SDS2 Steel Connection Design: Connection Cube Report

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

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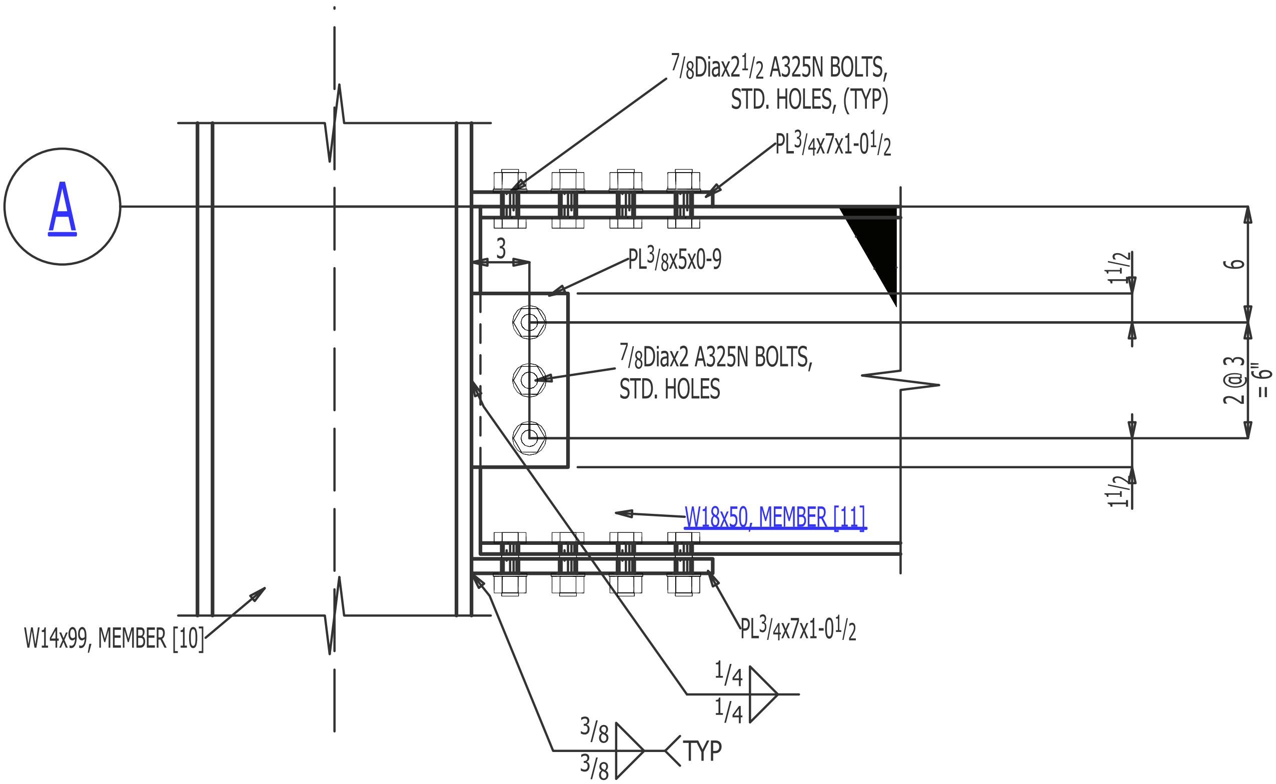
Ex. II.B-1 [5] at X=175-0, Y=100-0 Elev=-9



ELEVATION VIEW

Beam B_11 [11]

Design method

- AISC Steel Construction Manual, Sixteenth Edition (ASD)
- 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:	28.0 kips
Story shear:	0.0 kips
Moment:	168.0 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

Design loads

Shear:	28.0 kips
Moment:	168.0 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 66.7 % of the allowable plastic moment, Mp/OMEGA.

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, $R_n = \left(t_{fp} + 5 \cdot k_s\right) \cdot t_{w,s} \cdot F_{yw,s}$

$= (0.75 + 5 \cdot 1.38) \cdot 0.485 \cdot 50$

$= 185.512\ kips$

$\Omega = 1.5$

Web local yielding strength, $\frac{P_{f,yield}}{\Omega} = \frac{R_n}{\Omega}$

$= \frac{185.512}{1.5}$

$= 123.675\ kips$

Calculate flange local bending capacity

$\Omega = 1.67$

Flange local bending strength, $\frac{P_{f,bend}}{\Omega} = \frac{6.25 \cdot t_{f,s}^2 \cdot F_{yf,s}}{\Omega}$

$= \frac{6.25 \cdot 0.78^2 \cdot 50}{1.67}$

$= 113.847\ kips$

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, $M_a = 168\ kip \cdot ft$

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

$= \frac{\left(\left(\frac{|168|}{(18 + 0.75)}\right) \cdot 12 + max\ (0,0)\right)}{(123.675 + 0)}$

$= 0.869375$

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

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

$= \dfrac{\left(\left(\dfrac{|168|}{(18 + 0.75)}\right) \cdot 12 + 0\right)}{(113.847 + 0)}$

$= 0.944423$

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

$= max\ (0.869375, 0.944423)$

$= 0.944423$

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

$= min\ (123.675 - max\ (0,0) - 0)$

$= 113.847\ kips$

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

$= \dfrac{113.847 \cdot (18 + 0.75)}{12}$

$= 177.886\ kip \cdot ft$

$177.9\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(OK)**

$0.944 \leq 1$ **(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, $M_a = 168\ kip \cdot ft$

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

$= 1.5$

$\Omega = 2$

$F_{EXX} = 70\ ksi$

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

$= \dfrac{0.6 \cdot 70 \cdot 1.5}{2}$

$= 31.5\ ksi$

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

$\Omega = 2$

$F_{EXX} = 70\ ksi$

$\Omega = 2$

Maximum effective weld size, $w_{e,max} = \dfrac{\left(\dfrac{0.6 \cdot F_{u,pl} \cdot t_{pl}}{\Omega}\right)}{\left(\dfrac{2 \cdot 0.707 \cdot 0.6 \cdot F_{EXX}}{\Omega}\right)}$

$= \dfrac{\left(\dfrac{0.6 \cdot 65 \cdot 0.75}{2}\right)}{\left(\dfrac{2 \cdot 0.707 \cdot 0.6 \cdot 70}{2}\right)}$

$= 0.492524\ in$

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, $\dfrac{R_f}{\Omega} = \dfrac{F_w}{\Omega} \cdot t_{eff} \cdot W_{pl}$

$= 31.5 \cdot 0.53025 \cdot 7$

$= 116.92\ kips$

Applied member moment, $M_a = 168\ kip \cdot ft$

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

$= \left(\dfrac{\left(\dfrac{|168|}{(18 + 0.75)}\right)}{116.92}\right) \cdot 12$

$= 0.919602$

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

$Moment\ capacity = \dfrac{\left(\dfrac{R_f}{\Omega} - T_{a,f,h}\right) \cdot (d + t_{pl})}{12}$

$= \dfrac{(116.92 - 0) \cdot (18 + 0.75)}{12}$

$= 182.688\ kip \cdot ft$

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

$182.7\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(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, $w_p = 7\ in$

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, $A_n = t_{pl} \cdot (w_p - 2 \cdot d_h)$

$= 0.75 \cdot (7 - 2 \cdot 1)$

$= 3.75\ in^2$

$\Omega = 2$

$\Omega = 1.67$

Remaining tension capacity in plate, $\frac{T}{\Omega} = min\left(\frac{F_{y,pl} \cdot A_g}{\Omega}, \frac{F_u \cdot A_n}{\Omega}\right) - T_{a,f,h}$

$= min\left(\frac{50 \cdot 5.25}{1.67}, \frac{65 \cdot 3.75}{2}\right) - 0$

$= 121.875\ kips$

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, $F_n = F_{y,pl}$

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

$= 50\ ksi$

$\Omega = 1.67$

Remaining compression capacity in plate, $\frac{C}{\Omega} = \frac{F_n \cdot A_g}{\Omega} - C_{a,f,h}$

$= \frac{50 \cdot 5.25}{1.67} - 0$

$= 157.186\ kips$

$$Moment\ capacity = \frac{min\left(\frac{T}{\Omega}, \frac{C}{\Omega}\right) \cdot \left(d + t_{pl}\right)}{12}$$

$$= \frac{min\ (121.875, 157.186) \cdot (18 + 0.75)}{12}$$

$= 190.43\ kip \cdot ft$

Applied member moment, $M_a = 168\ kip \cdot ft$

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

$= \frac{|168|}{190.433}$

$= 0.882198$

$190.4\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(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, $A_w = d_c \cdot t_{w,eq}$

$= 14.2 \cdot 0.485$

$= 6.887\ in^2$

$\Omega = 1.67$

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

$= \frac{0.6 \cdot 50}{1.67}$

$= 17.9641\ ksi$

Web panel zone capacity, $R_v = \frac{F_v}{\Omega} \cdot A_w$

$= 17.9641 \cdot 6.887$

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

$= 123.719\ kips$

Allowable flange force, $F_f = R_v - V_s$

$= 123.719 - 0$

$= 123.719\ kips$

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

$$= \frac{123.719 \cdot (18 + 0.75)}{12}$$

$= 193.31\ kip \cdot ft$

Applied panel moment, $M_{a,z,p} = 168\ kip \cdot ft$

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

$$= \frac{|168|}{193.308}$$

$= 0.869078$

$193.3\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(OK)**

$0.869 \leq 1$ **(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, $F_{nv} = 54\ ksi$

$\Omega = 2$

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

$$= \frac{54 \cdot 0.60132 \cdot 1 \cdot 1}{2}$$

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

$= 16.2357\ kips$

Hole diameter, $d_h = 0.9375\ in$

$\Omega = 2$

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

$= \frac{2.4 \cdot 0.875 \cdot 0.75 \cdot 65}{2}$

$= 51.1875\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$

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

$= 3 - 0.9375$

$= 2.0625\ in$

$\Omega = 2$

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

$= \frac{1.2 \cdot 2.0625 \cdot 0.75 \cdot 65}{2}$

$= 60.3281\ kips$

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

$= min\ (60.3281, 51.1875, 16.2357)$

$= 16.2357\ kips$

Edge bolt capacity

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

$= 1.5 - 0.5 \cdot 0.9375$

$= 1.03125\ in$

$\Omega = 2$

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

$= \frac{1.2 \cdot 1.03125 \cdot 0.75 \cdot 65}{2}$

$= 30.1641\ kips$

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

$= min\ (30.1641, 51.1875, 16.2357)$

$= 16.2357\ kips$

Number of edge bolts, $N_e = m$

$= 2$

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

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

$= 2 \cdot 4 - 2$

$= 6$

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

$= 16.2357 \cdot 2 + 16.2357 \cdot 6$

$= 129.885\ kips$

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

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

$= \frac{(129.885 - 0) \cdot 18}{12}$

$= 194.828\ kip \cdot ft$

Applied member moment, $M_a = 168\ kip \cdot ft$

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

$= \frac{|168|}{194.825}$

$= 0.862312$

$194.8\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(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, $F_{nv} = 54\ ksi$

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

$\Omega = 2$

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

$= \frac{54 \cdot 0.60132 \cdot 1 \cdot 1}{2}$

$= 16.2357\ kips$

Hole diameter, $d_h = 0.9375\ in$

$\Omega = 2$

Bolt bearing capacity, $\frac{R_{n,b}}{\Omega} = \frac{2.4 \cdot d_b \cdot t_f \cdot F_{u,b}}{\Omega}$

$= \frac{2.4 \cdot 0.875 \cdot 0.57 \cdot 65}{2}$

$= 38.9025\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$

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

$= 3 - 0.9375$

$= 2.0625\ in$

$\Omega = 2$

Tearout load capacity, $\frac{R_{n,to}}{\Omega} = \frac{1.2 \cdot L_{c,int} \cdot t_f \cdot F_{u,b}}{\Omega}$

$= \frac{1.2 \cdot 2.0625 \cdot 0.57 \cdot 65}{2}$

$= 45.8494\ kips$

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

$= min\ (45.8494, 38.9025, 16.2357)$

$= 16.2357\ kips$

Edge bolt capacity

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

$= 1.5375 - 0.5 \cdot 0.9375$

$= 1.06875\ in$

$\Omega = 2$

Tearout load capacity, $\frac{R_{n,to}}{\Omega} = \frac{1.2 \cdot L_{c,edge} \cdot t_f \cdot F_{u,b}}{\Omega}$

$= \frac{1.2 \cdot 1.06875 \cdot 0.57 \cdot 65}{2}$

$= 23.7583\ kips$

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

$= min\ (23.7583, 38.9025, 16.2357)$

Edge bolt capacity (continued)

$= 16.2357\ kips$

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, $\frac{F_{f,allow}}{\Omega} = \frac{R_{n,e}}{\Omega} \cdot N_e + \frac{R_{n,i}}{\Omega} \cdot N_i$

$= 16.2357 \cdot 2 + 16.2357 \cdot 6$

$= 129.885\ kips$

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

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

$= \frac{(129.885 - 0) \cdot 18}{12}$

$= 194.828\ kip \cdot ft$

Applied member moment, $M_a = 168\ kip \cdot ft$

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

$= \frac{|168|}{194.825}$

$= 0.862312$

$194.8\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(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, $M_a = 168\ kip \cdot ft$

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

Bolt area, $A_b = 0.60132\ in^2$
Allowable shear stress, $F_{nv} = 54\ ksi$
$\Omega = 2$

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

$= \frac{54 \cdot 0.60132 \cdot 1 \cdot 1}{2}$

$= 16.2357\ kips$

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

$$= \left(\frac{\left(\frac{|168|}{18} \right)}{16.2357 \cdot 4 \cdot 2} \right) \cdot 12$$

$= 0.8623$

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

$= \frac{16.2357 \cdot 4 \cdot 2 \cdot 18}{12}$

$= 194.828\ kip \cdot ft$

$194.8\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(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, $M_a = 168\ kip \cdot ft$
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, $A_{fg} = b_f \cdot t_f$

$= 7.5 \cdot 0.57$

$= 4.275\ in^2$

Calculate the strong axis moment capacity (continued)

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

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

$= 3.135\ in^2$

Hole reduction coefficient, $Y_t = 1$

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

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

$= 353.131\ kip \cdot ft$

$\Omega = 1.67$

Allowable moment, $\dfrac{M}{\Omega} = \dfrac{M_n}{\Omega}$

$= \dfrac{353.131}{1.67}$

$= 211.455\ kip \cdot ft$

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

$= \dfrac{|168|}{211.455}$

$= 0.794494$

$Moment\ capacity = \dfrac{M}{\Omega}$

$= 211.455\ kip \cdot ft$

$211.5\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(OK)**

$0.794 \leq 1$ **(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, $E = 29000\ ksi$

Chord-stress interaction parameter, $Q_f = 1$

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

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

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

$= 309.69\ kips$

$\Omega = 2$

Allowable flange force, $F_f = \frac{R_n}{\Omega}$

$= \frac{309.69}{2}$

$= 154.845\ kips$

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

$= \frac{154.845 \cdot (18 + 0.75)}{12}$

$= 241.946\ kip \cdot ft$

Applied member moment, $M_a = 168\ kip \cdot ft$

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

$= \frac{|168|}{241.942}$

$= 0.694382$

$241.9\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(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, $A_{nv} = t_{conn} \cdot (d_{pl} - n \cdot d_h)$

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

$= 2.25\ in^2$

$\Omega = 2$

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

$= \frac{0.6 \cdot 65 \cdot 2.25}{2}$

$= 43.875\ kips$

$Shear\ capacity = \frac{V_n}{\Omega}$

$= 43.875\ kips$

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

Applied member shear, $V_a = 28\ kips$

$$Unity = \frac{V_a}{Shear\ capacity}$$
$$= \frac{28}{43.9}$$
$$= 0.637813$$

$43.9\ kips \geq 28\ kips$ **(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, $F_{nv} = 54\ ksi$

$\Omega = 2$

Bolt shear capacity, $\frac{R_{n,v}}{\Omega} = \frac{F_{nv} \cdot A_b \cdot N_s}{\Omega}$
$$= \frac{54 \cdot 0.60132 \cdot 1}{2}$$
$$= 16.2357\ kips$$

Hole diameter, $d_h = 0.9375\ in$

$\Omega = 2$

Bolt bearing capacity, $\frac{R_{n,b}}{\Omega} = \frac{2.4 \cdot d_b \cdot t_{pl} \cdot F_u}{\Omega}$
$$= \frac{2.4 \cdot 0.875 \cdot 0.375 \cdot 65}{2}$$
$$= 25.5938\ kips$$

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)

$\Omega = 2$

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

$= \frac{1.2 \cdot 2.0625 \cdot 0.375 \cdot 65}{2}$

$= 30.1641\ kips$

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

$= min\ (30.1641, 25.5938, 16.2357)$

$= 16.2357\ kips$

Edge bolt capacity

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

$= 1.5 - 0.5 \cdot 0.9375$

$= 1.03125\ in$

$\Omega = 2$

Tearout load capacity, $\frac{R_{n,to}}{\Omega} = \frac{1.2 \cdot L_{c,edge} \cdot t_{pl} \cdot F_u}{\Omega}$

$= \frac{1.2 \cdot 1.03125 \cdot 0.375 \cdot 65}{2}$

$= 15.082\ kips$

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

$= min\ (15.082, 25.5938, 16.2357)$

$= 15.082\ kips$

Number of edge bolts, $N_e = m$

$= 1$

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

$= 1 \cdot 3 - 1$

$= 2$

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

$= 15.082 \cdot 1 + 16.2357 \cdot 2$

$= 47.5533\ kips$

Applied member shear, $V_a = 28\ kips$

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

$= \frac{28}{47.6}$

$= 0.588235$

$47.6\ kips \geq 28\ kips$ **(OK)**

$0.588 \leq 1$ **(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, $F_{nv} = 54\ ksi$
$\Omega = 2$

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

$= \frac{54 \cdot 0.60132 \cdot 1}{2}$

$= 16.2357\ kips$

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

$= 16.2357 \cdot 3 \cdot 1$

$= 48.707\ kips$

Applied member shear, $V_a = 28\ kips$

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

$= \frac{28}{48.7}$

$= 0.574949$

$48.7\ kips \geq 28\ kips$ **(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, $F_{nv} = 54\ ksi$
$\Omega = 2$

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

$= \frac{54 \cdot 0.60132 \cdot 1}{2}$

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

$= 16.2357\ kips$

Hole diameter, $d_h = 0.9375\ in$

$\Omega = 2$

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

$= \frac{2.4 \cdot 0.875 \cdot 0.355 \cdot 65}{2}$

$= 24.2288\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$

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

$= 3 - 0.9375$

$= 2.0625\ in$

$\Omega = 2$

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

$= \frac{1.2 \cdot 2.0625 \cdot 0.355 \cdot 65}{2}$

$= 28.5553\ kips$

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

$= min\ (28.5553, 24.2288, 16.2357)$

$= 16.2357\ kips$

Edge bolt capacity

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

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

$= min\ (24.2288, 16.2357)$

$= 16.2357\ kips$

Number of edge bolts, $N_e = m$

$= 1$

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

$= 1 \cdot 3 - 1$

$= 2$

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

$= 16.2357 \cdot 1 + 16.2357 \cdot 2$

$= 48.707\ kips$

Applied member shear, $V_a = 28\ kips$

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

$Unity = \dfrac{V_a}{Shear\ capacity}$
$= \dfrac{28}{48.7}$
$= 0.574949$

$48.7\ kips \geq 28\ kips$ **(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, $T_{a,f,M} = 115.663\ kips$
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, $R_{gv} = 0.6 \cdot F_y \cdot A_{gv}$
$= 0.6 \cdot 50 \cdot 12.0128$
$= 360.382\ kips$

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

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, $R_n = min\ (R_{gv}, R_{nv}) + R_t$

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

$= 405.512\ kips$

$\Omega = 2$

Flange block shear strength, $\frac{R_{bs}}{\Omega} = \frac{R_n}{\Omega}$

$= \frac{405.512}{2}$

$= 202.756\ kips$

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

$= \frac{(0 + 115.663)}{202.756}$

$= 0.570452$

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

$= \frac{202.756 \cdot (18 - 0.57)}{12}$

$= 294.503\ kip \cdot ft$

$294.5\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(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, $F_y = 50\ ksi$

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

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

$= 0.6 \cdot 50 \cdot 15.75$

$= 472.5\ kips$

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, $R_n = min\ (R_{gv}, R_{nv}) + R_t$

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

$= 555.75\ kips$

$\Omega = 2$

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

$= \dfrac{555.75}{2}$

$= 277.875\ kips$

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

$= 0.6 \cdot 65 \cdot 10.5$

$= 409.5\ kips$

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, $R_n = min\ (R_{gv}, R_{nv}) + R_t$

$= min\ (472.5, 409.5) + 97.5$

$= 507\ kips$

$\Omega = 2$

Block shear capacity (2 L-shaped patterns), $\frac{R_{bs2}}{\Omega} = \frac{R_n}{\Omega}$

$= \frac{507}{2}$

$= 253.5\ kips$

L-shaped failure pattern

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

Connection length, $L = s_{total}$

$= 9\ in$

L-shaped failure pattern (continued)

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, $R_n = min\ (R_{gv}, R_{nv}) + R_t$

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

$= 399.75\ kips$

$\Omega = 2$

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

$= \frac{399.75}{2}$

$= 199.875\ kips$

Applied member moment, $M_a = 168\ kip \cdot ft$

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

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

$$= \frac{\left(\left(\frac{|168|}{(18 + 0.75)}\right) \cdot 12 + 0\right)}{min\ (277.875, 253.5, 199.875)}$$

$$= 0.537936$$

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

$$= \frac{min\ (277.875, 253.5, 199.875) \cdot (18 + 0.75)}{12}$$

$$= 312.305\ kip \cdot ft$$

$312.3\ kip \cdot ft \geq (|168| = 168\ kip \cdot ft)$ **(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, $A_{gv} = t_{pl} \cdot (s_{total} + L_{ev})$

$$= 0.375 \cdot (6 + 1.5)$$

$$= 2.8125\ in^2$$

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

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

$$= 0.553125\ in^2$$

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

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, $R_n = min\ (R_{gv}, R_{nv}) + R_t$
$= min\ (84.375, 73.125) + 35.9531$
$= 109.078\ kips$
$\Omega = 2$
$Shear\ capacity = \frac{R_n}{\Omega}$
$= \frac{109.078}{2}$
$= 54.5391\ kips$
Applied member shear, $V_a = 28\ kips$
$Unity = \frac{V_a}{Shear\ capacity}$
$= \frac{28}{54.5}$
$= 0.513761$
$54.5\ kips \geq 28\ kips$ **(OK)**
$0.514 \leq 1$ **(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, $L = 9\ in$
Gross shear area, $A_{gv} = t \cdot L$
$= 0.375 \cdot 9$
$= 3.375\ in^2$
$\Omega = 1.5$
$Shear\ capacity = \frac{0.6 \cdot F_{y,conn} \cdot A_{gv}}{\Omega}$
$= \frac{0.6 \cdot 50 \cdot 3.375}{1.5}$
$= 67.5\ kips$
Applied member shear, $V_a = 28\ kips$

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

$$Unity = \frac{V_a}{Shear\ capacity}$$
$$= \frac{28}{67.5}$$
$$= 0.414815$$

$67.5\ kips \geq 28\ kips$ **(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, $V_a = 28\ kips$

$\Omega = 1.5$

Allowable shear stress, $\frac{F_v}{\Omega} = \frac{0.6 \cdot F_y}{\Omega}$
$$= \frac{0.6 \cdot 50}{1.5}$$
$$= 20\ ksi$$

Web shear area, $A_w = d \cdot t_w$
$$= 18 \cdot 0.355$$
$$= 6.39\ in^2$$

$$Unity = \frac{V_a}{\frac{F_v}{\Omega} \cdot A_w}$$
$$= \frac{28}{20 \cdot 6.39}$$
$$= 0.219092$$

$$Shear\ capacity = \frac{F_v}{\Omega} \cdot A_w$$
$$= 20 \cdot 6.39$$
$$= 127.8\ kips$$

$127.8\ kips \geq 28\ kips$ **(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, $V_a = 28\ kips$

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

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

$= 2 \cdot 9 \cdot 0.78$

$= 14.04\ in^2$

$\Omega = 2$

$\Omega = 1.5$

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

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

$$= min\left(\frac{0.6 \cdot 50 \cdot 14.04}{1.5}, \frac{0.6 \cdot 65 \cdot 14.04}{2}\right)$$

$$= 273.78\ kips$$

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

$$= \frac{28}{273.78}$$

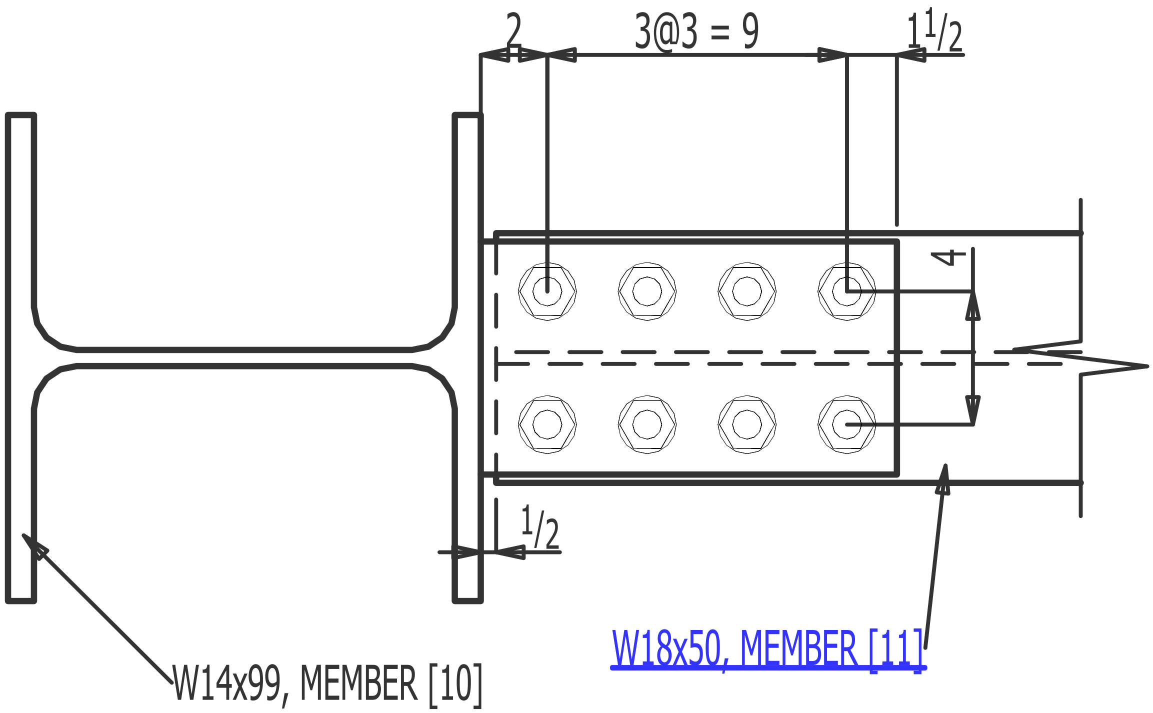
$$= 0.102272$$

$$Shear\ capacity = R_v$$

$$= 273.78\ kips$$

$273.8\ kips \geq 28\ kips$ **(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:	168.00 kip·ft
d_2:	0.00 in
M_2:	0.00 kip·ft
Story shear:	0.00 kips
σ_F:	107.52 ksi
Column resisting moment, F_y*Z_x/ω:	431.64 kip·ft

Unstiffened column strength

Flange bending:	113.85 kips	(AISC Spec J10.1)
Web yielding:	123.67 kips	(AISC Spec J10.2)
Web crippling:	154.85 kips	(AISC Spec J10.3)
Web buckling:	172.58 kips	(AISC Spec J10.5)
Panel zone shear:	123.72 kips	(AISC Spec J10.6)
Computed flange force:	107.52 kips	

Limit state summary

	Calc. Num.	Unity ratio	Rn/OMEGA	AISC Ref
Strength of column flange:	83.3	0.944	177.9 kip·ft	J10
Rupture of flange plate to support weld:	212	0.920	182.7 kip·ft	J2, Table J2.5
Tension/comp. of flange plate:	46	0.882	190.4 kip·ft	D,E3,J4
Panel zone shear of column web:	395	0.869	193.3 kip·ft	J10.6
Bolt bearing on flange plate:	69	0.862	194.8 kip·ft	J3.11
Bolt bearing on beam flange:	69	0.862	194.8 kip·ft	J3.11
Bolt shear of flange bolts:	68	0.862	194.8 kip·ft	J,Table J3.2
Flexural rupture of beam:	211	0.794	211.5 kip·ft	F
Crippling of column web:	25	0.694	241.9 kip·ft	J10.3
Shear rupture of plate:	21	0.638	43.9 kips	J4.2
Bolt bearing on plate:	110	0.588	47.6 kips	J3.11
Bolt shear of web bolts:	1	0.575	48.7 kips	J3.7, J3.9
Bolt bearing on beam web:	110	0.575	48.7 kips	J3.11
Block shear rupture of beam flange:	259	0.570	294.5 kip·ft	J4.3

Limit state summary (continued)

Block shear rupture of flange plate:	85	0.538	312.3 kip·ft	J4.3
Block shear rupture of plate:	6	0.514	54.5 kips	J4.3
Shear yielding of plate:	15	0.415	67.5 kips	J4.2
Shear yielding of beam web:	2	0.219	127.8 kips	G2.1
Flange plate width to thickness ratio:	461	0.149	NA	B4.1
Shear of support:	36	0.102	273.8 kips	J4.2

Connection strength

	Value:	Unity ratio:
Shear:	43.9 kips	0.638
Moment:	177.9 kip·ft	0.944
Panel moment:	193.3 kip·ft	0.869

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.