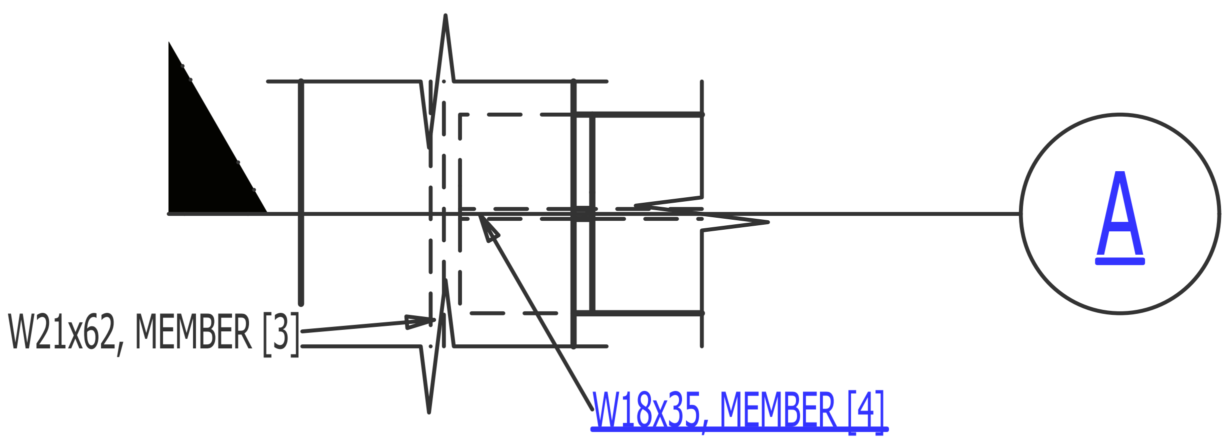
SDS2 Steel Connection Design: Connection Cube Report

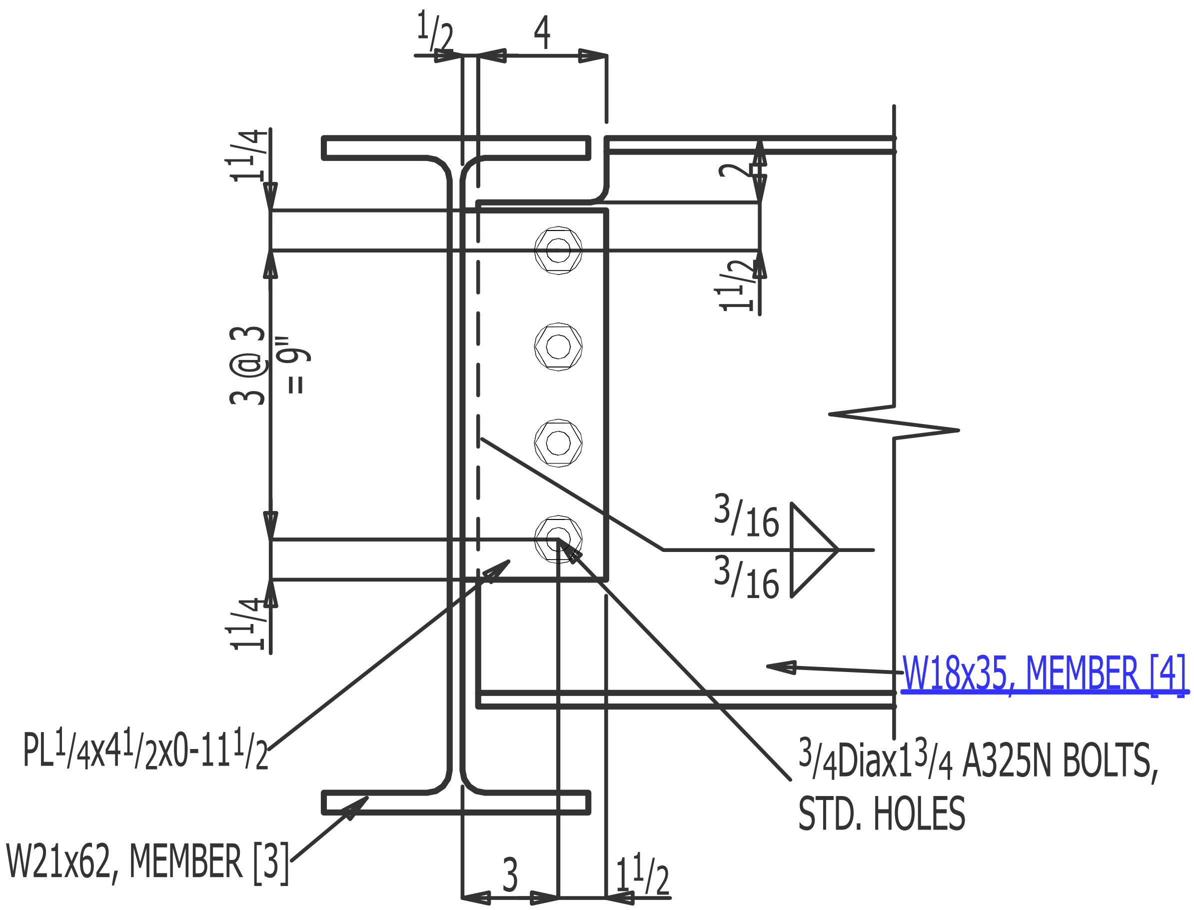
Cube: Ex. II.A-18
Revision: 0
Project: ASD16ValidationExamples
Engineer:
Fabricator: ASD16ValidationExamples

Generated by SDS2 x 2025.03 on Friday, Oct 11, 2024

Ex. II.A-18 [2] at X=25-0, Y=25-0 Elev=-8 7/8



TOP SIDE VIEW


1/2
4
1 1/4
2
1 1/2
3 @ 3
= 9"
3/16
3/16
1 1/4
W18x35, MEMBER [4]
PL1/4x4 1/2x0-11 1/2
3/4Diax1 3/4 A325N BOLTS,
STD. HOLES
W21x62, MEMBER [3]
3
1 1/2


A
Section A
ELEVATION

Beam B_4 [4]

Design method

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

Overview

Section size:	W18x35
Sequence:	1
ABM:	N/Assign
Plan length:	25-0
Camber:	0.00 in
Span length:	25-0
Slope:	0.00 °
Material length:	24-11 5/16
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	17.7 in
Web thickness, t_w:	0.3 in
Flange width, b_f:	6 in
Flange thickness, t_f:	0.425 in
Design k distance, k_{des}:	0.827 in
Detail k distance, k_{det}:	1.125 in
Distance between web toes of fillets, T:	15.45 in
Moment of inertia about the major axis, I_x:	510 in^4

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: Automatic
	One bolt column
	Bevel shear tab: Automatic
	Attach to: Supporting
Elevation:	0
Minus Dim:	0.6875 in
Mtrl Setback:	0.6875 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	26.5 kips
Moment:	0.0 kip·ft (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_4 [4] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_3 [3]
Section size:	W21x62
End 0 elevation:	0
End 1 elevation:	0
Support intersection elevation:	0
Supporting beam rotation:	0.00 degrees
	(looking toward left end)
Material grade:	A992
Detail k distance, k_{det}:	1.3125 in
Design k distance, k_{des}:	1.12 in
Supporting member thickness, t_{sup}:	0.4 in

Design loads

$Shear$:	26.5 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 49.9 % of the allowable uniform steel beam load.

Connection summary

- SINGLE PLATE SHEAR CONNECTION

Connection details

Plate:	**Grade:**	A572-50
	Tensile strength, F_u:	65 ksi
	Yield stress, F_y:	50 ksi
	Thickness, t:	0.25 in
	Width, b:	4.475 in
	Depth, d:	11.5 in
	Weld line to bolt group c.g., a:	2.9875 in
Weld:	**Weld type:**	Double fillet
	Weld leg size, w:	0.1875 in
	Total effective weld throat, t_e:	0.27 in
	Weld metal strength, F_{exx}:	70 ksi
Bolts:	**Bolt type:**	A325N
	Hole type in connection:	Standard round
	Bolt diameter, d_b:	3/4
	Bolt rows, n:	4
	Bolt row spacing, s:	3 in
	Bolt columns, m:	1
	Web end distance, $L_{e,w}$:	2.5 in
Connection geometry:	**Dihedral angle, θ:**	90.00 °

Connection design lock summary

Locked Via Member Edit:	16
(at dd) Not Locked:	263

Cope information

Top cope depth, d_{ct}:	2 in
Top cope length, C_t:	4 in

Cope notes

- Cope length dimension is from the end of the beam web.
- At coped section : S_{net} = 18.20 in^3, h_o = 15.70 in AISC Table 9-2
- L_h = 2.50 in, L_v = 1.50 in

Expanded design calculation

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

Connection tensile strength, $F_u = 65\ ksi$
Bolt rows, $n = 4$
Connection thickness, $t_{conn} = 0.25\ in$
Connection depth, $d_{pl} = 11.5\ in$
Hole diameter, $d_h = 0.875\ in$
Net shear area, $A_{nv} = t_{conn} \cdot (d_{pl} - n \cdot d_h)$
$= 0.25 \cdot (11.5 - 4 \cdot 0.875)$
$= 2\ in^2$
$\Omega = 2$
Shear capacity, $\frac{V_n}{\Omega} = \frac{0.6 \cdot F_u \cdot A_{nv}}{\Omega}$
$= \frac{0.6 \cdot 65 \cdot 2}{2}$
$= 39\ kips$
$Shear\ capacity = \frac{V_n}{\Omega}$
$= 39\ kips$
Applied member shear, $V_a = 26.5\ kips$
$Unity = \frac{V_a}{Shear\ capacity}$
$= \frac{26.5}{39}$
$= 0.679487$
$39.0\ kips \geq 26.5\ kips$ **(OK)**
$0.679 \leq 1$ **(OK)**

Bolt shear of web bolts (3). Reference J3.7, J3.9

Number of shear planes, $N_s = 1$
Coefficient, $C = 3.56171$
Bolt area, $A_b = 0.441786\ in^2$
Allowable shear stress, $F_{nv} = 54\ ksi$
$\Omega = 2$
Bolt shear capacity, $\frac{R_{n,v}}{\Omega} = \frac{F_{nv} \cdot A_b \cdot N_s}{\Omega}$
$= \frac{54 \cdot 0.441786 \cdot 1}{2}$
$= 11.9282\ kips$
$Shear\ capacity = C \cdot \frac{R_{n,v}}{\Omega}$
$= 3.56171 \cdot 11.9282$
$= 42.485\ kips$
Applied member shear, $V_a = 26.5\ kips$

Bolt shear of web bolts (3). Reference J3.7, J3.9 (continued)

$Unity = \frac{V_a}{Shear\ capacity}$
$= \frac{26.5}{42.5}$
$= 0.623529$

$42.5\ kips \geq 26.5\ kips$ **(OK)**

$0.624 \leq 1$ **(OK)**

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

Plate thickness, $t_{pl} = 0.25\ in$
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, $n = 4$
Column edge distance, $L_{eh} = 1.4875\ in$
Row edge distance, $L_{ev} = 1.25\ in$
Bolt columns, $m = 1$
Hole diameter, $d_h = 0.875\ in$
Hole length, $l_h = 0.875\ in$
Total length of bolt group, $s_{total} = 9\ in$
Gross shear area, $A_{gv} = t_{pl} \cdot (s_{total} + L_{ev})$
$= 0.25 \cdot (9 + 1.25)$
$= 2.5625\ in^2$
Net shear area, $A_{nv} = t_{pl} \cdot (s_{total} + L_{ev}) - t_{pl} \cdot (n - 0.5) \cdot d_h$
$= 0.25 \cdot (9 + 1.25) - 0.25 \cdot (4 - 0.5) \cdot 0.875$
$= 1.79688\ in^2$
Gross tensile area, $A_{gt} = t_{pl} \cdot (s_{col} \cdot (m - 1) + L_{eh})$
$= 0.25 \cdot (0 \cdot (1 - 1) + 1.4875)$
$= 0.371875\ in^2$
Net tensile area, $A_{nt} = t_{pl} \cdot (s_{col} \cdot (m - 1) + L_{eh}) - t_{pl} \cdot (m - 0.5) \cdot l_h$
$= 0.25 \cdot (0 \cdot (1 - 1) + 1.4875) - 0.25 \cdot (1 - 0.5) \cdot 0.875$
$= 0.2625\ in^2$
Reduction coefficient, $U_{bs} = 1$
Shear yield load, $R_{gv} = 0.6 \cdot F_y \cdot A_{gv}$
$= 0.6 \cdot 50 \cdot 2.5625$
$= 76.875\ kips$
Shear rupture load, $R_{nv} = 0.6 \cdot F_u \cdot A_{nv}$
$= 0.6 \cdot 65 \cdot 1.79688$
$= 70.0781\ kips$

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

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

$= 1 \cdot 65 \cdot 0.2625$

$= 17.0625\ kips$

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

$= min\ (76.875, 70.0781) + 17.0625$

$= 87.1406\ kips$

$\Omega = 2$

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

$= \frac{87.1406}{2}$

$= 43.5703\ kips$

Applied member shear, $V_a = 26.5\ kips$

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

$= \frac{26.5}{43.6}$

$= 0.607798$

$43.6\ kips \geq 26.5\ kips$ **(OK)**

$0.608 \leq 1$ **(OK)**

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

Tensile strength, $F_u = 65\ ksi$

Plate thickness, $t_{pl} = 0.25\ in$

Bolt row spacing, $s = 3\ in$

Row edge distance, $L_{ev} = 1.25\ in$

Bolt diameter, $d_b = 0.75\ in$

Number of shear planes, $N_s = 1$

Bolt columns, $m = 1$

Bolt rows, $n = 4$

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

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

$= 9\ in$

Bolt area, $A_b = 0.441786\ in^2$

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

$\Omega = 2$

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

$= \frac{54 \cdot 0.441786 \cdot 1}{2}$

$= 11.9282\ kips$

Hole diameter, $d_h = 0.8125\ in$

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

$\Omega = 2$

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

$= \frac{2.4 \cdot 0.75 \cdot 0.25 \cdot 65}{2}$

$= 14.625\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$

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

$= 3 - 0.8125$

$= 2.1875\ in$

$\Omega = 2$

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

$= \frac{1.2 \cdot 2.1875 \cdot 0.25 \cdot 65}{2}$

$= 21.3281\ kips$

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

$= min\ (21.3281, 14.625, 11.9282)$

$= 11.9282\ kips$

Edge bolt capacity

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

$= 1.25 - 0.5 \cdot 0.8125$

$= 0.84375\ in$

$\Omega = 2$

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

$= \frac{1.2 \cdot 0.84375 \cdot 0.25 \cdot 65}{2}$

$= 8.22656\ kips$

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

$= min\ (8.22656, 14.625, 11.9282)$

$= 8.22656\ kips$

Number of edge bolts, $N_e = m$

$= 1$

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

$= 1 \cdot 4 - 1$

$= 3$

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

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

$= 8.22656 \cdot 1 + 11.9282 \cdot 3$

$= 44.0113\ kips$

Applied member shear, $V_a = 26.5\ kips$

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

$= \frac{26.5}{44}$

$= 0.602273$

$44.0\ kips \geq 26.5\ kips$ **(OK)**

$0.602 \leq 1$ **(OK)**

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

Tensile strength, $F_u = 65\ ksi$

Plate thickness, $t_{pl} = 0.3\ in$

Bolt row spacing, $s = 3\ in$

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

Bolt diameter, $d_b = 0.75\ in$

Number of shear planes, $N_s = 1$

Bolt columns, $m = 1$

Bolt rows, $n = 4$

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

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

$= 9\ in$

Bolt area, $A_b = 0.441786\ in^2$

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

$\Omega = 2$

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

$= \frac{54 \cdot 0.441786 \cdot 1}{2}$

$= 11.9282\ kips$

Hole diameter, $d_h = 0.8125\ in$

$\Omega = 2$

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

$= \frac{2.4 \cdot 0.75 \cdot 0.3 \cdot 65}{2}$

$= 17.55\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$

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

$= 3 - 0.8125$

$= 2.1875\ in$

$\Omega = 2$

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

$= \frac{1.2 \cdot 2.1875 \cdot 0.3 \cdot 65}{2}$

$= 25.5938\ kips$

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

$= min\ (25.5938, 17.55, 11.9282)$

$= 11.9282\ kips$

Edge bolt capacity

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

$= 1.5 - 0.5 \cdot 0.8125$

$= 1.09375\ in$

$\Omega = 2$

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

$= \frac{1.2 \cdot 1.09375 \cdot 0.3 \cdot 65}{2}$

$= 12.7969\ kips$

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

$= min\ (12.7969, 17.55, 11.9282)$

$= 11.9282\ kips$

Number of edge bolts, $N_e = m$

$= 1$

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

$= 1 \cdot 4 - 1$

$= 3$

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

$= 11.9282 \cdot 1 + 11.9282 \cdot 3$

$= 47.7129\ kips$

Applied member shear, $V_a = 26.5\ kips$

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

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

$= \frac{26.5}{47.7}$

$= 0.555556$

$47.7\ kips \geq 26.5\ kips$ **(OK)**

$0.556 \leq 1$ **(OK)**

Shear yielding of plate (38). Reference J4.2

Depth, $d = 11.5\ in$

Plate thickness, $t_{pl} = 0.25\ in$

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

Applied member shear, $V_a = 26.5\ kips$

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

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

Gross area, $A_g = d \cdot t_{pl}$

$= 11.5 \cdot 0.25$

$= 2.875\ in^2$

$\Omega = 1.5$

Plate capacity in pure shear, $\frac{R_v}{\Omega} = \frac{0.6 \cdot F_{y,pl} \cdot A_g}{\Omega}$

$= \frac{0.6 \cdot 50 \cdot 2.875}{1.5}$

$= 57.5\ kips$

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

$= \frac{26.5}{57.5}$

$= 0.46087$

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

$= 57.5\ kips$

$57.5\ kips \geq 26.5\ kips$ **(OK)**

$0.461 \leq 1$ **(OK)**

Block shear of beam web (6). Reference J4.3

Plate thickness, $Web\ thickness = 0.3\ in$

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, $n = 4$

Column edge distance, $L_{eh} = 2.5\ in$

Block shear of beam web (6). Reference J4.3 (continued)

Row edge distance, $L_{ev} = 1.5\ in$
Bolt columns, $m = 1$
Hole diameter, $d_h = 0.875\ in$
Hole length, $l_h = 0.875\ in$
Total length of bolt group, $s_{total} = 9\ in$
Gross shear area, $A_{gv} = Web\ thickness \cdot (s_{total} + L_{ev})$
$= 0.3 \cdot (9 + 1.5)$
$= 3.15\ in^2$
Net shear area, $A_{nv} = Web\ thickness \cdot (s_{total} + L_{ev}) - Web\ thickness \cdot (n - 0.5) \cdot d_h$
$= 0.3 \cdot (9 + 1.5) - 0.3 \cdot (4 - 0.5) \cdot 0.875$
$= 2.23125\ in^2$
Gross tensile area, $A_{gt} = Web\ thickness \cdot (s_{col} \cdot (m - 1) + L_{eh})$
$= 0.3 \cdot (0 \cdot (1 - 1) + 2.5)$
$= 0.75\ in^2$
Net tensile area, $A_{nt} = Web\ thickness \cdot (s_{col} \cdot (m - 1) + L_{eh}) - Web\ thickness \cdot (m - 0.5) \cdot l_h$
$= 0.3 \cdot (0 \cdot (1 - 1) + 2.5) - 0.3 \cdot (1 - 0.5) \cdot 0.875$
$= 0.61875\ in^2$
Reduction coefficient, $U_{bs} = 1$
Shear yield load, $R_{gv} = 0.6 \cdot F_y \cdot A_{gv}$
$= 0.6 \cdot 50 \cdot 3.15$
$= 94.5\ kips$
Shear rupture load, $R_{nv} = 0.6 \cdot F_u \cdot A_{nv}$
$= 0.6 \cdot 65 \cdot 2.23125$
$= 87.0187\ kips$
Tension load, $R_t = U_{bs} \cdot F_u \cdot A_{nt}$
$= 1 \cdot 65 \cdot 0.61875$
$= 40.2188\ kips$
Nominal block shear capacity, $R_n = min\ (R_{gv}, R_{nv}) + R_t$
$= min\ (94.5, 87.0187) + 40.2188$
$= 127.237\ kips$
$\Omega = 2$
$Shear\ capacity = \frac{R_n}{\Omega}$
$= \frac{127.237}{2}$
$= 63.6187\ kips$
Applied member shear, $V_a = 26.5\ kips$
$Unity = \frac{V_a}{Shear\ capacity}$
$= \frac{26.5}{63.6}$

Block shear of beam web (6). Reference J4.3 (continued)

$= 0.416667$

$63.6\ kips \geq 26.5\ kips$ **(OK)**

$0.417 \leq 1$ **(OK)**

Shear rupture of beam web (4). Reference J4.2

Tensile strength, $F_u = 65\ ksi$

Bottom cope depth, $d_{cb} = 0\ in$

Top cope depth, $d_{ct} = 2\ in$

Bolt rows, $n = 4$

Web thickness, $t_w = 0.3\ in$

Full section depth, $d = 17.7\ in$

Gross shear area, $A_g = t_w \cdot (d - d_{ct} - d_{cb})$

$= 0.3 \cdot (17.7 - 2 - 0)$

$= 4.71\ in^2$

Hole diameter, $d_h = 0.875\ in$

Net shear area, $A_n = A_g - n \cdot d_h \cdot t_w$

$= 4.71 - 4 \cdot 0.875 \cdot 0.3$

$= 3.66\ in^2$

$\Omega = 2$

$Shear\ capacity = \frac{0.6 \cdot F_u \cdot A_n}{\Omega}$

$= \frac{0.6 \cdot 65 \cdot 3.66}{2}$

$= 71.37\ kips$

Applied member shear, $V_a = 26.5\ kips$

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

$= \frac{26.5}{71.4}$

$= 0.371148$

$71.4\ kips \geq 26.5\ kips$ **(OK)**

$0.371 \leq 1$ **(OK)**

Shear yielding of beam web (5). Reference G2.1

Yield stress, $F_y = 50\ ksi$

Bottom cope depth, $d_{cb} = 0\ in$

Top cope depth, $d_{ct} = 2\ in$

Web thickness, $t_w = 0.3\ in$

Full section depth, $d = 17.7\ in$

Gross area, $A_g = t_w \cdot (d - d_{ct} - d_{cb})$

$= 0.3 \cdot (17.7 - 2 - 0)$

$= 4.71\ in^2$

Shear yielding of beam web (5). Reference G2.1 (continued)

$\Omega = 1.5$

$$Shear\ capacity = \frac{0.6 \cdot F_y \cdot A_g}{\Omega}$$
$$= \frac{0.6 \cdot 50 \cdot 4.71}{1.5}$$
$$= 94.2\ kips$$

Applied member shear, $V_a = 26.5\ kips$

$$Unity = \frac{V_a}{Shear\ capacity}$$
$$= \frac{26.5}{94.2}$$
$$= 0.281316$$

$94.2\ kips \geq 26.5\ kips$ **(OK)**

$0.281 \leq 1$ **(OK)**

Flexure of plate (314). Reference Pg 10-51, 12-7

Shear tab yield stress, $F_y = 50\ ksi$

Eccentricity, $e = 1.49375\ in$

Shear tab thickness, $t = 0.25\ in$

Shear tab depth, $d = 11.5\ in$

Plastic section modulus about the major axis, $Z_x = \frac{t \cdot d^2}{4}$

$$= \frac{0.25 \cdot 11.5^2}{4}$$
$$= 8.26562\ in^3$$

$\Omega = 1.67$

$$Shear\ capacity = \frac{\left(\frac{F_y \cdot Z_x}{\Omega}\right)}{e}$$
$$= \frac{\left(\frac{50 \cdot 8.26562}{1.67}\right)}{1.49375}$$
$$= 165.673\ kips$$

Applied member shear, $V_a = 26.5\ kips$

$$Unity = \frac{V_a}{Shear\ capacity}$$
$$= \frac{26.5}{165.7}$$
$$= 0.159928$$

$165.7\ kips \geq 26.5\ kips$ **(OK)**

$0.160 \leq 1$ **(OK)**

Flexure of coped beam (41). Reference Pg 9-7

Material setback, $m_s = 0.4875\ in$

Yield stress, $F_y = 50\ ksi$

Bottom cope length, $C_b = 0\ in$

Bottom cope depth, $d_{cb} = 0\ in$

Top cope length, $C_t = 4\ in$

Top cope depth, $d_{ct} = 2\ in$

k distance, $k = 0.827\ in$

Web thickness, $t_w = 0.3\ in$

Beam depth, $d = 17.7\ in$

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

Effective top cope depth, $d_{ct,e} = max\ (d_{ct}, 0)$
$= max\ (2, 0)$
$= 2\ in$

Effective top cope length, $c_{t,e} = C_t$
$= 4\ in$

Effective bottom cope depth, $d_{cb,e} = max\ (d_{cb}, 0)$
$= max\ (0, 0)$
$= 0\ in$

Effective bottom cope length, $c_{b,e} = C_b$
$= 0\ in$

Top flange is coped.

Depth of coped section, $h_c = d - d_{ct,e} - d_{cb,e}$
$= 17.7 - 2 - 0$
$= 15.7\ in$

Maximum distance from supporting face to end of cope, $e = c_{t,e} + m_s$
$= 4 + 0.4875$
$= 4.4875\ in$

Web slenderness, $\lambda = \frac{h_c}{t_w}$
$= \frac{15.7}{0.3}$
$= 52.3333$

Cope length, $C = c_{t,e}$
$= 4\ in$

Buckling adjustment factor, $f = \frac{2 \cdot C}{d}$
$= \frac{2 \cdot 4}{17.7}$
$= 0.451977$

Plate buckling coefficient, $k = 2.2 \cdot \left(\frac{h_c}{C}\right)^{1.65}$

Flexure of coped beam (41). Reference Pg 9-7 (continued)

$= 2.2 \cdot \left(\frac{15.7}{4}\right)^{1.65}$

$= 21.0019$

Modified plate buckling coefficient, $k_1 = max\ (f \cdot k, 1.61)$

$= max\ (0.451977 \cdot 21.0019, 1.61)$

$= 9.49237$

Limiting slenderness for a compact web, $\lambda_p = 0.475 \cdot \sqrt{\left(\frac{k_1 \cdot E}{F_y}\right)}$

$= 0.475 \cdot \sqrt{\left(\frac{9.49237 \cdot 29000}{50}\right)}$

$= 35.2448$

Plastic section modulus at the cope, $Z_c = 32.0982\ in^3$

Plastic bending moment, $M_p = \frac{F_y \cdot Z_c}{12}$

$= \frac{50 \cdot 32.0982}{12}$

$= 133.742\ kip \cdot ft$

Elastic section modulus at the cope, $S_c = 18.2016\ in^3$

Flexural yield moment, $M_y = \frac{F_y \cdot S_c}{12}$

$= \frac{50 \cdot 18.2016}{12}$

$= 75.8398\ kip \cdot ft$

Flexural strength at the coped section, $M_n = M_p - (M_p - M_y) \cdot \left(\frac{\lambda}{\lambda_p} - 1\right)$

$= 133.742 - (133.742 - 75.8398) \cdot \left(\frac{52.3333}{35.2448} - 1\right)$

$= 105.668\ kip \cdot ft$

$\Omega = 1.67$

Controlling strength, $R_r = \left(\frac{\left(\frac{M_n}{\Omega}\right)}{e}\right) \cdot 12$

$= \left(\frac{\left(\frac{105.668}{1.67}\right)}{4.4875}\right) \cdot 12$

$= 169.202\ kips$

$Shear\ capacity = R_r$

$= 169.202\ kips$

Applied member shear, $V_a = 26.5\ kips$

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

Flexure of coped beam (41). Reference Pg 9-7 (continued)

$= \dfrac{26.5}{169.2}$

$= 0.156619$

$169.2\ kips \geq 26.5\ kips$ **(OK)**

$0.157 \leq 1$ **(OK)**

Shear of support (36). Reference J4.2

Connection depth, $d_{conn} = 11.5\ in$

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

Supporting member thickness, $t_{sup} = 0.4\ in$

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

Applied member shear, $V_a = 26.5\ kips$

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

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

$= 2 \cdot 11.5 \cdot 0.4$

$= 9.2\ in^2$

$\Omega = 2$

$\Omega = 1.5$

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

$= min\left(\dfrac{0.6 \cdot 50 \cdot 9.2}{1.5}, \dfrac{0.6 \cdot 65 \cdot 9.2}{2}\right)$

$= 179.4\ kips$

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

$= \dfrac{26.5}{179.4}$

$= 0.147715$

$Shear\ capacity = R_v$

$= 179.4\ kips$

$179.4\ kips \geq 26.5\ kips$ **(OK)**

$0.148 \leq 1$ **(OK)**

Rupture of weld to supporting member (112). Reference J2, Table J2.5

Shear tab thickness, $t_{conn} = 0.25\ in$

FS Weld leg size, $w_{fs} = 0.1875\ in$

NS Weld leg size, $w_{ns} = 0.1875\ in$

Total effective weld throat, $t_{eff} = 0.707 \cdot (w_{ns} + w_{fs})$

$= 0.707 \cdot (0.1875 + 0.1875)$

$= 0.265125\ in$

Minimum specified weld size, $w_{min} = 0.625 \cdot t_{conn}$

$= 0.625 \cdot 0.25$

Rupture of weld to supporting member (112). Reference J2, Table J2.5 (continued)

$= 0.15625\ in$

Minimum specified total weld throat thickness, $t_{min} = 2 \cdot 0.707 \cdot w_{min}$

$= 2 \cdot 0.707 \cdot 0.15625$

$= 0.220938\ in$

$\left(t_{eff} = 0.265125\ in\right) \geq \left(t_{min} = 0.220938\ in\right)$

Weld is sized to develop the full strength of the plate.

Results summary

Shear Tab on left end of Beam B_4 [4]

AISC manual conventional configuration and design method

	t $d_b/2$ + 1/16 in
	t_w $d_b/2$ + 1/16 in

Limit state summary

	Calc. Num.	Unity ratio	Rn/OMEGA	AISC Ref
Shear rupture of plate:	21	0.679	39.0 kips	J4.2
Bolt shear of web bolts:	3	0.624	42.5 kips	J3.7, J3.9
Block shear rupture of plate:	6	0.608	43.6 kips	J4.3
Bolt bearing on plate:	110	0.602	44.0 kips	J3.11
Bolt bearing on beam web:	110	0.556	47.7 kips	J3.11
Shear yielding of plate:	38	0.461	57.5 kips	J4.2
Block shear rupture of beam web:	6	0.417	63.6 kips	J4.3
Shear rupture of beam web:	4	0.371	71.4 kips	J4.2
Shear yielding of beam web:	5	0.281	94.2 kips	G2.1
Flexure of plate:	314	0.160	165.7 kips	Pg 10-51, 12-7
Flexure of coped beam:	41	0.157	169.2 kips	Pg 9-7
Shear of support:	36	0.148	179.4 kips	J4.2

Connection strength

	Value:	**Unity ratio:**
Shear:	39.0 kips	0.679

Notes and conclusions

- Weld sized to develop the full plate strength.
- See 'Single-plate connection' notes for design criteria applicable to this connection. ⚠
- CONNECTION IS OK
  - Strength equals or exceeds design loads.