$ORIGIN := 1$

Reference to AISC 14th Edition shapes database:

$T1 :=$ AISC Shapes Database v14.1.xls

$$Row(shape) := \left| \begin{array}{l} \text{for } i \in 1 .. rows(T1) - 1 \\ \quad R \leftarrow i \ \text{ if } \left(T1^{\langle 2 \rangle}\right)_i = shape \\ R \end{array} \right.$$

Solve Example 5.9 Chevron Brace Connection

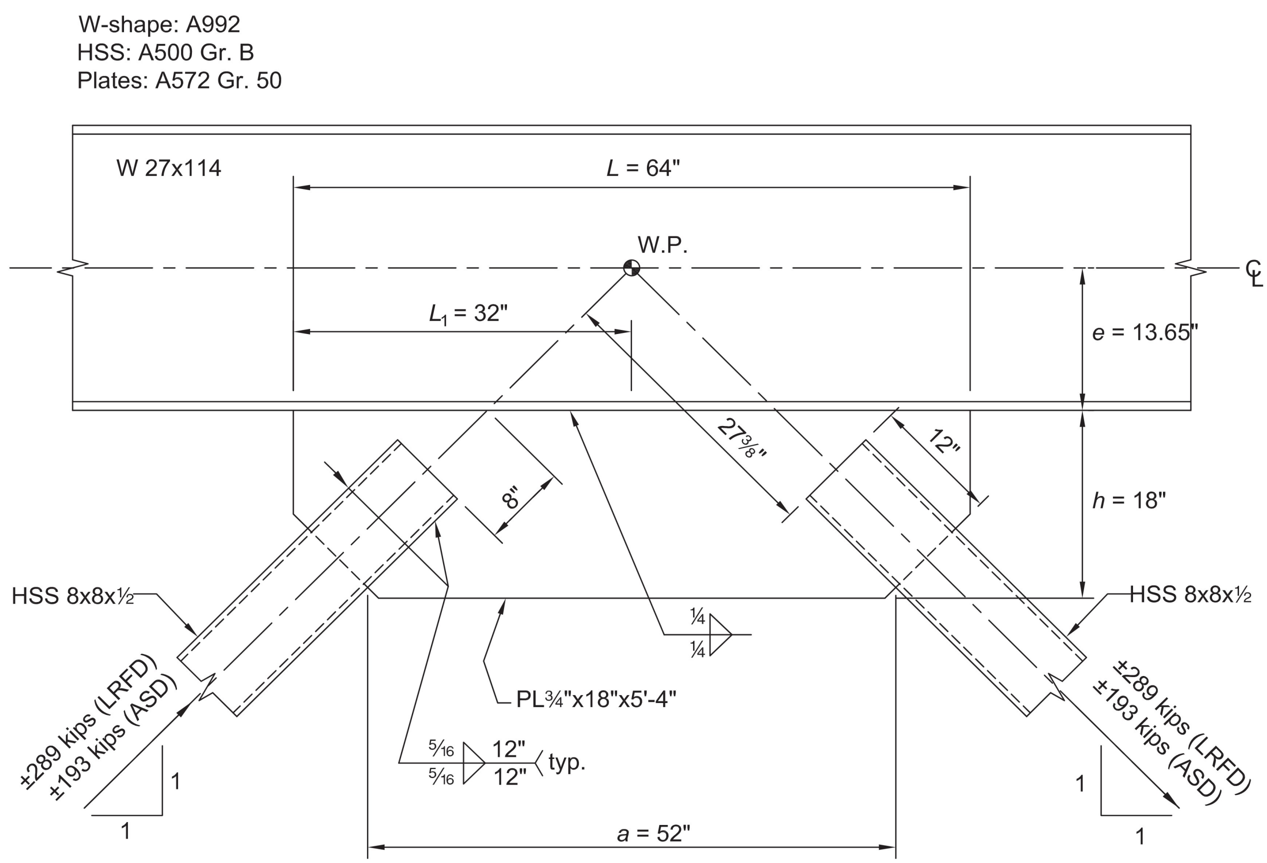

Fig. 5-19. Typical chevron brace connection.

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Loads

LRFD Brace axial force
note : similar in both braces

$P_u := 289kip$

Material Properties

Beams steel grade A992 — $Fy_{beam} := 50 \cdot ksi$ — $Fu_{beam} := 65 \cdot ksi$

Brace steel grade A500 GrB — $Fy_{brace} := 46 \cdot ksi$ — $Fu_{brace} := 58 \cdot ksi$

Plate steel grade A572 Gr. 50 — $Fy_{PL} := 50ksi$ — $Fu_{PL} := 65ksi$

Modulus of Elasticity — $E := 29000ksi$

weld strength — $F_{EXX} := 70ksi$ — $C_1 := 1.0$

Member Properties

Beam Properties

Beam section — $\boxed{Beam := \text{"W27X114"}}$

depth of beam — $d_{beam} := T1_{(Row(Beam),7)} \cdot in = 27.3 \cdot in$

width of flange — $bf_{beam} := T1_{(Row(Beam),12)} \cdot in = 10.1 \cdot in$

thickness of flange — $tf_{beam} := T1_{(Row(Beam),20)}\ in = 0.93\ in$

thickness of web — $tw_{beam} := T1_{(Row(Beam),17)} \cdot in = 0.57 \cdot in$

design weld depth — $k\_des_{brace} := T1_{(Row(Beam),25)} \cdot in = 1.53 \cdot in$

Brace Properties

Brace section — $\boxed{Brace := \text{"HSS8X8X.500"}}$

cross sectional area — $Ag_{brace} := T1_{(Row(Brace),6)} \cdot in^2 = 13.5 \cdot in^2$

radius of gyration — $r_{brace} := T1_{(Row(Brace),42)} \cdot in = 3 \cdot in$ — this is the r.x value (sim to r.y for square shapes)

depth to end of weld — $t\_des_{brace} := T1_{(Row(Brace),24)} \cdot in = 0.465 \cdot in$

$B := T1_{(Row(Brace),9)} \cdot in = 8\ in$

$H := T1_{(Row(Brace),14)} \cdot in = 8\ in$

slope of brace — $slope := atan\left(\frac{12}{12}\right) = 45 \cdot deg$

Plate Properties

thickness $t_{PL} := 0.75in$

length $L := 64in$

height $h := 18in$

cut length $a := 52in$

distance brace to W.P. $e_{WP} := 27.375in$

eccentricity of brace CL to W.P. $e := 13.65\ in$

end of PL to W.P. $L_1 := 32in$

length (centered) $L_2 := L_1$

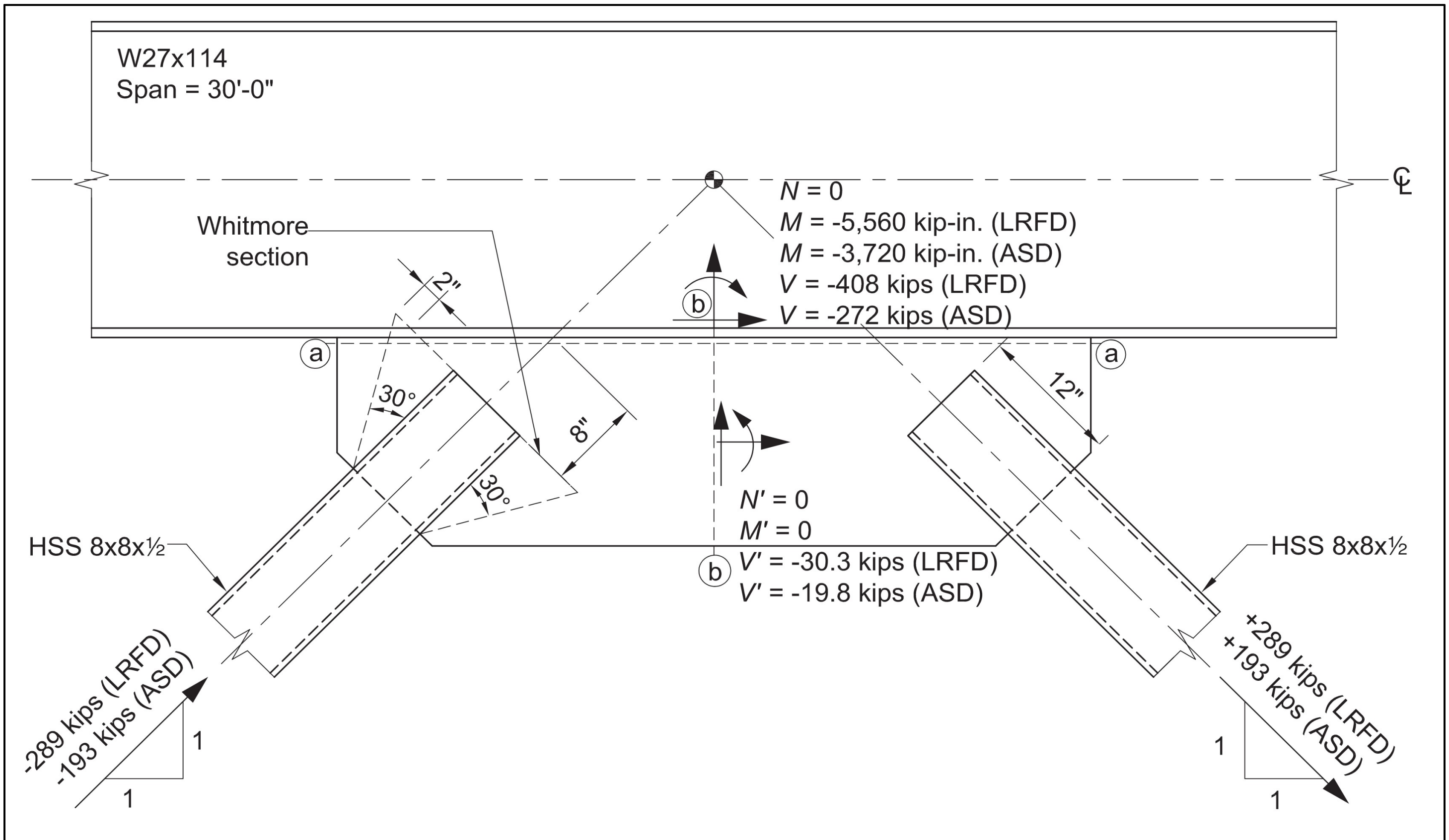

Fig. 5-20. Admissible force fields.

subscript “1” denotes Brace 1, on the left side of Figure 5-19, and the subscript “2” denotes Brace 2 on the right side

$$\Delta := \frac{L_1 - L_2}{2} = \frac{32 \cdot in - 32 \cdot in}{2} = 0\,in$$

$P_{u1} := -289kip$

$H_{u1} := P_{u1} \cdot \cos(slope) = -204.35 \cdot kip$

$V_{u1} := P_{u1} \cdot \sin(slope) = -204.4 \cdot kip$

$M_{u1} := H_{u1} \cdot e + V_{u1} \cdot \Delta = -2789.4 \cdot kip \cdot in$

$M'_{u1} := \frac{1}{8} \cdot V_{u1} \cdot L - \frac{1}{4} \cdot H_{u1} \cdot h - \frac{1}{2} \cdot M_{u1} = 679 \cdot kip \cdot in$

$P_{u2} := 289kip$

$H_{u2} := P_{u2} \cdot \cos(slope) = 204.4 \cdot kip$

$V_{u2} := P_{u2} \cdot \sin(slope) = 204.4 \cdot kip$

$M_{u2} := H_{u2} \cdot e + V_{u2} \cdot \Delta = 2789.4 \cdot kip \cdot in$

$M'_{u2} := \frac{1}{8} \cdot V_{u2} \cdot L - \frac{1}{4} \cdot H_{u2} \cdot h - \frac{1}{2} \cdot M_{u2} = -679 \cdot kip \cdot in$

Axial	$N_u := V_{u1} + V_{u2} = 0 \cdot kip$	recall
Shear	$V_u := H_{u1} - H_{u2} = -408.7 \cdot kip$	$V_{u1} = -204.4\,kip$
Moment	$M_u := M_{u1} - M_{u2} = -5578.9 \cdot kip \cdot in$	$V_{u2} = 204.4\,kip$
		$H_{u1} = -204.4\,kip$
Axial	$N'_u := \frac{1}{2} \cdot (H_{u1} + H_{u2}) = 0 \cdot kip$	$H_{u2} = 204.4\,kip$
		$L = 64\,in$
Shear	$V'_u := \frac{1}{2} \cdot (V_{u1} - V_{u2}) - \frac{2M_u}{L} = -30 \cdot kip$	$M'_{u1} = 679.5\,in \cdot kip$
		$M'_{u2} = -679.5\,in \cdot kip$
Moment	$M'_u := M'_{u1} + M'_{u2} = 0 \cdot kip \cdot in$	

Check tension yielding on the brace (J4.1a)

feduction factor $\Phi := 0.90$

tensile yealding $\Phi R_n := \Phi \cdot Fy_{brace} \cdot Ag_{brace} = 558.9 \cdot kip$

$\text{if}\left(P_u < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

recall: $Fy_{brace} = 46 \cdot ksi$, $Ag_{brace} = 13.5\,in^2$, $P_u = 289\,kip$

shear rupture in the brace wall (J4.2b)

feduction factor $\Phi := 0.75$

$P_u \le \Phi \cdot 0.6 \cdot F_u \cdot A_{nv} = \Phi \cdot 0.6 \cdot F_u \cdot t\_des_{brace} \cdot (4) \cdot l_{weld}$

calculated weld length
(4 lengths of weld)

$$l_{weld} := \frac{P_u}{\left[0.6 \cdot \left[\Phi \cdot Fu_{brace} \cdot t\_des_{brace} \cdot (4)\right]\right]} = 6\,in$$

recall: $P_u = 289\,kip$, $Fu_{brace} = 58 \cdot ksi$, $t\_des_{brace} = 0.5\,in$

check weld strength (EQ: 8-1)

feduction factor $\Phi := 0.75$

try weld size 5/16 inch $w_{weld} := 5in$ 16 th of an inch

weld strength

$$P_u \le \Phi \cdot R_n = \Phi \cdot 0.60 \cdot F_{EXX} \cdot \left(\frac{1}{\sqrt{2}}\right) \cdot w_{weld} \cdot (4) \cdot l_{weld}$$

recall: $P_u = 289\,kip$, $F_{EXX} = 70\,ksi$

minimum weld length

$$l_{weld} \ge \frac{P_u}{\left[\Phi \cdot 0.60 \cdot F_{EXX} \cdot \left(\frac{1}{\sqrt{2}}\right) \cdot w_{weld} \cdot (4)\right]}$$

$$l_{weld} := \frac{P_u}{\left[\Phi \cdot 0.60 \cdot F_{EXX} \cdot \left(\frac{1}{\sqrt{2}}\right) \cdot \frac{w_{weld}}{16} \cdot (4)\right]} = 10.4\,in$$

check weld strength (EQ: 8-2)

weld strength	$\Phi \cdot P_n = 1.392D \cdot l_{weld} \cdot (4)$	recall
minimum weld length	$l_{weld} := \dfrac{P_u}{1.392ksi \cdot w_{weld} \cdot (4)} = 10.4\,in$	$P_u = 289\,kip$ $w_{weld} = 5\,in$
use l_{weld} = 12"	$l_{weld} := 12in$	

Check tensile rupture on the brace (J4.1b)

		recall
slot width	$d_{slot} := t_{PL} + \dfrac{2}{16}in = 0.9\,in$	$t_{PL} = 0.75\,in$
Net area	$An_{brace} := Ag_{brace} - 2 \cdot t\_des_{brace} \cdot d_{slot} = 12.69\,in^2$	$Ag_{brace} = 13.5\,in^2$ $t\_des_{brace} = 0.465\,in$
eccentricity of connection Table D3.1, Case 6	$x\_bar := \dfrac{B^2 + 2 \cdot B \cdot H}{4(B + H)} = 3\,in$	$B = 8\,in$ $H = 8\,in$
shear lag factor Table D3.1, Case 6	$U := 1 - \left(\dfrac{x\_bar}{l_{weld}}\right) = 0.75$	$l_{weld} = 12\,in$ $Fu_{brace} = 58\,ksi$ $P_u = 289\,kip$
effective net area	$Ae_{brace} := An_{brace} \cdot U = 9.51\,in^2$	
feduction factor	$\Phi := 0.75$	
tensile rupture strength	$\Phi R_n := \Phi \cdot Fu_{brace} \cdot Ae_{brace} = 413.9 \cdot kip$	
	$\text{if}\left(P_u < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$	

Check block shear rupture on the gusset plate (J4.3)

		recall
Block shear strength	$R_n = 0.60F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \le 0.60F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$	$t_{PL} = 0.75\,in$
gross shear area	$Agv_{PL} := 2 \cdot t_{PL} \cdot l_{weld} = 18\,in^2$	$l_{weld} = 12\,in$
net shear area	$Anv_{PL} := Agv_{PL} = 18\,in^2$	$Fy_{PL} = 50\,ksi$ $Fu_{PL} = 65\,ksi$
	$0.60 \cdot Fy_{PL} \cdot Agv_{PL} = 540 \cdot kip$	$B = 8\,in$
	$0.60 \cdot Fu_{PL} \cdot Anv_{PL} = 702 \cdot kip$	$P_u = 289\,kip$
	$U_{bs} := 1$	
gross tensile area	$Agt_{PL} := t_{PL} \cdot B = 6\,in^2$	
net tensile area	$Ant_{PL} := Agt_{PL} = 6\,in^2$	

$U_{bs} \cdot Fu_{PL} \cdot Ant_{PL} = 390 \cdot kip$

feduction factor $\Phi := 0.75$

Block shear strength $\Phi R_n := \Phi \cdot \left(\min\left(0.60 \cdot Fy_{PL} \cdot Agv_{PL}, 0.60 \cdot Fu_{PL} \cdot Anv_{PL}\right) + U_{bs} \cdot Fu_{PL} \cdot Ant_{PL}\right) = 697.5 \cdot kip$

$\text{if}\left(P_u < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

Check the gusset plate for tensile yielding on the Whitmore section (J4.1a)

width of Whitmore section (pg: 9-3)
see sketch above $l_w := B + 2 \cdot l_{weld} \cdot \tan(30deg) = 21.9\,in$

gross tensile area $A_w := \left(l_w - 2in\right) \cdot t_{PL} + 2in \cdot \left(tw_{beam}\right) = 16\,in^2$

feduction factor $\Phi := 0.90$

tensile yielding strength $\Phi R_n := \Phi \cdot Fy_{PL} \cdot A_w = 721.5 \cdot kip$

$\text{if}\left(P_u < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

recall

$B = 8\,in$

$l_{weld} = 12\,in$

$t_{PL} = 0.75\,in$

$tw_{beam} = 0.57\,in$

$P_u = 289\,kip$

Check the gusset plate for buckling on the Whitmore section

effective length factor $K_{PL} := 0.65$

length of plate to buckle $L_{PL} := 8in$

radius of gyration $r_{PL} := \sqrt{\dfrac{L_{PL} \cdot t_{PL}^{\,3}}{12 \cdot t_{PL} \cdot L_{PL}}} = 0.217\,in$

$\dfrac{K \cdot L}{r}$ $\dfrac{K_{PL} \cdot L_{PL}}{r_{PL}} = 24$ < 25

recall

$B = 8\,in$

$t_{PL} = 0.8\,in$

Gusset-to-Beam Connection
Check the gusset plate for tensile yielding and shear yielding along the beam flange (J4.1a & J4.2a)

recall

$t_{PL} = 0.75\,in$

$L = 64\,in$

$V_u = -408.7\cdot kip$

$N_u = 0\cdot kip$

$M_u = -5578.9\cdot kip\cdot in$

$Fy_{PL} = 50\,ksi$

gross area $Ag_{PL} := t_{PL}\cdot L = 48\,in^2$

Shear yielding stress $f_{uv} := \dfrac{|V_u|}{Ag_{PL}} = 8.5\cdot ksi$

feduction factor $\Phi := 1.00$

$\Phi R_n := \Phi\cdot 0.60\cdot Fy_{PL} = 30\,ksi$

$if(f_{uv} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$

Tensile yielding stress (due to tension) $f_{ua} := \dfrac{N_u}{Ag_{PL}} = 0\cdot ksi$

plastic section modulus $Z_{PL} := \dfrac{t_{PL}\cdot L^2}{4} = 768\,in^3$

Tensile yielding stress (due to moment) $f_{ub} := \dfrac{|M_u|}{Z_{PL}} = 7.3\cdot ksi$

Total tensile yielding stress $f_{un} := f_{uv} + f_{ub} = 15.8\cdot ksi$

feduction factor $\Phi := 0.90$

$\Phi R_n := \Phi\cdot Fy_{PL} = 45\,ksi$

$if(f_{un} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$

Design weld at gusset-to-beam flange connection (EQ: 8-13)

recall

$M_u = -5578.9\,in\cdot kip$

$V_u = -408.7\,kip$

$L = 64\,in$

$C_1 = 1$

effective eccentricity of the shear force $e_{effective} := \dfrac{M_u}{V_u} = 13.65\,in$

value in table 8-4 $a_{weld} := \dfrac{e_{effective}}{L} = 0.213$

Table 8-4 $C := 3.458$

additioanl ductility (pg: 13-11) $ductility := 1.25$

feduction factor $\Phi := 0.75$

weld strength $V_u < \dfrac{\Phi R_n}{ductility} = \dfrac{(\Phi\cdot C\cdot C_1\cdot D\cdot L)}{ductility}$ $\quad D_{min} > \dfrac{V_u\cdot ductility}{(\Phi\cdot C\cdot C_1\cdot L)}$

minimum weld size $D_{min} := \dfrac{(|V_u| \div kip)\cdot ductility}{\Phi\cdot C\cdot C_1\cdot(L \div in)} = 3.1$

use D = 4 /16th $D_{weld} := 4in$ 16 th of an inch alternative method to determine the required weld size (DG29 Appendix B), (EQ: 8-2 & pg: 8-9)

recall

$M_u = -5578.9\,in \cdot kip$

$V_u = -408.7\,kip$

$N_u = 0\,kip$

$L = 64\,in$

$D_{min} = 3.1$

$ductility = 1.25$

maximum equivalent normal force

$$N_{max} := |N_u| + \left|\frac{2 \cdot M_u}{L \div 2}\right| = 348.7 \cdot kip$$

minimum equivalent normal force

$$N_{min} := \left| |N_u| - \left|\frac{4M_u}{L}\right| \right| = 348.7 \cdot kip$$

peak weld resultant force

$$R_{peak} := \sqrt{V_u^2 + N_{max}^2} = 537.2 \cdot kip$$

average weld resultant force

$$R_{avg} := \sqrt{V_u^2 + \left(\frac{N_{max} + N_{min}}{2}\right)^2} = 537.2 \cdot kip$$

angle of weld loading

$$\theta := atan\left(\frac{N_{max}}{|V_u|}\right) = 40.5 \cdot deg$$

weld strength

$$R_{avg} < \frac{\Phi R_n}{ductility} = \frac{\Phi \cdot 1.392 \cdot \left(1 + 0.5 \cdot \sin(\theta)^{1.5}\right) \cdot D \cdot L \cdot (2)}{ductility}$$

$$1 + 0.5 \cdot \sin(\theta)^{1.5} = 1.26$$

% strength increase

minimum weld size

$$D_{min.2} := \frac{ductility \cdot (R_{avg} \div kip)}{1.392 \cdot \left(1 + 0.5 \cdot \sin(\theta)^{1.5}\right) \cdot (2) \cdot (L \div in)} = 2.99$$

% difference in calculated weld size

$$\frac{D_{min} - D_{min.2}}{D_{min}} = 2.93 \cdot \%$$

Check gusset internal strength

Check the gusset plate for shear yielding (J4.2a)

recall

$V'_u = -30.01 \cdot kip$

$Agv_{PL} = 18\,in^2$

$Fy_{PL} = 50\,ksi$

$$\Phi := 1.00$$

$$\Phi R_n := \Phi \cdot 0.60 \cdot Fy_{PL} \cdot Agv_{PL} = 540 \cdot kip$$

$$if\left(V'_u < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$$

Check gusset stresses on Section b-b for a hypothetical load case (P.1 = P.2= compression)

recall

$N'_u = 0 \cdot kip$

$M'_u = 0 \cdot kip \cdot in$

$M_{u1} = -2789.4 \cdot kip \cdot in$

$H_{u1} = -204.4\, kip$

$e = 13.7\, in$

$L = 64\, in$

$V_{u1} = -204.4\, kip$

$h = 18\, in$

$a = 52\, in$

$t_{PL} = 0.75\, in$

$Fy_{PL} = 50\, ksi$

$$M_{u1\_hypothetical} := H_{u1} \cdot e = -2789.4 \cdot kip \cdot in$$

$$M_{u2\_hypothetical} := M_{u1\_hypothetical} = -2789.4 \cdot kip \cdot in$$

$$M'_{u1\_hypothetical} := \left(\frac{1}{8} \cdot H_{u1} \cdot L - \frac{1}{4} \cdot V_{u1} \cdot h\right) - \frac{1}{2} \cdot M_{u1\_hypothetical}$$

$$M'_{u1\_hypothetical} = 679.5\, in \cdot kip$$

$$M'_{u2\_hypothetical} := M'_{u1\_hypothetical} = 679.5 \cdot kip \cdot in$$

$$N_{u\_hypothetical} := H_{u1} + V_{u1} = -408.7 \cdot kip$$

$$V_{u\_hypothetical} := H_{u1} - V_{u1} = -0 \cdot kip$$

$$M_{u\_hypothetcal} := M_{u1\_hypothetical} - M_{u2\_hypothetical} = 0 \cdot kip \cdot in$$

$$N'_{\_hypothetical} := \frac{1}{2} \cdot \left(H_{u1} + V_{u1}\right) = -204.4 \cdot kip$$

$$V'_{\_hypothetical} := \frac{1}{2} \cdot \left(H_{u1} - V_{u1}\right) + \frac{2in}{L} \cdot \left(V_{u\_hypothetical}\right) = -0 \cdot kip$$

$$M'_{hypothetical} := M'_{u1\_hypothetical} + M'_{u2\_hypothetical} = 1359 \cdot kip \cdot in$$

worst case equivalent normal force (compression)
DG 29 Appendix B

$$N_{ue\_hypothetical} := \left|N'_{\_hypothetical}\right| + \frac{M'_{hypothetical}}{h \div 2} \cdot (2) = 506.3 \cdot kip$$

DG 29 Appendix C.4

$$b := h = 18\, in$$

$$\lambda := \frac{\left(b \div t_{PL}\right) \cdot \sqrt{Fy_{PL} \div ksi}}{5 \cdot \sqrt{475 + \frac{1120}{(a \div b)^2}}} = 1.4$$

$$Q := 1.34 - 0.486 \cdot \lambda = 0.7$$

$$\Phi := 0.9$$

critical buckling stress
DG 29 EQ:C-6

$$\Phi F_{cr} := \Phi \cdot Q \cdot Fy_{PL} = 30.2 \cdot ksi$$

actual (worst case) compressive stress

$$f_{ua\_theoretical} := \frac{N_{ue\_hypothetical}}{t_{PL} \cdot h} = 37.5 \cdot ksi$$

$$\text{if}\left(f_{ua\_theoretical} < \Phi F_{cr}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"NOT OK"}$$

using 7/8" plate

$$t_{PL} := \frac{7}{8} in$$

$$\lambda := \frac{(b \div t_{PL}) \cdot \sqrt{Fy_{PL} \div ksi}}{5 \cdot \sqrt{475 + \frac{1120}{(a \div b)^2}}} = 1.2$$

$$Q := 1.34 - 0.486 \cdot \lambda = 0.8$$

critical buckling stress
DG 29 EQ:C-6

$$\Phi F_{cr} := \Phi \cdot Q \cdot Fy_{PL} = 34.5 \cdot ksi$$

actual (worst case)
compressive stress

$$f_{ua\_theoretical} := \frac{N_{ue\_hypothetical}}{t_{PL} \cdot h} = 32.1 \cdot ksi$$

$$if(f_{ua\_theoretical} < \Phi F_{cr}, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$$

plate thickness

$$t_{PL} := \frac{3}{4} in$$

$$b := 13.6 in$$

see Tamboli (2010)

$$f_u := 29.9 ksi$$

$$\lambda := \frac{(b \div t_{PL}) \cdot \sqrt{Fy_{PL} \div ksi}}{5 \cdot \sqrt{475 + \frac{1120}{(a \div b)^2}}} = 1.1$$

$$Q := 1.34 - 0.486 \cdot \lambda = 0.8$$

critical buckling stress
DG 29 EQ:C-6

$$\Phi F_{cr} := \Phi \cdot Q \cdot Fy_{PL} = 36.4 \cdot ksi$$

$$if(f_u < \Phi F_{cr}, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$$

Check the gusset plate for buckling on the Whitmore section

recall

$L_{PL} = 8 in$

$r_{PL} = 0.217 in$

$A_w = 16.03 in^2$

$P_u = 289 kip$

effective length factor (conservative)

$$K_{buckle} := 1.2$$

length of bucklling member

$$L_{buckle} := \frac{L_{PL}}{\sqrt{2}} = 5.66 in$$

$\frac{K \cdot L}{r}$

$$\frac{K_{buckle} \cdot L_{buckle}}{r_{PL}} = 31.4$$

> 25

Available critical stress (Table 4-22)

$$\Phi F_{cr} := 41.86 ksi$$

Compressive strength (E.3)

$$\Phi P_n := \Phi F_{cr} \cdot A_w = 671.1 \cdot kip$$

$$if(P_u < \Phi P_n, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$$

Check the gusset plate for sidesway buckling

Compressive strength (E.3)

$\Phi P_n := \Phi F_{cr} \cdot t_{PL} \cdot L = 2009.3 \cdot kip$

$\text{if}\left(\left|N_{u\_hypothetical}\right| < \Phi P_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

recall

$\Phi F_{cr} = 41.9\,ksi$

$t_{PL} = 0.75\,in$

$L = 64\,in$

$\left|N_{u\_hypothetical}\right| = 408.7 \cdot kip$

check the beam for web local yielding and local crippling

Check beam web local yielding (J10.2)

length of bearing

strength reduction factor $\Phi := 1.0$

wel local yielding strength $\Phi R_n := \Phi \cdot Fy_{beam} \cdot tw_{beam} \cdot \left(5k\_des_{brace} + L\right) = 2042 \cdot kip$

$\text{if}\left(N_{max} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

recall

$N_{max} = 348.7 \cdot kip$

$k\_des_{brace} = 1.53\,in$

$tw_{beam} = 0.57\,in$

$Fy_{beam} = 50\,ksi$

$L = 64\,in$

Check beam web shear yielding (J4.2a)

strength reduction factor $\Phi := 1.0$

$\Phi R_n = \Phi \cdot 0.60 \cdot Fy_{beam} \cdot Agv_{beam}$

shear yielding strength $\Phi R_n := \Phi \cdot 0.60 \cdot Fy_{beam} \cdot tw_{beam} \cdot L = 1094.4 \cdot kip$

$\text{if}\left(\left|N_{u\_hypothetical}\right| < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

recall

$Fy_{beam} = 50\,ksi$

$tw_{beam} = 0.57\,in$

$L = 64\,in$

$N_{u\_hypothetical} = -408.7\,kip$

Check beam web local crippling (J10.3)

recall

$E = 29000\,ksi$

$tf_{beam} = 0.93\,in$

$d_{beam} = 27.3\,in$

strength reduction factor $\Phi := 0.75$

web local crippling strength

$$\Phi R_n := \Phi \cdot 0.80 \cdot tw_{beam}^{\,2} \cdot \left[1 + 3 \cdot \left(\frac{L}{d_{beam}}\right)\left(\frac{tw_{beam}}{tf_{beam}}\right)^{1.5}\right] \cdot \sqrt{\frac{E \cdot Fy_{beam} \cdot tf_{beam}}{tw_{beam}}} = 1311.7 \cdot kip$$

$\text{if}\left(N_{max} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

Check transverse section web yielding (G2.1)

$V_u := H_{u2} + V'_u = 174.3 \cdot kip$

Area of beam web $Aw_{beam} := tw_{beam} \cdot d_{beam} = 15.6\,in^2$

strength reduction factor $\Phi := 1.0$ $C_v := 1.0$

shear yielding strength $\Phi V_n := \Phi \cdot 0.6 \cdot Fy_{beam} \cdot Aw_{beam} \cdot C_v = 466.8 \cdot kip$

$\text{if}\left(V_u < \Phi V_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

recall

$H_{u2} = 204.4 \cdot kip$

$V'_u = -30 \cdot kip$

$Fy_{beam} = 50\,ksi$

$tw_{beam} = 0.57\,in$

$d_{beam} = 27.3\,in$