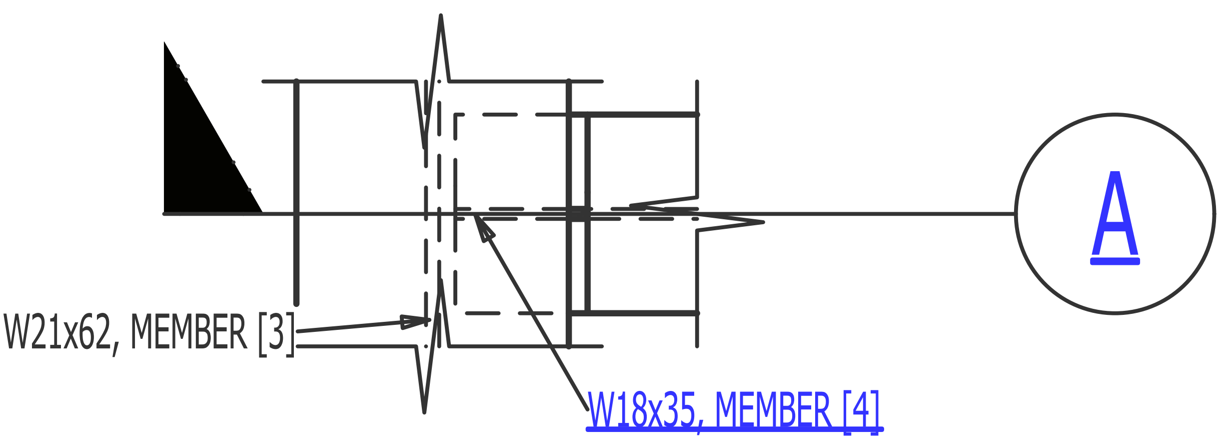
SDS2 Steel Connection Design: Connection Cube Report

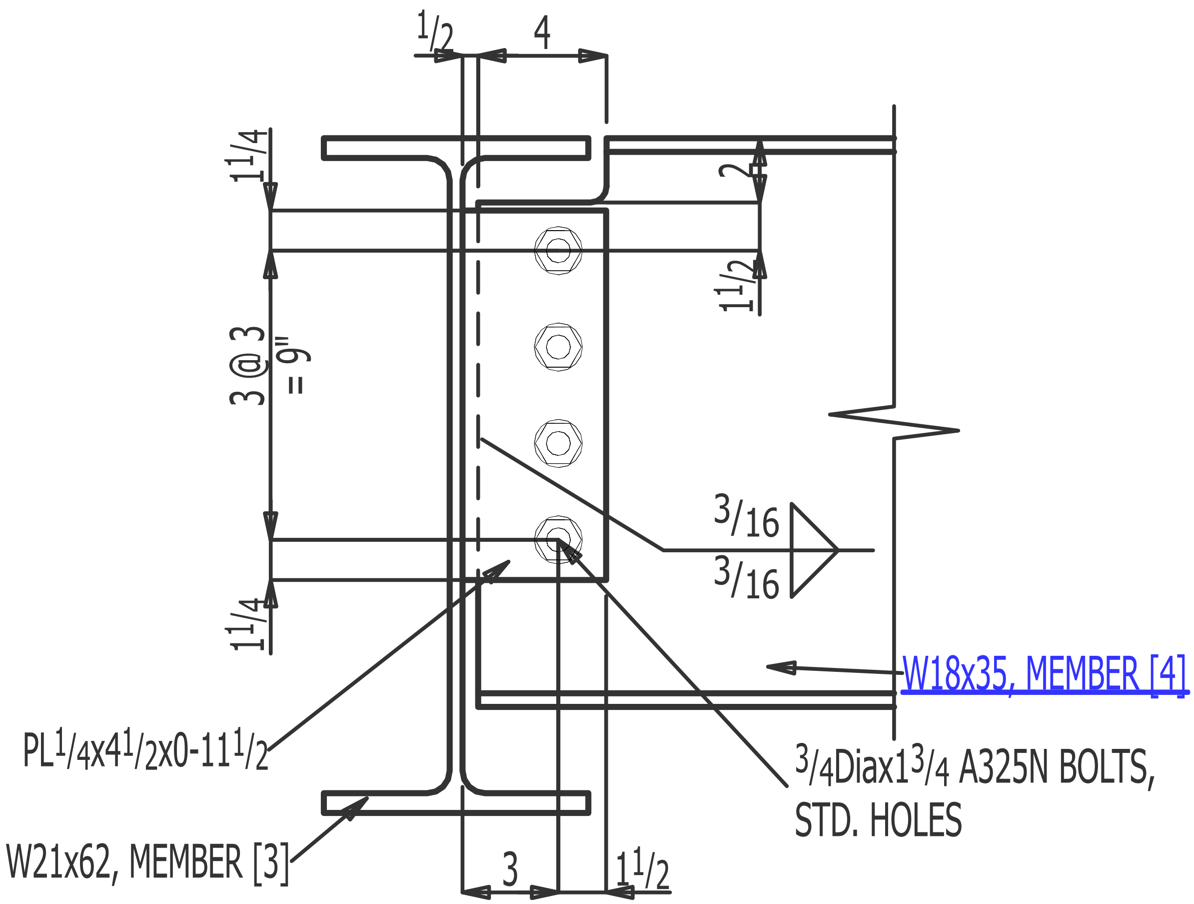
Cube: Ex. II.A-18
Revision: 0
Project: LRFD16ValidationExamples
Engineer:
Fabricator: ASD16ValidationExamples

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



A

Section A

ELEVATION

Beam B_4 [4]

Design method

- AISC Steel Construction Manual, Sixteenth Edition (LRFD)
- 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:	39.8 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

Factored loads

Shear:	39.8 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, $A_{nv} = t_{conn} \cdot (d_{pl} - n \cdot d_h)$
$= 0.25 \cdot (11.5 - 4 \cdot 0.875)$
$= 2\ in^2$

$\phi = 0.75$

Shear capacity, $\phi V_n = \phi \cdot 0.6 \cdot F_u \cdot A_{nv}$
$= 0.75 \cdot 0.6 \cdot 65 \cdot 2$
$= 58.5\ kips$

$Shear\ capacity = \phi V_n$
$= 58.5\ kips$

Applied member shear, $V_a = 39.8\ kips$

$Unity = \frac{V_a}{Shear\ capacity}$
$= \frac{39.8}{58.5}$
$= 0.680342$

$58.5\ kips \geq 39.8\ kips$ **(OK)**

$0.680 \leq 1$ **(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, $F_{nv} = 54\ ksi$

$\phi = 0.75$

Bolt shear capacity, $\phi R_{n,v} = \phi \cdot F_{nv} \cdot A_b \cdot N_s$
$= 0.75 \cdot 54 \cdot 0.441786 \cdot 1$
$= 17.8924\ kips$

$Shear\ capacity = C \cdot \phi R_{n,v}$
$= 3.56171 \cdot 17.8924$
$= 63.7274\ kips$

Applied member shear, $V_a = 39.8\ kips$

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

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

$= \frac{39.8}{63.7}$

$= 0.624804$

$63.7\ kips \geq 39.8\ kips$ **(OK)**

$0.625 \leq 1$ **(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, $R_{nv} = 0.6 \cdot F_u \cdot A_{nv}$

$= 0.6 \cdot 65 \cdot 1.79688$

$= 70.0781\ kips$

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

$= 1 \cdot 65 \cdot 0.2625$

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

$= 17.0625\ kips$

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

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

$= 87.1406\ kips$

$\phi = 0.75$

$Shear\ capacity = \phi \cdot R_n$

$= 0.75 \cdot 87.1406$

$= 65.3555\ kips$

Applied member shear, $V_a = 39.8\ kips$

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

$= \frac{39.8}{65.4}$

$= 0.608563$

$65.4\ kips \geq 39.8\ kips$ **(OK)**

$0.609 \leq 1$ **(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, $F_{nv} = 54\ ksi$

$\phi = 0.75$

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

$= 0.75 \cdot 54 \cdot 0.441786 \cdot 1$

$= 17.8924\ kips$

Hole diameter, $d_h = 0.8125\ in$

$\phi = 0.75$

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

$= 0.75 \cdot 2.4 \cdot 0.75 \cdot 0.25 \cdot 65$

$= 21.9375\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$

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

$= 3 - 0.8125$

$= 2.1875\ in$

$\phi = 0.75$

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

$= 0.75 \cdot 1.2 \cdot 2.1875 \cdot 0.25 \cdot 65$

$= 31.9922\ kips$

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

$= min\ (31.9922, 21.9375, 17.8924)$

$= 17.8924\ kips$

Edge bolt capacity

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

$= 1.25 - 0.5 \cdot 0.8125$

$= 0.84375\ in$

$\phi = 0.75$

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

$= 0.75 \cdot 1.2 \cdot 0.84375 \cdot 0.25 \cdot 65$

$= 12.3398\ kips$

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

$= min\ (12.3398, 21.9375, 17.8924)$

$= 12.3398\ kips$

Number of edge bolts, $N_e = m$

$= 1$

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

$= 1 \cdot 4 - 1$

$= 3$

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

$= 12.3398 \cdot 1 + 17.8924 \cdot 3$

$= 66.0169\ kips$

Applied member shear, $V_a = 39.8\ kips$

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

$= \dfrac{39.8}{66}$

$= 0.60303$

$66.0\ kips \geq 39.8\ kips$ **(OK)**

$0.603 \leq 1$ **(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, $F_{nv} = 54\ ksi$
$\phi = 0.75$
Bolt shear capacity, $\phi R_{n,v} = \phi \cdot F_{nv} \cdot A_b \cdot N_s$
$= 0.75 \cdot 54 \cdot 0.441786 \cdot 1$
$= 17.8924\ kips$
Hole diameter, $d_h = 0.8125\ in$
$\phi = 0.75$
Bolt bearing capacity, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_{pl} \cdot F_u$
$= 0.75 \cdot 2.4 \cdot 0.75 \cdot 0.3 \cdot 65$
$= 26.325\ kips$

Interior bolt capacity

Bolt row spacing, $s = 3\ in$
Clear distance from bolt hole to bolt hole, $L_{c,int} = s - d_h$
$= 3 - 0.8125$
$= 2.1875\ in$
$\phi = 0.75$
Tearout load capacity, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_{pl} \cdot F_u$
$= 0.75 \cdot 1.2 \cdot 2.1875 \cdot 0.3 \cdot 65$
$= 38.3906\ kips$
Controlling bearing/tearout strength of interior bolt, $\phi R_{n,i} = min\left(\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v}\right)$
$= min\left(38.3906, 26.325, 17.8924\right)$
$= 17.8924\ kips$

Edge bolt capacity

Clear distance from hole to edge of material, $L_{c,edge} = L_{ev} - 0.5 \cdot d_h$
$= 1.5 - 0.5 \cdot 0.8125$
$= 1.09375\ in$

Edge bolt capacity (continued)

$\phi = 0.75$

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

$= 0.75 \cdot 1.2 \cdot 1.09375 \cdot 0.3 \cdot 65$

$= 19.1953\ kips$

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

$= min\left(19.1953, 26.325, 17.8924\right)$

$= 17.8924\ kips$

Number of edge bolts, $N_e = m$

$= 1$

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

$= 1 \cdot 4 - 1$

$= 3$

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

$= 17.8924 \cdot 1 + 17.8924 \cdot 3$

$= 71.5694\ kips$

Applied member shear, $V_a = 39.8\ kips$

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

$= \dfrac{39.8}{71.6}$

$= 0.555866$

$71.6\ kips \geq 39.8\ kips$ **(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, $V_a = 39.8\ kips$

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

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

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

$= 11.5 \cdot 0.25$

$= 2.875\ in^2$

$\phi = 1$

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

$= 1 \cdot 0.6 \cdot 50 \cdot 2.875$

$= 86.25\ kips$

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

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

$= \frac{39.8}{86.25}$

$= 0.461449$

$Shear\ capacity = \phi R_v$

$= 86.25\ kips$

$86.3\ kips \geq 39.8\ kips$ **(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, $L_{eh} = 2.5\ in$

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

$= 0.6 \cdot 65 \cdot 2.23125$

$= 87.0187\ kips$

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

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, $R_n = min\ (R_{gv}, R_{nv}) + R_t$
$= min\ (94.5, 87.0187) + 40.2188$
$= 127.237\ kips$

$\phi = 0.75$

Shear capacity $= \phi \cdot R_n$
$= 0.75 \cdot 127.237$
$= 95.4281\ kips$

Applied member shear, $V_a = 39.8\ kips$

$Unity = \frac{V_a}{Shear\ capacity}$
$= \frac{39.8}{95.4}$
$= 0.417191$

$95.4\ kips \geq 39.8\ kips$ **(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, $A_n = A_g - n \cdot d_h \cdot t_w$
$= 4.71 - 4 \cdot 0.875 \cdot 0.3$
$= 3.66\ in^2$

$\phi = 0.75$

Shear capacity $= \phi \cdot 0.6 \cdot F_u \cdot A_n$
$= 0.75 \cdot 0.6 \cdot 65 \cdot 3.66$
$= 107.055\ kips$

Applied member shear, $V_a = 39.8\ kips$

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

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

$= \frac{39.8}{107.1}$

$= 0.371615$

$107.1\ kips \geq 39.8\ kips$ **(OK)**

$0.372 \leq 1$ **(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, $A_g = t_w \cdot (d - d_{ct} - d_{cb})$

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

$= 4.71\ in^2$

$\phi = 1$

$Shear\ capacity = \phi \cdot 0.6 \cdot F_y \cdot A_g$

$= 1 \cdot 0.6 \cdot 50 \cdot 4.71$

$= 141.3\ kips$

Applied member shear, $V_a = 39.8\ kips$

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

$= \frac{39.8}{141.3}$

$= 0.28167$

$141.3\ kips \geq 39.8\ kips$ **(OK)**

$0.282 \leq 1$ **(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, $Z_x = \frac{t \cdot d^2}{4}$

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

$= 8.26562\ in^3$

$\phi = 0.9$

$Shear\ capacity = \frac{\phi \cdot F_y \cdot Z_x}{e}$

$= \frac{0.9 \cdot 50 \cdot 8.26562}{1.49375}$

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

$= 249.006\ kips$

Applied member shear, $V_a = 39.8\ kips$

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

$= \dfrac{39.8}{249}$

$= 0.159839$

$249.0\ kips \geq 39.8\ kips$ **(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, $\lambda = \dfrac{h_c}{t_w}$

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

$= \frac{15.7}{0.3}$

$= 52.3333$

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

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

$= 21.0019$

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, $M_n = M_p - (M_p - M_y) \cdot \left(\frac{\lambda}{\lambda_p} - 1\right)$

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

$= 105.668\ kip \cdot ft$

$\phi = 0.9$

Controlling strength, $R_{r} = \left(\frac{\phi \cdot M_n}{e}\right) \cdot 12$

$= \left(\frac{0.9 \cdot 105.668}{4.4875}\right) \cdot 12$

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

$= 254.31\ kips$

$Shear\ capacity = R_r$

$= 254.31\ kips$

Applied member shear, $V_a = 39.8\ kips$

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

$= \frac{39.8}{254.3}$

$= 0.156508$

$254.3\ kips \geq 39.8\ kips$ **(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, $V_a = 39.8\ kips$

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

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

$= 2 \cdot 11.5 \cdot 0.4$

$= 9.2\ in^2$

$\phi = 0.75$

$\phi = 1$

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

$= min\left(1 \cdot 0.6 \cdot 50 \cdot 9.2, 0.75 \cdot 0.6 \cdot 65 \cdot 9.2\right)$

$= 269.1\ kips$

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

$= \frac{39.8}{269.1}$

$= 0.1479$

$Shear\ capacity = R_v$

$= 269.1\ kips$

$269.1\ kips \geq 39.8\ kips$ **(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, $t_{eff} = 0.707 \cdot \left(w_{ns} + w_{fs}\right)$

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

$= 0.707 \cdot (0.1875 + 0.1875)$

$= 0.265125\ in$

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

$= 0.625 \cdot 0.25$

$= 0.15625\ in$

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

$= 2 \cdot 0.707 \cdot 0.15625$

$= 0.220938\ in$

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

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

$$\frac{t \; d_b/2 + 1/16 \text{ in}}{t_w \; d_b/2 + 1/16 \text{ in}}$$

Limit state summary

	Calc. Num.	Unity ratio	PHI*Rn	AISC Ref
Shear rupture of plate:	21	0.680	58.5 kips	J4.2
Bolt shear of web bolts:	3	0.625	63.7 kips	J3.7, J3.9
Block shear rupture of plate:	6	0.609	65.4 kips	J4.3
Bolt bearing on plate:	110	0.603	66.0 kips	J3.11
Bolt bearing on beam web:	110	0.556	71.6 kips	J3.11
Shear yielding of plate:	38	0.461	86.3 kips	J4.2
Block shear rupture of beam web:	6	0.417	95.4 kips	J4.3
Shear rupture of beam web:	4	0.372	107.1 kips	J4.2
Shear yielding of beam web:	5	0.282	141.3 kips	G2.1
Flexure of plate:	314	0.160	249.0 kips	Pg 10-51, 12-7
Flexure of coped beam:	41	0.157	254.3 kips	Pg 9-7
Shear of support:	36	0.148	269.1 kips	J4.2

Connection strength

	Value:	**Unity ratio:**
Shear:	58.5 kips	0.680

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.