ORIGIN := 1

Reference to AISC 14th Edition shapes database:

T1 := AISC Shapes Database v14.1.xls

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

Solve Example 5.1 Corner tensile Flange: General Uniform Force Method

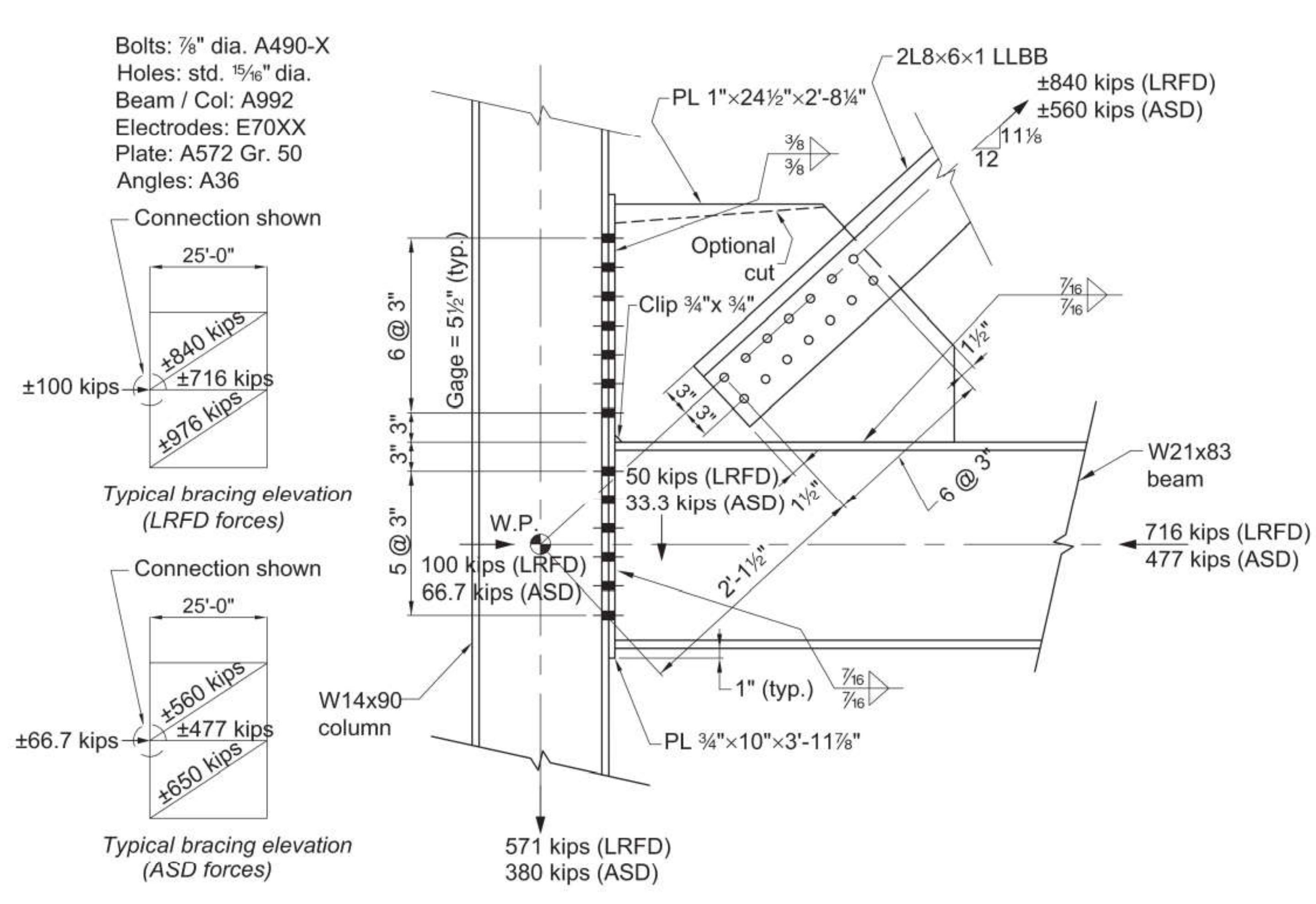


Fig. 5-1. Strong axis bracing connection—general uniform force method.

Loads

LRFD Brace axial force	$P_u := 840\text{kip}$
horizontal nodal force	$Hu_{node} := 100\text{kip}$
beam axial reaction	$Au_{beam} := 716\text{kip}$
beam shear reaction	$Vu_{beam} := 50\text{kip}$

Material Properties

Beams steel grade A992	$Fy_{beam} := 50 \cdot \text{ksi}$	$Fu_{beam} := 65 \cdot \text{ksi}$
Brace steel grade A36	$Fy_{brace} := 36 \cdot \text{ksi}$	$Fu_{brace} := 58 \cdot \text{ksi}$
Plate steel grade A572 Gr. 50	$Fy_{PL} := 50\text{ksi}$	$Fu_{PL} := 65\text{ksi}$
Modulus of Elasticity	$E := 29000\text{ksi}$	
weld strength	$F_{EXX} := 70\text{ksi}$	

Member Properties

Beam Properties

Beam section	$\boxed{\text{Beam} := \text{"W21X83"}}$	$\text{Span} := 25\text{ft}$
depth of beam	$d_{beam} := T1_{(\text{Row}(\text{Beam}),7)} \cdot \text{in} = 21.4 \cdot \text{in}$	
width of flange	$bf_{beam} := T1_{(\text{Row}(\text{Beam}),12)} \cdot \text{in} = 8.4 \cdot \text{in}$	
thickness of flange	$tf_{beam} := T1_{(\text{Row}(\text{Beam}),20)}\, \text{in} = 0.835\, \text{in}$	
thickness of web	$tw_{beam} := T1_{(\text{Row}(\text{Beam}),17)} \cdot \text{in} = 0.515 \cdot \text{in}$	
design weld depth	$k\_des_{brace} := T1_{(\text{Row}(\text{Beam}),25)} \cdot \text{in} = 1.34 \cdot \text{in}$	
center of web to flange toe of fillet	$k\_1_{beam} := T1_{(\text{Row}(\text{Beam}),27)} \cdot \text{in} = 0.875\, \text{in}$	
strong moment of inertia	$Ix_{beam} := T1_{(\text{Row}(\text{Beam}),39)} \cdot \text{in}^4 = 1830\, \text{in}^4$	

Column Properties

$\boxed{\text{Column} := \text{"W14X90"}}$

depth	$d_{column} := T1_{(\text{Row}(\text{Column}),7)} \cdot \text{in} = 14 \cdot \text{in}$
flange width	$bf_{column} := T1_{(\text{Row}(\text{Column}),12)} \cdot \text{in} = 14.5 \cdot \text{in}$
flange thickness	$tf_{column} := T1_{(\text{Row}(\text{Column}),20)}\, \text{in} = 0.71\, \text{in}$
web thickness	$tw_{column} := T1_{(\text{Row}(\text{Column}),17)} \cdot \text{in} = 0.44 \cdot \text{in}$
strong moment of inertia	$Ix_{column} := T1_{(\text{Row}(\text{Column}),39)} \cdot \text{in}^4 = 999\, \text{in}^4$

Brace Properties

Brace section $Brace := \text{"2L8X6X1LLBB"}$

cross sectional area $Ag_{brace} := T1_{(Row(Brace),6)} \cdot in^2 = 26.2 \cdot in^2$

x distance to centroid $x\_bar_{brace} := T1_{(Row(L1),28)} \cdot in = 1.65 \cdot in$

$L1 := \text{"L8X6X1"}$
value for single angle

$t_{brace} := T1_{(Row(Brace),22)} \cdot in = 1\,in$

Plate Properties

thickness $t_{PL} := 1in$

clip thickness $t_{clip} := \frac{3}{4}in$

end PL width $h_{endPL} := 10in$

Member Design

Brace-to-Gusset Connection

Determine required number of bolts (J3.6)

bolt pitch (spacing) $p := 3in$

bolt diameter $d_{bolt} := \frac{7}{8}in$

area of bolt $A_{bolt} := d_{bolt}^2 \cdot \frac{\pi}{4} = 0.6\,in^2$

bolt shear strength $Fnv_{bolt} := 84ksi$

bolt tensile strength $Fnt_{bolt} := 113ksi$

Table J3.2 Group A
threads X

strength reduction factor $\Phi := 0.75$

bolt shear strength (single shear) $\Phi r_{nv} := \Phi\, Fnv_{bolt} \cdot A_{bolt} = 37.9\,kip$ EQ : J3-1

bolt tensile strength $\Phi r_{nt} := \Phi \cdot Fnt_{bolt} \cdot A_{bolt} = 51\,kip$ EQ : J3-1

number of bolts
double shear

$N_{bolts} := \frac{P_u}{\Phi r_{nv} \cdot (2)} = 11.1$

use 14 bolts
$N_{bolts} := 14$

recall
$P_u = 840\,kip$

Check tensile yielding on the brace gross section (J4.1a)

recall

$Fy_{brace} = 36\,ksi$

$Ag_{brace} = 26.2\,in^2$

$P_u = 840\,kip$

strength reduction factor $\Phi := 0.9$

tensile yielding strength of brace $\Phi R_n := \Phi \cdot Fy_{brace} \cdot Ag_{brace} = 848.9\,kip$

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

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

recall

$Ag_{brace} = 26.2\,in^2$

$t_{brace} = 1\,in$

$x\_bar_{brace} = 1.65\,in$

$Fy_{brace} = 36\,ksi$

$Fu_{brace} = 58\,ksi$

$Fy_{PL} = 50\,ksi$

$Fu_{PL} = 65\,ksi$

$N_{bolts} = 14$

$d_{bolt} = 0.875\,in$

$p = 3\,in$

$t_{PL} = 1\,in$

$P_u = 840\,kip$

net area of brace $An_{brace} := Ag_{brace} - 4 \cdot t_{brace} \cdot \left(d_{bolt} + \frac{2}{16} in\right) = 22.2\,in^2$

length of bolted connection $l_{brace} := \left(\frac{14}{2} - 1\right) \cdot 3in = 18\,in$

shear lag factor (Tbl D3.1 case 2) $U := 1 - \frac{x\_bar_{brace}}{l_{brace}} = 0.908$

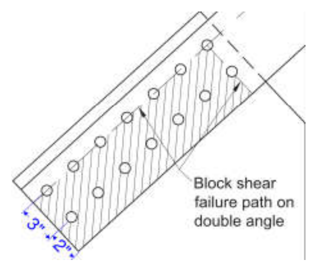
effective area of brace $Ae_{brace} := U \cdot An_{brace} = 20.2\,in^2$

strength reduction factor $\Phi := 0.75$

tensile rupture strength of brace $\Phi R_n := \Phi \cdot Fu_{brace} \cdot Ae_{brace} = 877.2\,kip$

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

Check block shear rupture on the brace (J4.3)



block shear strength $R_n = 0.60F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \le 0.60F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$

gross shear area $Agv_{brace} := (2) \cdot t_{brace} \cdot (l_{brace} + 1.5in) = 39\,in^2$

net area for shear $Anv_{brace} := Agv_{brace} - (2)\left(\frac{14}{2} - 0.5\right) \cdot t_{brace} \cdot \left(d_{bolt} + \frac{2}{16} in\right) = 26\,in^2$

net area for tension $Ant_{brace} := (2) \cdot t_{brace} \left[(p + 2in) - 1.5 \cdot \left(d_{bolt} + \frac{2}{16} in\right)\right] = 7\,in^2$

uniformly loaded $U_{bs} := 1$

$0.60Fu_{brace} \cdot Anv_{brace} = 904.8\,kip$

$0.60Fy_{brace} \cdot Agv_{brace} = 842.4\,kip$

$U_{bs} \cdot Fu_{brace} \cdot Ant_{brace} = 406\,kip$

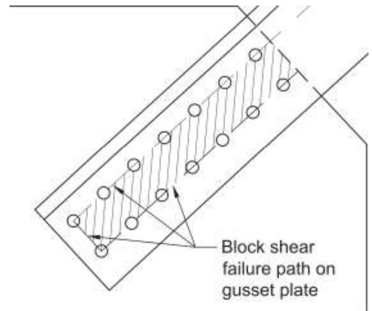
strength reduction factor $\Phi := 0.75$

block shear strength $\Phi R_n := \Phi \cdot \left(0.6 \min\left(Fu_{brace} \cdot Anv_{brace}, Fy_{brace} \cdot Agv_{brace}\right) + U_{bs} \cdot Fu_{brace} \cdot Ant_{brace}\right)$

$\Phi R_n = 936.3\,kip$

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

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



gross shear area $Agv_{PL} := 2 \cdot t_{PL} \cdot (l_{brace} + 1.5\text{in}) = 39\,\text{in}^2$

net area for shear $Anv_{PL} := Agv_{PL} - (2)\left(\frac{14}{2} - 0.5\right) \cdot t_{PL} \cdot \left(d_{bolt} + \frac{2}{16}\text{in}\right)$

$Anv_{PL} = 26\,\text{in}^2$

net area for tension $Ant_{PL} := t_{PL} \cdot \left[3\text{in} - \left(d_{bolt} + \frac{2}{16}\text{in}\right)\right] = 2\,\text{in}^2$

$0.60 Fu_{PL} \cdot Anv_{PL} = 1014\,\text{kip}$

$0.60 Fy_{PL} \cdot Agv_{PL} = 1170\,\text{kip}$

$U_{bs} \cdot Fu_{PL} \cdot Ant_{PL} = 130\,\text{kip}$

uniformly loaded $U_{bs} := 1$

strength reduction factor $\Phi := 0.75$

block shear strength $\Phi R_n := \Phi \cdot \left(0.6 \min\left(Fu_{PL} \cdot Anv_{PL}, Fy_{PL} \cdot Agv_{PL}\right) + U_{bs} \cdot Fu_{PL} \cdot Ant_{PL}\right) = 858\,\text{kip}$

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

Check bolt bearing on the gusset plate (J3.10)

recall

$\Phi r_{nv} = 37.9\,\text{kip}$

$d_{bolt} = 0.875\,\text{in}$

$Fu_{PL} = 65\,\text{ksi}$

$N_{bolts} = 14$

$P_u = 840\,\text{kip}$

bolt bearing strength $R_n = 1.2 \cdot l_c \cdot t \cdot F_u < 2.4 \cdot d \cdot t \cdot F_u$

clear distance (between bolts) $l_{c\_inner} := 3\text{in} - \left(d_{bolt} + \frac{1}{16}\text{in}\right) = 2.06\,\text{in}$

clear distance (between edge) $l_{c\_edge} := 1.5\text{in} - 0.5 \cdot \left(d_{bolt} + \frac{1}{16}\text{in}\right) = 1.03\,\text{in}$

strength reduction factor $\Phi := 0.75$

edge bolt bearing strength $\Phi R_{n\_edge} := \min\left[\Phi \cdot 1.2 \cdot l_{c\_edge} \cdot t_{PL} \cdot Fu_{PL}, \Phi \cdot 2.4 \cdot d_{bolt} \cdot t_{PL} \cdot Fu_{PL}, (2)\Phi r_{nv}\right] = 60.3\,\text{kip}$

inner bolt bearing strength $\Phi R_{n\_inner} := \min\left[\Phi \cdot 1.2 \cdot l_{c\_inner} \cdot t_{PL} \cdot Fu_{PL}, \Phi \cdot 2.4 \cdot d_{bolt} \cdot t_{PL} \cdot Fu_{PL}, (2)\Phi r_{nv}\right] = 75.8\,\text{kip}$

Total bolt bearing strength $\Phi R_n := (2) \cdot \Phi R_{n\_edge} + \left(N_{bolts} - 2\right) \cdot \Phi R_{n\_inner} = 1029.9\,\text{kip}$

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

Check the gusset plate for tensile yielding on the Whitmore section (pg: 9-3)

recall

$l_{brace} = 18\,\text{in}$

$tw_{beam} = 0.515\,\text{in}$

$t_{PL} = 1\,\text{in}$

$Fy_{PL} = 50\,\text{ksi}$

$P_u = 840\,\text{kip}$

whitmore section width $l_w := p + \tan(30\text{deg}) \cdot 2 \cdot l_{brace} = 23.8\,\text{in}$

$l_{w\_web} := 4.70\text{in}$

effective area of the Whitmore section $A_w := \left(l_w - l_{w\_web}\right) \cdot t_{PL} + \left(l_{w\_web}\right) \cdot tw_{beam} = 21.5\,\text{in}^2$

tensile yielding strength of the gusset plate (J4.1a)

strength reduction factor $\Phi := 0.90$

tensile yielding strength $\Phi R_n := \Phi \cdot Fy_{PL} \cdot A_w = 967.7\,\text{kip}$

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

Check the gusset plate for compression buckling on the Whitmore section (J4.4)

recall
$t_{PL} = 1\,\text{in}$
$Fy_{PL} = 50\,\text{ksi}$
$P_u = 840\,\text{kip}$

effective length factor $K_{PL} := 0.5$

length of plate to buckle $L_{PL} := 9.76\text{in}$

radius of gyration $r_{PL} := \sqrt{\frac{L_{PL} \cdot t_{PL}^3}{12 \cdot t_{PL} \cdot L_{PL}}} = 0.289\,\text{in}$

$\frac{K \cdot L}{r}$ $\frac{K_{PL} \cdot L_{PL}}{r_{PL}} = 16.9$ < 25

gross area $Ag_{PL} := t_{PL} \cdot 20.9\text{in} = 20.9\,\text{in}^2$

strength reduction factor $\Phi := 0.90$

compressive strength $\Phi P_n := \Phi \cdot Fy_{PL} \cdot Ag_{PL} = 940.5\,\text{kip}$

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

Connection Interface Forces (chapter 13)

recall
$d_{beam} = 21.4\,\text{in}$
$d_{column} = 14\,\text{in}$
$P_u = 840\,\text{kip}$

beam half depth $e_{beam} := \frac{d_{beam}}{2} = 10.7\,\text{in}$

column half depth $e_{column} := \frac{d_{column}}{2} = 7\,\text{in}$

slope of brace $\theta_{brace} := \text{atan}\left(\frac{12}{11.125}\right) = 47.2 \cdot \text{deg}$

$\alpha - \beta \cdot \tan(\theta) = e_{beam} \cdot \tan(\theta) - e_{column}$ Eq : 13-1

$\beta\_bar := 12\text{in}$

$\beta := \beta\_bar$

$\alpha := e_{beam} \cdot \tan(\theta_{brace}) - e_{column} + \beta \cdot \tan(\theta_{brace}) = 17.5\,\text{in}$

plate thickness $t_p := 1\text{in}$

$$\alpha\_bar = \alpha = \frac{l_h + t_{clip}}{2} + t_p$$

$$l_h := 2(\alpha - t_p) - t_{clip} = 32.2\,\text{in}$$

$$r := \sqrt{(\alpha + e_{column})^2 + (\beta + e_{beam})^2} = 33.4\,\text{in}$$ Eq. 13-6

$$V_{column} := \frac{\beta}{r} \cdot P_u = 301.9\,\text{kip}$$ Eq. 13-2

$$V_{beam} := \frac{e_{beam}}{r} \cdot P_u = 269.2\,\text{kip}$$ Eq. 13-4

$$P_u \cdot \cos(\theta_{brace}) - (V_{column} + V_{beam}) = -0\,\text{kip}$$

$$H_{column} := \frac{e_{column}}{r} \cdot P_u = 176.1\,\text{kip}$$ Eq. 13-3

$$H_{beam} := \frac{\alpha}{r} \cdot P_u = 439.9\,\text{kip}$$ Eq. 13-5

$$P_u \cdot \sin(\theta_{brace}) - (H_{column} + H_{beam}) = -0\,\text{kip}$$

Gusset-to-Beam Connection

Check gusset plate for shear yielding (J4.2a) and tensile yielding (J4.1a) along the beam flange

weld length $l_{weld} := l_h - 0.75\text{in} = 31.5\,\text{in}$

strength reduction factor $\Phi := 1.00$

shear yielding strength $\Phi R_n := \Phi \cdot 0.60 \cdot Fy_{PL} \cdot t_{PL} \cdot l_{weld} = 944.1\,\text{kip}$

check capacity $\text{if}(H_{beam} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$

strength reduction factor $\Phi := 0.90$

Tensile yielding strength $\Phi R_n := \Phi \cdot Fy_{PL} \cdot t_{PL} \cdot l_{weld} = 1416.2\,\text{kip}$

check capacity $\text{if}(V_{beam} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$

recall

$l_h = 32.2\,\text{in}$

$t_{PL} = 1\,\text{in}$

$Fy_{PL} = 50\,\text{ksi}$

$H_{beam} = 439.9\,\text{kip}$

$V_{beam} = 269.2\,\text{kip}$

Consider force interaction for gusset plate

interaction Eq plasticity theory (Neal, 1977) and suggested by Astaneh-Asl (1998)

$$\left(\frac{M_{ub}}{\Phi M_n}\right) + \left(\frac{V_{ub}}{\Phi N_n}\right)^2 + \left(\frac{H_{ub}}{\Phi V_n}\right)^4 < 1$$

recall

$Fy_{PL} = 50\,ksi$

$l_{weld} = 31.5\,in$

$t_{PL} = 1\,in$

$H_{beam} = 439.9\,kip$

$V_{beam} = 269.2\,kip$

strength reduction factor

$$\Phi := 0.90$$

$$\Phi Mn_{PL} = \Phi \cdot F_y \cdot Z_x$$

nominal moment strength (EQ: F2-1)

$$\Phi Mn_{PL} := \Phi \cdot Fy_{PL} \cdot \left(\frac{t_{PL} \cdot l_{weld}^2}{4}\right) = 11142.1\,in \cdot kip$$

interaction equation

$$\left(\frac{0}{\Phi Mn_{PL}}\right) + \left[\frac{V_{beam}}{0.90 \cdot \left(Fy_{PL} \cdot t_{PL} \cdot l_{weld}\right)}\right]^2 + \left[\frac{H_{beam}}{1.0 \cdot \left(0.60 \cdot Fy_{PL} \cdot t_{PL} \cdot l_{weld}\right)}\right]^4 = 0.08$$

Design weld at gusset-to-beam flange connection

recall

$l_{weld} = 31.5\,in$

$H_{beam} = 439.9\,kip$

$V_{beam} = 269.2\,kip$

axial stress in weld

$$f_a := \frac{V_{beam}}{l_{weld}} = 8.6 \cdot \frac{kip}{in}$$

shear stress in weld

$$f_v := \frac{H_{beam}}{l_{weld}} = 14 \cdot \frac{kip}{in}$$

bending stress in weld

$$f_b := 0$$

peak stress

$$f_{peak} := \sqrt{\left(f_a + f_b\right)^2 + f_v^2} = 16.4 \cdot \frac{kip}{in}$$

average stress

$$f_{avg} := \frac{1}{2} \cdot \left[\sqrt{\left(f_a - f_b\right)^2 + f_v^2} + \sqrt{\left(f_a + f_b\right)^2 + f_v^2}\right] = 16.4 \cdot \frac{kip}{in}$$

load angle

$$\theta_{load} := \operatorname{atan}\left(\frac{f_a}{f_v}\right) = 31.5 \cdot deg$$

additional ductility (pg: 13-11)

$$ductility := 1.25$$

stress on weld

$$f_{weld} := \max\left(ductility \cdot f_{avg}, f_{peak}\right) = 20.5 \cdot \frac{kip}{in}$$

gusset to beam flange weld size

$$D := \frac{f_{weld}}{2 \cdot \left(1.392 \frac{kip}{in}\right) \cdot \left(1.0 + 0.50 \cdot \sin\left(\theta_{load}\right)^{1.5}\right)} = 6.2$$

$$D := ceil(D) \cdot in = 7\,in$$ 16 th of an inch

Check beam web local yielding (J10.2)

strength reduction factor: $\Phi := 1.00$

web local yielding strength:

$\Phi R_n = \Phi \cdot F_{yw} \cdot t_w \cdot (2.5k + l_b)$

$\Phi R_n := \Phi \cdot Fy_{beam} \cdot tw_{beam} \cdot (2.5k\_des_{brace} + l_{weld}) = 896.6\,kip$

check capacity: $if(V_{beam} < \Phi R_n, "OK", "NOT\ OK") = "OK"$

recall

$Fy_{beam} = 50\,ksi$

$tw_{beam} = 0.5\,in$

$k\_des_{brace} = 1.3\,in$

$l_{weld} = 31.5\,in$

$V_{beam} = 269.2\,kip$

Check equivalent normal force, Ne

$N_{ue} := V_{beam} + \dfrac{2 \cdot 0}{(l_{weld} \div 2)} = 269.2\,kip$

recall

$tw_{beam} = 0.5\,in$

$l_{weld} = 31.5\,in$

$d_{beam} = 21.4\,in$

recall

$tf_{beam} = 0.84\,in$

$Fy_{beam} = 50\,ksi$

$E = 29000\,ksi$

$V_{beam} = 269.2\,kip$

Check beam web local crippling (J10.3)

strength reduction factor: $\Phi := 0.75$

web local crippling strength:

$\Phi R_n := \Phi \cdot 0.80 \cdot tw_{beam}^{\,2} \cdot \left[1 + 3 \cdot \left(\dfrac{l_{weld}}{d_{beam}}\right) \cdot \left(\dfrac{tw_{beam}}{tf_{beam}}\right)^{1.5}\right] \cdot \sqrt{\dfrac{E \cdot Fy_{beam} \cdot tf_{beam}}{tw_{beam}}} = 765.4\,kip$

check capacity: $if(V_{beam} < \Phi R_n, "OK", "NOT\ OK") = "OK"$

Gusset-to-Column Connection

Design bolts at gusset-to-column connection (J3.6 & J3.7)

number of bolts: $N_{bolts.end\_PL} := 14$

ultimate shear force per bolt: $r_{uv} := \dfrac{V_{column}}{N_{bolts.end\_PL}} = 21.6\,kip$

check capacity: $if(r_{uv} < \Phi r_{nv}, "OK", "NOT\ OK") = "OK"$

ultimate tensile force per bolt: $r_{ut} := \dfrac{H_{column}}{N_{bolts.end\_PL}} = 12.6\,kip$

check capacity: $if(r_{ut} < \Phi r_{nt}, "OK", "NOT\ OK") = "OK"$

recall

$Fnv_{bolt} = 84\,ksi$

$Fnt_{bolt} = 113\,ksi$

$\Phi r_{nv} = 37.9\,kip$

$\Phi r_{nt} = 51\,kip$

$H_{column} = 176.1\,kip$

$V_{column} = 301.9\,kip$

$A_{bolt} = 0.6\,in^2$

strength reduction factor: $\Phi := 0.75$

modified nominal tensile stress factored to include shear stress efects:

$F'_{nt} = 1.3 \cdot F_{nt} - \dfrac{F_{nt}}{\Phi \cdot F_{nv}} \cdot f_{rv} < F_{nt}$

$F'nt_{bolt} := \min\left[1.3 \cdot Fnt_{bolt} - \dfrac{Fnt_{bolt}}{\Phi \cdot Fnv_{bolt}} \cdot \left(\dfrac{r_{uv}}{A_{bolt}}\right), Fnt_{bolt}\right] = 82.6\,ksi$

combined loading strength: $\Phi R_n := \Phi \cdot F'nt_{bolt} \cdot A_{bolt} = 37.2\,kip$

check capacity: $if(r_{ut} < \Phi R_n, "OK", "NOT\ OK") = "OK"$

Design gusset-to-end plate weld (Eq: 8-2)

recall
$H_{column} = 176.1\,kip$
$V_{column} = 301.9\,kip$
$N_{bolts.end\_PL} = 14$
$p = 3\,in$

resultant load on weld $R_u := \sqrt{H_{column}^2 + V_{column}^2} = 349.5\,kip$

resultant weld load angle $\theta_{resultant} := atan\left(\frac{H_{column}}{V_{column}}\right) = 30.3\cdot deg$

end plate effective length of weld $l_{weld.end\_PL} := p\cdot\frac{N_{bolts.end\_PL}}{2} = 21\,in$

end plate weld size $D_{end\_PL} := \frac{R_u}{(2)\cdot\left(1.392\frac{kip}{in}\right)\cdot l_{weld.end\_PL}\cdot\left(1.0 + 0.50\cdot\sin\left(\theta_{resultant}\right)^{1.5}\right)} = 5.1$

$D_{end\_PL} := ceil\left(D_{end\_PL}\right) = 6$

Check gusset plate tensile and shear yielding at the gusset-to-end-plate interface (J4.1a & J4.2a)

recall
$l_w = 23.8\,in$
$t_{PL} = 1\,in$
$Fy_{PL} = 50\,ksi$
$H_{column} = 176.1\,kip$
$V_{column} = 301.9\,kip$

strength reduction factor $\Phi := 0.90$

Tensile yielding strength $\Phi N_n := \Phi\cdot Fy_{PL}\cdot\left(t_{PL}\cdot l_w\right) = 1070.3\,kip$

check capacity $if\left(H_{column} < \Phi N_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

strength reduction factor $\Phi := 1.00$

shear yielding strength $\Phi V_n := \Phi\cdot 0.60\cdot Fy_{PL}\cdot\left(t_{PL}\cdot l_w\right) = 713.5\,kip$

check capacity $if\left(V_{column} < \Phi N_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

Check prying action on bolts at the end plate (Part 9 and Figure 5-4a)

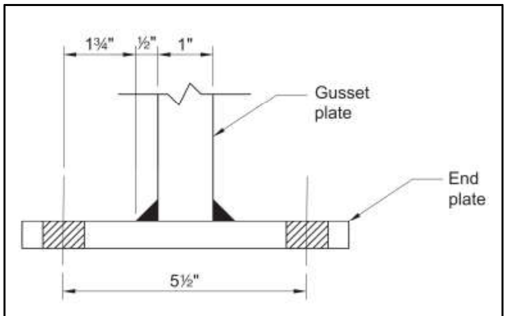


Fig. 5-4a. Washer clearance for end-plate bolts.

bolt gage $gage := 5.5in$

$b_{endPL} := \frac{gage - t_{PL}}{2} = 2.3\,in$

Eq 9-21 $b'_{endPL} := b_{endPL} - \frac{d_{bolt}}{2} = 1.8\,in$

$a_{endPL} := \frac{h_{endPL} - gage}{2} = 2.25\,in$

recall
$t_{PL} = 1\,in$
$d_{bolt} = \frac{7}{8}\,in$
$h_{endPL} = 10\,in$
$\Phi r_{nt} = 51\,kip$
$p = 3\,in$
$\Phi r_{nt} = 51\,kip$
$Fu_{PL} = 65\,ksi$
$r_{ut} = 12.6\,kip$

Eq 9-27 $a'_{endPL} := min\left(a_{endPL} + \frac{d_{bolt}}{2}, 1.25\cdot b_{endPL} + \frac{d_{bolt}}{2}\right) = 2.69\,in$

$\rho_{endPL} := \frac{b'_{endPL}}{a'_{endPL}} = 0.674$

$d' := d_{bolt} + \frac{1}{16}in = \frac{15}{16}\,in$

Eq 9-24 $\delta_{endPL} := 1 - \frac{d'}{p} = 0.69$

available tensile bolt strength $B := \Phi r_{nt} = 51\,\text{kip}$

strength reduction factor $\Phi := 0.90$ $B := 24.2\text{kip}$

thickness to prevent prying action
Eq 9-30 $t_c := \sqrt{\dfrac{4 \cdot B \cdot b'_{endPL}}{\Phi \cdot p \cdot Fu_{PL}}} = 1\,\text{in}$

Eq 9-35 $\alpha' := \dfrac{1}{\delta_{endPL} \cdot \left(1 + \rho_{endPL}\right)} \cdot \left[\left(\dfrac{t_c}{t_{PL}}\right)^2 - 1\right] = -0$

Eq 9-33 $Q := \left(\dfrac{t_{PL}}{t_c}\right)^2 \cdot \left(1 + \delta_{endPL} \cdot \alpha'\right) = 1$

available strength $T_{avail} := B \cdot Q = 24.2\,\text{kip}$

check capcity $\text{if}\left(r_{ut} < T_{avail}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

try t_endPL=5/8" $t_{endPL} := \dfrac{5}{8}\text{in}$

$$\alpha' := \frac{1}{\delta_{endPL} \cdot \left(1 + \rho_{endPL}\right)} \cdot \left[\left(\frac{t_c}{t_{endPL}}\right)^2 - 1\right] = 1.35$$

$$Q := \left(\frac{t_{endPL}}{t_c}\right)^2 \cdot \left(1 + \delta_{endPL}\right) = 0.66$$

available strength $T_{avail} := B \cdot Q = 16\,\text{kip}$

check capcity $\text{if}\left(r_{ut} < T_{avail}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

Check bolt bearing at bolt holes on end plate (J3.10a)

recall
$\Phi r_{nv} = 37.9\,\text{kip}$
$d_{bolt} = 0.875\,\text{in}$
$t_{endPL} = 0.625\,\text{in}$
$Fu_{PL} = 65\,\text{ksi}$

clear distance $l_{c\_endPL} := 1.75\text{in} - 0.5 \cdot \left(d_{bolt} + \dfrac{1}{16}\text{in}\right) = 1.28\,\text{in}$

strength reduction factor $\Phi := 0.75$

bolt bearing strength $\Phi R_n := \min\left(\Phi \cdot 1.2 \cdot l_{c\_endPL} \cdot t_{endPL} \cdot Fu_{PL}, \Phi \cdot 2.4 \cdot d_{bolt} \cdot t_{endPL} \cdot Fu_{PL}, \Phi r_{nv}\right) = 37.9\,\text{kip}$

check capcity $\text{if}\left(\Phi R_n \ge \Phi r_{nv}, \text{"bolt shear governs"}, \text{"check bolt bearing"}\right) = \text{"bolt shear governs"}$

Check block shear rupture of the end plate (J4.3)

recall

$V_{column} = 301.9\,kip$

$N_{bolts.end\_PL} = 14$

$t_{endPL} = 0.625\,in$

block shear strength $R_n = 0.60F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \le 0.60F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$

gross shear area (1 side) $Agv_{endPL} := \left[\left(\frac{N_{bolts.end\_PL}}{2} - 1\right) \cdot p + 1.75in\right] \cdot t_{endPL} = 12.3\,in^2$

net shear area (1 side) $Anv_{endPL} := Agv_{endPL} - t_{endPL} \cdot \left(\frac{N_{bolts.end\_PL}}{2} - 0.5\right) \cdot \left(d_{bolt} + \frac{2}{16}in\right) = 8.28\,in^2$

net tensile area (1 side) $Ant_{endPL} := t_{endPL} \cdot \left[\frac{h_{endPL} - gage}{2} - 0.5 \cdot \left(d_{bolt} + \frac{2}{16}in\right)\right] = 1.09\,in^2$

$0.60Fu_{PL} \cdot Anv_{endPL} = 323\,kip$

$U_{bs} := 1$

$0.60Fy_{PL} \cdot Agv_{endPL} = 370.3\,kip$

$U_{bs} \cdot Fu_{PL} \cdot Ant_{endPL} = 71.1\,kip$

strength reduction factor $\Phi := 0.75$

block shear strength $\Phi R_n := \Phi \cdot (2)\left(0.6\min\left(Fu_{PL} \cdot Anv_{endPL}, Fy_{PL} \cdot Agv_{endPL}\right) + U_{bs} \cdot Fu_{PL} \cdot Ant_{endPL}\right) = 591.1\,kip$

check capcity $\text{if}\left(V_{column} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

Check prying action on column flange

$b_{column} := \frac{gage - tw_{column}}{2} = 2.5\,in$

recall

$gage = 5.5\,in$

$tw_{column} = 0.44\,in$

$d_{bolt} = 0.875\,in$

$bf_{column} = 14.5\,in$

$tf_{column} = 0.71\,in$

$p = 3\,in$

$Fu_{beam} = 65\,ksi$

$r_{ut} = 12.6\,kip$

$b'_{column} := b_{column} - \frac{d_{bolt}}{2} = 2.1\,in$

$a_{column} := \frac{bf_{column} - gage}{2} = 4.5\,in$ > use $a_{endPL} = 2.25\,in$

$a'_{column} := \min\left(a_{endPL} + \frac{d_{bolt}}{2}, 1.25 \cdot b_{column} + \frac{d_{bolt}}{2}\right) = 2.69\,in$

$\rho_{column} := \frac{b'_{column}}{a'_{column}} = 0.78$

$\delta_{column} := 1 - \frac{d'}{p} = 0.69$

$\Phi := 0.90$

$B = 24.2\,kip$

$t_c := \sqrt{\frac{4 \cdot B \cdot b'_{column}}{\Phi \cdot p \cdot Fu_{beam}}} = 1.074\,in$

$\alpha' := \frac{1}{\delta_{column} \cdot \left(1 + \rho_{column}\right)} \cdot \left[\left(\frac{t_c}{tf_{column}}\right)^2 - 1\right] = 1.05$

$Q := \left(\frac{tf_{column}}{t_c}\right)^2 \cdot \left(1 + \delta_{column}\right) = 0.74$

available strength $T_{avail} := B \cdot Q = 17.8\,kip$

check capacity $\text{if}\left(r_{ut} < T_{avail}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

more realistic model

number of bolts in row $n := N_{bolts.end\_PL} \div 2 = 7$

$$a\_bar := \frac{bf_{column} - gage}{2}$$

$$b\_bar := b_{column} = 2.5\,in$$

replace p with p.eff
$$p_{eff} := \frac{(n-1)p + \pi \cdot b\_bar + 2 \cdot a\_bar}{n} = 5\,in$$

$$\delta_{column\_eff} := 1 - \frac{d'}{p_{eff}} = 0.81$$

$$t_c := \sqrt{\frac{4 \cdot B \cdot b'_{column}}{\Phi \cdot p_{eff} \cdot Fu_{beam}}} = 0.833\,in$$

$$\alpha' := \frac{1}{\delta_{column\_eff} \cdot (1 + \rho_{column})} \cdot \left[\left(\frac{t_c}{tf_{column}} \right)^2 - 1 \right] = 0.26$$

$$Q_{eff} := \left(\frac{tf_{column}}{t_c} \right)^2 \cdot (1 + \delta_{endPL} \cdot \alpha') = 0.86$$

available trength $T_{avail.eff} := B \cdot Q_{eff} = 20.7\,kip$

check capacity $\text{if}(r_{ut} < T_{avail.eff}, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$

recall
$N_{bolts.end\_PL} = 14$
$bf_{column} = 14.5\,in$
$b_{column} = 2.5\,in$
$gage = 5.5\,in$
$Fu_{beam} = 65\,ksi$
$tf_{column} = 0.7\,in$
$T_{avail} = 17.8\,kip$

$B = 24.2\,kip$

percent change
$$\frac{T_{avail.eff} - T_{avail}}{T_{avail}} = 16.3 \cdot \%$$

Check bearing on column flange

$tf_{column} = 0.7\,in$ > $t_{endPL} = \frac{5}{8}\,in$

Beam-to-Column Connection

$$b := \frac{Span}{2} = 150\,in$$

$$Height := \frac{Span}{\tan(\theta_{brace})} = 278.1\,in$$

$$c := \frac{Height}{2} = 139.1\,in$$

DG 29 EQ 4-12
$$M_{uD} = 6 \cdot \left(\frac{P}{A \cdot b \cdot c} \right) \left(\frac{I_b \cdot I_c}{\frac{I_b}{b} + \frac{2I_c}{c}} \right) \cdot \left(\frac{b^2 + c^2}{bc} \right)$$

$$M_{uD} := 6 \cdot \left(\frac{P_u}{Ag_{brace} \cdot b \cdot c} \right) \left(\frac{Ix_{beam} \cdot Ix_{column}}{\frac{Ix_{beam}}{b} + \frac{2Ix_{column}}{c}} \right) \cdot \left(\frac{b^2 + c^2}{b \cdot c} \right)$$

$$M_{uD} = 1272.8\,in \cdot kip$$

recall
$Span = 25 \cdot ft$
$\theta_{brace} = 47.2 \cdot deg$
$Ix_{beam} = 1830\,in^4$
$Ix_{column} = 999\,in^4$
$Ag_{brace} = 26.2\,in^2$
$e_{beam} = 10.7\,in$
$\beta\_bar = 12\,in$
$H_{column} = 176.1\,kip$
$P_u = 840\,kip$
$V_{beam} = 269.2\,kip$

$$H_{uD} := \frac{M_{uD}}{\beta\_bar + e_{beam}} = 56.1\,kip$$

$$H_u := H_{column} - H_{uD} + Hu_{node} = 220\,kip$$

required shear strength $V_u := V_{beam} + Vu_{beam} = 319.2\,kip$

required axial strength $T_u := H_u = 220\,kip$

Design bolts at beam-to-column connection (J3.6 & J3.7)

recall
$\Phi r_{nv} = 37.9\,kip$
$\Phi r_{nt} = 51\,kip$
$Fnt_{bolt} = 113\,ksi$
$Fnv_{bolt} = 84\,ksi$
$A_{bolt} = 0.6\,in^2$
$V_u = 319.2\,kip$
$T_u = 220\,kip$

number of bolts $N_{bolts\_beam} := 12$

ultimate shear force per bolt $$r_{uv} := \frac{V_u}{N_{bolts\_beam}} = 26.6\,kip$$

check capacity $\text{if}\left(r_{uv} < \Phi r_{nv}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

ultimate tensile force per bolt $$r_{ut} := \frac{T_u}{N_{bolts\_beam}} = 18.3\,kip$$

check capacity $\text{if}\left(r_{ut} < \Phi r_{nt}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

strength reduction factor $\Phi := 0.75$

modified nominal tensile stress factored to include shear stress efects

$$F'_{nt} = 1.3 \cdot F_{nt} - \frac{F_{nt}}{\Phi \cdot F_{nv}} \cdot f_{rv} < F_{nt}$$

$$F'nt_{bolt} := \min\left[1.3 \cdot Fnt_{bolt} - \frac{Fnt_{bolt}}{\Phi \cdot Fnv_{bolt}} \cdot \left(\frac{r_{uv}}{A_{bolt}}\right), Fnt_{bolt}\right] = 67.6\,ksi$$

combined loading strength $\Phi R_n := \Phi \cdot F'nt_{bolt} \cdot A_{bolt} = 30.5\,kip$

check capacity $\text{if}\left(r_{ut} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

try Gr 490 bolts

nominal tensile stress $Fnt_{bolt\_A490} := 113\,ksi$ $\Phi := 0.75$ factored tensile strength per bolt

nominal shear stress $Fnv_{bolt\_A490} := 84\,ksi$ $B := \Phi \cdot Fnt_{bolt\_A490} \cdot A_{bolt} = 51\,kip$

modified nominal tensile stress factored to include shear stress efects

$$F'nt_{bolt\_A490} := \min\left[1.3 \cdot Fnt_{bolt\_A490} - \frac{Fnt_{bolt\_A490}}{\Phi \cdot Fnv_{bolt\_A490}} \cdot \left(\frac{r_{uv}}{A_{bolt}}\right), Fnt_{bolt\_A490}\right] = 67.6\,ksi$$

strength reduction factor $\Phi := 0.75$

combined loading strength $\Phi R_{n\_A490} := \Phi \cdot F'nt_{bolt\_A490} \cdot A_{bolt} = 30.5\,kip$

check capacity $\text{if}\left(r_{ut} < \Phi R_{n\_A490}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

Design beam web-to-end plate weld (Eq 8-2)

resultant force $R_u := \sqrt{V_u^2 + T_u^2} = 387.7\,\text{kip}$

angle of load $\theta_{b\_endPL} := \text{atan}\left(\frac{T_u}{V_u}\right) = 34.6 \cdot \text{deg}$

effective length of connection $l_{b\_endPL} := \frac{N_{bolts\_beam}}{2} \cdot p$

end plate weld size

$$D_{b\_endPL} := \frac{R_u}{(2)\cdot\left(1.392\,\frac{\text{kip}}{\text{in}}\right)\cdot l_{b\_endPL}\cdot\left(1.0 + 0.50\cdot\sin\left(\theta_{b\_endPL}\right)^{1.5}\right)} = 6.4$$

$D_{b\_endPL} := \text{ceil}\left(D_{b\_endPL}\right)\cdot\text{in} = 7\,\text{in}$

recall
$N_{bolts\_beam} = 12$
$V_u = 319.2\,\text{kip}$
$T_u = 220\,\text{kip}$

Check the 5.5in. gage with 7/16in. fillet welds (Table 7-15)

required clearance $C_{3\_req} := \frac{7}{8}\text{in}$

clearance $\text{clearance} := \frac{\text{gage}}{2} - \frac{tw_{beam}}{2} - \frac{D_{b\_endPL}}{16} = 2.05\,\text{in}$

check clearance $\text{if}\left(C_{3\_req} < \text{clearance}, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$

recall
$tw_{beam} = 0.515\,\text{in}$
$\text{gage} = 5.5\,\text{in}$
$D_{b\_endPL} = 7\,\text{in}$

Check prying action on bolts and end plate (Chapter 9)

beam end plate thickness $t_{b\_endPL} := \frac{5}{8}\text{in}$

$b_{beam} := \frac{\text{gage} - tw_{beam}}{2} = 2.5\,\text{in}$

$b'_{beam} := b_{beam} - \frac{d_{bolt}}{2} = 2.1\,\text{in}$

$a_{beam} := \frac{h_{endPL} - \text{gage}}{2} = 2.3\,\text{in}$

$a'_{beam} := \min\left(a_{beam} + \frac{d_{bolt}}{2}, 1.25 b_{beam} + \frac{d_{bolt}}{2}\right) = 2.7\,\text{in}$

$\rho_{b\_endPL} := \frac{b'_{beam}}{a'_{beam}} = 0.76$

$\delta_{b\_endPL} := 1 - \frac{d'}{p} = 0.69$

recall
$tw_{beam} = 0.515\,\text{in}$
$\text{gage} = 5.5\,\text{in}$
$d_{bolt} = 0.875\,\text{in}$
$h_{endPL} = 10\,\text{in}$
$p = 3\,\text{in}$
$Fu_{PL} = 65\,\text{ksi}$
$r_{ut} = 18.3\,\text{kip}$

$\Phi := 0.90$

$B = 51\,\text{kip}$

$B := 30.4\text{kip}$

$$t_c := \sqrt{\frac{4 \cdot B \cdot b'_{beam}}{\Phi \cdot p \cdot Fu_{PL}}} = 1.19\,\text{in}$$

$$\alpha'_{b\_endPL} := \frac{1}{\delta_{b\_endPL} \cdot (1 + \rho_{b\_endPL})} \cdot \left[\left(\frac{t_c}{t_{b\_endPL}}\right)^2 - 1\right] = 2.2$$

$$Q_{b\_endPL} := \left(\frac{t_{b\_endPL}}{t_c}\right)^2 \cdot (1 + \delta_{b\_endPL}) = 0.46$$

$$T_{avail.b\_endPL} := B \cdot Q_{b\_endPL} = 14.1\,\text{kip}$$

$$\text{if}(r_{ut} < T_{avail.b\_endPL}, \text{"OK"}, \text{"NOT OK"}) = \text{"NOT OK"}$$

use thicker plate

$$t_{b\_endPL} := \frac{3}{4}\text{in}$$

$$\alpha'_{b\_endPL} := \frac{1}{\delta_{b\_endPL} \cdot (1 + \rho_{b\_endPL})} \cdot \left[\left(\frac{t_c}{t_{b\_endPL}}\right)^2 - 1\right] = 1.3$$

$$Q_{b\_endPL} := \left(\frac{t_{b\_endPL}}{t_c}\right)^2 \cdot (1 + \delta_{b\_endPL}) = 0.67$$

available strength

$$T_{avail.b\_endPL} := B \cdot Q_{b\_endPL} = 20.3\,\text{kip}$$

check capcity

$$\text{if}(r_{ut} < T_{avail.b\_endPL}, \text{"OK"}, \text{"NOT OK"}) = \text{"OK"}$$

Check prying action on column flange

$$t_c := \sqrt{\frac{4 \cdot B \cdot b'_{column}}{\Phi \cdot p \cdot Fu_{beam}}} = 1.2\,\text{in}$$

$$\alpha'_{column} := \frac{1}{\delta_{column} \cdot (1 + \rho_{column})} \cdot \left[\left(\frac{t_c}{tf_{column}}\right)^2 - 1\right] = 1.5$$

$$Q_{column} := \left(\frac{tf_{column}}{t_c}\right)^2 \cdot (1 + \delta_{column} \cdot 1) = 0.59$$

$$T_{avail.column} := B \cdot Q_{column} = 17.8\,\text{kip}$$

$$\text{if}(r_{ut} < T_{avail.column}, \text{"OK"}, \text{"NOT OK"}) = \text{"NOT OK"}$$

recall

$b'_{column} = 2.09\,\text{in}$

$a'_{column} = 2.7\,\text{in}$

$\rho_{column} = 0.8$

$p = 3\,\text{in}$

$\delta_{column} = 0.688$

$Fu_{beam} = 65\,\text{ksi}$

$tf_{column} = 0.7\,\text{in}$

$r_{ut} = 18.3\,\text{kip}$

$B = 30.4\,\text{kip}$

Check bolt bearing at end plate

Check block shear rupture on end plate (J4.3)

recall

$h_{endPL} = 10\,in$

$t_{b\_endPL} = 0.8\,in$

$N_{bolts\_beam} = 12$

$d_{bolt} = 0.875\,in$

$gage = 5.5\,in$

block shear strength $R_n = 0.60F_u \cdot A_{nv} + U_{bs} \cdot F_u \cdot A_{nt} \le 0.60F_y \cdot A_{gv} + U_{bs} \cdot F_u \cdot A_{nt}$

fross area in shear (1 side) $Agv_{b\_endPL} := \left[\left(\frac{N_{bolts\_beam}}{2} - 1\right) \cdot p + 4.40in\right] \cdot t_{b\_endPL}$

$Agv_{b\_endPL} = 14.5\,in^2$

net shear area (1 side) $Anv_{b\_endPL} := Agv_{b\_endPL} - t_{b\_endPL} \cdot \left[\left(\frac{N_{bolts\_beam}}{2} - 0.5\right) \cdot \left(d_{bolt} + \frac{2}{16}in\right)\right] = 10.4\,in^2$

net tensile area (1 side) $Ant_{b\_endPL} := t_{b\_endPL} \cdot \left[\frac{h_{endPL} - gage}{2} - 0.5 \cdot \left(d_{bolt} + \frac{2}{16}in\right)\right] = 1.31\,in^2$

$0.60Fu_{PL} \cdot Anv_{b\_endPL} = 406.6\,kip$

$U_{bs} := 1$ $0.60Fy_{PL} \cdot Agv_{b\_endPL} = 436.5\,kip$

$U_{bs} \cdot Fu_{PL} \cdot Ant_{b\_endPL} = 85.3\,kip$

strength reduction factor $\Phi := 0.75$

block shear strength $\Phi R_n := \Phi \cdot (2)\left(0.6\,\min\left(Fu_{PL} \cdot Anv_{b\_endPL}, Fy_{PL} \cdot Agv_{b\_endPL}\right) + U_{bs} \cdot Fu_{PL} \cdot Ant_{b\_endPL}\right)$

$\Phi R_n = 737.8\,kip$

recall

$V_u = 319.2\,kip$

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

Check beam shear strength (J4.2)

recall

$d_{beam} = 21.4\,in$

$tw_{beam} = 0.515\,in$

$Fy_{beam} = 50\,ksi$

$V_u = 319.2\,kip$

gross shear area $Agv_{beam} := d_{beam} \cdot tw_{beam} = 11\,in^2$

strength reduction factor $\Phi := 1.00$

shear strength $\Phi R_n := \Phi \cdot 0.60 \cdot Fy_{beam} \cdot Agv_{beam} = 330.6\,kip$

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

Check column shear strength (J4.2)

recall

$d_{column} = 14\,in$

$tw_{column} = 0.44\,in$

$Fy_{beam} = 50\,ksi$

$H_{column} = 176.1\,kip$

gross shear area $Agv_{column} := d_{column} \cdot tw_{column} = 6.2\,in^2$

strength reduction factor $\Phi := 1.00$

shear strength $\Phi R_n := \Phi \cdot 0.60 \cdot Fy_{beam} \cdot Agv_{column} = 184.8\,kip$

check capcity $if\left(H_{column} < \Phi R_n, \text{"OK"}, \text{"NOT OK"}\right) = \text{"OK"}$