 ksi
	Thickness, t:	0.375 in
	Depth, d:	9 in
Weld:	**Weld leg size, w:**	0.25 in
Web bolts:	**Bolt type:**	A325N
	Hole type in connection:	Standard round
	Bolt diameter, d_b:	7/8
	Bolt rows, n:	3
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
Flange plates:	**Grade:**	A572-50
	Tensile strength, F_u:	65 ksi
	Yield stress, F_y:	50 ksi
	Thickness, t:	0.75 in
	Width, b:	7 in
Flange bolts:	**Bolt type:**	A325N
	Hole type in connection:	Standard round
	Bolt diameter, d_b:	7/8
	Bolt rows, n:	4
	Bolt row spacing, s:	3 in
	Bolt gage, g:	4 in
Flange connection welds:	**Weld type:**	Double fillet
	Weld leg size, w:	0.375 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection notes

- Flange connection Fy does not match beam flange Fy.

Connection design lock summary

Locked Via Member Edit: 39

Expanded design calculation

Strength of column flange (83). Reference J10

Beam section depth, $d = 18\ in$
Column flange thickness, $t_{f,s} = 0.78\ in$
Column k distance, $k_s = 1.38\ in$
Flange plate thickness, $t_{fp} = 0.75\ in$
Column web thickness, $t_{w,s} = 0.485\ in$
Column flange yield stress, $F_{yf,s} = 50\ ksi$
Column web yield stress, $F_{yw,s} = 50\ ksi$

Calculate web local yielding capacity

Nominal web yielding capacity, $R_n = \left(t_{fp} + 5 \cdot k_s\right) \cdot t_{w,s} \cdot F_{yw,s}$
$= (0.75 + 5 \cdot 1.38) \cdot 0.485 \cdot 50$
$= 185.512\ kips$

$\phi = 1$

Web local yielding strength, $\phi P_{f,yield} = \phi \cdot R_n$
$= 1 \cdot 185.512$
$= 185.512\ kips$

Calculate flange local bending capacity

$\phi = 0.9$

Flange local bending strength, $\phi P_{f,bend} = \phi \cdot 6.25 \cdot t_{f,s}^2 \cdot F_{yf,s}$
$= 0.9 \cdot 6.25 \cdot 0.78^2 \cdot 50$
$= 171.113\ kips$

Yield capacity of stiffeners, $P_{y,st} = 0\ kips$
Flange tension load, horizontal component, $T_{a,f,h} = 0\ kips$
Flange compression load, horizontal component, $C_{a,f,h} = 0\ kips$
Applied member moment, $M_a = 252\ kip \cdot ft$

Unity ratio for web yielding, $U_{yield} = \dfrac{\left(\left(\dfrac{|M_a|}{(d + t_{fp})}\right) \cdot 12 + max\ \left(T_{a,f,h}, C_{a,f,h}\right)\right)}{\left(\phi P_{f,yield} + P_{y,st}\right)}$

$= \dfrac{\left(\left(\dfrac{|252|}{(18 + 0.75)}\right) \cdot 12 + max\ (0,0)\right)}{(185.512 + 0)}$

$= 0.869375$

Strength of column flange (83). Reference J10 (continued)

Unity ratio for flange bending, $U_{bend} = \frac{\left(\left(\frac{|M_a|}{(d + t_{fp})}\right) \cdot 12 + T_{a,f,h}\right)}{(\phi P_{f,bend} + P_{y,st})}$

$= \frac{\left(\left(\frac{|252|}{(18 + 0.75)}\right) \cdot 12 + 0\right)}{(171.113 + 0)}$

$= 0.942538$

$Unity = max\ (U_{yield}, U_{bend})$

$= max\ (0.869375, 0.942538)$

$= 0.942538$

Remaining column capacity, $P_{f,all} = min\ (\phi P_{f,yield} - max\ (T_{a,f,h}, C_{a,f,h}), \phi P_{f,bend} - T_{a,f,h})$

$= min\ (185.512 - max\ (0,0) - 0)$

$= 171.113\ kips$

$Moment\ capacity = \frac{P_{f,all} \cdot (d + t_{fp})}{12}$

$= \frac{171.113 \cdot (18 + 0.75)}{12}$

$= 267.363\ kip \cdot ft$

$267.4\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.943 \leq 1$ **(OK)**

Rupture of flange plate to support weld (212). Reference J2, Table J2.5

Flange plate tensile strength, $F_{u,pl} = 65\ ksi$

Flange plate width, $W_{pl} = 7\ in$

Flange plate thickness, $t_{pl} = 0.75\ in$

Full section depth, $d = 18\ in$

Weld leg size, $w = 0.375\ in$

Applied member moment, $M_a = 252\ kip \cdot ft$

Weld adjustment for angle of loading, $factor = 1 + 0.5 \cdot sin\ (90)^{1.5}$

$= 1.5$

$F_{EXX} = 70\ ksi$

$\phi = 0.75$

Allowable weld stress, $\phi F_w = \phi \cdot 0.6 \cdot F_{EXX} \cdot factor$

$= 0.75 \cdot 0.6 \cdot 70 \cdot 1.5$

$= 47.25\ ksi$

$F_{EXX} = 70\ ksi$

$\phi = 0.75$

Rupture of flange plate to support weld (212). Reference J2, Table J2.5 (continued)

$\phi = 0.75$

Maximum effective weld size, $w_{e,max} = \dfrac{\phi \cdot 0.6 \cdot F_{u,pl} \cdot t_{pl}}{2 \cdot 0.707 \cdot \phi \cdot 0.6 \cdot F_{EXX}}$

$= \dfrac{0.75 \cdot 0.6 \cdot 65 \cdot 0.75}{2 \cdot 0.707 \cdot 0.75 \cdot 0.6 \cdot 70}$

$= 0.492524\ in$

Effective weld size, $w_e = w$

$= 0.375\ in$

Calculate total effective transverse weld throat

Total effective transverse weld throat, $t_{eff} = 0.707 \cdot (w_e + w_e)$

$= 0.707 \cdot (0.375 + 0.375)$

$= 0.53025\ in$

Flange connection capacity, $\phi R_f = \phi F_w \cdot t_{eff} \cdot W_{pl}$

$= 47.25 \cdot 0.53025 \cdot 7$

$= 175.38\ kips$

Applied member moment, $M_a = 252\ kip \cdot ft$

$$Unity = \left(\frac{\left(\frac{|M_a|}{(d + t_{pl})} \right)}{\phi R_f} \right) \cdot 12$$

$$= \left(\frac{\left(\frac{|252|}{(18 + 0.75)} \right)}{175.38} \right) \cdot 12$$

$= 0.919602$

Flange tension load, horizontal component, $T_{a,f,h} = 0\ kips$

$$Moment\ capacity = \frac{(\phi R_f - T_{a,f,h}) \cdot (d + t_{pl})}{12}$$

$$= \frac{(175.38 - 0) \cdot (18 + 0.75)}{12}$$

$= 274.032\ kip \cdot ft$

$274.0\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.920 \leq 1$ **(OK)**

Tension/compression of flange plate (46). Reference D,E3,J4

Unsupported flange plate length, $L_b = 2\ in$

Connection tensile strength, $F_u = 65\ ksi$

Plate yield stress, $F_{y,pl} = 50\ ksi$

Plate thickness, $t_{pl} = 0.75\ in$

Plate width, $w_p = 7\ in$

Tension/compression of flange plate (46). Reference D,E3,J4 (continued)

Beam depth, $d = 18\ in$
Flange tension load, horizontal component, $T_{a,f,h} = 0\ kips$
Flange compression load, horizontal component, $C_{a,f,h} = 0\ kips$
Gross area, $A_g = w_p \cdot t_{pl}$
$= 7 \cdot 0.75$
$= 5.25\ in^2$
Hole diameter, $d_h = 1\ in$
Net area, $A_n = t_{pl} \cdot (w_p - 2 \cdot d_h)$
$= 0.75 \cdot (7 - 2 \cdot 1)$
$= 3.75\ in^2$
$\phi = 0.75$
$\phi = 0.9$
Remaining tension capacity in plate, $\phi T = min\ (\phi \cdot F_{y,pl} \cdot A_g, \phi \cdot F_u \cdot A_n) - T_{a,f,h}$
$= min\ (0.9 \cdot 50 \cdot 5.25, 0.75 \cdot 65 \cdot 3.75) - 0$
$= 182.812\ kips$
Effective length factor, $K = 0.65$
Unsupported length, $L = max\ (2, L_b)$
$= max\ (2,2)$
$= 2\ in$
Radius of gyration, $r = \frac{t_{pl}}{\sqrt{12}}$
$= \frac{0.75}{\sqrt{12}}$
$= 0.216506\ in$
Slenderness ratio, $Kl/r = \frac{K \cdot L}{r}$
$= \frac{0.65 \cdot 2}{0.216506}$
$= 6.00444$
Nominal stress, $F_n = F_{y,pl}$
$= 50\ ksi$
$\phi = 0.9$
Remaining compression capacity in plate, $\phi C = \phi \cdot F_n \cdot A_g - C_{a,f,h}$
$= 0.9 \cdot 50 \cdot 5.25 - 0$
$= 236.25\ kips$
$Moment\ capacity = \frac{min\ (\phi T, \phi C) \cdot (d + t_{pl})}{12}$
$= \frac{min\ (182.812, 236.25) \cdot (18 + 0.75)}{12}$
$= 285.645\ kip \cdot ft$
Applied member moment, $M_a = 252\ kip \cdot ft$

Tension/compression of flange plate (46). Reference D,E3,J4 (continued)

$Unity = \frac{|M_a|}{Moment\ capacity}$

$= \frac{|252|}{285.642}$

$= 0.882224$

$285.6\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.882 \leq 1$ **(OK)**

Panel zone shear of column web (395). Reference J10.6

Story shear, $V_s = 0\ kips$

Beam depth, $d_b = 18\ in$

Flange connection thickness, $t = 0.75\ in$

Column yield stress, $F_{y,c} = 50\ ksi$

Column web thickness, $t_{w,c} = 0.485\ in$

Column depth, $d_c = 14.2\ in$

Equivalent web thickness, $t_{w,eq} = t_{w,c}$

$= 0.485\ in$

Web panel zone area, $A_w = d_c \cdot t_{w,eq}$

$= 14.2 \cdot 0.485$

$= 6.887\ in^2$

$\phi = 0.9$

Allowable shear stress, $\phi F_v = \phi \cdot 0.6 \cdot F_{y,c}$

$= 0.9 \cdot 0.6 \cdot 50$

$= 27\ ksi$

Web panel zone capacity, $R_v = \phi F_v \cdot A_w$

$= 27 \cdot 6.887$

$= 185.949\ kips$

Allowable flange force, $F_f = R_v - V_s$

$= 185.949 - 0$

$= 185.949\ kips$

$Panel\ moment\ capacity = \frac{F_f \cdot (d_b + t)}{12}$

$= \frac{185.949 \cdot (18 + 0.75)}{12}$

$= 290.545\ kip \cdot ft$

Applied panel moment, $M_{a,z,p} = 252\ kip \cdot ft$

$Unity = \frac{|M_{a,z,p}|}{Panel\ moment\ capacity}$

$= \frac{|252|}{290.542}$

$= 0.867345$

Panel zone shear of column web (395). Reference J10.6 (continued)

$290.5\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.867 \leq 1$ **(OK)**

Bolt bearing on flange plate (69). Reference J3.11

Moment arm, $L_m = 18\ in$

Row edge distance, $L_e = 1.5\ in$

Bolt row spacing, $s = 3\ in$

Number of shear planes, $N_s = 1$

Plate tensile strength, $F_{u,p} = 65\ ksi$

Flange plate thickness, $t_p = 0.75\ in$

Bolt diameter, $d_b = 0.875\ in$

Bolt columns, $m = 2$

Bolt rows, $n = 4$

Total length of bolt group, $s_{total} = 9\ in$

Length of joint, $L = s_{total}$

$= 9\ in$

$(L = 9\ in) \leq 38\ in$

No reduction for connection length.

Bolt pattern length reduction factor, $k_r = 1$

Bolt area, $A_b = 0.60132\ in^2$

Allowable shear stress, $F_{nv} = 54\ ksi$

$\phi = 0.75$

Bolt shear capacity, $\phi R_{n,v} = \phi \cdot F_{nv} \cdot A_b \cdot N_s \cdot k_r$

$= 0.75 \cdot 54 \cdot 0.60132 \cdot 1 \cdot 1$

$= 24.3535\ kips$

Hole diameter, $d_h = 0.9375\ in$

$\phi = 0.75$

Bolt bearing capacity, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_p \cdot F_{u,p}$

$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.75 \cdot 65$

$= 76.7812\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$

Clear distance from bolt hole to bolt hole, $L_{c,int} = s - d_h$

$= 3 - 0.9375$

$= 2.0625\ in$

$\phi = 0.75$

Tearout load capacity, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_p \cdot F_{u,p}$

$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.75 \cdot 65$

$= 90.4922\ kips$

Interior bolt capacity (continued)

Controlling bearing/tearout strength of interior bolt, $\phi R_{n,i} = min\left(\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v}\right)$

$= min\left(90.4922, 76.7812, 24.3535\right)$

$= 24.3535\ kips$

Edge bolt capacity

Clear distance from hole to edge of material, $L_{c,edge} = L_e - 0.5 \cdot d_h$

$= 1.5 - 0.5 \cdot 0.9375$

$= 1.03125\ in$

$\phi = 0.75$

Tearout load capacity, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,edge} \cdot t_p \cdot F_{u,p}$

$= 0.75 \cdot 1.2 \cdot 1.03125 \cdot 0.75 \cdot 65$

$= 45.2461\ kips$

Controlling bearing/tearout strength of exterior bolt, $\phi R_{n,e} = min\left(\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v}\right)$

$= min\left(45.2461, 76.7812, 24.3535\right)$

$= 24.3535\ kips$

Number of edge bolts, $N_e = m$

$= 2$

Number of interior bolts, $N_i = m \cdot n - N_e$

$= 2 \cdot 4 - 2$

$= 6$

Bolt bearing capacity of flange plate, $\phi F_{f,allow} = \phi R_{n,e} \cdot N_e + \phi R_{n,i} \cdot N_i$

$= 24.3535 \cdot 2 + 24.3535 \cdot 6$

$= 194.828\ kips$

Force on flange due to axial load, $P_{a,f} = 0\ kips$

$$Moment\ capacity = \frac{\left(\phi F_{f,allow} - P_{a,f}\right) \cdot L_m}{12}$$

$$= \frac{\left(194.828 - 0\right) \cdot 18}{12}$$

$= 292.242\ kip \cdot ft$

Applied member moment, $M_a = 252\ kip \cdot ft$

$$Unity = \frac{\left|M_a\right|}{Moment\ capacity}$$

$$= \frac{\left|252\right|}{292.242}$$

$= 0.8623$

$292.2\ kip \cdot ft \geq \left(\left|252\right| = 252\ kip \cdot ft\right)$ **(OK)**

$0.862 \leq 1$ **(OK)**

Bolt bearing on beam flange (69). Reference J3.11

Moment arm, $L_m = 18\ in$
End distance, $L_e = 1.5375\ in$
Bolt row spacing, $s = 3\ in$
Number of shear planes, $N_s = 1$
Beam tensile strength, $F_{u,b} = 65\ ksi$
Flange thickness, $t_f = 0.57\ in$
Bolt diameter, $d_b = 0.875\ in$
Bolt columns, $m = 2$
Bolt rows, $n = 4$
Total length of bolt group, $s_{total} = 9\ in$
Length of joint, $L = s_{total}$
$= 9\ in$
$(L = 9\ in) \leq 38\ in$
No reduction for connection length.
Bolt pattern length reduction factor, $k_r = 1$
Bolt area, $A_b = 0.60132\ in^2$
Allowable shear stress, $F_{nv} = 54\ ksi$
$\phi = 0.75$
Bolt shear capacity, $\phi R_{n,v} = \phi \cdot F_{nv} \cdot A_b \cdot N_s \cdot k_r$
$= 0.75 \cdot 54 \cdot 0.60132 \cdot 1 \cdot 1$
$= 24.3535\ kips$
Hole diameter, $d_h = 0.9375\ in$
$\phi = 0.75$
Bolt bearing capacity, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_f \cdot F_{u,b}$
$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.57 \cdot 65$
$= 58.3537\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$
Clear distance from bolt hole to bolt hole, $L_{c,int} = s - d_h$
$= 3 - 0.9375$
$= 2.0625\ in$
$\phi = 0.75$
Tearout load capacity, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_f \cdot F_{u,b}$
$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.57 \cdot 65$
$= 68.7741\ kips$
Controlling bearing/tearout strength of interior bolt, $\phi R_{n,i} = min\ (\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v})$
$= min\ (68.7741, 58.3537, 24.3535)$
$= 24.3535\ kips$

Edge bolt capacity

Clear distance from hole to edge of material, $L_{c,edge} = L_e - 0.5 \cdot d_h$
$= 1.5375 - 0.5 \cdot 0.9375$
$= 1.06875\ in$

$\phi = 0.75$

Tearout load capacity, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,edge} \cdot t_f \cdot F_{u,b}$
$= 0.75 \cdot 1.2 \cdot 1.06875 \cdot 0.57 \cdot 65$
$= 35.6375\ kips$

Controlling bearing/tearout strength of exterior bolt, $\phi R_{n,e} = min\ (\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v})$
$= min\ (35.6375, 58.3537, 24.3535)$
$= 24.3535\ kips$

Number of edge bolts, $N_e = m$
$= 2$

Number of interior bolts, $N_i = m \cdot n - N_e$
$= 2 \cdot 4 - 2$
$= 6$

Bolt bearing capacity of flange plate, $\phi F_{f,allow} = \phi R_{n,e} \cdot N_e + \phi R_{n,i} \cdot N_i$
$= 24.3535 \cdot 2 + 24.3535 \cdot 6$
$= 194.828\ kips$

Force on flange due to axial load, $P_{a,f} = 0\ kips$

$$Moment\ capacity = \frac{(\phi F_{f,allow} - P_{a,f}) \cdot L_m}{12}$$

$$= \frac{(194.828 - 0) \cdot 18}{12}$$

$= 292.242\ kip \cdot ft$

Applied member moment, $M_a = 252\ kip \cdot ft$

$$Unity = \frac{|M_a|}{Moment\ capacity}$$

$$= \frac{|252|}{292.242}$$

$= 0.8623$

$292.2\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.862 \leq 1$ **(OK)**

Bolt shear of flange bolts (68). Reference J,Table J3.2

Smaller section depth, $d = 18\ in$

Number of shear planes, $N_s = 1$

Bolt row spacing, $s = 3\ in$

Bolt columns, $m = 2$

Bolt rows, $n = 4$

Force on flange due to axial load, $P_{a,f} = 0\ kips$

Applied member moment, $M_a = 252\ kip \cdot ft$

Bolt shear of flange bolts (68). Reference J,Table J3.2 (continued)

Total length of bolt group, $s_{total} = 9\ in$

Joint length, $l_j = s_{total}$

$= 9\ in$

$(l_j = 9\ in) \leq 38\ in$

No reduction for connection length.

Bolt pattern length reduction factor, $k_r = 1$

Bolt area, $A_b = 0.60132\ in^2$

Allowable shear stress, $F_{nv} = 54\ ksi$

$\phi = 0.75$

Bolt shear capacity, $\phi R_{n,v} = \phi \cdot F_{nv} \cdot A_b \cdot N_s \cdot k_r$

$= 0.75 \cdot 54 \cdot 0.60132 \cdot 1 \cdot 1$

$= 24.3535\ kips$

$$Unity = \left(\frac{\left(\frac{|M_a|}{d}\right)}{\phi R_{n,v} \cdot n \cdot m}\right) \cdot 12$$

$$= \left(\frac{\left(\frac{|252|}{18}\right)}{24.3535 \cdot 4 \cdot 2}\right) \cdot 12$$

$= 0.8623$

$$Moment\ capacity = \frac{\phi R_{n,v} \cdot n \cdot m \cdot d}{12}$$

$$= \frac{24.3535 \cdot 4 \cdot 2 \cdot 18}{12}$$

$= 292.242\ kip \cdot ft$

$292.2\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.862 \leq 1$ **(OK)**

Flexural rupture of beam (211). Reference F

Flange bolt columns, $m_f = 2$

Flange thickness, $t_f = 0.57\ in$

Flange width, $b_f = 7.5\ in$

Tensile strength, $F_u = 65\ ksi$

Elastic section modulus about the major axis, $S_x = 88.9\ in^3$

Applied member moment, $M_a = 252\ kip \cdot ft$

Applied tension load, horizontal component, $T_{a,h} = 0\ kips$

Applied compression load, horizontal component, $C_{a,h} = 0\ kips$

Steel modulus of elasticity, $E = 29000\ ksi$

Hole diameter flange, $d_{h,f} = 1\ in$

Hole diameter web, $d_{h,w} = 1\ in$

Calculate the strong axis moment capacity

Gross flange area, $A_{fg} = b_f \cdot t_f$

$= 7.5 \cdot 0.57$

$= 4.275\ in^2$

Net flange area, $A_{fn} = (b_f - m_f \cdot d_{h,f}) \cdot t_f$

$= (7.5 - 2 \cdot 1) \cdot 0.57$

$= 3.135\ in^2$

Hole reduction coefficient, $Y_t = 1$

Nominal moment capacity, $M_n = \dfrac{\left(\dfrac{F_u \cdot A_{fn} \cdot S_x}{A_{fg}}\right)}{12}$

$= \dfrac{\left(\dfrac{65 \cdot 3.135 \cdot 88.9}{4.275}\right)}{12}$

$= 353.131\ kip \cdot ft$

$\phi = 0.9$

Allowable moment, $\phi M = \phi \cdot M_n$

$= 0.9 \cdot 353.131$

$= 317.817\ kip \cdot ft$

$Unity = \dfrac{|M_a|}{\phi M}$

$= \dfrac{|252|}{317.817}$

$= 0.792908$

$Moment\ capacity = \phi M$

$= 317.817\ kip \cdot ft$

$317.8\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.793 \leq 1$ **(OK)**

Crippling of column web (25). Reference J10.3

Flange plate thickness, $t_{fp} = 0.75\ in$

Beam section depth, $d_b = 18\ in$

Column flange thickness, $t_{f,s} = 0.78\ in$

Column section depth, $d_s = 14.2\ in$

Column web thickness, $t_{w,s} = 0.485\ in$

Column yield stress, $F_{y,s} = 50\ ksi$

Applied compression load, horizontal component, $C_{a,h} = 0\ kips$

Bearing length, $l_b = t_{fp}$

$= 0.75\ in$

Modulus of Elasticity, $E = 29000\ ksi$

Crippling of column web (25). Reference J10.3 (continued)

Chord-stress interaction parameter, $Q_f = 1$

Nominal web crippling strength, $R_n = 0.8 \cdot t_{w,s}^2 \cdot \left(1 + 3 \cdot \left(\frac{l_b}{d_s}\right) \cdot \left(\frac{t_{w,s}}{t_{f,s}}\right)^{1.5}\right) \cdot \sqrt{\left(\frac{E \cdot F_{y,s} \cdot t_{f,s}}{t_{w,s}}\right)} \cdot Q_f$

$= 0.8 \cdot 0.485^2 \cdot \left(1 + 3 \cdot \left(\frac{0.75}{14.2}\right) \cdot \left(\frac{0.485}{0.78}\right)^{1.5}\right) \cdot \sqrt{\left(\frac{29000 \cdot 50 \cdot 0.78}{0.485}\right)} \cdot 1$

$= 309.69$ *kips*

$\phi = 0.75$

Allowable flange force, $F_f = \phi \cdot R_n$

$= 0.75 \cdot 309.69$

$= 232.268$ *kips*

$Moment\ capacity = \frac{F_f \cdot (d_b + t_{fp})}{12}$

$= \frac{232.268 \cdot (18 + 0.75)}{12}$

$= 362.918\ kip \cdot ft$

Applied member moment, $M_a = 252\ kip \cdot ft$

$Unity = \frac{|M_a|}{Moment\ capacity}$

$= \frac{|252|}{362.917}$

$= 0.694374$

$362.9\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.694 \leq 1$ **(OK)**

Shear rupture of plate (21). Reference J4.2

Connection tensile strength, $F_u = 65\ ksi$

Bolt rows, $n = 3$

Connection thickness, $t_{conn} = 0.375\ in$

Connection depth, $d_{pl} = 9\ in$

Hole diameter, $d_h = 1\ in$

Net shear area, $A_{nv} = t_{conn} \cdot (d_{pl} - n \cdot d_h)$

$= 0.375 \cdot (9 - 3 \cdot 1)$

$= 2.25\ in^2$

$\phi = 0.75$

Shear capacity, $\phi V_n = \phi \cdot 0.6 \cdot F_u \cdot A_{nv}$

$= 0.75 \cdot 0.6 \cdot 65 \cdot 2.25$

$= 65.8125$ *kips*

$Shear\ capacity = \phi V_n$

$= 65.8125$ *kips*

Applied member shear, $V_a = 42$ *kips*

Shear rupture of plate (21). Reference J4.2 (continued)

$Unity = \frac{V_a}{Shear\ capacity}$
$= \frac{42}{65.8}$
$= 0.638298$

$65.8\ kips \geq 42\ kips$ **(OK)**

$0.638 \leq 1$ **(OK)**

Bolt bearing on plate (110). Reference J3.11

Tensile strength, $F_u = 65\ ksi$
Plate thickness, $t_{pl} = 0.375\ in$
Bolt row spacing, $s = 3\ in$
Row edge distance, $L_{ev} = 1.5\ in$
Bolt diameter, $d_b = 0.875\ in$
Number of shear planes, $N_s = 1$
Bolt columns, $m = 1$
Bolt rows, $n = 3$
Total length of bolt group, $s_{total} = 6\ in$
Length of joint, $L = s_{total}$
$= 6\ in$
Bolt area, $A_b = 0.60132\ in^2$
Allowable shear stress, $F_{nv} = 54\ ksi$
$\phi = 0.75$
Bolt shear capacity, $\phi R_{n,v} = \phi \cdot F_{nv} \cdot A_b \cdot N_s$
$= 0.75 \cdot 54 \cdot 0.60132 \cdot 1$
$= 24.3535\ kips$
Hole diameter, $d_h = 0.9375\ in$
$\phi = 0.75$
Bolt bearing capacity, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_{pl} \cdot F_u$
$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.375 \cdot 65$
$= 38.3906\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$
Clear distance from bolt hole to bolt hole, $L_{c,int} = s - d_h$
$= 3 - 0.9375$
$= 2.0625\ in$
$\phi = 0.75$
Tearout load capacity, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_{pl} \cdot F_u$
$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.375 \cdot 65$
$= 45.2461\ kips$

Interior bolt capacity (continued)

Controlling bearing/tearout strength of interior bolt, $\phi R_{n,i} = min\ (\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v})$
$= min\ (45.2461, 38.3906, 24.3535)$
$= 24.3535\ kips$

Edge bolt capacity

Clear distance from hole to edge of material, $L_{c,edge} = L_{ev} - 0.5 \cdot d_h$
$= 1.5 - 0.5 \cdot 0.9375$
$= 1.03125\ in$
$\phi = 0.75$
Tearout load capacity, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,edge} \cdot t_{pl} \cdot F_u$
$= 0.75 \cdot 1.2 \cdot 1.03125 \cdot 0.375 \cdot 65$
$= 22.623\ kips$
Controlling bearing/tearout strength of exterior bolt, $\phi R_{n,e} = min\ (\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v})$
$= min\ (22.623, 38.3906, 24.3535)$
$= 22.623\ kips$

Number of edge bolts, $N_e = m$
$= 1$
Number of interior bolts, $N_i = m \cdot n - N_e$
$= 1 \cdot 3 - 1$
$= 2$
$Shear\ capacity = \phi R_{n,e} \cdot N_e + \phi R_{n,i} \cdot N_i$
$= 22.623 \cdot 1 + 24.3535 \cdot 2$
$= 71.33\ kips$
Applied member shear, $V_a = 42\ kips$
$Unity = \frac{V_a}{Shear\ capacity}$
$= \frac{42}{71.3}$
$= 0.58906$
$71.3\ kips \geq 42\ kips$ **(OK)**
$0.589 \leq 1$ **(OK)**

Bolt shear of web bolts (1). Reference J3.7, J3.9

Number of shear planes, $N_s = 1$
Bolt columns, $m = 1$
Bolt rows, $n = 3$
Bolt area, $A_b = 0.60132\ in^2$
Allowable shear stress, $F_{nv} = 54\ ksi$
$\phi = 0.75$
Bolt shear capacity, $\phi R_{n,v} = \phi \cdot F_{nv} \cdot A_b \cdot N_s$
$= 0.75 \cdot 54 \cdot 0.60132 \cdot 1$

Bolt shear of web bolts (1). Reference J3.7, J3.9 (continued)

$= 24.3535\ kips$

$Shear\ capacity = \phi R_{n,v} \cdot n \cdot m$

$= 24.3535 \cdot 3 \cdot 1$

$= 73.0604\ kips$

Applied member shear, $V_a = 42\ kips$

$Unity = \frac{V_a}{Shear\ capacity}$

$= \frac{42}{73.1}$

$= 0.574555$

$73.1\ kips \geq 42\ kips$ **(OK)**

$0.575 \leq 1$ **(OK)**

Bolt bearing on beam web (110). Reference J3.11

Tensile strength, $F_u = 65\ ksi$

Plate thickness, $t_{pl} = 0.355\ in$

Bolt row spacing, $s = 3\ in$

Bolt diameter, $d_b = 0.875\ in$

Number of shear planes, $N_s = 1$

Bolt columns, $m = 1$

Bolt rows, $n = 3$

Total length of bolt group, $s_{total} = 6\ in$

Length of joint, $L = s_{total}$

$= 6\ in$

Bolt area, $A_b = 0.60132\ in^2$

Allowable shear stress, $F_{nv} = 54\ ksi$

$\phi = 0.75$

Bolt shear capacity, $\phi R_{n,v} = \phi \cdot F_{nv} \cdot A_b \cdot N_s$

$= 0.75 \cdot 54 \cdot 0.60132 \cdot 1$

$= 24.3535\ kips$

Hole diameter, $d_h = 0.9375\ in$

$\phi = 0.75$

Bolt bearing capacity, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_{pl} \cdot F_u$

$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.355 \cdot 65$

$= 36.3431\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$

Clear distance from bolt hole to bolt hole, $L_{c,int} = s - d_h$

$= 3 - 0.9375$

$= 2.0625\ in$

Interior bolt capacity (continued)

$\phi = 0.75$

Tearout load capacity, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_{pl} \cdot F_u$

$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.355 \cdot 65$

$= 42.833\ kips$

Controlling bearing/tearout strength of interior bolt, $\phi R_{n,i} = min\ (\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v})$

$= min\ (42.833, 36.3431, 24.3535)$

$= 24.3535\ kips$

Edge bolt capacity

Tear out will not occur, so the bearing capacity controls.

Controlling bearing/tearout strength of exterior bolt, $\phi R_{n,e} = min\ (\phi R_{n,b}, \phi R_{n,v})$

$= min\ (36.3431, 24.3535)$

$= 24.3535\ kips$

Number of edge bolts, $N_e = m$

$= 1$

Number of interior bolts, $N_i = m \cdot n - N_e$

$= 1 \cdot 3 - 1$

$= 2$

$Shear\ capacity = \phi R_{n,e} \cdot N_e + \phi R_{n,i} \cdot N_i$

$= 24.3535 \cdot 1 + 24.3535 \cdot 2$

$= 73.0604\ kips$

Applied member shear, $V_a = 42\ kips$

$Unity = \frac{V_a}{Shear\ capacity}$

$= \frac{42}{73.1}$

$= 0.574555$

$73.1\ kips \geq 42\ kips$ **(OK)**

$0.575 \leq 1$ **(OK)**

Block shear rupture of beam flange (259). Reference J4.3

Full section depth, $d = 18\ in$

Bolt row spacing, $s = 3\ in$

Edge distance, $L_e = 1.5375\ in$

Bolt gage, $g = 4\ in$

Bolt rows, $n = 4$

Flange thickness, $t_f = 0.57\ in$

Flange width, $b_f = 7.5\ in$

Tensile strength, $F_u = 65\ ksi$

Yield stress, $F_y = 50\ ksi$

Applied flange tension due to moment, $T_{a,f,M} = 173.494\ kips$

Block shear rupture of beam flange (259). Reference J4.3 (continued)

Applied tension in flange, $T_{a,f} = 0\ kips$

Total length of bolt group, $s_{total} = 9\ in$

Connection length, $L = s_{total}$

$= 9\ in$

Hole diameter, $d_h = 1\ in$

Gross tensile area, $A_{gt} = 2 \cdot 0.5 \cdot (b_f - g) \cdot t_f$

$= 2 \cdot 0.5 \cdot (7.5 - 4) \cdot 0.57$

$= 1.995\ in^2$

Net tensile area, $A_{nt} = A_{gt} - 2 \cdot (1 - 0.5) \cdot d_h \cdot t_f$

$= 1.995 - 2 \cdot (1 - 0.5) \cdot 1 \cdot 0.57$

$= 1.425\ in^2$

Gross shear area, $A_{gv} = 2 \cdot (L + L_e) \cdot t_f$

$= 2 \cdot (9 + 1.5375) \cdot 0.57$

$= 12.0128\ in^2$

Net shear area, $A_{nv} = A_{gv} - 2 \cdot (n - 0.5) \cdot d_h \cdot t_f$

$= 12.0128 - 2 \cdot (4 - 0.5) \cdot 1 \cdot 0.57$

$= 8.02275\ in^2$

Reduction coefficient, $U_{bs} = 1$

Shear yield load, $R_{gv} = 0.6 \cdot F_y \cdot A_{gv}$

$= 0.6 \cdot 50 \cdot 12.0128$

$= 360.382\ kips$

Shear rupture load, $R_{nv} = 0.6 \cdot F_u \cdot A_{nv}$

$= 0.6 \cdot 65 \cdot 8.02275$

$= 312.887\ kips$

Tension load, $R_t = U_{bs} \cdot F_u \cdot A_{nt}$

$= 1 \cdot 65 \cdot 1.425$

$= 92.625\ kips$

Nominal block shear capacity, $R_n = min\ (R_{gv}, R_{nv}) + R_t$

$= min\ (360.382, 312.887) + 92.625$

$= 405.512\ kips$

$\phi = 0.75$

Flange block shear strength, $\phi R_{bs} = \phi \cdot R_n$

$= 0.75 \cdot 405.512$

$= 304.134\ kips$

$$Unity = \frac{(T_{a,f} + T_{a,f,M})}{\phi R_{bs}}$$

$$= \frac{(0 + 173.494)}{304.134}$$

$= 0.570452$

Block shear rupture of beam flange (259). Reference J4.3 (continued)

$Moment\ capacity = \dfrac{\phi R_{bs} \cdot (d - t_f)}{12}$

$= \dfrac{304.134 \cdot (18 - 0.57)}{12}$

$= 441.755\ kip \cdot ft$

$441.8\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.570 \leq 1$ **(OK)**

Block shear rupture of flange plate (85). Reference J4.3

Full section depth, $d = 18\ in$

Bolt row spacing, $s = 3\ in$

Edge distance, $L_e = 1.5\ in$

Gage, $g = 4\ in$

Bolt rows, $n = 4$

Flange plate thickness, $t_{fp} = 0.75\ in$

Flange plate width, $b_{conn} = 7\ in$

Tensile strength, $F_u = 65\ ksi$

Yield stress, $F_y = 50\ ksi$

Applied tension in flange, $T_{a,f} = 0\ kips$

C-shaped failure pattern

Total length of bolt group, $s_{total} = 9\ in$

Connection length, $L = s_{total}$

$= 9\ in$

Hole diameter, $d_h = 1\ in$

Gross tensile area, $A_{gt} = 1 \cdot g \cdot t_{fp}$

$= 1 \cdot 4 \cdot 0.75$

$= 3\ in^2$

Net tensile area, $A_{nt} = A_{gt} - 1 \cdot (2 - 1) \cdot d_h \cdot t_{fp}$

$= 3 - 1 \cdot (2 - 1) \cdot 1 \cdot 0.75$

$= 2.25\ in^2$

Gross shear area, $A_{gv} = 1 \cdot 2 \cdot (L + L_e) \cdot t_{fp}$

$= 1 \cdot 2 \cdot (9 + 1.5) \cdot 0.75$

$= 15.75\ in^2$

Net shear area, $A_{nv} = A_{gv} - 1 \cdot 2 \cdot (n - 0.5) \cdot d_h \cdot t_{fp}$

$= 15.75 - 1 \cdot 2 \cdot (4 - 0.5) \cdot 1 \cdot 0.75$

$= 10.5\ in^2$

Reduction coefficient, $U_{bs} = 1$

Shear yield load, $R_{gv} = 0.6 \cdot F_y \cdot A_{gv}$

$= 0.6 \cdot 50 \cdot 15.75$

C-shaped failure pattern (continued)

$= 472.5\ kips$

Shear rupture load, $R_{nv} = 0.6 \cdot F_u \cdot A_{nv}$

$= 0.6 \cdot 65 \cdot 10.5$

$= 409.5\ kips$

Tension load, $R_t = U_{bs} \cdot F_u \cdot A_{nt}$

$= 1 \cdot 65 \cdot 2.25$

$= 146.25\ kips$

Nominal block shear capacity, $R_n = min\ (R_{gv}, R_{nv}) + R_t$

$= min\ (472.5, 409.5) + 146.25$

$= 555.75\ kips$

$\phi = 0.75$

Block shear capacity (C-shaped pattern), $\phi R_{bs1} = \phi \cdot R_n$

$= 0.75 \cdot 555.75$

$= 416.812\ kips$

2 L-shaped failure pattern

Total length of bolt group, $s_{total} = 9\ in$

Connection length, $L = s_{total}$

$= 9\ in$

Hole diameter, $d_h = 1\ in$

Gross tensile area, $A_{gt} = 2 \cdot 0.5 \cdot (b_{conn} - g) \cdot t_{fp}$

$= 2 \cdot 0.5 \cdot (7 - 4) \cdot 0.75$

$= 2.25\ in^2$

Net tensile area, $A_{nt} = A_{gt} - 2 \cdot (1 - 0.5) \cdot d_h \cdot t_{fp}$

$= 2.25 - 2 \cdot (1 - 0.5) \cdot 1 \cdot 0.75$

$= 1.5\ in^2$

Gross shear area, $A_{gv} = 2 \cdot (L + L_e) \cdot t_{fp}$

$= 2 \cdot (9 + 1.5) \cdot 0.75$

$= 15.75\ in^2$

Net shear area, $A_{nv} = A_{gv} - 2 \cdot (n - 0.5) \cdot d_h \cdot t_{fp}$

$= 15.75 - 2 \cdot (4 - 0.5) \cdot 1 \cdot 0.75$

$= 10.5\ in^2$

Reduction coefficient, $U_{bs} = 1$

Shear yield load, $R_{gv} = 0.6 \cdot F_y \cdot A_{gv}$

$= 0.6 \cdot 50 \cdot 15.75$

$= 472.5\ kips$

Shear rupture load, $R_{nv} = 0.6 \cdot F_u \cdot A_{nv}$

$= 0.6 \cdot 65 \cdot 10.5$

$= 409.5\ kips$

2 L-shaped failure pattern (continued)

Tension load, $R_t = U_{bs} \cdot F_u \cdot A_{nt}$
$= 1 \cdot 65 \cdot 1.5$
$= 97.5\ kips$

Nominal block shear capacity, $R_n = min\ (R_{gv}, R_{nv}) + R_t$
$= min\ (472.5, 409.5) + 97.5$
$= 507\ kips$

$\phi = 0.75$

Block shear capacity (2 L-shaped patterns), $\phi R_{bs2} = \phi \cdot R_n$
$= 0.75 \cdot 507$
$= 380.25\ kips$

L-shaped failure pattern

Total length of bolt group, $s_{total} = 9\ in$

Connection length, $L = s_{total}$
$= 9\ in$

Hole diameter, $d_h = 1\ in$

Gross tensile area, $A_{gt} = 1 \cdot (g + 0.5 \cdot (b_{conn} - g)) \cdot t_{fp}$
$= 1 \cdot (4 + 0.5 \cdot (7 - 4)) \cdot 0.75$
$= 4.125\ in^2$

Net tensile area, $A_{nt} = A_{gt} - 1 \cdot (2 - 0.5) \cdot d_h \cdot t_{fp}$
$= 4.125 - 1 \cdot (2 - 0.5) \cdot 1 \cdot 0.75$
$= 3\ in^2$

Gross shear area, $A_{gv} = 1 \cdot (L + L_e) \cdot t_{fp}$
$= 1 \cdot (9 + 1.5) \cdot 0.75$
$= 7.875\ in^2$

Net shear area, $A_{nv} = A_{gv} - 1 \cdot (n - 0.5) \cdot d_h \cdot t_{fp}$
$= 7.875 - 1 \cdot (4 - 0.5) \cdot 1 \cdot 0.75$
$= 5.25\ in^2$

Reduction coefficient, $U_{bs} = 1$

Shear yield load, $R_{gv} = 0.6 \cdot F_y \cdot A_{gv}$
$= 0.6 \cdot 50 \cdot 7.875$
$= 236.25\ kips$

Shear rupture load, $R_{nv} = 0.6 \cdot F_u \cdot A_{nv}$
$= 0.6 \cdot 65 \cdot 5.25$
$= 204.75\ kips$

Tension load, $R_t = U_{bs} \cdot F_u \cdot A_{nt}$
$= 1 \cdot 65 \cdot 3$
$= 195\ kips$

Nominal block shear capacity, $R_n = min\ (R_{gv}, R_{nv}) + R_t$

L-shaped failure pattern (continued)

$= min\ (236.25, 204.75) + 195$

$= 399.75\ kips$

$\phi = 0.75$

Block shear capacity (L-shaped pattern), $\phi R_{bs3} = \phi \cdot R_n$

$= 0.75 \cdot 399.75$

$= 299.812\ kips$

Applied member moment, $M_a = 252\ kip \cdot ft$

$$Unity = \frac{\left(\left(\frac{|M_a|}{(d + t_{fp})}\right) \cdot 12 + T_{a,f}\right)}{min\ (\phi R_{bs1}, \phi R_{bs2}, \phi R_{bs3})}$$

$$= \frac{\left(\left(\frac{|252|}{(18 + 0.75)}\right) \cdot 12 + 0\right)}{min\ (416.812, 380.25, 299.812)}$$

$= 0.537936$

$$Moment\ capacity = \frac{min\ (\phi R_{bs1}, \phi R_{bs2}, \phi R_{bs3}) \cdot (d + t_{fp})}{12}$$

$$= \frac{min\ (416.812, 380.25, 299.812) \cdot (18 + 0.75)}{12}$$

$= 468.457\ kip \cdot ft$

$468.5\ kip \cdot ft \geq (|252| = 252\ kip \cdot ft)$ **(OK)**

$0.538 \leq 1$ **(OK)**

Block shear rupture of plate (6). Reference J4.3

Plate thickness, $t_{pl} = 0.375\ in$

Yield stress, $F_y = 50\ ksi$

Tensile strength, $F_u = 65\ ksi$

Bolt column spacing, $s_{col} = 0\ in$

Bolt row spacing, $s = 3\ in$

Bolt rows, $n = 3$

Column edge distance, $L_{eh} = 1.975\ in$

Row edge distance, $L_{ev} = 1.5\ in$

Bolt columns, $m = 1$

Hole diameter, $d_h = 1\ in$

Hole length, $l_h = 1\ in$

Total length of bolt group, $s_{total} = 6\ in$

Gross shear area, $A_{gv} = t_{pl} \cdot (s_{total} + L_{ev})$

$= 0.375 \cdot (6 + 1.5)$

Block shear rupture of plate (6). Reference J4.3 (continued)

$= 2.8125\ in^2$

Net shear area, $A_{nv} = t_{pl} \cdot (s_{total} + L_{ev}) - t_{pl} \cdot (n - 0.5) \cdot d_h$

$= 0.375 \cdot (6 + 1.5) - 0.375 \cdot (3 - 0.5) \cdot 1$

$= 1.875\ in^2$

Gross tensile area, $A_{gt} = t_{pl} \cdot (s_{col} \cdot (m - 1) + L_{eh})$

$= 0.375 \cdot (0 \cdot (1 - 1) + 1.975)$

$= 0.740625\ in^2$

Net tensile area, $A_{nt} = t_{pl} \cdot (s_{col} \cdot (m - 1) + L_{eh}) - t_{pl} \cdot (m - 0.5) \cdot l_h$

$= 0.375 \cdot (0 \cdot (1 - 1) + 1.975) - 0.375 \cdot (1 - 0.5) \cdot 1$

$= 0.553125\ in^2$

Reduction coefficient, $U_{bs} = 1$

Shear yield load, $R_{gv} = 0.6 \cdot F_y \cdot A_{gv}$

$= 0.6 \cdot 50 \cdot 2.8125$

$= 84.375\ kips$

Shear rupture load, $R_{nv} = 0.6 \cdot F_u \cdot A_{nv}$

$= 0.6 \cdot 65 \cdot 1.875$

$= 73.125\ kips$

Tension load, $R_t = U_{bs} \cdot F_u \cdot A_{nt}$

$= 1 \cdot 65 \cdot 0.553125$

$= 35.9531\ kips$

Nominal block shear capacity, $R_n = min\ (R_{gv}, R_{nv}) + R_t$

$= min\ (84.375, 73.125) + 35.9531$

$= 109.078\ kips$

$\phi = 0.75$

$Shear\ capacity = \phi \cdot R_n$

$= 0.75 \cdot 109.078$

$= 81.8086\ kips$

Applied member shear, $V_a = 42\ kips$

$Unity = \frac{V_a}{Shear\ capacity}$

$= \frac{42}{81.8}$

$= 0.513447$

$81.8\ kips \geq 42\ kips$ **(OK)**

$0.513 \leq 1$ **(OK)**

Shear yielding of plate (15). Reference J4.2

Connection yield stress, $F_{y,conn} = 50\ ksi$

Connection thickness, $t = 0.375\ in$

Connection length, $L = 9\ in$

Shear yielding of plate (15). Reference J4.2 (continued)

Gross shear area, $A_{gv} = t \cdot L$

$= 0.375 \cdot 9$

$= 3.375\ in^2$

$\phi = 1$

$Shear\ capacity = \phi \cdot 0.6 \cdot F_{y,conn} \cdot A_{gv}$

$= 1 \cdot 0.6 \cdot 50 \cdot 3.375$

$= 101.25\ kips$

Applied member shear, $V_a = 42\ kips$

$Unity = \frac{V_a}{Shear\ capacity}$

$= \frac{42}{101.3}$

$= 0.41461$

$101.3\ kips \geq 42\ kips$ **(OK)**

$0.415 \leq 1$ **(OK)**

Shear yielding of beam web (2). Reference G2.1

Yield stress, $F_y = 50\ ksi$

Web thickness, $t_w = 0.355\ in$

Full section depth, $d = 18\ in$

Applied member shear, $V_a = 42\ kips$

$\phi = 1$

Allowable shear stress, $\phi F_v = \phi \cdot 0.6 \cdot F_y$

$= 1 \cdot 0.6 \cdot 50$

$= 30\ ksi$

Web shear area, $A_w = d \cdot t_w$

$= 18 \cdot 0.355$

$= 6.39\ in^2$

$Unity = \frac{V_a}{\phi F_v \cdot A_w}$

$= \frac{42}{30 \cdot 6.39}$

$= 0.219092$

$Shear\ capacity = \phi F_v \cdot A_w$

$= 30 \cdot 6.39$

$= 191.7\ kips$

$191.7\ kips \geq 42\ kips$ **(OK)**

$0.219 \leq 1$ **(OK)**

Flange plate width to thickness ratio (461). Reference B4.1

Flange plate width, $b_{conn} = 7\ in$

Flange gage, $g = 4\ in$

Flange plate thickness, $t_{fp} = 0.75\ in$

Flange plate yield stress, $F_{y,pl} = 50\ ksi$

Check the slenderness of the stiffened portion of the plate

Stiffened plate width, $b_{sp} = g$

$= 4\ in$

Modulus of elasticity, $E = 29000\ ksi$

$$\left(\frac{b_{sp}}{t_{fp}} = \frac{4}{0.75} = 5.33333\right) \leq \left(1.49 \cdot \sqrt{\left(\frac{E}{F_{y,pl}}\right)} = 1.49 \cdot \sqrt{\left(\frac{29000}{50}\right)} = 35.884\right) \quad \textbf{(OK)}$$

Check the slenderness of the unstiffened portion of the plate

Unstiffened plate width, $b_{up} = 0.5 \cdot b_{conn} - 0.5 \cdot g$

$= 0.5 \cdot 7 - 0.5 \cdot 4$

$= 1.5\ in$

Modulus of elasticity, $E = 29000\ ksi$

$$\left(\frac{b_{up}}{t_{fp}} = \frac{1.5}{0.75} = 2\right) \leq \left(0.56 \cdot \sqrt{\left(\frac{E}{F_{y,pl}}\right)} = 0.56 \cdot \sqrt{\left(\frac{29000}{50}\right)} = 13.4866\right) \quad \textbf{(OK)}$$

$$Unity = max\left(\frac{\left(\frac{b_{sp}}{t_{fp}}\right)}{1.49 \cdot \sqrt{\left(\frac{E}{F_{y,pl}}\right)}}, \frac{\left(\frac{b_{up}}{t_{fp}}\right)}{0.56 \cdot \sqrt{\left(\frac{E}{F_{y,pl}}\right)}}\right)$$

$$= max\left(\frac{\left(\frac{4}{0.75}\right)}{1.49 \cdot \sqrt{\left(\frac{29000}{50}\right)}}, \frac{\left(\frac{1.5}{0.75}\right)}{0.56 \cdot \sqrt{\left(\frac{29000}{50}\right)}}\right)$$

$= 0.148627$

Shear of support (36). Reference J4.2

Connection depth, $d_{conn} = 9\ in$

Supporting member tensile strength, $F_{u,s} = 65\ ksi$

Supporting member thickness, $t_{sup} = 0.78\ in$

Web axial load, horizontal component, $P_{a,w,h} = 0\ kips$

Applied member shear, $V_a = 42\ kips$

Supporting member yield stress, $F_{y,s} = 50\ ksi$

Shear area, $A_v = 2 \cdot d_{conn} \cdot t_{sup}$

$= 2 \cdot 9 \cdot 0.78$

Shear of support (36). Reference J4.2 (continued)

$= 14.04\ in^2$

$\phi = 0.75$

$\phi = 1$

Gross shear capacity of support, $R_v = min\ (\phi \cdot 0.6 \cdot F_{y,s} \cdot A_v, \phi \cdot 0.6 \cdot F_{u,s} \cdot A_v)$

$= min\ (1 \cdot 0.6 \cdot 50 \cdot 14.04, 0.75 \cdot 0.6 \cdot 65 \cdot 14.04)$

$= 410.67\ kips$

$Unity = \frac{V_a}{R_v}$

$= \frac{42}{410.67}$

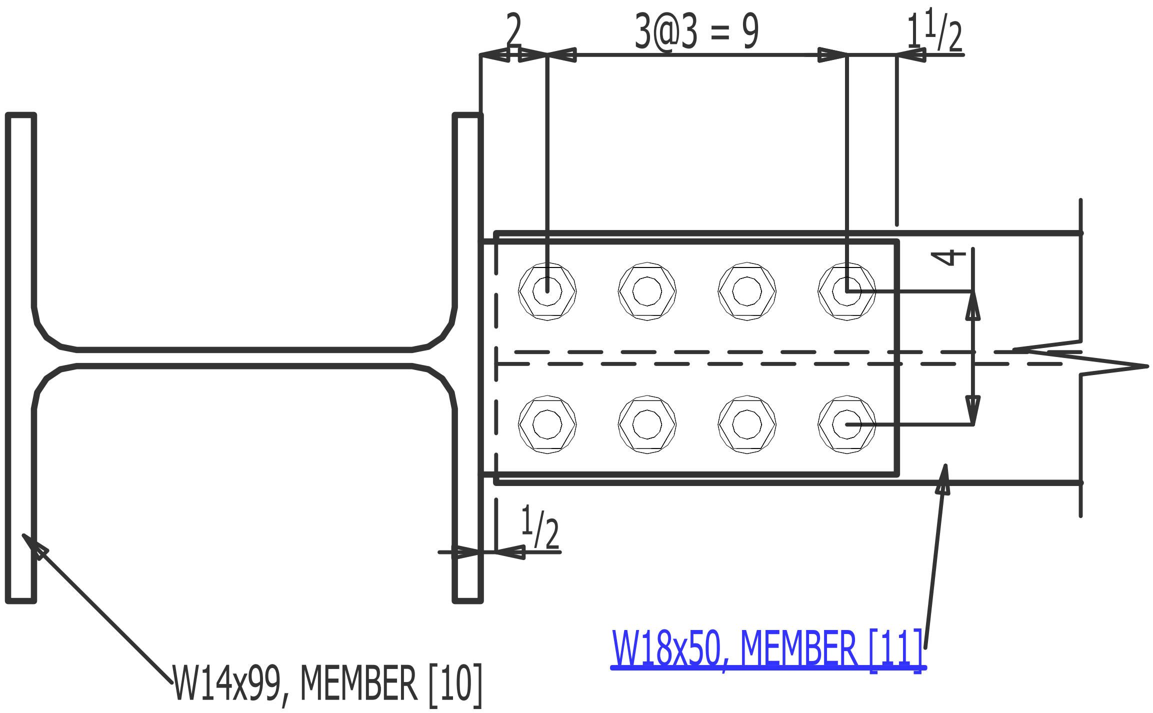
$= 0.102272$

$Shear\ capacity = R_v$

$= 410.67\ kips$

$410.7\ kips \geq 42\ kips$ **(OK)**

$0.102 \leq 1$ **(OK)**


2
3@3 = 9
1 1/2
4
1/2
W14x99, MEMBER [10]
W18x50, MEMBER [11]


A
Section A
Unknown

Design summary for member [11]'s left end

See Page 4

B_11 [11] Connection strength check: left end

See Page 6

Results summary

Shear Moment Plate on left end of Beam B_11 [11]

Beam to column flange moment connection

Minimum column web thickness:	0.421 in
	J10.6, Fig. C-J10.4
d_1:	18.00 in
M_1:	252.00 kip·ft
d_2:	0.00 in
M_2:	0.00 kip·ft
Story shear:	0.00 kips
σ_F:	161.28 ksi
Column resisting moment, $\varphi * F_y / Z_x$:	648.75 kip·ft

Unstiffened column strength

Flange bending:	171.11 kips	(AISC Spec J10.1)
Web yielding:	185.51 kips	(AISC Spec J10.2)
Web crippling:	232.27 kips	(AISC Spec J10.3)
Web buckling:	259.38 kips	(AISC Spec J10.5)
Panel zone shear:	185.95 kips	(AISC Spec J10.6)
Computed flange force:	161.28 kips	

Limit state summary

	Calc. Num.	Unity ratio	PHI*Rn	AISC Ref
Strength of column flange:	83.3	0.943	267.4 kip·ft	J10
Rupture of flange plate to support weld:	212	0.920	274.0 kip·ft	J2, Table J2.5
Tension/comp. of flange plate:	46	0.882	285.6 kip·ft	D,E3,J4
Panel zone shear of column web:	395	0.867	290.5 kip·ft	J10.6
Bolt bearing on flange plate:	69	0.862	292.2 kip·ft	J3.11
Bolt bearing on beam flange:	69	0.862	292.2 kip·ft	J3.11
Bolt shear of flange bolts:	68	0.862	292.2 kip·ft	J,Table J3.2
Flexural rupture of beam:	211	0.793	317.8 kip·ft	F
Crippling of column web:	25	0.694	362.9 kip·ft	J10.3
Shear rupture of plate:	21	0.638	65.8 kips	J4.2
Bolt bearing on plate:	110	0.589	71.3 kips	J3.11
Bolt shear of web bolts:	1	0.575	73.1 kips	J3.7, J3.9
Bolt bearing on beam web:	110	0.575	73.1 kips	J3.11
Block shear rupture of beam flange:	259	0.570	441.8 kip·ft	J4.3

Limit state summary (continued)

Block shear rupture of flange plate:	85	0.538	468.5 kip·ft	J4.3
Block shear rupture of plate:	6	0.513	81.8 kips	J4.3
Shear yielding of plate:	15	0.415	101.3 kips	J4.2
Shear yielding of beam web:	2	0.219	191.7 kips	G2.1
Flange plate width to thickness ratio:	461	0.149	NA	B4.1
Shear of support:	36	0.102	410.7 kips	J4.2

Connection strength

	Value:	Unity ratio:
Shear:	65.8 kips	0.638
Moment:	267.4 kip·ft	0.943
Panel moment:	290.5 kip·ft	0.867

Notes and conclusions

- Tab weld sized to develop the full plate strength.
- Eccentricity is neglected in the shear connection, misc note 33.
- Column checked for web doublers.
- Column checked for stiffeners.
- CONNECTION IS OK
    - Strength equals or exceeds design loads.