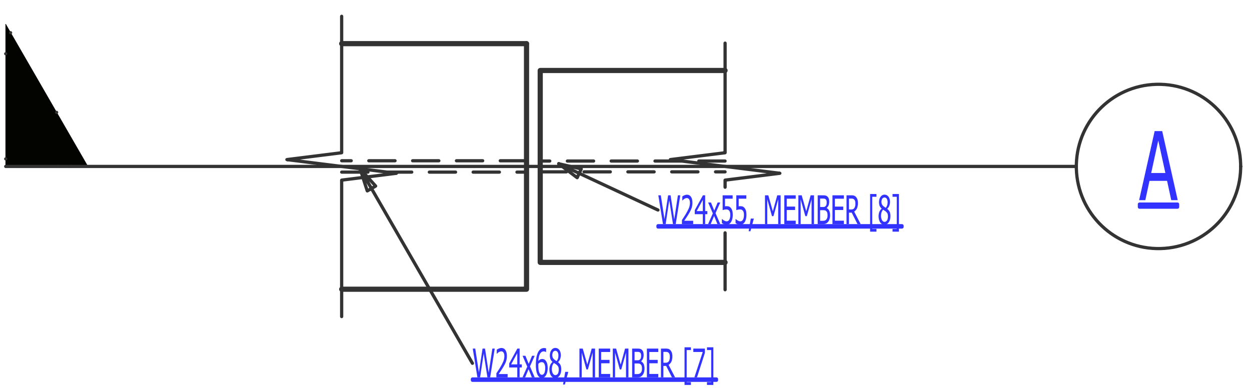
SDS2 Steel Connection Design: Connection Cube Report

Cube: Ex. II.A-20
Revision: 0
Project: LRFD16ValidationExamples
Engineer:
Fabricator: ASD16ValidationExamples

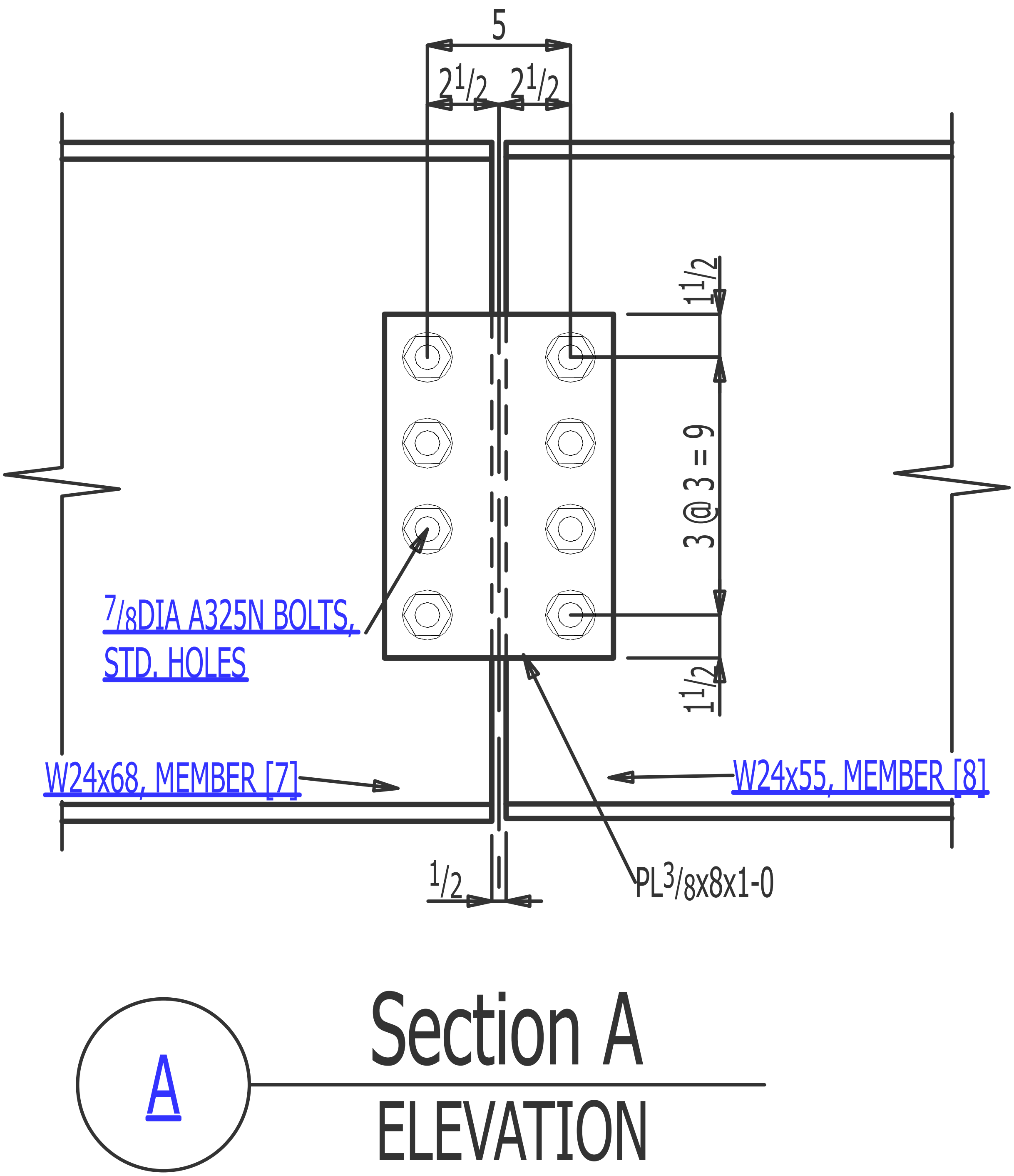
Generated by [SDS2 x 2025.03] on Friday, Oct 11, 2024

Ex. II.A-20 [4] at X=125-0, Y=75-0 Elev=-11 7/8



TOP SIDE VIEW

Ex. II.A-20


5
2 1/2 2 1/2
1 1/2
3 @ 3 = 9
1 1/2
7/8DIA A325N BOLTS,
STD. HOLES
W24x68, MEMBER [7]
W24x55, MEMBER [8]
1/2
PL 3/8x8x1-0
A
Section A
ELEVATION

Beam B_8 [7]

Design method

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

Overview

Section size:	W24x68
Sequence:	1
ABM:	N/Assign
Plan length:	25-0
Camber:	0.00 in
Span length:	25-0
Slope:	0.00 °
Material length:	24-11 3/4
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	23.7 in
Web thickness, t_w:	0.415 in
Flange width, b_f:	8.97 in
Flange thickness, t_f:	0.585 in
Design k distance, k_{des}:	1.09 in
Detail k distance, k_{det}:	1.875 in
Distance between web toes of fillets, T:	19.95 in
Moment of inertia about the major axis, I_x:	1830 in^4

Design summary

Right end

Connection:	Splice plate
	Plates on left end, Near side
Elevation:	0
Minus Dim:	0.25 in
Mtrl Setback:	0.25 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	60.0 kips
Moment:	0.0 kip·ft (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_8 [7] Connection strength check: RIGHT END

Member end summary

Connecting nodes

Node 1

Beam:	B_7 [8]
Section size:	W24x55
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}:	1.4375 in
Design k distance, k_{des}:	1.01 in
Depth, d:	23.6 in
Web thickness, t_w:	0.395 in
Flange thickness, t_f:	0.505 in

Factored loads

***Shear*:**	60.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 28.2 % of the allowable uniform steel beam load.

Connection summary

- BOLTED BEAM SHEAR PLATE SPLICE
- (Splice plate on one side of web)

Connection details

Plates:	**Grade:**	A572-50
	Tensile strength, F_u:	65 ksi
	Yield stress, F_y:	50 ksi
Web plates:	**Thickness, t:**	0.375 in
	Depth, d:	12 in
Web 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 columns, m:	1
Gap between members, g:	0.5 in	

Connection design lock summary

Locked Via Member Edit:	20
(at dd) Not Locked:	106

Expanded design calculation

Bolt bearing on web plate(s) (20). Reference J3.11

Number of shear planes, $N_s = 1$
Number of sides, $N = 1$
Row edge distance, $L_e = 1.5\ in$
Connection thickness, $t = 0.375\ in$
Connection tensile strength, $F_u = 65\ ksi$
Bolt row spacing, $s = 3\ in$
Bolt columns, $m = 1$
Bolt rows, $n = 4$
Bolt diameter, $d_b = 0.875\ in$
$C = 3.07968$
Total number of bolts, $N = n \cdot m$
$= 4 \cdot 1$
$= 4$
Number of edge bolts, $N_{edge} = m$
$= 1$
Number of interior bolts, $N_{int} = N - m$
$= 4 - 1$
$= 3$
Total length of bolt group, $s_{total} = 9\ in$
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, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t \cdot F_u$
$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.375 \cdot 65$
$= 38.3906\ kips$

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, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t \cdot F_u$

Interior bolt capacity (continued)

$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.375 \cdot 65$

$= 45.2461\ kips$

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

$= min\ (45.2461, 38.3906, 24.3535)$

$= 24.3535\ kips$

Edge bolt capacity

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

$= 1.5 - 0.5 \cdot 0.9375$

$= 1.03125\ in$

$\phi = 0.75$

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

$= 0.75 \cdot 1.2 \cdot 1.03125 \cdot 0.375 \cdot 65$

$= 22.623\ kips$

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

$= min\ (22.623, 38.3906, 24.3535)$

$= 22.623\ kips$

Average bolt bearing/tearout, $\phi R_{v,ave} = \dfrac{(\phi R_{n,e} \cdot N_{edge} + \phi R_{n,i} \cdot N_{int})}{N}$

$= \dfrac{(22.623 \cdot 1 + 24.3535 \cdot 3)}{4}$

$= 23.9209\ kips$

Shear capacity, $\phi V_n = N \cdot \phi R_{v,ave} \cdot C$

$= 1 \cdot 23.9209 \cdot 3.07968$

$= 73.6687\ kips$

$Shear\ capacity = \phi V_n$

$= 73.6687\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \dfrac{60}{73.7}$

$= 0.814111$

$73.7\ kips \geq 60\ kips$ **(OK)**

$0.814 \leq 1$ **(OK)**

Bolt bearing on beam web (20). Reference J3.11

Bolt diameter, $d_b = 0.875\ in$

Number of shear planes, $N_s = 1$

Number of sides, $N = 1$

Bolt rows, $n = 4$

Bolt bearing on beam web (20). Reference J3.11 (continued)

Bolt columns, $m = 1$
Vertical bolt spacing, $s = 3\ in$
This beam tensile strength, $F_u = 65\ ksi$
This beam web thickness, $t_w = 0.415\ in$
Other beam tensile strength, $F_{u,s} = 65\ ksi$
Other beam web thickness, $t_{w,s} = 0.395\ in$

This beam

$C = 3.07968$
Total number of bolts, $N = n \cdot m$
$= 4 \cdot 1$
$= 4$
Number of edge bolts, $N_{edge} = m$
$= 1$
Number of interior bolts, $N_{int} = N - m$
$= 4 - 1$
$= 3$
Total length of bolt group, $s_{total} = 9\ in$
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, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_w \cdot F_u$
$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.415 \cdot 65$
$= 42.4856\ kips$

Interior bolt capacity

Vertical bolt 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, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_w \cdot F_u$
$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.415 \cdot 65$
$= 50.0723\ kips$
Controlling bearing/tearout strength of interior bolt, $\phi R_{n,i} = min\ (\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v})$

Interior bolt capacity (continued)

$= min\ (50.0723, 42.4856, 24.3535)$

$= 24.3535\ kips$

Edge bolt capacity

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

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

$= min\ (42.4856, 24.3535)$

$= 24.3535\ kips$

Average bolt bearing/tearout, $\phi R_{v,ave} = \frac{(\phi R_{n,e} \cdot N_{edge} + \phi R_{n,i} \cdot N_{int})}{N}$

$= \frac{(24.3535 \cdot 1 + 24.3535 \cdot 3)}{4}$

$= 24.3535\ kips$

Shear capacity, $\phi V_n = N \cdot \phi R_{v,ave} \cdot C$

$= 1 \cdot 24.3535 \cdot 3.07968$

$= 75.0009\ kips$

$Shear\ capacity = \phi V_n$

$= 75.0009\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \frac{60}{75}$

$= 0.8$

Bearing on beam web, $\phi P_{brg} = Shear\ capacity$

$= 75\ kips$

This beam unity ratio, $U = Unity$

$= 0.8$

Other beam

$C = 3.07968$

Total number of bolts, $N = n \cdot m$

$= 4 \cdot 1$

$= 4$

Number of edge bolts, $N_{edge} = m$

$= 1$

Number of interior bolts, $N_{int} = N - m$

$= 4 - 1$

$= 3$

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

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

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

Other beam (continued)

$\phi = 0.75$

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, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_{w,s} \cdot F_{u,s}$

$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.395 \cdot 65$

$= 40.4381\ kips$

Interior bolt capacity

Vertical bolt 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, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_{w,s} \cdot F_{u,s}$

$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.395 \cdot 65$

$= 47.6592\ kips$

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

$= min\ (47.6592, 40.4381, 24.3535)$

$= 24.3535\ kips$

Edge bolt capacity

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

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

$= min\ (40.4381, 24.3535)$

$= 24.3535\ kips$

Average bolt bearing/tearout, $\phi R_{v,ave} = \dfrac{(\phi R_{n,e} \cdot N_{edge} + \phi R_{n,i} \cdot N_{int})}{N}$

$= \dfrac{(24.3535 \cdot 1 + 24.3535 \cdot 3)}{4}$

$= 24.3535\ kips$

Shear capacity, $\phi V_n = N \cdot \phi R_{v,ave} \cdot C$

$= 1 \cdot 24.3535 \cdot 3.07968$

$= 75.0009\ kips$

$Shear\ capacity = \phi V_n$

$= 75.0009\ kips$

Applied member shear, $V_a = 60\ kips$

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

Other beam (continued)

$= \frac{60}{75}$

$= 0.8$

Bearing on other web, $\phi P_{brg,s} = Shear\ capacity$

$= 75\ kips$

Other beam unity ratio, $U_o = Unity$

$= 0.8$

$Unity = max\ (U, U_o)$

$= max\ (0.8, 0.8)$

$= 0.8$

$Shear\ capacity = min\ (\phi P_{brg}, \phi P_{brg,s})$

$= min\ (75, 75)$

$= 75\ kips$

$75.0\ kips \geq 60\ kips$ **(OK)**

$0.800 \leq 1$ **(OK)**

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

Number of shear planes, $N_s = 1$

Coefficient, $C = 3.07968$

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

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

$\phi = 0.75$

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

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

$= 24.3535\ kips$

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

$= 3.07968 \cdot 24.3535$

$= 75.0009\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \frac{60}{75}$

$= 0.8$

$75.0\ kips \geq 60\ kips$ **(OK)**

$0.800 \leq 1$ **(OK)**

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

Connection tensile strength, $F_{u,conn} = 65\ ksi$

FS bolt rows, $n_{FS} = 4$

NS bolt rows, $n_{NS} = 4$

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

FS connection thickness, $t_{fs} = 0\ in$
NS connection thickness, $t_{ns} = 0.375\ in$
FS connection depth, $d_{fs} = 0\ in$
NS connection depth, $d_{ns} = 12\ in$
Hole diameter, $d_h = 1\ in$
NS Net shear area, $A_{nv,ns} = t_{ns} \cdot (d_{ns} - n_{NS} \cdot d_h)$
$= 0.375 \cdot (12 - 4 \cdot 1)$
$= 3\ in^2$
FS Net shear area, $A_{nv,fs} = t_{fs} \cdot (d_{fs} - n_{FS} \cdot d_h)$
$= 0 \cdot (0 - 4 \cdot 1)$
$= 0\ in^2$
Total net shear area, $A_{nv,total} = A_{nv,ns} + A_{nv,fs}$
$= 3 + 0$
$= 3\ in^2$
$\phi = 0.75$
Shear capacity, $\phi V_n = \phi \cdot 0.6 \cdot F_{u,conn} \cdot A_{nv,total}$
$= 0.75 \cdot 0.6 \cdot 65 \cdot 3$
$= 87.75\ kips$
$Shear\ capacity = \phi V_n$
$= 87.75\ kips$
Applied member shear, $V_a = 60\ kips$
$Unity = \dfrac{V_a}{Shear\ capacity}$
$= \dfrac{60}{87.8}$
$= 0.683371$
$87.8\ kips \geq 60\ kips$ **(OK)**
$0.683 \leq 1$ **(OK)**

Block shear rupture of web plate(s) (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, $s_{col} = 5\ in$
Bolt row spacing, $s = 3\ in$
Bolt rows, $n = 4$
Column edge distance, $L_{eh} = 1.5\ in$
Row edge distance, $L_{ev} = 1.5\ in$
Bolt columns, $m = 1$
Hole diameter, $d_h = 1\ in$

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

Hole length, $l_h = 1\ in$

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

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

$= 0.375 \cdot (9 + 1.5)$

$= 3.9375\ in^2$

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

$= 0.375 \cdot (9 + 1.5) - 0.375 \cdot (4 - 0.5) \cdot 1$

$= 2.625\ in^2$

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

$= 0.375 \cdot (5 \cdot (1 - 1) + 1.5)$

$= 0.5625\ in^2$

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

$= 0.375 \cdot (5 \cdot (1 - 1) + 1.5) - 0.375 \cdot (1 - 0.5) \cdot 1$

$= 0.375\ in^2$

Reduction coefficient, $U_{bs} = 1$

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

$= 0.6 \cdot 50 \cdot 3.9375$

$= 118.125\ kips$

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

$= 0.6 \cdot 65 \cdot 2.625$

$= 102.375\ kips$

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

$= 1 \cdot 65 \cdot 0.375$

$= 24.375\ kips$

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

$= min\ (118.125, 102.375) + 24.375$

$= 126.75\ kips$

$\phi = 0.75$

$Shear\ capacity = \phi \cdot R_n$

$= 0.75 \cdot 126.75$

$= 95.0625\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \dfrac{60}{95.1}$

$= 0.630915$

$95.1\ kips \geq 60\ kips$ **(OK)**

$0.631 \leq 1$ **(OK)**

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

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

FS connection thickness, $t_{fs} = 0\ in$

NS connection thickness, $t_{ns} = 0.375\ in$

FS connection depth, $d_{fs} = 0\ in$

NS connection depth, $d_{ns} = 12\ in$

Gross shear area, $A_{gv} = d_{ns} \cdot t_{ns} + d_{fs} \cdot t_{fs}$

$= 12 \cdot 0.375 + 0 \cdot 0$

$= 4.5\ in^2$

$\phi = 1$

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

$= 1 \cdot 0.6 \cdot 50 \cdot 4.5$

$= 135\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \dfrac{60}{135}$

$= 0.444444$

$135.0\ kips \geq 60\ kips$ **(OK)**

$0.444 \leq 1$ **(OK)**

Flexure of web plate(s) (19). Reference F11

$F_{u,conn} = 65\ ksi$

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

Bolt row spacing, $s = 3\ in$

Bolt rows, $n = 4$

Number of connection sides, $N = 1$

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

Connection depth, $d_{pl} = 12\ in$

Eccentricity in x-direction, $e_x = 2.5\ in$

Hole diameter, $d_h = 1\ in$

> **Gross moment capacity**
>
> Steel modulus of elasticity, $E = 29000\ ksi$
>
> Unbraced Length, $L_b = e_x$
>
> $= 2.5\ in$
>
> Plastic section modulus, $Z = \dfrac{t_{pl} \cdot d_{pl}^2}{4}$
>
> $= \dfrac{0.375 \cdot 12^2}{4}$
>
> $= 13.5\ in^3$

Gross moment capacity (continued)

Elastic section modulus, $S = \dfrac{t_{pl} \cdot d_{pl}^2}{6}$

$= \dfrac{0.375 \cdot 12^2}{6}$

$= 9\ in^3$

Plastic bending moment, $M_p = \dfrac{F_{y,p} \cdot Z}{12}$

$= \dfrac{50 \cdot 13.5}{12}$

$= 56.25\ kip \cdot ft$

$\left(M_p = 56.25\ kip \cdot ft\right) \leq \left(\dfrac{1.5 \cdot F_{y,p} \cdot S}{12} = \dfrac{1.5 \cdot 50 \cdot 9}{12} = 56.25\ kip \cdot ft\right)$

$\left(\dfrac{L_b \cdot d_{pl}}{t_{pl}^2} = \dfrac{2.5 \cdot 12}{0.375^2} = 213.333\right) > \left(\dfrac{0.08 \cdot E}{F_{y,p}} = \dfrac{0.08 \cdot 29000}{50} = 46.4\right)$

Lateral-torsional buckling modification factor, $C_b = 1.84$

$\left(\dfrac{L_b \cdot d_{pl}}{t_{pl}^2} = \dfrac{2.5 \cdot 12}{0.375^2} = 213.333\right) \leq \left(\dfrac{1.9 \cdot E}{F_{y,p}} = \dfrac{1.9 \cdot 29000}{50} = 1102\right)$

Flexural yield moment, $M_y = \dfrac{F_{y,p} \cdot S}{12}$

$= \dfrac{50 \cdot 9}{12}$

$= 37.5\ kip \cdot ft$

Nominal flexural strength, $M_n = min\left(C_b \cdot \left(1.52 - 0.274 \cdot \left(\dfrac{L_b \cdot d_{pl}}{t_{pl}^2}\right) \cdot \left(\dfrac{F_{y,p}}{E}\right)\right) \cdot M_y, M_p\right)$

$= min\left(1.84 \cdot \left(1.52 - 0.274 \cdot \left(\dfrac{2.5 \cdot 12}{0.375^2}\right) \cdot \left(\dfrac{50}{29000}\right)\right) \cdot 37.5, 56.25\right)$

$= 56.25\ kip \cdot ft$

$\phi = 0.9$

Gross moment capacity, $\phi M_{n,gross} = N \cdot \phi \cdot M_n$

$= 1 \cdot 0.9 \cdot 56.25$

$= 50.625\ kip \cdot ft$

Net moment capacity

$\phi = 0.75$

Bending stress, $\phi F_b = \phi \cdot F_{u,conn}$

$= 0.75 \cdot 65$

$= 48.75\ ksi$

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

Row edge distance top, $L_{e,top} = \dfrac{\left(d_{pl} - s_{total}\right)}{2}$

Net moment capacity (continued)

$= \frac{(12 - 9)}{2}$

$= 1.5\ in$

Row edge distance bottom, $L_{e,bot} = L_{e,top}$

$= 1.5\ in$

Bolt row spacing, $s = 3\ in$

Net plastic section modulus, $Z_{x,net} = \frac{t_{pl} \cdot (s - d_h) \cdot n^2 \cdot s}{4}$

$= \frac{0.375 \cdot (3 - 1) \cdot 4^2 \cdot 3}{4}$

$= 9\ in^3$

Bolt row spacing, $s = 3\ in$

Deduction of net section modulus due to the bolt holes, $S_{deduct} = \frac{\left(\frac{s^2 \cdot n \cdot (n^2 - 1) \cdot t_{pl} \cdot d_h}{6}\right)}{d_{pl}}$

$= \frac{\left(\frac{3^2 \cdot 4 \cdot (4^2 - 1) \cdot 0.375 \cdot 1}{6}\right)}{12}$

$= 2.8125\ in^3$

Net elastic section modulus, $S_{x,net} = \frac{t_{pl} \cdot d_{pl}^2}{6} - S_{deduct}$

$= \frac{0.375 \cdot 12^2}{6} - 2.8125$

$= 6.1875\ in^3$

$(Z_{x,net} = 9\ in^3) \leq (1.5 \cdot S_{x,net} = 1.5 \cdot 6.1875 = 9.28125\ in^3)$

Net moment capacity, $\phi M_{n,net} = \frac{N \cdot \phi F_b \cdot Z_{x,net}}{12}$

$= \frac{1 \cdot 48.75 \cdot 9}{12}$

$= 36.5625\ kip \cdot ft$, Reference: (9-8)

$Shear\ capacity = \left(\frac{min\ (\phi M_{n,gross}, \phi M_{n,net})}{e_x}\right) \cdot 12$

$= \left(\frac{min\ (50.625, 36.5625)}{2.5}\right) \cdot 12$

$= 175.5\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \frac{60}{175.5}$

Flexure of web plate(s) (19). Reference F11 (continued)

$= 0.34188$

$175.5\ kips \geq 60\ kips$ **(OK)**

$0.342 \leq 1$ **(OK)**

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

This beam depth, $d = 23.7\ in$

This beam web thickness, $t_w = 0.415\ in$

This beam yield stress, $F_y = 50\ ksi$

Other beam depth, $d_s = 23.6\ in$

Other beam web thickness, $t_{w,s} = 0.395\ in$

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

<u>This beam</u>

Applied member shear, $V_a = 60\ kips$

$\phi = 1$

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

$= 1 \cdot 0.6 \cdot 50$

$= 30\ ksi$

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

$= 23.7 \cdot 0.415$

$= 9.8355\ in^2$

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

$$= \frac{60}{30 \cdot 9.8355}$$

$= 0.203345$

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

$= 30 \cdot 9.8355$

$= 295.065\ kips$

Beam gross shear, $\phi V_g = Shear\ capacity$

$= 295.1\ kips$

This beam unity ratio, $U = Unity$

$= 0.203345$

<u>Other beam</u>

Applied member shear, $V_a = 60\ kips$

$\phi = 1$

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

$= 1 \cdot 0.6 \cdot 50$

$= 30\ ksi$

Web shear area, $A_w = d_s \cdot t_{w,s}$

$= 23.6 \cdot 0.395$

Other beam (continued)

$= 9.322\ in^2$

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

$= \dfrac{60}{30 \cdot 9.322}$

$= 0.214546$

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

$= 30 \cdot 9.322$

$= 279.66\ kips$

Other beam gross shear, $\phi V_{g,s} = Shear\ capacity$

$= 279.7\ kips$

Other beam unity ratio, $U_o = Unity$

$= 0.214546$

$Unity = max\ (U, U_o)$

$= max\ (0.203345, 0.214546)$

$= 0.214546$

$Shear\ capacity = min\ (\phi V_g, \phi V_{g,s})$

$= min\ (295.1, 279.7)$

$= 279.7\ kips$

$279.7\ kips \geq 60\ kips$ **(OK)**

$0.215 \leq 1$ **(OK)**

Beam B_7 [8]

Design method

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

Overview

Section size:	W24x55
Sequence:	1
ABM:	N/Assign
Plan length:	25-0
Camber:	0.00 in
Span length:	25-0
Slope:	0.00 °
Material length:	24-11 3/4
Plan rotation:	0.00 °

Section properties

Material grade:	A992
Yield stress, F_y:	50 ksi
Tensile strength, F_u:	65 ksi
Depth, d:	23.6 in
Web thickness, t_w:	0.395 in
Flange width, b_f:	7.01 in
Flange thickness, t_f:	0.505 in
Design k distance, k_{des}:	1.01 in
Detail k distance, k_{det}:	1.4375 in
Distance between web toes of fillets, T:	20.725 in
Moment of inertia about the major axis, I_x:	1350 in^4

Design summary

Left end

Connection:	Splice plate
	Plates on left end, Near side
Elevation:	0
Minus Dim:	0.25 in
Mtrl Setback:	0.25 in (AUTO)
Std Detail:	None
Web:	Web vertical
End rotation:	0.00 °
Shear:	60.0 kips
Moment:	0.0 kip·ft (AUTO)
Tension:	0.0 kips
Compress:	0.0 kips
Tying:	0.0 kips (AUTO)

B_7 [8] Connection strength check: LEFT END

Member end summary

Connecting nodes

Node 1

Beam:	B_8 [7]
Section size:	W24x68
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}:	1.875 in
Design k distance, k_{des}:	1.09 in
Depth, d:	23.7 in
Web thickness, t_w:	0.415 in
Flange thickness, t_f:	0.585 in

Factored loads

***Shear*:**	60.0 kips

Design load notes

- Non-composite design
- Reaction has been input
- Design reaction is 37.3 % of the allowable uniform steel beam load.

Connection summary

- BOLTED BEAM SHEAR PLATE SPLICE
- (Splice plate on one side of web)

Connection details

Plates:	**Grade:**	A572-50
	Tensile strength, F_u:	65 ksi
	Yield stress, F_y:	50 ksi
Web plates:	**Thickness, t:**	0.375 in
	Depth, d:	12 in
Web 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 columns, m:	1
Gap between members, g:	0.5 in	

Connection design lock summary

Locked Via Member Edit:	20
(at dd) Not Locked:	106

Expanded design calculation

Bolt bearing on web plate(s) (20). Reference J3.11

Number of shear planes, $N_s = 1$
Number of sides, $N = 1$
Row edge distance, $L_e = 1.5\ in$
Connection thickness, $t = 0.375\ in$
Connection tensile strength, $F_u = 65\ ksi$
Bolt row spacing, $s = 3\ in$
Bolt columns, $m = 1$
Bolt rows, $n = 4$
Bolt diameter, $d_b = 0.875\ in$
$C = 3.07968$
Total number of bolts, $N = n \cdot m$
$= 4 \cdot 1$
$= 4$
Number of edge bolts, $N_{edge} = m$
$= 1$
Number of interior bolts, $N_{int} = N - m$
$= 4 - 1$
$= 3$
Total length of bolt group, $s_{total} = 9\ in$
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, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t \cdot F_u$
$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.375 \cdot 65$
$= 38.3906\ kips$

> **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, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t \cdot F_u$

Interior bolt capacity (continued)

$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.375 \cdot 65$

$= 45.2461\ kips$

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

$= min\ (45.2461, 38.3906, 24.3535)$

$= 24.3535\ kips$

Edge bolt capacity

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

$= 1.5 - 0.5 \cdot 0.9375$

$= 1.03125\ in$

$\phi = 0.75$

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

$= 0.75 \cdot 1.2 \cdot 1.03125 \cdot 0.375 \cdot 65$

$= 22.623\ kips$

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

$= min\ (22.623, 38.3906, 24.3535)$

$= 22.623\ kips$

Average bolt bearing/tearout, $\phi R_{v,ave} = \dfrac{(\phi R_{n,e} \cdot N_{edge} + \phi R_{n,i} \cdot N_{int})}{N}$

$= \dfrac{(22.623 \cdot 1 + 24.3535 \cdot 3)}{4}$

$= 23.9209\ kips$

Shear capacity, $\phi V_n = N \cdot \phi R_{v,ave} \cdot C$

$= 1 \cdot 23.9209 \cdot 3.07968$

$= 73.6687\ kips$

$Shear\ capacity = \phi V_n$

$= 73.6687\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \dfrac{60}{73.7}$

$= 0.814111$

$73.7\ kips \geq 60\ kips$ **(OK)**

$0.814 \leq 1$ **(OK)**

Bolt bearing on beam web (20). Reference J3.11

Bolt diameter, $d_b = 0.875\ in$

Number of shear planes, $N_s = 1$

Number of sides, $N = 1$

Bolt rows, $n = 4$

Bolt bearing on beam web (20). Reference J3.11 (continued)

Bolt columns, $m = 1$
Vertical bolt spacing, $s = 3\ in$
This beam tensile strength, $F_u = 65\ ksi$
This beam web thickness, $t_w = 0.395\ in$
Other beam tensile strength, $F_{u,s} = 65\ ksi$
Other beam web thickness, $t_{w,s} = 0.415\ in$

This beam

$C = 3.07968$
Total number of bolts, $N = n \cdot m$
$= 4 \cdot 1$
$= 4$
Number of edge bolts, $N_{edge} = m$
$= 1$
Number of interior bolts, $N_{int} = N - m$
$= 4 - 1$
$= 3$
Total length of bolt group, $s_{total} = 9\ in$
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, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_w \cdot F_u$
$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.395 \cdot 65$
$= 40.4381\ kips$

Interior bolt capacity

Vertical bolt 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, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_w \cdot F_u$
$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.395 \cdot 65$
$= 47.6592\ kips$
Controlling bearing/tearout strength of interior bolt, $\phi R_{n,i} = min\ (\phi R_{n,to}, \phi R_{n,b}, \phi R_{n,v})$

Interior bolt capacity (continued)

$= min\ (47.6592, 40.4381, 24.3535)$

$= 24.3535\ kips$

Edge bolt capacity

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

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

$= min\ (40.4381, 24.3535)$

$= 24.3535\ kips$

Average bolt bearing/tearout, $\phi R_{v,ave} = \frac{(\phi R_{n,e} \cdot N_{edge} + \phi R_{n,i} \cdot N_{int})}{N}$

$= \frac{(24.3535 \cdot 1 + 24.3535 \cdot 3)}{4}$

$= 24.3535\ kips$

Shear capacity, $\phi V_n = N \cdot \phi R_{v,ave} \cdot C$

$= 1 \cdot 24.3535 \cdot 3.07968$

$= 75.0009\ kips$

$Shear\ capacity = \phi V_n$

$= 75.0009\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \frac{60}{75}$

$= 0.8$

Bearing on beam web, $\phi P_{brg} = Shear\ capacity$

$= 75\ kips$

This beam unity ratio, $U = Unity$

$= 0.8$

Other beam

$C = 3.07968$

Total number of bolts, $N = n \cdot m$

$= 4 \cdot 1$

$= 4$

Number of edge bolts, $N_{edge} = m$

$= 1$

Number of interior bolts, $N_{int} = N - m$

$= 4 - 1$

$= 3$

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

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

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

Other beam (continued)

$\phi = 0.75$

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, $\phi R_{n,b} = \phi \cdot 2.4 \cdot d_b \cdot t_{w,s} \cdot F_{u,s}$

$= 0.75 \cdot 2.4 \cdot 0.875 \cdot 0.415 \cdot 65$

$= 42.4856\ kips$

Interior bolt capacity

Vertical bolt 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, $\phi R_{n,to} = \phi \cdot 1.2 \cdot L_{c,int} \cdot t_{w,s} \cdot F_{u,s}$

$= 0.75 \cdot 1.2 \cdot 2.0625 \cdot 0.415 \cdot 65$

$= 50.0723\ kips$

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

$= min\ (50.0723, 42.4856, 24.3535)$

$= 24.3535\ kips$

Edge bolt capacity

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

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

$= min\ (42.4856, 24.3535)$

$= 24.3535\ kips$

Average bolt bearing/tearout, $\phi R_{v,ave} = \dfrac{(\phi R_{n,e} \cdot N_{edge} + \phi R_{n,i} \cdot N_{int})}{N}$

$= \dfrac{(24.3535 \cdot 1 + 24.3535 \cdot 3)}{4}$

$= 24.3535\ kips$

Shear capacity, $\phi V_n = N \cdot \phi R_{v,ave} \cdot C$

$= 1 \cdot 24.3535 \cdot 3.07968$

$= 75.0009\ kips$

$Shear\ capacity = \phi V_n$

$= 75.0009\ kips$

Applied member shear, $V_a = 60\ kips$

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

Other beam (continued)

$= \frac{60}{75}$

$= 0.8$

Bearing on other web, $\phi P_{brg,s} = Shear\ capacity$

$= 75\ kips$

Other beam unity ratio, $U_o = Unity$

$= 0.8$

$Unity = max\ (U, U_o)$

$= max\ (0.8, 0.8)$

$= 0.8$

$Shear\ capacity = min\ (\phi P_{brg}, \phi P_{brg,s})$

$= min\ (75, 75)$

$= 75\ kips$

$75.0\ kips \geq 60\ kips$ **(OK)**

$0.800 \leq 1$ **(OK)**

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

Number of shear planes, $N_s = 1$

Coefficient, $C = 3.07968$

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

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

$\phi = 0.75$

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

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

$= 24.3535\ kips$

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

$= 3.07968 \cdot 24.3535$

$= 75.0009\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \frac{60}{75}$

$= 0.8$

$75.0\ kips \geq 60\ kips$ **(OK)**

$0.800 \leq 1$ **(OK)**

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

Connection tensile strength, $F_{u,conn} = 65\ ksi$

FS bolt rows, $n_{FS} = 4$

NS bolt rows, $n_{NS} = 4$

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

FS connection thickness, $t_{fs} = 0\ in$

NS connection thickness, $t_{ns} = 0.375\ in$

FS connection depth, $d_{fs} = 0\ in$

NS connection depth, $d_{ns} = 12\ in$

Hole diameter, $d_h = 1\ in$

NS Net shear area, $A_{nv,ns} = t_{ns} \cdot (d_{ns} - n_{NS} \cdot d_h)$

$= 0.375 \cdot (12 - 4 \cdot 1)$

$= 3\ in^2$

FS Net shear area, $A_{nv,fs} = t_{fs} \cdot (d_{fs} - n_{FS} \cdot d_h)$

$= 0 \cdot (0 - 4 \cdot 1)$

$= 0\ in^2$

Total net shear area, $A_{nv,total} = A_{nv,ns} + A_{nv,fs}$

$= 3 + 0$

$= 3\ in^2$

$\phi = 0.75$

Shear capacity, $\phi V_n = \phi \cdot 0.6 \cdot F_{u,conn} \cdot A_{nv,total}$

$= 0.75 \cdot 0.6 \cdot 65 \cdot 3$

$= 87.75\ kips$

$Shear\ capacity = \phi V_n$

$= 87.75\ kips$

Applied member shear, $V_a = 60\ kips$

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

$$= \frac{60}{87.8}$$

$= 0.683371$

$87.8\ kips \geq 60\ kips$ **(OK)**

$0.683 \leq 1$ **(OK)**

Block shear rupture of web plate(s) (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, $s_{col} = 5\ in$

Bolt row spacing, $s = 3\ in$

Bolt rows, $n = 4$

Column edge distance, $L_{eh} = 1.5\ in$

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

Bolt columns, $m = 1$

Hole diameter, $d_h = 1\ in$

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

Hole length, $l_h = 1\ in$

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

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

$= 0.375 \cdot (9 + 1.5)$

$= 3.9375\ in^2$

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

$= 0.375 \cdot (9 + 1.5) - 0.375 \cdot (4 - 0.5) \cdot 1$

$= 2.625\ in^2$

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

$= 0.375 \cdot (5 \cdot (1 - 1) + 1.5)$

$= 0.5625\ in^2$

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

$= 0.375 \cdot (5 \cdot (1 - 1) + 1.5) - 0.375 \cdot (1 - 0.5) \cdot 1$

$= 0.375\ in^2$

Reduction coefficient, $U_{bs} = 1$

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

$= 0.6 \cdot 50 \cdot 3.9375$

$= 118.125\ kips$

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

$= 0.6 \cdot 65 \cdot 2.625$

$= 102.375\ kips$

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

$= 1 \cdot 65 \cdot 0.375$

$= 24.375\ kips$

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

$= min\ (118.125, 102.375) + 24.375$

$= 126.75\ kips$

$\phi = 0.75$

$Shear\ capacity = \phi \cdot R_n$

$= 0.75 \cdot 126.75$

$= 95.0625\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \dfrac{60}{95.1}$

$= 0.630915$

$95.1\ kips \geq 60\ kips$ **(OK)**

$0.631 \leq 1$ **(OK)**

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

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

FS connection thickness, $t_{fs} = 0\ in$

NS connection thickness, $t_{ns} = 0.375\ in$

FS connection depth, $d_{fs} = 0\ in$

NS connection depth, $d_{ns} = 12\ in$

Gross shear area, $A_{gv} = d_{ns} \cdot t_{ns} + d_{fs} \cdot t_{fs}$

$= 12 \cdot 0.375 + 0 \cdot 0$

$= 4.5\ in^2$

$\phi = 1$

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

$= 1 \cdot 0.6 \cdot 50 \cdot 4.5$

$= 135\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \frac{60}{135}$

$= 0.444444$

$135.0\ kips \geq 60\ kips$ **(OK)**

$0.444 \leq 1$ **(OK)**

Flexure of web plate(s) (19). Reference F11

$F_{u,conn} = 65\ ksi$

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

Bolt row spacing, $s = 3\ in$

Bolt rows, $n = 4$

Number of connection sides, $N = 1$

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

Connection depth, $d_{pl} = 12\ in$

Eccentricity in x-direction, $e_x = 2.5\ in$

Hole diameter, $d_h = 1\ in$

Gross moment capacity

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

Unbraced Length, $L_b = e_x$

$= 2.5\ in$

Plastic section modulus, $Z = \frac{t_{pl} \cdot d_{pl}^2}{4}$

$= \frac{0.375 \cdot 12^2}{4}$

$= 13.5\ in^3$

Gross moment capacity (continued)

Elastic section modulus, $S = \frac{t_{pl} \cdot d_{pl}^2}{6}$

$= \frac{0.375 \cdot 12^2}{6}$

$= 9\ in^3$

Plastic bending moment, $M_p = \frac{F_{y,p} \cdot Z}{12}$

$= \frac{50 \cdot 13.5}{12}$

$= 56.25\ kip \cdot ft$

$\left(M_p = 56.25\ kip \cdot ft\right) \leq \left(\frac{1.5 \cdot F_{y,p} \cdot S}{12} = \frac{1.5 \cdot 50 \cdot 9}{12} = 56.25\ kip \cdot ft\right)$

$\left(\frac{L_b \cdot d_{pl}}{t_{pl}^2} = \frac{2.5 \cdot 12}{0.375^2} = 213.333\right) > \left(\frac{0.08 \cdot E}{F_{y,p}} = \frac{0.08 \cdot 29000}{50} = 46.4\right)$

Lateral-torsional buckling modification factor, $C_b = 1.84$

$\left(\frac{L_b \cdot d_{pl}}{t_{pl}^2} = \frac{2.5 \cdot 12}{0.375^2} = 213.333\right) \leq \left(\frac{1.9 \cdot E}{F_{y,p}} = \frac{1.9 \cdot 29000}{50} = 1102\right)$

Flexural yield moment, $M_y = \frac{F_{y,p} \cdot S}{12}$

$= \frac{50 \cdot 9}{12}$

$= 37.5\ kip \cdot ft$

Nominal flexural strength, $M_n = min\left(C_b \cdot \left(1.52 - 0.274 \cdot \left(\frac{L_b \cdot d_{pl}}{t_{pl}^2}\right) \cdot \left(\frac{F_{y,p}}{E}\right)\right) \cdot M_y, M_p\right)$

$= min\left(1.84 \cdot \left(1.52 - 0.274 \cdot \left(\frac{2.5 \cdot 12}{0.375^2}\right) \cdot \left(\frac{50}{29000}\right)\right) \cdot 37.5, 56.25\right)$

$= 56.25\ kip \cdot ft$

$\phi = 0.9$

Gross moment capacity, $\phi M_{n,gross} = N \cdot \phi \cdot M_n$

$= 1 \cdot 0.9 \cdot 56.25$

$= 50.625\ kip \cdot ft$

Net moment capacity

$\phi = 0.75$

Bending stress, $\phi F_b = \phi \cdot F_{u,conn}$

$= 0.75 \cdot 65$

$= 48.75\ ksi$

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

Row edge distance top, $L_{e,top} = \frac{\left(d_{pl} - s_{total}\right)}{2}$

Net moment capacity (continued)

$= \frac{(12 - 9)}{2}$

$= 1.5\ in$

Row edge distance bottom, $L_{e,bot} = L_{e,top}$

$= 1.5\ in$

Bolt row spacing, $s = 3\ in$

Net plastic section modulus, $Z_{x,net} = \frac{t_{pl} \cdot (s - d_h) \cdot n^2 \cdot s}{4}$

$= \frac{0.375 \cdot (3 - 1) \cdot 4^2 \cdot 3}{4}$

$= 9\ in^3$

Bolt row spacing, $s = 3\ in$

Deduction of net section modulus due to the bolt holes, $S_{deduct} = \frac{\left(\frac{s^2 \cdot n \cdot (n^2 - 1) \cdot t_{pl} \cdot d_h}{6}\right)}{d_{pl}}$

$= \frac{\left(\frac{3^2 \cdot 4 \cdot (4^2 - 1) \cdot 0.375 \cdot 1}{6}\right)}{12}$

$= 2.8125\ in^3$

Net elastic section modulus, $S_{x,net} = \frac{t_{pl} \cdot d_{pl}^2}{6} - S_{deduct}$

$= \frac{0.375 \cdot 12^2}{6} - 2.8125$

$= 6.1875\ in^3$

$(Z_{x,net} = 9\ in^3) \leq (1.5 \cdot S_{x,net} = 1.5 \cdot 6.1875 = 9.28125\ in^3)$

Net moment capacity, $\phi M_{n,net} = \frac{N \cdot \phi F_b \cdot Z_{x,net}}{12}$

$= \frac{1 \cdot 48.75 \cdot 9}{12}$

$= 36.5625\ kip \cdot ft$, Reference: (9-8)

$Shear\ capacity = \left(\frac{min\ (\phi M_{n,gross}, \phi M_{n,net})}{e_x}\right) \cdot 12$

$= \left(\frac{min\ (50.625, 36.5625)}{2.5}\right) \cdot 12$

$= 175.5\ kips$

Applied member shear, $V_a = 60\ kips$

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

$= \frac{60}{175.5}$

Flexure of web plate(s) (19). Reference F11 (continued)

$= 0.34188$

$175.5\ kips \geq 60\ kips$ **(OK)**

$0.342 \leq 1$ **(OK)**

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

This beam depth, $d = 23.6\ in$

This beam web thickness, $t_w = 0.395\ in$

This beam yield stress, $F_y = 50\ ksi$

Other beam depth, $d_s = 23.7\ in$

Other beam web thickness, $t_{w,s} = 0.415\ in$

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

This beam

Applied member shear, $V_a = 60\ kips$

$\phi = 1$

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

$= 1 \cdot 0.6 \cdot 50$

$= 30\ ksi$

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

$= 23.6 \cdot 0.395$

$= 9.322\ in^2$

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

$$= \frac{60}{30 \cdot 9.322}$$

$= 0.214546$

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

$= 30 \cdot 9.322$

$= 279.66\ kips$

Beam gross shear, $\phi V_g = Shear\ capacity$

$= 279.7\ kips$

This beam unity ratio, $U = Unity$

$= 0.214546$

Other beam

Applied member shear, $V_a = 60\ kips$

$\phi = 1$

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

$= 1 \cdot 0.6 \cdot 50$

$= 30\ ksi$

Web shear area, $A_w = d_s \cdot t_{w,s}$

$= 23.7 \cdot 0.415$

Other beam (continued)

$$= 9.8355\ in^2$$

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

$$= \frac{60}{30 \cdot 9.8355}$$

$$= 0.203345$$

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

$$= 30 \cdot 9.8355$$

$$= 295.065\ kips$$

Other beam gross shear, $\phi V_{g,s} = Shear\ capacity$

$= 295.1\ kips$

Other beam unity ratio, $U_o = Unity$

$= 0.203345$

$Unity = max\ (U, U_o)$

$= max\ (0.214546, 0.203345)$

$= 0.214546$

$Shear\ capacity = min\ (\phi V_g, \phi V_{g,s})$

$= min\ (279.7, 295.1)$

$= 279.7\ kips$

$279.7\ kips \geq 60\ kips$ **(OK)**

$0.215 \leq 1$ **(OK)**

Results summary

Beam Splice Plates on right end of Beam B_8 [7]

Limit state summary

	Calc. Num.	Unity ratio	PHI*Rn	AISC Ref
Bolt bearing on web plate(s):	20	0.814	73.7 kips	J3.11
Bolt bearing on beam web:	20	0.800	75.0 kips	J3.11
Bolt shear of web bolts:	3	0.800	75.0 kips	J3.7, J3.9
Shear rupture of web plate(s):	21	0.683	87.8 kips	J4.2
Block shear rupture of web plate(s):	6	0.631	95.1 kips	J4.3
Shear yielding of web plate(s):	15	0.444	135.0 kips	J4.2
Flexure of web plate(s):	19	0.342	175.5 kips	F11
Shear yielding of beam web:	2	0.215	279.7 kips	G2.1

Connection strength

	Value:	**Unity ratio:**
Shear:	73.7 kips	0.814

Notes and conclusions

- Splice design is based on the smaller beam load and moment.
- The effect of eccentricity is included in the web connection design:
    - La = 2.5 in (0.5 * dist. between C.G.'s of bolt groups).
- CONNECTION IS OK
    - Strength equals or exceeds design loads.

Beam Splice Plates on left end of Beam B_7 [8]

Limit state summary

	Calc. Num.	Unity ratio	PHI*Rn	AISC Ref
Bolt bearing on web plate(s):	20	0.814	73.7 kips	J3.11
Bolt bearing on beam web:	20	0.800	75.0 kips	J3.11
Bolt shear of web bolts:	3	0.800	75.0 kips	J3.7, J3.9

Limit state summary (continued)

Shear rupture of web plate(s):	21	0.683	87.8 kips	J4.2
Block shear rupture of web plate(s):	6	0.631	95.1 kips	J4.3
Shear yielding of web plate(s):	15	0.444	135.0 kips	J4.2
Flexure of web plate(s):	19	0.342	175.5 kips	F11
Shear yielding of beam web:	2	0.215	279.7 kips	G2.1

Connection strength

	Value:	**Unity ratio:**
Shear:	73.7 kips	0.814

Notes and conclusions

- Splice design is based on the smaller beam load and moment.
- The effect of eccentricity is included in the web connection design:
  - La = 2.5 in (0.5 * dist. between C.G.'s of bolt groups).
- CONNECTION IS OK
  - Strength equals or exceeds design loads.