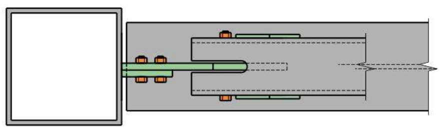
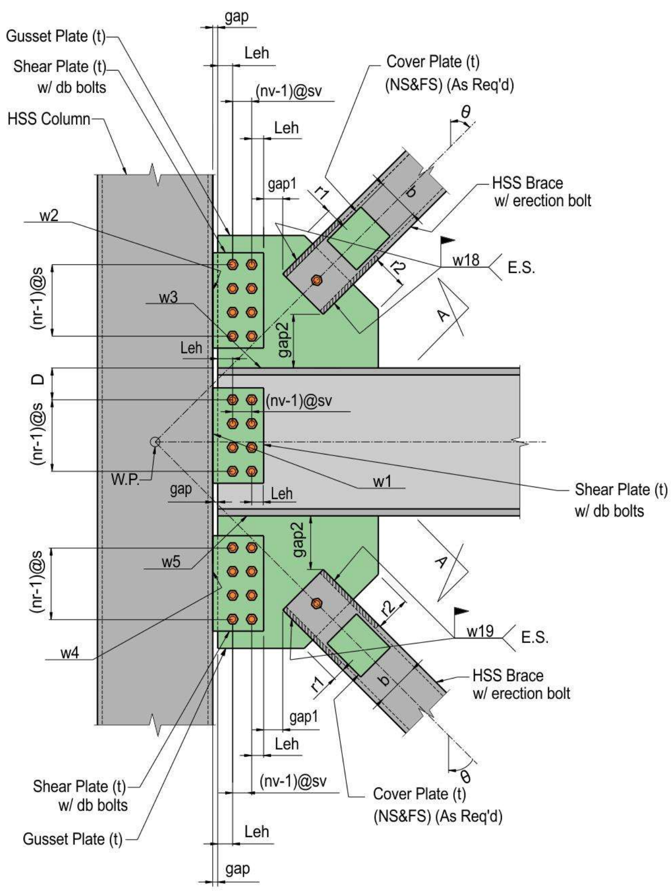
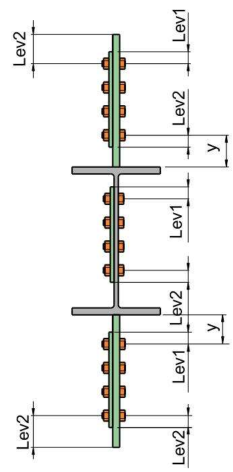
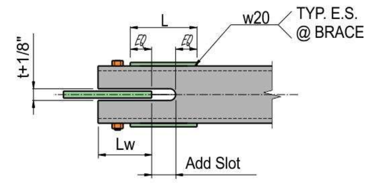
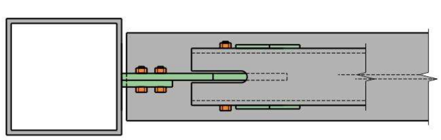








Section "A"



Note: Figure above does not represent actual design. Refer to connection schedule.

VERTICAL BRACE CONNECTION: HSS K-BRACE (DIRECTLY WELDED TO GUSSET PLATE) WITH SHEAR PLATE TWO-WAY GUSSET PLATE CONNECTION TO W BEAM AND RECTANGULAR HSS COLUMN

I. DESIGN DATA AND LOADS (LRFD-14th Edition)

COLUMN PROPERTIES : HSS14X6X5/8 - A500-B

Height,	H = 14 in	*Design Wall Thickness,*	tw = 0.581 in
Width,	B = 6 in	*Wall Thickness,*	tnom = 0.625 in
Area,	Ag = 21 in²		
Minimum Yield Stress,	Fy = 46 ksi	*Minimum Tensile Stress,*	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi		

BEAM PROPERTIES : W21X83 - A992

Depth,	d = 21.4 in	*Web Thickness,*	tw = 0.515 in
Flange Width,	bf = 8.36 in	*Flange Thickness,*	tf = 0.835 in
Distance k,	k = 1.5 in	*Distance k1,*	k1 = 0.875 in
Area,	Ag = 24.4 in²	*Distance k (Design),*	kdes = 1.34 in
Minimum Yield Stress,	Fy = 50 ksi	*Minimum Tensile Stress,*	Fu = 65 ksi
Modulus of Elasticity,	E = 29000 ksi	*Cut Distance from Web,*	z = 0 in
Top of Steel Elevation,	Elev = 12 ft + 0 in		
Span Length,	L = 30 ft	*Erection Clearance,*	gap = 0.5 in
Slope,	θsl = 0 deg	*Skew,*	θsk = 0 deg
		Inclination to the Column,	θin = 90 deg
Depth of Top Cope,	dcT = 0 in	*Depth of Bottom Cope,*	dcB = 0 in
Length of Top Cope,	cT = 0 in	*Length of Bottom Cope,*	cB = 0 in

BRACE 1 PROPERTIES : HSS6X5X3/8 - A500-B

Height,	H = 6 in	*Design Wall Thickness,*	tw = 0.349 in
Width,	B = 5 in	*Wall Thickness,*	tnom = 0.375 in
Area,	Ag = 6.88 in²		
Minimum Yield Stress,	Fy = 46 ksi	*Minimum Tensile Stress,*	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi		
Unbraced Length,	Lu = 16 ft + 3.375 in		
Angle from Vertical Member,	θ = 50 deg	*Additional Slot,*	Add Slot = 2 in

BRACE 2 PROPERTIES : HSS6X5X3/8 - A500-B

Height,	H = 6 in	*Design Wall Thickness,*	tw = 0.349 in
Width,	B = 5 in	*Wall Thickness,*	tnom = 0.375 in
Area,	Ag = 6.88 in^2		
Minimum Yield Stress,	Fy = 46 ksi	*Minimum Tensile Stress,*	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi		
Unbraced Length,	Lu = 16 ft + 3.375 in		
Angle from Vertical Member,	θ = 50 deg	*Additional Slot,*	Add Slot = 2 in

GUSSET PLATE 1 PROPERTIES : A36

Thickness,	t = 0.375 in	*Number of Plates,*	n = 1
Minimum Yield Stress,	Fy = 36 ksi	*Minimum Tensile Stress,*	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi		
Clip,	c = 0 in		

GUSSET PLATE 2 PROPERTIES : A36

Thickness,	t = 0.375 in	*Number of Plates,*	n = 1
Minimum Yield Stress,	Fy = 36 ksi	*Minimum Tensile Stress,*	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi		
Clip,	c = 0 in		

GUSSET SHEAR PLATE 1 PROPERTIES : A36

Thickness,	t = 0.375 in	*Number of Plates,*	n = 1
Minimum Yield Stress,	Fy = 36 ksi	*Minimum Tensile Stress,*	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi		
Clip,	c = 0 in		

GUSSET SHEAR PLATE 2 PROPERTIES : A36

Thickness,	t = 0.375 in	*Number of Plates,*	n = 1
Minimum Yield Stress,	Fy = 36 ksi	*Minimum Tensile Stress,*	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi		
Clip,	c = 0 in		

BEAM SHEAR PLATE PROPERTIES : A36

Thickness,	t = 0.375 in	*Number of Plates,*	n = 1
Minimum Yield Stress,	Fy = 36 ksi	*Minimum Tensile Stress,*	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi		
Clip,	c = 0 in		

BOLTS PROPERTIES : 3/4" - ø - A325-SC-SSLP-CLASS A

For Gusset Shear Plate 1 to Gusset Plate 1 Connection:

Bolt Diameter,	db = 0.75 in		
Bolt Shear Strength,	Λrv = 8.068 kips	*Bolt Tensile Strength,*	Λrn = 29.821 kips
Bolt Type,	Bolt_Type = A325-SC-SSLP-CLASS A	*Connection Type,*	Conn_type = Slip Critical Type
Number of Bolt Rows,	nr = 4	*Bolt Vertical Spacing,*	s = 3 in
Number of Bolt Column Lines,	nv = 1	*Bolt Horizontal Spacing,*	sv = 0 in
Total Number of Bolts (nr·nv),	nb = 4		
Holes at Gusset Plate,		*Holes at Gusset Shear Plate,*	
Vertical Hole Dimension,	hdv = 0.875 in	*Vertical Hole Dimension,*	hdv = 0.875 in
Horizontal Hole Dimension,	hdh = 0.875 in	*Horizontal Hole Dimension,*	hdh = 1.063 in
Vertical Edge Distance (Lev2),	Lev = 2.5 in	*Vertical Edge Distance min(Lev1, Lev2),*	Lev = 1.5 in
Horizontal Edge Distance,	Leh = 1.75 in	*Horizontal Edge Distance,*	Leh = 1.5 in

BOLTS PROPERTIES : 3/4" - ø - A325-SC-SSLP-CLASS A

For Gusset Shear Plate 2 to Gusset Plate 2 Connection:

Bolt Diameter,	db = 0.75 in		
Bolt Shear Strength,	Λrv = 8.068 kips	*Bolt Tensile Strength,*	Λrn = 29.821 kips
Bolt Type,	Bolt_Type = A325-SC-SSLP-CLASS A	*Connection Type,*	Conn_type = Slip Critical Type
Number of Bolt Rows,	nr = 4	*Bolt Vertical Spacing,*	s = 3 in
Number of Bolt Column Lines,	nv = 1	*Bolt Horizontal Spacing,*	sv = 0 in
Total Number of Bolts (nr·nv),	nb = 4		

Holes at Gusset Plate,		*Holes at Gusset Shear Plate,*	
Vertical Hole Dimension,	hdv = 0.875 in	*Vertical Hole Dimension,*	hdv = 0.875 in
Horizontal Hole Dimension,	hdh = 0.875 in	*Horizontal Hole Dimension,*	hdh = 1.063 in
Vertical Edge Distance (Lev2),	Lev = 2.5 in	*Vertical Edge Distance min(Lev1, Lev2),*	Lev = 1.5 in
Horizontal Edge Distance,	Leh = 1.75 in	*Horizontal Edge Distance,*	Leh = 1.5 in

BOLTS PROPERTIES : 3/4" - ø - A325-SC-SSLP-CLASS A

For Beam Shear Plate to Beam Web Connection:

Bolt Diameter,	db = 0.75 in		
Bolt Shear Strength,	Λrv = 8.068 kips	*Bolt Tensile Strength,*	Λrn = 29.821 kips
Bolt Type,	Bolt_Type = A325-SC-SSLP-CLASS A	*Connection Type,*	Conn_type = Slip Critical Type
Number of Bolt Rows,	nr = 5	*Bolt Vertical Spacing,*	s = 3 in
Number of Bolt Column Lines,	nv = 1	*Bolt Horizontal Spacing,*	sv = 0 in
Total Number of Bolts (nr·nv),	nb = 5		
Holes at Beam Web,		*Holes at Beam Shear Plate,*	
Vertical Hole Dimension,	hdv = 0.875 in	*Vertical Hole Dimension,*	hdv = 0.875 in
Horizontal Hole Dimension,	hdh = 0.875 in	*Horizontal Hole Dimension,*	hdh = 1.063 in
Bolt First Down from Top of Beam,	D = 3 in		
Vertical Edge Distance (D - dcT),	Lev = 3 in	*Vertical Edge Distance min(Lev1, Lev2),*	Lev = 1.5 in
Horizontal Edge Distance,	Leh = 1.75 in	*Horizontal Edge Distance,*	Leh = 1.5 in

WELDS PROPERTIES : E70xx LH

Minimum Tensile Stress, Fu = 70 ksi

For Brace 1 to Gusset Plate 1 Connection:

Preferred Weld Size (w18),	w = 0.25 in	*Length of Weld,*	Lw = 6 in

For Brace 2 to Gusset Plate 2 Connection:

Preferred Weld Size (w19)	w = 0.25 in	*Length of Weld,*	Lw = 6 in

For Gusset Shear Plate 1 to Column Wall Connection:

Preferred Weld Size (w2), w = 0.25 in

For Gusset Shear Plate 2 to Column Wall Connection:

Preferred Weld Size (w4), w = 0.25 in

For Gusset Plate 1 to Beam Flange Connection:

Preferred Weld Size (w3), w = 0.25 in *Length of Weld,* Lw = 16.312 in

For Gusset Plate 2 to Beam Flange Connection:

Preferred Weld Size (w5), w = 0.25 in *Length of Weld,* Lw = 16.312 in

For Beam Shear Plate to Column Wall Connection:

Preferred Weld Size (w1), w = 0.25 in

SAFETY AND RESISTANCE FACTORS:

Safety Factor, Ω(ASD) *Resistance Factor, ϕ(LRFD)*

Modification Factor,

$\Lambda = \frac{1}{\Omega}$ *(if ASD)* $\Lambda = \phi$ *(if LRFD)*

	safety factor	*resistance factor*	*modification factor*
For Member in Bearing/ Bolt Bearing (brg),	Ωbrg = 2.00	ϕbrg = 0.75	Λbrg = 0.75
For Block Shear (bs),	Ωbs = 2.00	ϕbs = 0.75	Λbs = 0.75
For Compression (c),	Ωc = 1.67	ϕc = 0.90	Λc = 0.90
For Fillet Weld Shear (vw),	Ωvw = 2.00	ϕvw = 0.75	Λvw = 0.75
For Flexural Local Buckling/Flexural Strength (b),	Ωb = 1.67	ϕb = 0.90	Λb = 0.90
For Flexural Rupture (fr),	Ωfr = 2.00	ϕfr = 0.75	Λfr = 0.75
For Partial Pen Weld - shear (vwp),	Ωvwp = 2.00	ϕvwp = 0.75	Λvwp = 0.75
For Partial Pen Weld - Tension (twp),	Ωtwp = 1.88	ϕtwp = 0.80	Λtwp = 0.80
For Shear Rupture (vr),	Ωvr = 2.00	ϕvr = 0.75	Λvr = 0.75
For Shear Yielding (vy),	Ωvy = 1.50	ϕvy = 1.00	Λvy = 1.00
For Tension Rupture (tr),	Ωtr = 2.00	ϕtr = 0.75	Λtr = 0.75

For Tension Yielding(ty),	$\Omega ty = 1.67$	$\phi ty = 0.90$	$\Lambda ty = 0.90$
For Web Crippling(cr),	$\Omega cr = 2.00$	$\phi cr = 0.75$	$\Lambda cr = 0.75$
For Member Shear Yielding for S, M, W, HSS (wy),	$\Omega wy = 1.50$	$\phi wy = 1.00$	$\Lambda wy = 1.00$
For Eccentric Weld (ew),	$\Omega ew = 2.00$	$\phi ew = 0.75$	$\Lambda ew = 0.75$
For Rectangular-Square HSS Chord Plastification (pHSS),	$\Omega pHSS = 1.67$	$\phi pHSS = 0.90$	$\Lambda pHSS = 1.00$

APPLIED LOADS:

Brace 1:

Given Tension Load,	Pt1 = 25 kips	*Given Compression Load,*	Pc1 = 25 kips

Given Load

Governing Tension Load,	Pt = 25 kips	*Governing Compression Load,*	Pc = 25 kips
Maximum Axial Load,	P = max(Pt,Pc)	P = 25 kips	

Brace 2:

Given Tension Load,	Pt1 = 25 kips	*Given Compression Load,*	Pc1 = 25 kips

Given Load

Governing Tension Load,	Pt = 25 kips	*Governing Compression Load,*	Pc = 25 kips
Maximum Axial Load,	P = max(Pt,Pc)	P = 25 kips	

Beam:

Given End Reaction

Shear Load,	V = 10 kips
Transfer Force,	TF = 0 kips

Column:

Axial Load,	P = 0 kips
Moment Load,	M = 0 kips·ft
Uplift Force (if any),	PUplift = 0 kips

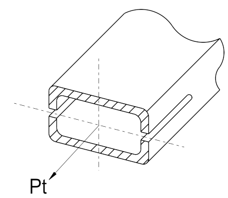
II. CALCULATIONS

A. BRACE 1 CHECK

1. Rupture Capacity

(AISC 14th Ed. Specifications, Chapter D, Section D2, pages 16.1-26 to 16.1-27)

Length of the Connection,



$Lcon = Lw$ $\quad\quad$ $Lcon = 6\ in$

Net Tension Area,

$$Ant = Ag - 2\cdot tw\cdot\left(t + \frac{1}{8} in\right) \quad\quad Ant = 6.531\ in^2$$

Eccentricity of the Connection,

$$econ = \frac{B^2 + 2\cdot B\cdot H}{4\cdot(B+H)} \quad\quad econ = 1.932\ in$$

Reduction Coefficient,

$$U = 1 - \frac{econ}{Lcon} \quad\quad U = 0.678$$

Effective Net Tension Area,

$$Ae = U\cdot Ant \quad\quad Ae = 4.428\ in^2$$

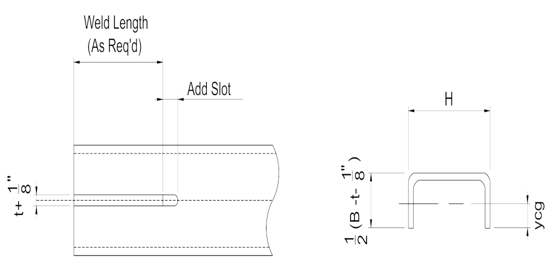
Tensile Rupture Capacity, (D2-2)

$Rtr = \Lambda tr\cdot Fu\cdot Ae$

$Rtr = 192.627\ kips$ $\quad\quad$ $Pt = 25\ kips$

Tensile Rupture Capacity > Applied Force, UCV = 0.13, OK

2. Additional Check for Slot of HSS



a. Local Check of C-Shape Section

Unstiffened Width,

$$b = \frac{1}{2}\cdot\left(B - t - \frac{1}{8} in\right) \quad\quad b = 2.25\ in$$

Limiting Width-to-Thickness Ratio,

(AISC 14th Ed. Specifications, Chapter B, Table B4.1a, page 16.1-16)

$$\frac{b}{tw} \leq 0.56 \cdot \left(\frac{E}{Fy}\right)^{0.5}$$

$$\frac{b}{tw} = 6.447 \qquad 0.56 \cdot \left(\frac{E}{Fy}\right)^{0.5} = 14.061$$

Section is Non-Slender

Q Factor,

(AISC 14th Ed. Specifications, Chapter E, Section E7.1, pages 16.1-40 to 16.1-41)

$$\frac{b}{tw} \leq 0.56 \cdot \left(\frac{E}{Fy}\right)^{0.5}$$

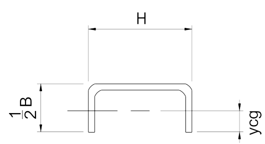
$Qs = 1$

b. Compression Capacity of C-Shape Section

(AISC 14th Ed. Specifications, Chapter E, Section E7, page 16.1-40)

Effective Length Factor,

$K = 1$



Laterally Unbraced Length,

$Lu = \text{Add Slot}$

Modulus of Elasticity

$E = 29000\ ksi$

Gross Area,

$A1 = 2 \cdot (b \cdot tw)$ $\qquad A1 = 1.57\ in^2$

$A2 = (H - 2 \cdot tw) \cdot tw$ $\qquad A2 = 1.85\ in^2$

$Ag = A1 + A2$ $\qquad Ag = 3.421\ in^2$

Centroid,

$$ycg = \frac{A1 \cdot \left(\frac{1}{2} \cdot b\right) + A2 \cdot \left(b - \frac{1}{2} \cdot tw\right)}{Ag} \qquad ycg = 1.639\ in$$

Moment of Inertia,

$$I1 = 2 \cdot \frac{tw \cdot b^3}{12} + A1 \cdot \left(\frac{1}{2} \cdot b - ycg\right)^2$$

$$I2 = \frac{(H - 2 \cdot tw) \cdot tw^3}{12} + A2 \cdot \left(b - ycg - \frac{1}{2} \cdot tw\right)^2$$

$I = I1 + I2$ $\qquad I = 1.449\ in^4$

Radius of Gyration,

$$r = \left(\frac{I}{Ag}\right)^{0.5} \qquad r = 0.651\ \text{in}$$

Slenderness Ratio,

$$KLr = \frac{K \cdot Lu}{r} \qquad KLr = 3.073$$

Elastic Critical Buckling Stress,

$$Fe = \frac{\pi^2 \cdot E}{(KLr)^2} \qquad Fe = 30304.774\ \text{ksi}$$

Flexural Buckling Stress,

$$KLr \le 4.71 \cdot \left(\frac{E}{Qs \cdot Fy}\right)^{0.5}$$

$$Fcr = Qs \cdot 0.658^{\frac{Qs \cdot Fy}{Fe}} \cdot Fy$$

$$Fcr = 45.971\ \text{ksi}$$

Compression Capacity,

$$Rcb = \Lambda c \cdot Fcr \cdot Ag$$

$$Rcb = 141.535\ \text{kips} \qquad \frac{1}{2} Pc = 12.5\ \text{kips}$$

Compression Capacity > Applied Force, UCV = 0.088, OK

B. BRACE 1 TO GUSSET PLATE 1 CHECK

1. Weld Capacity

(AISC 14th Ed. Specifications, Chapter J, pages 16.1-110 to 16.1-117)

(AISC 14th Ed. Manual, Part 8, pages 8-9 to 8-15)

a. Using Fillet Weld

Number of Weld Sides,

$$nws = 4$$

Minimum Weld Size,

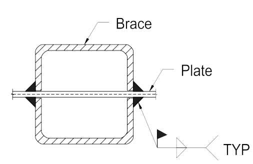
$$wmin = 0.187\ \text{in} \qquad w = 0.25\ \text{in}$$

Preferred Weld Size > Minimum Weld Size, OK

Shear Strength,

For Brace,

$$Rv1 = \Lambda vr \cdot 0.6 \cdot Fu \cdot tw \cdot nws \qquad Rv1 = 36.436\ \text{kips/in}$$



Number of Plates,

$$n1 = 1$$

For Gusset Plate,

$Rv2 = \Lambda vr \cdot 0.6 \cdot Fu \cdot t \cdot n1$ $Rv2 = 19.575$ kips/in

For Weld,

$Rv3 = \Lambda vw \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws$ $Rv3 = 89.095$ ksi

Maximum Effective Weld Size,

$$weff = \frac{\min(Rv1, Rv2)}{Rv3}$$ $weff = 0.22$ in

Length of Weld,

$Lw = 6$ in

Weld Capacity,

$Rw = \Lambda vw \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws \cdot Lw \cdot \min(w, weff)$

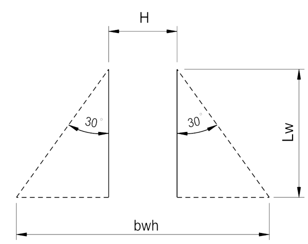
$Rw = 117.45$ kips $P = 25$ kips

Weld Capacity > Applied Force, UCV = 0.213, OK

C. GUSSET PLATE 1 CHECK

1. Whitmore Section

Width of Whitmore Section,



$bwh1 = 2 \cdot Lw \cdot \tan(30deg) + H$

$bwh1 = 12.928$ in

Width of Whitmore Section Outside Gusset Plate,

$bwhog = 0$ in

Available Width of Whitmore Section in Gusset Plate,

$bwh = bwh1 - 2 \cdot bwhog$ $bwh = 12.928$ in

Effective Length of Whitmore Section,

$Lwh = 5.125$ in

2. Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, page 16.1-128)

Width,

$b = bwh$ $b = 12.928$ in

Gross Tension Area,

$Ag = b \cdot t$

Number of Areas in Consideration,

$n1 = n$

Tensile Yielding Capacity, (J4-1)

$Rty = \Lambda ty \cdot n1 \cdot Fy \cdot Ag$

$Rty = 157.078$ kips $Pt = 25$ kips

Tensile Yielding Capacity > Applied Force, UCV = 0.159, OK

3. Compression Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.4, page 16.1-129 to 16.1-130)

Effective Length Factor,

(Commentary on the Specification for Structural Steel Building Table C-A-7.1)

$K = 0.65$

Laterally Unbraced Length,

$Lu = Lwh$ $\qquad$ $Lu = 5.125$ in

Gross Area,

$Ag = bwh \cdot t$ $\qquad$ $Ag = 4.848$ in²

Radius of Gyration,

$$r = \frac{t}{(12)^{0.5}}$$ $\qquad$ $r = 0.108$ in

Slenderness Ratio,

$$KLr = \frac{K \cdot Lu}{r}$$ $\qquad$ $KLr = 30.773$

Elastic Critical Buckling Stress,

$$Fe = \frac{\pi^2 \cdot E}{KLr^2}$$ $\qquad$ $Fe = 302.249$ ksi

Flexural Buckling Stress,

$KLr > 25$

$$KLr \le 4.71 \cdot \left(\frac{E}{Fy}\right)^{0.5}$$

$$Fcr = 0.658^{\frac{Fy}{Fe}} \cdot Fy$$ $\qquad$ $Fcr = 34.249$ ksi

Number of Areas in Consideration,

$n1 = n$

Compression Capacity,

$Rcb = \Lambda c \cdot n1 \cdot Fcr \cdot Ag$

$Rcb = 149.439$ kips $\qquad$ $Pc = 25$ kips

Compression Capacity > Applied Force, UCV = 0.167, OK

4. Block Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

Reduction Factor,

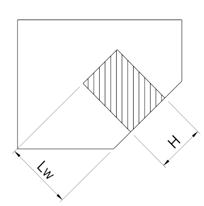
$Ubs = 1.0$ $\qquad$ *(tension stress is uniform)*

Gross Shear Area,

$Agv = 2 \cdot Lw \cdot t$ $\qquad$ $Agv = 4.5$ in²

Net Tension Area,

$Ant = H \cdot t$ $\qquad$ $Ant = 2.25$ in²



Net Shear Area,

Anv = Agv Anv = 4.5 in²

Number of Areas in Consideration,

n1 = n

Block Shear Capacity, (J4-5)

Rbs = Λbs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)

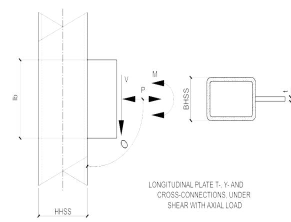
Rbs = 170.775 kips Pt = 25 kips

Block Shear Capacity > Applied Force, UCV = 0.146, OK

D. GUSSET PLATE 1 FORCE DISTRIBUTION

1. Gusset Plate Edge Forces

Connecting Face of HSS,



BHSS = B

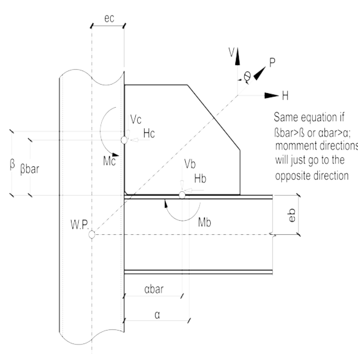
BHSS = 6 in

HHSS = H

HHSS = 14 in

(AISC 14th Ed. Manual Part 13, pages 13-3 to 13-11)

Uniform Force Method



Beam,

eb = 0.5·d

Horizontal Side,

αbar = 0.5·Lw + gap

αbar = 8.656 in

α = (βbar + eb)·tan(θ) - ec

α = 14.69 in

$$r = \frac{P}{\left[(\alpha + ec)^2 + (\beta + eb)^2\right]^{0.5}}$$

Column,

ec = 0.5HHSS

Vertical Side,

βbar = 0.5·(nr - 1)·s + y

βbar = 7.5 in

β = βbar

β = 7.5 in

r = 0.883 kips/in

Horizontal Side,

Hb = α·r

Hb = 12.97 kips

Vb = eb·r

Vb = 9.448 kips

Mb = |Vb·(α - αbar)|

Mb = 57.003 kips·in

Vertical Side,

Hc = ec·r

Hc = 6.181 kips

Vc = β·r

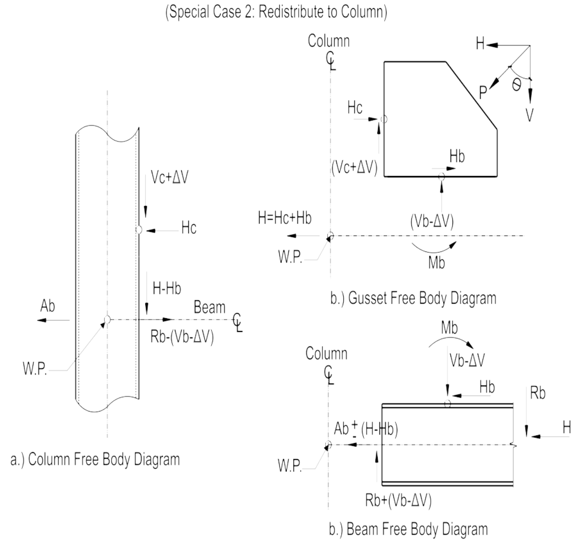
Vc = 6.622 kips

Mc = |Hc·(β - βbar)|

Mc = 0 kips·in

Redistribution of Forces,

(Special Case 2: Redistribute to Column)



a.) Column Free Body Diagram

b.) Gusset Free Body Diagram

b.) Beam Free Body Diagram

Shear Transfer,

ΔV = 0 kips

Gusset-to-Beam Connection,

Vb = Vb - ΔV

Vb = 9.448 kips

Hb = |Hb|

Hb = 12.97 kips

Mb = |ΔV·αbar + Mb|

Mb = 57.003 kips·in

Gusset-to-Column Connection,

Vc = |Vc + ΔV|

Vc = 6.622 kips

Hc = |Hc|

Hc = 6.181 kips

Mc = |Hc·(β - βbar)|

Mc = 0 kips·in

E. GUSSET PLATE 1 TO COLUMN WALL CHECK

Note: Since Mc is equal to 0 kips, limit states will only be checked due to forces Vc and Hc

1. Forces Acting on Connection

Vertical Force,

Vc = 6.622 kips

Horizontal Force,

Hc = 6.181 kips

Moment Force,

$Mc = 0\ kips \cdot in$

Resultant Force,

$Rc = \left(Vc^2 + Hc^2\right)^{0.5}$ $\qquad Rc = 9.058\ kips$

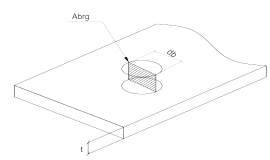
E.A. GUSSET PLATE 1 CHECK

1. Bolt Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)

a. Bolt Capacity due to Resultant Load

Bearing Area,



$Abrg = db \cdot t$ $\qquad Abrg = 0.281\ in^2$

Bolt Centerline Distance from Face of Support,

$ab = gap + Leh + 0.5 \cdot (nv - 1) \cdot sv$

$ab = 2.25\ in$

Eccentricity Distannce of End Reaction from Bolt Group Centerline,

$ebv = ab$ $\qquad ebv = 2.25\ in$

Bolt Vertical Centerline Distance from Beam Centerline,

$ah = |\beta - \beta bar|$ $\qquad ah = 0\ in$

Eccentricity distance of Axial Load from Bolt Group Centerline,

$Yo = ah$ $\qquad Yo = 0\ in$

Load Inclination from Vertical,

$\theta = atan\left(\frac{Hc}{Vc}\right)$ $\qquad \theta = 43.025\ deg$

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-8)

$C = 3.148$

Available Bearing Strength Using Edge Distance, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbe = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (Lev - 0.5 \cdot hdv) \cdot t \\ 1.2 \cdot (Leh - 0.5 \cdot hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$Fbe = min(Fbe)$ $\qquad Fbe = 20.798\ kips$

Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbs = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (s - hdv) \cdot t \\ 1.2 \cdot (sv - hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$nv > 1$

$Fbs = \min(Fbs_0, Fbs_2)$ $Fbs = 29.362$ kips

Number of Area in Consideration,

$n1 = n$

Bolt Capacity,

$Rbrg = C \cdot \min(n1 \cdot Fbe, n1 \cdot Fbs, n \cdot \Lambda rv)$

$Rbrg = 25.399$ kips $Rc = 9.058$ kips

Bolt Capacity > Applied Force, UCV = 0.357, OK

2. Block Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

a. Block Shear Capacity due to Shear Load

Reduction Factor,

$nv = 1$ *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,

$Agv = [(nr - 1) \cdot s + Lev] \cdot t$ $Agv = 4.312\ in^2$

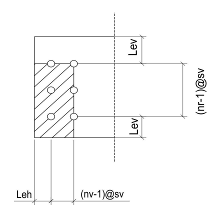
Net Tension Area,

$Ant = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot t$

$Ant = 0.492\ in^2$

Net Shear Area,

$Anv = Agv - [(nr - 0.5) \cdot hdv] \cdot t$ $Anv = 3.164\ in^2$



Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbss = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant, 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbss = 91.273$ kips $Vc = 6.622$ kips

Block Shear Capacity > Applied Force, UCV = 0.073, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

b. Block Shear Capacity due to Axial Load

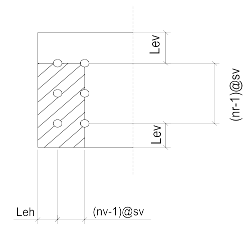
Pattern 1

Reduction Factor,

$ah = 0$ in *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,



$Agv = [Leh + (nv - 1) \cdot sv] \cdot t$ $\quad Agv = 0.656$ in²

Net Tension Area,

$Ant = [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hdv] \cdot t$

$Ant = 3.164$ in²

Net Shear Area,

$Anv = Agv - [(nv - 0.5) \cdot hdh] \cdot t$ $\quad Anv = 0.492$ in²

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs1 = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbs1 = 148.268$ kips $\quad Hc = 6.181$ kips

Block Shear Capacity > Applied Force, UCV = 0.042, OK

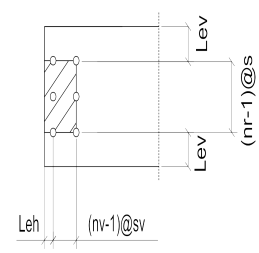
Pattern 2

Reduction Factor,

$ah = 0$ in *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,



$Agv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot t$ $\quad Agv = 1.312$ in²

Net Tension Area,

$Ant = [(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t$

$Ant = 2.391$ in²

Net Shear Area,

$Anv = Agv - 2 \cdot [(nv - 0.5) \cdot hdh] \cdot t$ $\quad Anv = 0.984$ in²

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs2 = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbs2 = 125.255$ kips $\quad Hc = 6.181$ kips

Block Shear Capacity > Applied Force, UCV = 0.049, OK

Governing Block Shear Capacity,

$Rbs = \min(Rbs1, Rbs2)$

Rbs = 125.255 kips   Hc = 6.181 kips

Block Shear Capacity > Applied Force, UCV = 0.049, OK

E.B. GUSSET PLATE 1 TO GUSSET SHEAR PLATE 1 CHECK

1. Bolt Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.6, page 16.1-125)

Load Inclination from Vertical,

$\theta = \text{atan}\left(\frac{Hc}{Vc}\right)$ θ = 43.025 deg

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-12)

C = 3.148

Shear Capacity Per Bolt,

Λrv = 8.068 kips

Bolt Shear Capacity,

$Rb = n \cdot C \cdot \Lambda rv$

Rb = 25.399 kips   Rc = 9.058 kips

Bolt Shear Capacity > Applied Force, UCV = 0.357, OK

2. Check for Spacing

(AISC 14th Ed. Specifications, Chapter J, Section J3.3 and J3.5, pages 16.1-122 to 16.1-124)

Shear Plate Thickness,

t1 = 0.375 in

Gusset Plate Thickness,

t2 = 0.375 in

a. Vertical Spacing,

Minimum Bolt Spacing,

s = 3 in

$smin = 2\frac{2}{3} \cdot db$ smin = 2 in

Maximum Bolt Spacing,

$smax = \min(12 \cdot in, 24 \cdot \min(t1, t2))$ smax = 9 in

Specified Bolt Spacing is acceptable, OK

3. Check for Edge Distance

(AISC 14th Ed. Specifications, Chapter J, Section J3.4 and J3.5, pages 16.1-122 to 16.1-124)

Shear Plate Edge Distances,

Lev1 = 1.5 in

$Leh1 = 1.5\ in$

Gusset Plate Edge Distances,

$Lev2 = 2.5\ in$

$Leh2 = 1.75\ in$

i) Minimum Vertical Edge Distance,

Connection Edge Distance,

$$Levcon = \begin{bmatrix} Lev1 \\ Lev2 \end{bmatrix} \qquad Levcon = \begin{bmatrix} 1.5 \\ 2.5 \end{bmatrix} in$$

Minimum Edge Distance,

$$Levmin = \begin{bmatrix} Levmin1 \\ Levmin2 \end{bmatrix} \qquad Levmin = \begin{bmatrix} 1 \\ 1 \end{bmatrix} in$$

Specified Edge Distance is Acceptable, OK

ii) Minimum Horizontal Edge Distance,

Connection Edge Distance,

$$Lehcon = \begin{bmatrix} Leh1 \\ Leh2 \end{bmatrix} \qquad Lehcon = \begin{bmatrix} 1.5 \\ 1.75 \end{bmatrix} in$$

Minimum Edge Distance,

$$Lehmin = \begin{bmatrix} Lehmin1 \\ Lehmin2 \end{bmatrix} \qquad Lehmin = \begin{bmatrix} 1.125 \\ 1 \end{bmatrix} in$$

Specified Edge Distance is Acceptable, OK

iii) Maximum Edge Distance,

Shear Plate Thickness,

$t1 = 0.375\ in$

Gusset Plate Thickness,

$t2 = 0.375\ in$

Nearest Connection Edge Distance,

$Lemin = \min(Lehcon, Levcon)$

$Lemin = 1.5\ in$

Maximum Edge Distance,

$Lemin = Levcon_0 \vee Lemin = Lehcon_0$

$Lemax = \min(6in, 12 \cdot t1)$

$Lemax = 4.5\ in$

Maximum Edge Distance Requirement is Satisfied, OK

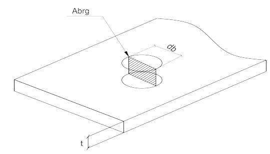
E.C. GUSSET SHEAR PLATE 1 CHECK

1. Bolt Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)

a. Bolt Capacity due to Resultant Load

Bearing Area,



$Abrg = db \cdot t$ $\qquad$ $Abrg = 0.281\ in^2$

Bolt Centerline Distance from Face of Support,

$ab = gap + Leh + 0.5 \cdot (nv - 1) \cdot sv$

$ab = 2.25\ in$

Eccentricity Distannce of End Reaction from Bolt Group Centerline,

$ebv = ab$ $\qquad$ $ebv = 2.25\ in$

Bolt Vertical Centerline Distance from Beam Centerline,

$ah = |\beta - \beta bar|$ $\qquad$ $ah = 0\ in$

Eccentricity distance of Axial Load from Bolt Group Centerline,

$Yo = ah$ $\qquad$ $Yo = 0\ in$

Load Inclination from Vertical,

$\theta = atan\left(\frac{Hc}{Vc}\right)$ $\qquad$ $\theta = 43.025\ deg$

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-8)

$C = 3.148$

Available Bearing Strength Using Edge Distance, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbe = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (Lev - 0.5 \cdot hdv) \cdot t \\ 1.2 \cdot (Leh - 0.5 \cdot hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$Fbe = min(Fbe)$ $\qquad$ $Fbe = 18.963\ kips$

Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbs = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (s - hdv) \cdot t \\ 1.2 \cdot (sv - hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$nv > 1$

$Fbs = min(Fbs_0, Fbs_2)$ $\qquad$ $Fbs = 29.362\ kips$

Number of Area in Consideration,

$n1 = n$

Bolt Capacity,

$Rbrg = C \cdot min(n1 \cdot Fbe, n1 \cdot Fbs, n \cdot \Lambda rv)$

Rbrg = 25.399 kips   Rc = 9.058 kips

Bolt Capacity > Applied Force, UCV = 0.357, OK

2. Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Yielding Capacity due to Shear Load

Length,

$L = (nr - 1)\cdot s + 2\cdot Lev$ L = 12 in

Number of Areas in Consideration,

$n1 = n$

Shear Yielding Capacity, (J4-3)

$Rvy = \Lambda vy\cdot n1\cdot 0.6\cdot Fy\cdot L\cdot t$

Rvy = 97.2 kips   Vc = 6.622 kips

Shear Yielding Capacity > Applied Force, UCV = 0.068, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, page 16.1-128)

b. Tensile Yielding Capacity due to Axial Load

Length,

$L = (nr - 1)\cdot s + 2\cdot Lev$ L = 12 in

Gross Tension Area,

$Ag = L\cdot t$

Number of Areas in Consideration,

$n1 = n$

Tensile Yielding Capacity, (J4-1)

$Rty = \Lambda ty\cdot n1\cdot Fy\cdot Ag$

Rty = 145.8 kips   Hc = 6.181 kips

Tensile Yielding Capacity > Applied Force, UCV = 0.042, OK

Interaction of Yielding Capacities,

$$\left(\frac{Vc}{Rvy}\right)^2 + \left(\frac{Hc}{Rty}\right)^2 \le 1.0$$

$$UCV = \left(\frac{Vc}{Rvy}\right)^2 + \left(\frac{Hc}{Rty}\right)^2$$ UCV = 0.006

Yielding Capacity > Applied Force, UCV = 0.006, OK

3. Rupture Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Rupture Capacity due to Shear Load

Net Shear Area,

$Anv = (L - nr \cdot hdv) \cdot t$

$Anv = 3.187\ in^2$

Number of Areas in Consideration,

$n1 = n$

Shear Rupture Capacity, (J4-4)

$Rvr = \Lambda vr \cdot n1 \cdot 0.6 \cdot Fu \cdot Anv$

$Rvr = 83.194\ kips$ $\quad Vc = 6.622\ kips$

Shear Rupture Capacity > Applied Force, UCV = 0.08, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, pages 16.1-128 to 16.1-129)

b. Tensile Rupture Capacity due to Axial Load

Effective Net Area,

$Ae = \min[(L - nr \cdot hdv) \cdot t,\ 0.85 \cdot L \cdot t]$ $\quad Ae = 3.187\ in^2$

Number of Areas in Consideration,

$n1 = n$

Tensile Rupture Capacity, (J4-2)

$Rtr = \Lambda tr \cdot n1 \cdot Fu \cdot Ae$

$Rtr = 138.656\ kips$ $\quad Hc = 6.181\ kips$

Tensile Rupture Capacity > Applied Force, UCV = 0.045, OK

Interaction of Rupture Capacities,

$$\left(\frac{Vc}{Rvr}\right)^2 + \left(\frac{Hc}{Rtr}\right)^2 \le 1.0$$

$$UCV = \left(\frac{Vc}{Rvr}\right)^2 + \left(\frac{Hc}{Rtr}\right)^2 \qquad UCV = 0.008$$

Rupture Capacity > Applied Force, UCV = 0.008, OK

4. Block Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

a. Block Shear Capacity due to Shear Load

Reduction Factor,

$nv = 1$ $\quad$ *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,

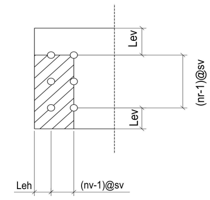
$Agv = [(nr - 1) \cdot s + Lev] \cdot t$ $\quad Agv = 3.938\ in^2$

Net Tension Area,

$Ant = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot t$

$Ant = 0.363\ in^2$

Net Shear Area,



$Anv = Agv - [(nr - 0.5)\cdot hdv]\cdot t$ $\qquad$ $Anv = 2.789\ in^2$

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbss = \Lambda bs\cdot n1\cdot \min(0.6\cdot Fu\cdot Anv + Ubs\cdot Fu\cdot Ant,\ 0.6\cdot Fy\cdot Agv + Ubs\cdot Fu\cdot Ant)$

$Rbss = 79.59\ kips$ $\qquad$ $Vc = 6.622\ kips$

Block Shear Capacity > Applied Force, UCV = 0.083, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

b. Block Shear Capacity due to Axial Load

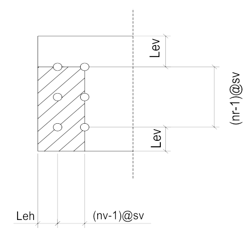
Pattern 1

Reduction Factor,

$ah = 0\ in$ $\qquad$ *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,



$Agv = [Leh + (nv - 1)\cdot sv]\cdot t$ $\qquad$ $Agv = 0.563\ in^2$

Net Tension Area,

$Ant = [(nr - 1)\cdot s + Lev - (nr - 0.5)\cdot hdv]\cdot t$

$Ant = 2.789\ in^2$

Net Shear Area,

$Anv = Agv - [(nv - 0.5)\cdot hdh]\cdot t$ $\qquad$ $Anv = 0.363\ in^2$

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbsa = \Lambda bs\cdot n1\cdot \min(0.6\cdot Fu\cdot Anv + Ubs\cdot Fu\cdot Ant,\ 0.6\cdot Fy\cdot Agv + Ubs\cdot Fu\cdot Ant)$

$Rbsa = 130.437\ kips$ $\qquad$ $Hc = 6.181\ kips$

Block Shear Capacity > Applied Force, UCV = 0.047, OK

Interaction of Bolt Shear Capacities,

$$\left(\frac{Vc}{Rbss}\right)^2 + \left(\frac{Hc}{Rbsa}\right)^2 \le 1.0$$

$$UCV = \left(\frac{Vc}{Rbss}\right)^2 + \left(\frac{Hc}{Rbsa}\right)^2 \qquad UCV = 0.009$$

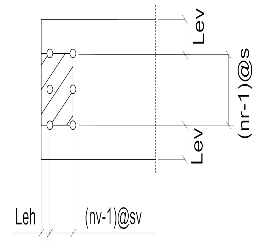
Block Shear Capacity > Applied Force, UCV = 0.009, OK

Pattern 2

Reduction Factor,

$ah = 0$ in *(tension stress is uniform)*

$Ubs = 1.0$



Gross Shear Area,

$Agv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot t$ $Agv = 1.125$ in^2

Net Tension Area,

$Ant = [(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t$

$Ant = 2.391$ in^2

Net Shear Area,

$Anv = Agv - 2 \cdot [(nv - 0.5) \cdot hdh] \cdot t$ $Anv = 0.727$ in^2

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs2 = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbs2 = 122.217$ kips $Hc = 6.181$ kips

Block Shear Capacity > Applied Force, UCV = 0.051, OK

5. Local Buckling Capacity

(AISC 14th Ed. Manual, Part 9, page 9-9)

Distance of Bolt Line to Support,

$ab = gap + Leh$ $ab = 2.25$ in

Coefficient,

$$\lambda = \frac{L \cdot Fy^{0.5}}{10 \cdot t \cdot \left[475 + 280 \cdot \left(\frac{L}{ab}\right)^2\right]^{0.5}} \cdot \frac{1}{ksi^{0.5}}$$ $\lambda = 0.209$

$\lambda \le 0.7$

$Q = 1$

Allowable Flexural Local Buckling Stress or Yielding Stress,

$Fcr = Q \cdot Fy$ $Fcr = 36$ ksi

Gross Plastic Section Modulus,

$$Zx = \frac{t \cdot L^2}{4}$$ $Zx = 13.5$ in^3

Eccentricity,

$e = ab$ $e = 2.25$ in

Local Buckling Capacity,

$$Rbc = \Lambda b \cdot \frac{Fcr \cdot Zx}{e}$$

Rbc = 194.4 kips Vc = 6.622 kips

Local Buckling Capacity > Applied Force, UCV = 0.034, OK

6. Shear and Flexural Yielding Capacity with Von-Mises Yield Criterion

(AISC 14th Ed., Manual Part 10, page 10-104)

(Muir, Larry and Hewitt,Christopher, "Design of Unstiffened Extended Single Plate Shear Connections", Engineering Journal, 2nd Quarter 2009, page 69)

Shear and Flexural Yielding Interaction Capacity,

$$Rfc = \frac{\Lambda b \cdot Fy \cdot L \cdot t}{\left[2.25 + 16 \cdot \left(\frac{e}{L}\right)^2\right]^{0.5}}$$

Rfc = 86.938 kips Vc = 6.622 kips

Yielding Capacity > Applied Force, UCV = 0.076, OK

7. Flexural Rupture Capacity

(AISC 14th Ed. Manual Part 15, page 15-4)

(AISC 14th Ed. Steel Construction Manual Design Examples, page IIA-104)

Net Plastic Section Modulus,

mod(nr, 2) = 0

$$Znet = \frac{t \cdot L^2}{4} - \frac{t \cdot hdv \cdot nr^2 \cdot s}{4}$$

Znet = 9.563 in^3

Flexural Rupture Capacity,

$$Rfr = \frac{\Lambda fr \cdot Fu \cdot Znet}{e}$$

Rfr = 184.875 kips Vc = 6.622 kips

Flexural Rupture Capacity > Applied Force, UCV = 0.036, OK

8. Compression Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.4, page 16.1-129 to 16.1-130)

Effective Length Factor,

(Commentary on the Specification for Structural Steel Building Table C-A-7.1)

K = 1.2

Gusset Horizontal Edge Distance,

Le1 = 1.75 in

Laterally Unbraced Length,

$Lu = gap + Le1$ $\qquad$ $Lu = 2.25$ in

Gross Area,

$Ag = L \cdot t$ $\qquad$ $Ag = 4.5$ in²

Radius of Gyration,

$$r = \frac{t}{(12)^{0.5}}$$ $\qquad$ $r = 0.108$ in

Slenderness Ratio,

$$KLr = \frac{K \cdot Lu}{r}$$ $\qquad$ $KLr = 24.942$

Elastic Critical Buckling Stress,

$$Fe = \frac{\pi^2 \cdot E}{KLr^2}$$ $\qquad$ $Fe = 460.099$ ksi

Flexural Buckling Stress,

$KLr \le 25$

$Fcr = Fy$ $\qquad$ $Fcr = 36$ ksi

Number of Areas in Consideration,

$n1 = n$

Compression Capacity,

$Rcb = \Lambda c \cdot n1 \cdot Fcr \cdot Ag$

$Rcb = 145.8$ kips $\qquad$ $Hc = 6.181$ kips

Compression Capacity > Applied Force, UCV = 0.042, OK

E.D. GUSSET SHEAR PLATE 1 TO COLUMN WALL CHECK

1. Weld Capacity

(AISC 14th Ed. Specifications, Chapter J, pages 16.1-110 to 16.1-117)

(AISC 14th Ed. Manual, Part 8, pages 8-9 to 8-15)

a. Using Fillet Weld

Number of Weld Sides,

$nws = 2$

Minimum Weld Size,

$wmin = 0.187$ in $\qquad$ $w = 0.25$ in

Preferred Weld Size > Minimum Weld Size, OK

Length of Weld,

$Lw = (nr - 1) \cdot s + 2 \cdot Lev$ $\qquad$ $Lw = 12$ in

Total Force Per Unit Length on Welds of Shear Plate to Column Connection,

$$Ruw = \left[\left(\frac{Vc}{Lw}\right)^2 + \left(\frac{Hc}{Lw}\right)^2\right]^{0.5}$$ $\qquad$ $Ruw = 0.755$ kips/in

Shear Strength,

For Column,

$Rv1 = \Lambda vr \cdot 0.6 \cdot Fu \cdot tw \cdot nws$ $\quad Rv1 = 30.328$ kips/in

For Shear Plate,

$Rv2 = \Lambda vr \cdot 0.6 \cdot Fu \cdot t$ $\quad Rv2 = 9.787$ kips/in

Effective Load Angle Factor,

$$\theta = \operatorname{atan}\left(\frac{Hc}{Vc}\right) \qquad \theta = 43.025 \text{ deg}$$

$$\mu = 1.0 + 0.50 \cdot \sin(\theta)^{1.5} \qquad \mu = 1.282$$

For Weld,

$Rv3 = \Lambda vw \cdot \mu \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws$

$Rv3 = 57.102$ ksi

Maximum effective weld size,

$$weff = \frac{\min(Rv1,\ Rv2)}{Rv3} \qquad weff = 0.171 \text{ in}$$

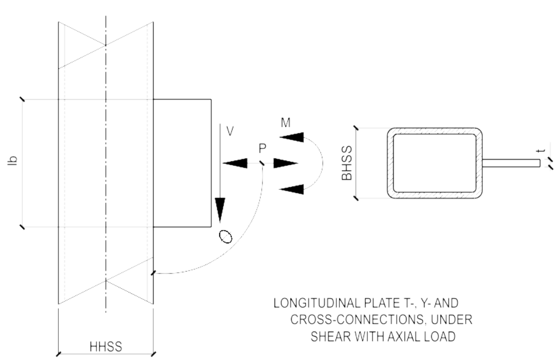
Weld Capacity,

$Rw = \Lambda vw \cdot \mu \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws \cdot \min(weff,\ w)$

$Rw = 9.787$ kips/in $\quad Ruw = 0.755$ kips/in

Weld Capacity > Applied Force, UCV = 0.077, OK

F. COLUMN WALL CHECK



1. Limits of Applicability

(AISC 14th Ed. Specifications, Chapter K, Table K1.2A, page 16.1-146)

a. Plate Load Angle

$\theta \geq 30deg$

Angle of Plate Load to the HSS Column Face,

$\theta in = 90$ deg

Connection is applicable, OK

b. HSS Wall Slenderness

$$\frac{BHSS}{tw} \le 40$$

$$\frac{BHSS}{tw} = 10.327$$

Connection is applicable, OK

c. HSS Wall Slenderness (For Branch Plate Shear Loading)

Width to Thickness Ratio of HSS,

$$WTR = \frac{BHSS - 3 \cdot tw}{tw} \qquad WTR = 7.327$$

Limiting Width to Thickness Ratio,

$$LWTR = 1.40 \cdot \left(\frac{E}{Fy}\right)^{0.5} \qquad LWTR = 35.152$$

Connection is applicable, OK

d. Material Strength

$$Fy \le 52\ ksi \qquad Fy = 46\ ksi$$

Connection is applicable, OK

e. Ductility

$$\frac{Fy}{Fu} \le 0.8$$

$$\frac{Fy}{Fu} = 0.793$$

Connection is applicable, OK

Applicability of Connection,

Connection is applicable, OK

2. HSS Local Check

a. HSS Punching Shear

(AISC Specification for the Design of Steel Hollow Structural Sections, page 15)

Thickness of Shear Plate,

$$t = 0.375\ in$$

Maximum Normal Stress in the Plate,

$$Nmax = \frac{Hc}{t \cdot L} \qquad Nmax = 1.373\ ksi$$

Maximum Shear Plate Thickness to Avoid Shear Tab Punching Thru Column Wall,

$$tPSmax = \frac{1.2 \cdot \Lambda vr \cdot Fu \cdot tw}{\Lambda ty \cdot Nmax}$$

$$tPSmax = 24.535\ in \qquad t = 0.375\ in$$

Plate thickness < Maximum Plate Thickness, OK

b. HSS Wall Plastification Capacity

(AISC 14th Ed. Specifications, Chapter K, Table K1.2, page 16.1-144)

Axial Load Required,

PuHSS = Hbm PuHSS = 0 kips

For Required Axial Strength,

Code = LRFD

$$PUR = \frac{P}{Ag \cdot Fy} \qquad PUR = 0$$

For Required Flexural Strength,

Code = LRFD

$$MUR = \frac{M}{S \cdot Fy} \qquad MUR = 0$$

For Uplift Force (if any),

Code = LRFD

$$ULUR = \frac{PUplift}{Ag \cdot Fy} \qquad ULUR = 0$$

Utilization Ratio,

(CIDECT Design Guide 3 Second Edition, Table 7.1, page 78)

$$n = \begin{bmatrix} -PUR - MUR + ULUR \\ -PUR + MUR + ULUR \\ PUR + MUR + ULUR \\ PUR - MUR + ULUR \end{bmatrix} \quad \begin{matrix} \text{(Axial in Compression, Moment in Compression)} \\ \text{(Axial in Compression, Moment in Tension)} \\ \text{(Axial in Tension, Moment in Tension)} \\ \text{(Axial in Tension, Moment in Compression)} \end{matrix}$$

Coefficient of Chord Stress Functions,

if $n < 0$ then $Cs = 0.20$

if $n \geq 0$ then $Cs = 0.10$

Chord-stress Interaction Parameter,

for $i \in 0..3$

if $n_i < 0$ then $Cs_i = 0.20$

if $n_i \geq 0$ then $Cs_i = 0.10$

for $i \in 0..3$

$$x_i = (1 - |n_i|)^{Cs_i}$$

$Qf = \min(x_i)$ $Qf = 1$

Branch Angle from the HSS Chord Face,

$\theta = 88.429$ deg

Nominal HSS Wall Plastification Capacity,

$$Rn = \frac{\frac{Fy \cdot tw^2}{1 - \frac{t}{BHSS}} \cdot \left[\frac{2L}{BHSS} + 4\left(1 - \frac{t}{BHSS}\right)^{0.5} \cdot Qf\right]}{\sin(\theta)}$$

HSS Wall Plastification Capacity,

$RpHSS = \Lambda pHSS \cdot Rn$

$RpHSS = 130.449$ kips $HcT = 6.181$ kips

Wall Plastification Capacity > Applied Force, UCV = 0.047, OK

G. GUSSET PLATE 1 TO BEAM FLANGE CHECK

1. Forces Acting on Connection

Vertical Force,

$Vb = 9.448$ kips

Horizontal Force,

$Hb = 12.97$ kips

Moment Force,

$Mb = 57.003$ kips·in

Resultant Force,

$Rb = \left(Vb^2 + Hb^2\right)^{0.5}$ $Rb = 16.046$ kips

2. Weld Capacity

(AISC 14th Ed. Specifications, Chapter J, pages 16.1-110 to 16.1-117)

(AISC 14th Ed. Manual, Part 8, pages 8-9 to 8-15)

a. Using Fillet Weld

Number of Weld Sides,

$nws = 2$

Minimum Weld Size,

$wmin = 0.187$ in $w = 0.25$ in

Preferred Weld Size > Minimum Weld Size, OK

Maximum Force on Welds Per Unit Length,

$$fmax = \left[\left(\frac{Hb}{Lw}\right)^2 + \left(\frac{Vb}{Lw} + \frac{4 \cdot Mb}{Lw^2}\right)^2\right]^{0.5}$$ $fmax = 1.641$ kips/in

Average Force on Welds Per Unit Length,

$$fave = \frac{1}{2}\cdot\left[\left[\left(\frac{Hb}{Lw}\right)^2+\left(\frac{Vb}{Lw}+\frac{4\cdot Mb}{Lw^2}\right)^2\right]^{0.5}+\left[\left(\frac{Hb}{Lw}\right)^2+\left(\frac{Vb}{Lw}-\frac{4\cdot Mb}{Lw^2}\right)^2\right]^{0.5}\right]$$

fave = 1.242 kips/in

Total Force Per Unit Length on Welds of Gusset Plate to Beam Connection,

$Ruw = \max(fmax, 1.25\cdot fave)$ Ruw = 1.641 kips/in

Shear Strength,

For Beam,

$Rv1 = \Lambda vr\cdot 0.6\cdot Fu\cdot tf\cdot nws$ Rv1 = 48.847 kips/in

For Gusset Plate,

$Rv2 = \Lambda vr\cdot 0.6\cdot Fu\cdot t$ Rv2 = 9.787 kips/in

Effective Load Angle Factor,

$$\theta = \operatorname{atan}\left(\frac{Vb+\frac{4\cdot Mb}{Lw}}{Hb}\right)$$ θ = 61.027 deg

$\mu = 1.0 + 0.50\cdot\sin(\theta)^{1.5}$ μ = 1.409

For Weld,

$Rv3 = \Lambda vw\cdot\mu\cdot 0.6\cdot Fu\cdot\sin(45deg)\cdot nws$

Rv3 = 62.774 ksi

Maximum effective weld size,

$$weff = \frac{\min(Rv1, Rv2)}{Rv3}$$ weff = 0.156 in

Weld Capacity,

$Rw = \Lambda vw\cdot\mu\cdot 0.6\cdot Fu\cdot\sin(45deg)\cdot nws\cdot\min(weff, w)$

Rw = 9.787 kips/in Ruw = 1.641 kips/in

Weld Capacity > Applied Force, UCV = 0.168, OK

G.A. GUSSET PLATE 1 CHECK

1. Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Yielding Capacity due to Shear Load

Length,

L = Lw L = 16.312 in

Number of Areas in Consideration,

n1 = n

Shear Yielding Capacity, (J4-3)

$Rvy = \Lambda vy \cdot n1 \cdot 0.6 \cdot Fy \cdot L \cdot t$

$Rvy = 132.131$ kips $\quad$ $Hb = 12.97$ kips

Shear Yielding Capacity > Applied Force, UCV = 0.098, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, page 16.1-128)

b. Tensile Yielding Capacity due to Axial Load

Length,

$L = Lw$ $\quad$ $L = 16.312$ in

Equivalent Normal Force,

$$Nb = Vb + \frac{4 \cdot Mb}{L}$$ $\quad$ $Nb = 23.425$ kips

Gross Tension Area,

$Ag = L \cdot t$

Tensile Yielding Capacity, (J4-1)

$Rty = \Lambda ty \cdot n \cdot Fy \cdot Ag$

$Rty = 198.197$ kips $\quad$ $Nb = 23.425$ kips

Tensile Yielding Capacity > Applied Force, UCV = 0.118, OK

Interaction of Yielding Capacities,

$$\left(\frac{Hb}{Rvy}\right)^2 + \left(\frac{Nb}{Rty}\right)^2 \le 1.0$$

$$UCV = \left(\frac{Hb}{Rvy}\right)^2 + \left(\frac{Nb}{Rty}\right)^2$$ $\quad$ $UCV = 0.024$

Yielding Capacity > Applied Force, UCV = 0.024, OK

H. BEAM WEB CHECK DUE TO GUSSET PLATE 1 STRESSES

1. Force Acting on Connection

Equivalent Normal Force Acting on the Connection,

$Nb = 23.425$ kips

2. Web Local Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J10.2, page 16.1-134)

Distance of Force to Beam End,

$De = 0.5 \cdot Lw$ $\quad$ $De = 8.156$ in

Bearing Length,

$N = Lw$ $\quad$ $N = 16.312$ in

Web Local Yielding Capacity, (J10-2, J10-3)

$De \le d$

$Rwy = \Lambda wy \cdot Fy \cdot tw \cdot (N + 2.5 \cdot kdes)$

$Rwy = 506.309$ kips $Nb = 23.425$ kips

Web Local Yielding Capacity > Applied Force, UCV = 0.046, OK

3. Web Local Crippling Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J10.3, pages 16.1-134 to 16.1-135)

Bearing Length,

$N = L$ $N = 16.312$ in

Web Crippling Capacity, (J10-4, J10-5a, J10-5b)

$$Esq = \left(\frac{E \cdot Fy \cdot tf}{tw}\right)^{0.5}$$ $Esq = 1533.288$ ksi

$$N2 = 1 + \left(\frac{4 \cdot N}{d} - 0.2\right) \cdot \left(\frac{tw}{tf}\right)^{1.5}$$ $N2 = 2.38$

$$De < \frac{d}{2} \wedge \frac{N}{d} > 0.2$$

$Rwc = \Lambda cr \cdot 0.4 \cdot tw^2 \cdot N2 \cdot Esq$

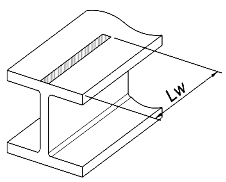
$Rwc = 290.361$ kips $Nb = 23.425$ kips

Web Local Crippling Capacity > Applied Force, UCV = 0.081, OK

4. Web Horizontal Shear Capacity

Force Acting on the Beam,

Horizontal Shear Force,



$$Vw = Hb \cdot \left(1 - \frac{tf \cdot bf}{Ag}\right)$$ $Vw = 9.26$ kips

Web Horizontal Shear Capacity,

$Rv = \Lambda vy \cdot 0.6 \cdot Fy \cdot Lw \cdot tw$

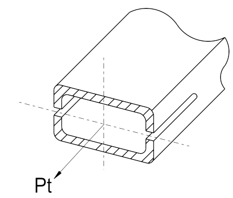
$Rv = 252.028$ kips $Vw = 9.26$ kips

Web Horizontal Shear Capacity > Applied Force, UCV = 0.037, OK

A. BRACE 2 CHECK

1. Rupture Capacity

(AISC 14th Ed. Specifications, Chapter D, Section D2, pages 16.1-26 to 16.1-27)



Length of the Connection,

$$Lcon = Lw \qquad Lcon = 6\ in$$

Net Tension Area,

$$Ant = Ag - 2 \cdot tw \cdot \left(t + \frac{1}{8} in\right) \qquad Ant = 6.531\ in^2$$

Eccentricity of the Connection,

$$econ = \frac{B^2 + 2 \cdot B \cdot H}{4 \cdot (B+H)} \qquad econ = 1.932\ in$$

Reduction Coefficient,

$$U = 1 - \frac{econ}{Lcon} \qquad U = 0.678$$

Effective Net Tension Area,

$$Ae = U \cdot Ant \qquad Ae = 4.428\ in^2$$

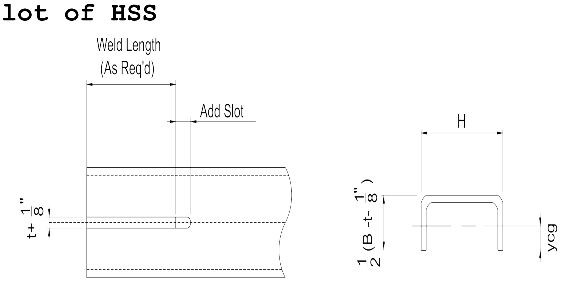
Tensile Rupture Capacity, (D2-2)

$$Rtr = \Lambda tr \cdot Fu \cdot Ae$$

$$Rtr = 192.627\ kips \qquad Pt = 25\ kips$$

Tensile Rupture Capacity > Applied Force, UCV = 0.13, OK

2. Additional Check for Slot of HSS



a. Local Check of C-Shape Section

Unstiffened Width,

$$b = \frac{1}{2} \cdot \left(B - t - \frac{1}{8} in\right) \qquad b = 2.25\ in$$

Limiting Width-to-Thickness Ratio,

(AISC 14th Ed. Specifications, Chapter B, Table B4.1a, page 16.1-16)

$$\frac{b}{tw} \le 0.56 \cdot \left(\frac{E}{Fy}\right)^{0.5}$$

$$\frac{b}{tw} = 6.447 \qquad 0.56\cdot\left(\frac{E}{Fy}\right)^{0.5} = 14.061$$

Section is Non-Slender

Q Factor,

(AISC 14th Ed. Specifications, Chapter E, Section E7.1, pages 16.1-40 to 16.1-41)

$$\frac{b}{tw} \le 0.56\cdot\left(\frac{E}{Fy}\right)^{0.5}$$

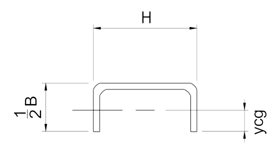
$Qs = 1$

b. Compression Capacity of C-Shape Section

(AISC 14th Ed. Specifications, Chapter E, Section E7, page 16.1-40)

Effective Length Factor,

$K = 1$



Laterally Unbraced Length,

Lu = Add Slot

Modulus of Elasticity

$E = 29000$ ksi

Gross Area,

$A1 = 2\cdot(b\cdot tw)$ $A1 = 1.57\ in^2$

$A2 = (H - 2\cdot tw)\cdot tw$ $A2 = 1.85\ in^2$

$Ag = A1 + A2$ $Ag = 3.421\ in^2$

Centroid,

$$ycg = \frac{A1\cdot\left(\frac{1}{2}\cdot b\right) + A2\cdot\left(b - \frac{1}{2}\cdot tw\right)}{Ag} \qquad ycg = 1.639\ in$$

Moment of Inertia,

$$I1 = 2\cdot\frac{tw\cdot b^3}{12} + A1\cdot\left(\frac{1}{2}\cdot b - ycg\right)^2$$

$$I2 = \frac{(H - 2\cdot tw)\cdot tw^3}{12} + A2\cdot\left(b - ycg - \frac{1}{2}\cdot tw\right)^2$$

$I = I1 + I2$ $I = 1.449\ in^4$

Radius of Gyration,

$$r = \left(\frac{I}{Ag}\right)^{0.5} \qquad r = 0.651\ in$$

Slenderness Ratio,

$$KLr = \frac{K \cdot Lu}{r} \qquad KLr = 3.073$$

Elastic Critical Buckling Stress,

$$Fe = \frac{\pi^2 \cdot E}{(KLr)^2} \qquad Fe = 30304.774\ ksi$$

Flexural Buckling Stress,

$$KLr \le 4.71 \cdot \left(\frac{E}{Qs \cdot Fy}\right)^{0.5}$$

$$Fcr = Qs \cdot 0.658^{\frac{Qs \cdot Fy}{Fe}} \cdot Fy$$

$$Fcr = 45.971\ ksi$$

Compression Capacity,

$$Rcb = \Lambda c \cdot Fcr \cdot Ag$$

$$Rcb = 141.535\ kips \qquad \frac{1}{2} Pc = 12.5\ kips$$

Compression Capacity > Applied Force, UCV = 0.088, OK

B. BRACE 2 TO GUSSET PLATE 2 CHECK

1. Weld Capacity

(AISC 14th Ed. Specifications, Chapter J, pages 16.1-110 to 16.1-117)

(AISC 14th Ed. Manual, Part 8, pages 8-9 to 8-15)

a. Using Fillet Weld

Number of Weld Sides,

$$nws = 4$$

Minimum Weld Size,

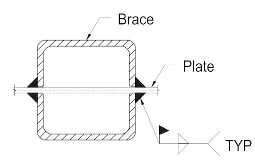
$$wmin = 0.187\ in \qquad w = 0.25\ in$$

Preferred Weld Size > Minimum Weld Size, OK

Shear Strength,

For Brace,

$$Rv1 = \Lambda vr \cdot 0.6 \cdot Fu \cdot tw \cdot nws \qquad Rv1 = 36.436\ kips/in$$



Number of Plates,

$$n1 = 1$$

For Gusset Plate,

$$Rv2 = \Lambda vr \cdot 0.6 \cdot Fu \cdot t \cdot n1 \qquad Rv2 = 19.575\ kips/in$$

For Weld,

$$Rv3 = \Lambda vw \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws \qquad Rv3 = 89.095\ ksi$$

Maximum Effective Weld Size,

$$weff = \frac{\min(Rv1, Rv2)}{Rv3}$$ weff = 0.22 in

Length of Weld,

Lw = 6 in

Weld Capacity,

$Rw = \Lambda vw \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws \cdot Lw \cdot \min(w, weff)$

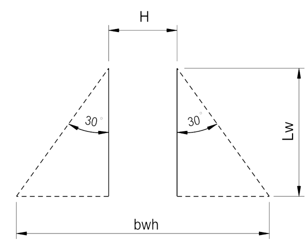
Rw = 117.45 kips P = 25 kips

Weld Capacity > Applied Force, UCV = 0.213, OK

C. GUSSET PLATE 2 CHECK

1. Whitmore Section

Width of Whitmore Section,



$bwh1 = 2 \cdot Lw \cdot \tan(30deg) + H$

bwh1 = 12.928 in

Width of Whitmore Section Outside Gusset Plate,

bwhog = 0 in

Available Width of Whitmore Section in Gusset Plate,

$bwh = bwh1 - 2 \cdot bwhog$ bwh = 12.928 in

Effective Length of Whitmore Section,

Lwh = 5.125 in

2. Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, page 16.1-128)

Width,

b = bwh b = 12.928 in

Gross Tension Area,

$Ag = b \cdot t$

Number of Areas in Consideration,

n1 = n

Tensile Yielding Capacity, (J4-1)

$Rty = \Lambda ty \cdot n1 \cdot Fy \cdot Ag$

Rty = 157.078 kips Pt = 25 kips

Tensile Yielding Capacity > Applied Force, UCV = 0.159, OK

3. Compression Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.4, page 16.1-129 to 16.1-130)

Effective Length Factor,

(Commentary on the Specification for Structural Steel Building Table C-A-7.1)

$K = 0.65$

Laterally Unbraced Length,

$Lu = Lwh$ $\quad$ $Lu = 5.125$ in

Gross Area,

$Ag = bwh \cdot t$ $\quad$ $Ag = 4.848$ in²

Radius of Gyration,

$$r = \frac{t}{(12)^{0.5}} \qquad r = 0.108 \text{ in}$$

Slenderness Ratio,

$$KLr = \frac{K \cdot Lu}{r} \qquad KLr = 30.773$$

Elastic Critical Buckling Stress,

$$Fe = \frac{\pi^2 \cdot E}{KLr^2} \qquad Fe = 302.249 \text{ ksi}$$

Flexural Buckling Stress,

$KLr > 25$

$$KLr \le 4.71 \cdot \left(\frac{E}{Fy}\right)^{0.5}$$

$$Fcr = 0.658^{\frac{Fy}{Fe}} \cdot Fy \qquad Fcr = 34.249 \text{ ksi}$$

Number of Areas in Consideration,

$n1 = n$

Compression Capacity,

$Rcb = \Lambda c \cdot n1 \cdot Fcr \cdot Ag$

$Rcb = 149.439$ kips $\quad$ $Pc = 25$ kips

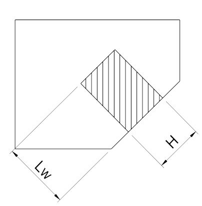
Compression Capacity > Applied Force, UCV = 0.167, OK

4. Block Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

Reduction Factor,

$Ubs = 1.0$ $\quad$ *(tension stress is uniform)*



Gross Shear Area,

$Agv = 2 \cdot Lw \cdot t$ $\quad$ $Agv = 4.5$ in²

Net Tension Area,

$Ant = H \cdot t$ $\quad$ $Ant = 2.25$ in²

Net Shear Area,

$Anv = Agv$ $\quad$ $Anv = 4.5$ in²

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

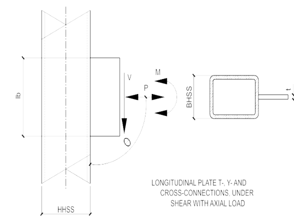
$Rbs = 170.775$ kips $Pt = 25$ kips

Block Shear Capacity > Applied Force, UCV = 0.146, OK

D. GUSSET PLATE 2 FORCE DISTRIBUTION

1. Gusset Plate Edge Forces

Connecting Face of HSS,



$BHSS = B$

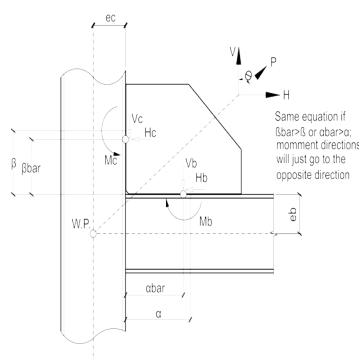
$BHSS = 6$ in

$HHSS = H$

$HHSS = 14$ in

(AISC 14th Ed. Manual Part 13, pages 13-3 to 13-11)

Uniform Force Method



Beam,

$eb = 0.5 \cdot d$

Horizontal Side,

$\alpha bar = 0.5 \cdot Lw + gap$

$\alpha bar = 8.656$ in

$\alpha = (\beta bar + eb) \cdot \tan(\theta) - ec$

$\alpha = 14.69$ in

$$r = \frac{P}{\left[(\alpha + ec)^2 + (\beta + eb)^2\right]^{0.5}}$$

Column,

$ec = 0.5HHSS$

Vertical Side,

$\beta bar = 0.5 \cdot (nr - 1) \cdot s + y$

$\beta bar = 7.5$ in

$\beta = \beta bar$

$\beta = 7.5$ in

$r = 0.883$ kips/in

Horizontal Side,

$Hb = \alpha \cdot r$

$Hb = 12.97$ kips

Vertical Side,

$Hc = ec \cdot r$

$Hc = 6.181$ kips

$Vb = eb \cdot r$ | $Vc = \beta \cdot r$

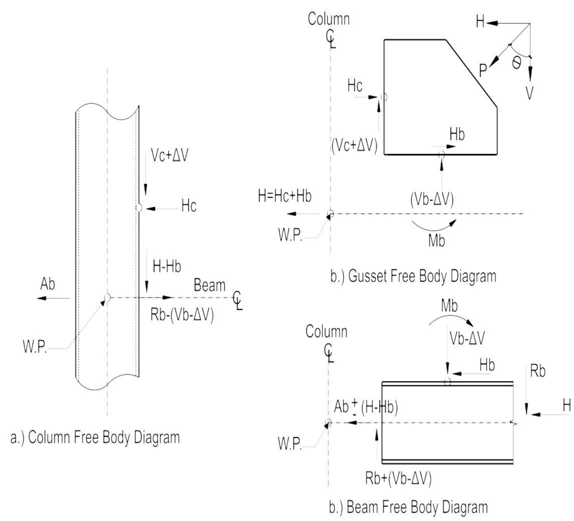
$Vb = 9.448$ kips | $Vc = 6.622$ kips

$Mb = |Vb \cdot (\alpha - \alpha bar)|$ | $Mc = |Hc \cdot (\beta - \beta bar)|$

$Mb = 57.003$ kips·in | $Mc = 0$ kips·in

Redistribution of Forces,

(Special Case 2: Redistribute to Column)



Shear Transfer,

$\Delta V = 0$ kips

Gusset-to-Beam Connection,

$Vb = Vb - \Delta V$ | $Hb = |Hb|$ | $Mb = |\Delta V \cdot \alpha bar + Mb|$

$Vb = 9.448$ kips | $Hb = 12.97$ kips | $Mb = 57.003$ kips·in

Gusset-to-Column Connection,

$Vc = |Vc + \Delta V|$ | $Hc = |Hc|$ | $Mc = |Hc \cdot (\beta - \beta bar)|$

$Vc = 6.622$ kips | $Hc = 6.181$ kips | $Mc = 0$ kips·in

E. GUSSET PLATE 2 TO COLUMN WALL CHECK

Note: Since Mc is equal to 0 kips, limit states will only be checked due to forces Vc and Hc

1. Forces Acting on Connection

Vertical Force,

$Vc = 6.622$ kips

Horizontal Force,

$Hc = 6.181$ kips

Moment Force,

$Mc = 0$ kips·in

Resultant Force,

$Rc = \left(Vc^2 + Hc^2\right)^{0.5}$ $\qquad Rc = 9.058$ kips

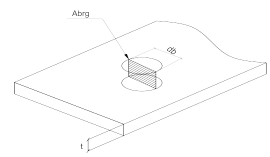
E.A. GUSSET PLATE 2 CHECK

1. Bolt Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)

a. Bolt Capacity due to Resultant Load

Bearing Area,



$Abrg = db \cdot t$ $\qquad Abrg = 0.281\ in^2$

Bolt Centerline Distance from Face of Support,

$ab = gap + Leh + 0.5 \cdot (nv - 1) \cdot sv$

$ab = 2.25$ in

Eccentricity Distannce of End Reaction from Bolt Group Centerline,

$ebv = ab$ $\qquad ebv = 2.25$ in

Bolt Vertical Centerline Distance from Beam Centerline,

$ah = |\beta - \beta bar|$ $\qquad ah = 0$ in

Eccentricity distance of Axial Load from Bolt Group Centerline,

$Yo = ah$ $\qquad Yo = 0$ in

Load Inclination from Vertical,

$\theta = atan\left(\frac{Hc}{Vc}\right)$ $\qquad \theta = 43.025$ deg

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-8)

$C = 3.148$

Available Bearing Strength Using Edge Distance, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbe = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (Lev - 0.5 \cdot hdv) \cdot t \\ 1.2 \cdot (Leh - 0.5 \cdot hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$Fbe = min(Fbe)$ $\qquad Fbe = 20.798$ kips

Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbs = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (s - hdv) \cdot t \\ 1.2 \cdot (sv - hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$nv > 1$

$Fbs = min(Fbs_0, Fbs_2)$ $\qquad Fbs = 29.362$ kips

Number of Area in Consideration,

$n1 = n$

Bolt Capacity,

$Rbrg = C \cdot \min(n1 \cdot Fbe,\ n1 \cdot Fbs,\ n \cdot \Lambda rv)$

$Rbrg = 25.399$ kips $\quad$ $Rc = 9.058$ kips

Bolt Capacity > Applied Force, UCV = 0.357, OK

2. Block Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

a. Block Shear Capacity due to Shear Load

Reduction Factor,

$nv = 1$ $\quad$ *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,

$Agv = [(nr - 1) \cdot s + Lev] \cdot t$ $\quad$ $Agv = 4.312$ in^2

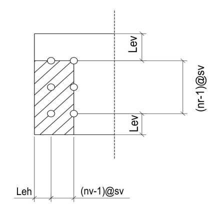
Net Tension Area,

$Ant = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot t$

$Ant = 0.492$ in^2

Net Shear Area,

$Anv = Agv - [(nr - 0.5) \cdot hdv] \cdot t$ $\quad$ $Anv = 3.164$ in^2



Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbss = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbss = 91.273$ kips $\quad$ $Vc = 6.622$ kips

Block Shear Capacity > Applied Force, UCV = 0.073, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

b. Block Shear Capacity due to Axial Load

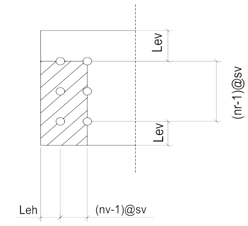
Pattern 1

Reduction Factor,

$ah = 0$ in *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,



$Agv = [Leh + (nv - 1) \cdot sv] \cdot t$ $Agv = 0.656\ in^2$

Net Tension Area,

$Ant = [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hdv] \cdot t$

$Ant = 3.164\ in^2$

Net Shear Area,

$Anv = Agv - [(nv - 0.5) \cdot hdh] \cdot t$ $Anv = 0.492\ in^2$

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs1 = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbs1 = 148.268$ kips $Hc = 6.181$ kips

Block Shear Capacity > Applied Force, UCV = 0.042, OK

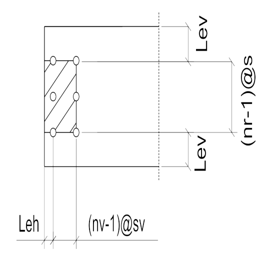
Pattern 2

Reduction Factor,

$ah = 0$ in *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,



$Agv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot t$ $Agv = 1.312\ in^2$

Net Tension Area,

$Ant = [(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t$

$Ant = 2.391\ in^2$

Net Shear Area,

$Anv = Agv - 2 \cdot [(nv - 0.5) \cdot hdh] \cdot t$ $Anv = 0.984\ in^2$

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs2 = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbs2 = 125.255$ kips $Hc = 6.181$ kips

Block Shear Capacity > Applied Force, UCV = 0.049, OK

Governing Block Shear Capacity,

$Rbs = \min(Rbs1, Rbs2)$

Rbs = 125.255 kips Hc = 6.181 kips

Block Shear Capacity > Applied Force, UCV = 0.049, OK

E.B. GUSSET PLATE 2 TO GUSSET SHEAR PLATE 2 CHECK

1. Bolt Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.6, page 16.1-125)

Load Inclination from Vertical,

$\theta = \operatorname{atan}\left(\frac{Hc}{Vc}\right)$ θ = 43.025 deg

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-12)

C = 3.148

Shear Capacity Per Bolt,

Λrv = 8.068 kips

Bolt Shear Capacity,

$Rb = n \cdot C \cdot \Lambda rv$

Rb = 25.399 kips Rc = 9.058 kips

Bolt Shear Capacity > Applied Force, UCV = 0.357, OK

2. Check for Spacing

(AISC 14th Ed. Specifications, Chapter J, Section J3.3 and J3.5, pages 16.1-122 to 16.1-124)

Shear Plate Thickness,

t1 = 0.375 in

Gusset Plate Thickness,

t2 = 0.375 in

a. Vertical Spacing,

Minimum Bolt Spacing,

s = 3 in

$smin = 2\frac{2}{3} \cdot db$ smin = 2 in

Maximum Bolt Spacing,

$smax = \min(12 \cdot in, 24 \cdot \min(t1, t2))$ smax = 9 in

Specified Bolt Spacing is acceptable, OK

3. Check for Edge Distance

(AISC 14th Ed. Specifications, Chapter J, Section J3.4 and J3.5, pages 16.1-122 to 16.1-124)

Shear Plate Edge Distances,

Lev1 = 1.5 in

Leh1 = 1.5 in

Gusset Plate Edge Distances,

Lev2 = 2.5 in

Leh2 = 1.75 in

i) Minimum Vertical Edge Distance,

Connection Edge Distance,

$$Levcon = \begin{bmatrix} Lev1 \\ Lev2 \end{bmatrix} \qquad Levcon = \begin{bmatrix} 1.5 \\ 2.5 \end{bmatrix} in$$

Minimum Edge Distance,

$$Levmin = \begin{bmatrix} Levmin1 \\ Levmin2 \end{bmatrix} \qquad Levmin = \begin{bmatrix} 1 \\ 1 \end{bmatrix} in$$

Specified Edge Distance is Acceptable, OK

ii) Minimum Horizontal Edge Distance,

Connection Edge Distance,

$$Lehcon = \begin{bmatrix} Leh1 \\ Leh2 \end{bmatrix} \qquad Lehcon = \begin{bmatrix} 1.5 \\ 1.75 \end{bmatrix} in$$

Minimum Edge Distance,

$$Lehmin = \begin{bmatrix} Lehmin1 \\ Lehmin2 \end{bmatrix} \qquad Lehmin = \begin{bmatrix} 1.125 \\ 1 \end{bmatrix} in$$

Specified Edge Distance is Acceptable, OK

iii) Maximum Edge Distance,

Shear Plate Thickness,

t1 = 0.375 in

Gusset Plate Thickness,

t2 = 0.375 in

Nearest Connection Edge Distance,

Lemin = min(Lehcon, Levcon)

Lemin = 1.5 in

Maximum Edge Distance,

$Lemin = Levcon_0 \vee Lemin = Lehcon_0$

$Lemax = \min(6in, 12 \cdot t1)$

Lemax = 4.5 in

Maximum Edge Distance Requirement is Satisfied, OK

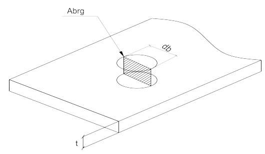
E.C. GUSSET SHEAR PLATE 2 CHECK

1. Bolt Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)

a. Bolt Capacity due to Resultant Load

Bearing Area,



$Abrg = db \cdot t$ $\qquad$ $Abrg = 0.281\ in^2$

Bolt Centerline Distance from Face of Support,

$ab = gap + Leh + 0.5 \cdot (nv - 1) \cdot sv$

$ab = 2.25\ in$

Eccentricity Distannce of End Reaction from Bolt Group Centerline,

$ebv = ab$ $\qquad$ $ebv = 2.25\ in$

Bolt Vertical Centerline Distance from Beam Centerline,

$ah = |\beta - \beta bar|$ $\qquad$ $ah = 0\ in$

Eccentricity distance of Axial Load from Bolt Group Centerline,

$Yo = ah$ $\qquad$ $Yo = 0\ in$

Load Inclination from Vertical,

$\theta = atan\left(\frac{Hc}{Vc}\right)$ $\qquad$ $\theta = 43.025\ deg$

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-8)

$C = 3.148$

Available Bearing Strength Using Edge Distance, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbe = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (Lev - 0.5 \cdot hdv) \cdot t \\ 1.2 \cdot (Leh - 0.5 \cdot hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$Fbe = min(Fbe)$ $\qquad$ $Fbe = 18.963\ kips$

Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbs = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (s - hdv) \cdot t \\ 1.2 \cdot (sv - hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$nv > 1$

$Fbs = min(Fbs_0, Fbs_2)$ $\qquad$ $Fbs = 29.362\ kips$

Number of Area in Consideration,

$n1 = n$

Bolt Capacity,

$Rbrg = C \cdot min(n1 \cdot Fbe, n1 \cdot Fbs, n \cdot \Lambda rv)$

Rbrg = 25.399 kips   Rc = 9.058 kips

Bolt Capacity > Applied Force, UCV = 0.357, OK

2. Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Yielding Capacity due to Shear Load

Length,

$L = (nr - 1)\cdot s + 2\cdot Lev$ $L = 12$ in

Number of Areas in Consideration,

$n1 = n$

Shear Yielding Capacity, (J4-3)

$Rvy = \Lambda vy\cdot n1\cdot 0.6\cdot Fy\cdot L\cdot t$

Rvy = 97.2 kips   Vc = 6.622 kips

Shear Yielding Capacity > Applied Force, UCV = 0.068, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, page 16.1-128)

b. Tensile Yielding Capacity due to Axial Load

Length,

$L = (nr - 1)\cdot s + 2\cdot Lev$ $L = 12$ in

Gross Tension Area,

$Ag = L\cdot t$

Number of Areas in Consideration,

$n1 = n$

Tensile Yielding Capacity, (J4-1)

$Rty = \Lambda ty\cdot n1\cdot Fy\cdot Ag$

Rty = 145.8 kips   Hc = 6.181 kips

Tensile Yielding Capacity > Applied Force, UCV = 0.042, OK

Interaction of Yielding Capacities,

$$\left(\frac{Vc}{Rvy}\right)^2 + \left(\frac{Hc}{Rty}\right)^2 \le 1.0$$

$$UCV = \left(\frac{Vc}{Rvy}\right)^2 + \left(\frac{Hc}{Rty}\right)^2$$ UCV = 0.006

Yielding Capacity > Applied Force, UCV = 0.006, OK

3. Rupture Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Rupture Capacity due to Shear Load

Net Shear Area,

$Anv = (L - nr \cdot hdv) \cdot t$

$Anv = 3.187\ in^2$

Number of Areas in Consideration,

$n1 = n$

Shear Rupture Capacity, (J4-4)

$Rvr = \Lambda vr \cdot n1 \cdot 0.6 \cdot Fu \cdot Anv$

$Rvr = 83.194\ kips$ $Vc = 6.622\ kips$

Shear Rupture Capacity > Applied Force, UCV = 0.08, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, pages 16.1-128 to 16.1-129)

b. Tensile Rupture Capacity due to Axial Load

Effective Net Area,

$Ae = \min[(L - nr \cdot hdv) \cdot t,\ 0.85 \cdot L \cdot t]$ $Ae = 3.187\ in^2$

Number of Areas in Consideration,

$n1 = n$

Tensile Rupture Capacity, (J4-2)

$Rtr = \Lambda tr \cdot n1 \cdot Fu \cdot Ae$

$Rtr = 138.656\ kips$ $Hc = 6.181\ kips$

Tensile Rupture Capacity > Applied Force, UCV = 0.045, OK

Interaction of Rupture Capacities,

$$\left(\frac{Vc}{Rvr}\right)^2 + \left(\frac{Hc}{Rtr}\right)^2 \le 1.0$$

$$UCV = \left(\frac{Vc}{Rvr}\right)^2 + \left(\frac{Hc}{Rtr}\right)^2$$ $UCV = 0.008$

Rupture Capacity > Applied Force, UCV = 0.008, OK

4. Block Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

a. Block Shear Capacity due to Shear Load

Reduction Factor,

$nv = 1$ *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,

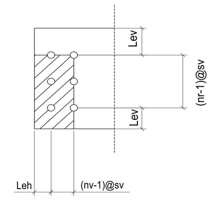
$Agv = [(nr - 1) \cdot s + Lev] \cdot t$ $Agv = 3.938\ in^2$

Net Tension Area,

$Ant = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot t$

$Ant = 0.363\ in^2$

Net Shear Area,



$Anv = Agv - [(nr - 0.5) \cdot hdv] \cdot t$ $\qquad$ $Anv = 2.789\ in^2$

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbss = \Lambda bs \cdot n1 \cdot min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbss = 79.59\ kips$ $\qquad$ $Vc = 6.622\ kips$

Block Shear Capacity > Applied Force, UCV = 0.083, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

b. Block Shear Capacity due to Axial Load

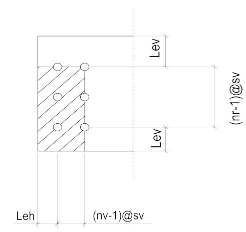
Pattern 1

Reduction Factor,

$ah = 0\ in$ $\qquad$ *(tension stress is uniform)*

$Ubs = 1.0$

Gross Shear Area,



$Agv = [Leh + (nv - 1) \cdot sv] \cdot t$ $\qquad$ $Agv = 0.563\ in^2$

Net Tension Area,

$Ant = [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hdv] \cdot t$

$Ant = 2.789\ in^2$

Net Shear Area,

$Anv = Agv - [(nv - 0.5) \cdot hdh] \cdot t$ $\qquad$ $Anv = 0.363\ in^2$

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbsa = \Lambda bs \cdot n1 \cdot min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbsa = 130.437\ kips$ $\qquad$ $Hc = 6.181\ kips$

Block Shear Capacity > Applied Force, UCV = 0.047, OK

Interaction of Bolt Shear Capacities,

$$\left(\frac{Vc}{Rbss}\right)^2 + \left(\frac{Hc}{Rbsa}\right)^2 \le 1.0$$

$$UCV = \sqrt{\left(\frac{Vc}{Rbss}\right)^2 + \left(\frac{Hc}{Rbsa}\right)^2} \qquad UCV = 0.009$$

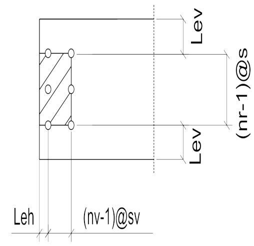
Block Shear Capacity > Applied Force, UCV = 0.009, OK

Pattern 2

Reduction Factor,

$ah = 0$ in (tension stress is uniform)

$Ubs = 1.0$



Gross Shear Area,

$Agv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot t$ $Agv = 1.125$ in²

Net Tension Area,

$Ant = [(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t$

$Ant = 2.391$ in²

Net Shear Area,

$Anv = Agv - 2 \cdot [(nv - 0.5) \cdot hdh] \cdot t$ $Anv = 0.727$ in²

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs2 = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbs2 = 122.217$ kips $Hc = 6.181$ kips

Block Shear Capacity > Applied Force, UCV = 0.051, OK

5. Local Buckling Capacity

(AISC 14th Ed. Manual, Part 9, page 9-9)

Distance of Bolt Line to Support,

$ab = gap + Leh$ $ab = 2.25$ in

Coefficient,

$$\lambda = \frac{L \cdot Fy^{0.5}}{10 \cdot t \cdot \left[475 + 280 \cdot \left(\frac{L}{ab}\right)^2\right]^{0.5}} \cdot \frac{1}{ksi^{0.5}}$$ $\lambda = 0.209$

$\lambda \leq 0.7$

$Q = 1$

Allowable Flexural Local Buckling Stress or Yielding Stress,

$Fcr = Q \cdot Fy$ $Fcr = 36$ ksi

Gross Plastic Section Modulus,

$$Zx = \frac{t \cdot L^2}{4}$$ $Zx = 13.5$ in³

Eccentricity,

$e = ab$ $e = 2.25$ in

Local Buckling Capacity,

$$Rbc = \Lambda b \cdot \frac{Fcr \cdot Zx}{e}$$

Rbc = 194.4 kips Vc = 6.622 kips

Local Buckling Capacity > Applied Force, UCV = 0.034, OK

6. Shear and Flexural Yielding Capacity with Von-Mises Yield Criterion

(AISC 14th Ed., Manual Part 10, page 10-104)

(Muir, Larry and Hewitt,Christopher, "Design of Unstiffened Extended Single Plate Shear Connections", Engineering Journal, 2nd Quarter 2009, page 69)

Shear and Flexural Yielding Interaction Capacity,

$$Rfc = \frac{\Lambda b \cdot Fy \cdot L \cdot t}{\left[2.25 + 16 \cdot \left(\frac{e}{L}\right)^2\right]^{0.5}}$$

Rfc = 86.938 kips Vc = 6.622 kips

Yielding Capacity > Applied Force, UCV = 0.076, OK

7. Flexural Rupture Capacity

(AISC 14th Ed. Manual Part 15, page 15-4)

(AISC 14th Ed. Steel Construction Manual Design Examples, page IIA-104)

Net Plastic Section Modulus,

mod(nr, 2) = 0

$$Znet = \frac{t \cdot L^2}{4} - \frac{t \cdot hdv \cdot nr^2 \cdot s}{4}$$

Znet = 9.563 in^3

Flexural Rupture Capacity,

$$Rfr = \frac{\Lambda fr \cdot Fu \cdot Znet}{e}$$

Rfr = 184.875 kips Vc = 6.622 kips

Flexural Rupture Capacity > Applied Force, UCV = 0.036, OK

8. Compression Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.4, page 16.1-129 to 16.1-130)

Effective Length Factor,

(Commentary on the Specification for Structural Steel Building Table C-A-7.1)

K = 1.2

Gusset Horizontal Edge Distance,

Le1 = 1.75 in

Laterally Unbraced Length,

$Lu = gap + Le1$ $\quad$ $Lu = 2.25$ in

Gross Area,

$Ag = L \cdot t$ $\quad$ $Ag = 4.5$ in²

Radius of Gyration,

$$r = \frac{t}{(12)^{0.5}} \qquad r = 0.108 \text{ in}$$

Slenderness Ratio,

$$KLr = \frac{K \cdot Lu}{r} \qquad KLr = 24.942$$

Elastic Critical Buckling Stress,

$$Fe = \frac{\pi^2 \cdot E}{KLr^2} \qquad Fe = 460.099 \text{ ksi}$$

Flexural Buckling Stress,

$KLr \le 25$

$Fcr = Fy$ $\quad$ $Fcr = 36$ ksi

Number of Areas in Consideration,

$n1 = n$

Compression Capacity,

$Rcb = \Lambda c \cdot n1 \cdot Fcr \cdot Ag$

$Rcb = 145.8$ kips $\quad$ $Hc = 6.181$ kips

Compression Capacity > Applied Force, UCV = 0.042, OK

E.D. GUSSET SHEAR PLATE 2 TO COLUMN WALL CHECK

1. Weld Capacity

(AISC 14th Ed. Specifications, Chapter J, pages 16.1-110 to 16.1-117)

(AISC 14th Ed. Manual, Part 8, pages 8-9 to 8-15)

a. Using Fillet Weld

Number of Weld Sides,

$nws = 2$

Minimum Weld Size,

$wmin = 0.187$ in $\quad$ $w = 0.25$ in

Preferred Weld Size > Minimum Weld Size, OK

Length of Weld,

$Lw = (nr - 1) \cdot s + 2 \cdot Lev$ $\quad$ $Lw = 12$ in

Total Force Per Unit Length on Welds of Shear Plate to Column Connection,

$$Ruw = \left[\left(\frac{Vc}{Lw}\right)^2 + \left(\frac{Hc}{Lw}\right)^2\right]^{0.5} \qquad Ruw = 0.755 \text{ kips/in}$$

Shear Strength,

For Column,

$Rv1 = \Lambda vr \cdot 0.6 \cdot Fu \cdot tw \cdot nws$ $\quad Rv1 = 30.328$ kips/in

For Shear Plate,

$Rv2 = \Lambda vr \cdot 0.6 \cdot Fu \cdot t$ $\quad Rv2 = 9.787$ kips/in

Effective Load Angle Factor,

$$\theta = \operatorname{atan}\left(\frac{Hc}{Vc}\right) \qquad \theta = 43.025\ deg$$

$$\mu = 1.0 + 0.50 \cdot \sin(\theta)^{1.5} \qquad \mu = 1.282$$

For Weld,

$Rv3 = \Lambda vw \cdot \mu \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws$

$Rv3 = 57.102$ ksi

Maximum effective weld size,

$$weff = \frac{\min(Rv1,\ Rv2)}{Rv3} \qquad weff = 0.171\ in$$

Weld Capacity,

$Rw = \Lambda vw \cdot \mu \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws \cdot \min(weff,\ w)$

$Rw = 9.787$ kips/in $\quad Ruw = 0.755$ kips/in

Weld Capacity > Applied Force, UCV = 0.077, OK

F. COLUMN WALL CHECK

1. HSS Local Check

a. HSS Punching Shear

(AISC Specification for the Design of Steel Hollow Structural Sections, page 15)

Thickness of Shear Plate,

$t = 0.375$ in

Maximum Normal Stress in the Plate,

$$Nmax = \frac{Hc}{t \cdot L} \qquad Nmax = 1.373\ ksi$$

Maximum Shear Plate Thickness to Avoid Shear Tab Punching Thru Column Wall,

$$tPSmax = \frac{1.2 \cdot \Lambda vr \cdot Fu \cdot tw}{\Lambda ty \cdot Nmax}$$

$tPSmax = 24.535$ in $\quad t = 0.375$ in

Plate thickness < Maximum Plate Thickness, OK

b. HSS Wall Plastification Capacity

(AISC 14th Ed. Specifications, Chapter K, Table K1.2, page 16.1-144)

Axial Load Required,

$PuHSS = Hbm$ $\quad PuHSS = 0$ kips

For Required Axial Strength,

Code = LRFD

$$PUR = \frac{P}{Ag \cdot Fy}$$ PUR = 0

For Required Flexural Strength,

Code = LRFD

$$MUR = \frac{M}{S \cdot Fy}$$ MUR = 0

For Uplift Force (if any),

Code = LRFD

$$ULUR = \frac{PUplift}{Ag \cdot Fy}$$ ULUR = 0

Utilization Ratio,

(CIDECT Design Guide 3 Second Edition, Table 7.1, page 78)

$$n = \begin{bmatrix} -PUR - MUR + ULUR \\ -PUR + MUR + ULUR \\ PUR + MUR + ULUR \\ PUR - MUR + ULUR \end{bmatrix}$$

(Axial in Compression, Moment in Compression)
(Axial in Compression, Moment in Tension)
(Axial in Tension, Moment in Tension)
(Axial in Tension, Moment in Compression)

Coefficient of Chord Stress Functions,

if $n < 0$ then $Cs = 0.20$

if $n \geq 0$ then $Cs = 0.10$

Chord-stress Interaction Parameter,

for $i \in 0..3$

if $n_i < 0$ then $Cs_i = 0.20$

if $n_i \geq 0$ then $Cs_i = 0.10$

for $i \in 0..3$

$$x_i = (1 - |n_i|)^{Cs_i}$$

$Qf = \min(x_i)$ Qf = 1

Branch Angle from the HSS Chord Face,

$\theta = 88.429$ deg

Nominal HSS Wall Plastification Capacity,

$$Rn = \frac{\dfrac{Fy \cdot tw^2}{1 - \dfrac{t}{BHSS}} \cdot \left[\dfrac{2L}{BHSS} + 4\left(1 - \dfrac{t}{BHSS}\right)^{0.5} \cdot Qf\right]}{\sin(\theta)}$$

HSS Wall Plastification Capacity,

$RpHSS = \Lambda pHSS \cdot Rn$

RpHSS = 130.449 kips HcB = 6.181 kips

Wall Plastification Capacity > Applied Force, UCV = 0.047, OK

G. GUSSET PLATE 2 TO BEAM FLANGE CHECK

1. Forces Acting on Connection

Vertical Force,

Vb = 9.448 kips

Horizontal Force,

Hb = 12.97 kips

Moment Force,

Mb = 57.003 kips·in

Resultant Force,

$$Rb = \left(Vb^2 + Hb^2\right)^{0.5}$$ Rb = 16.046 kips

2. Weld Capacity

(AISC 14th Ed. Specifications, Chapter J, pages 16.1-110 to 16.1-117)

(AISC 14th Ed. Manual, Part 8, pages 8-9 to 8-15)

a. Using Fillet Weld

Number of Weld Sides,

nws = 2

Minimum Weld Size,

wmin = 0.187 in w = 0.25 in

Preferred Weld Size > Minimum Weld Size, OK

Maximum Force on Welds Per Unit Length,

$$fmax = \left[\left(\frac{Hb}{Lw}\right)^2 + \left(\frac{Vb}{Lw} + \frac{4 \cdot Mb}{Lw^2}\right)^2\right]^{0.5}$$ fmax = 1.641 kips/in

Average Force on Welds Per Unit Length,

$$fave = \frac{1}{2} \cdot \left[\left[\left(\frac{Hb}{Lw}\right)^2 + \left(\frac{Vb}{Lw} + \frac{4 \cdot Mb}{Lw^2}\right)^2\right]^{0.5} + \left[\left(\frac{Hb}{Lw}\right)^2 + \left(\frac{Vb}{Lw} - \frac{4 \cdot Mb}{Lw^2}\right)^2\right]^{0.5}\right]$$

fave = 1.242 kips/in

Total Force Per Unit Length on Welds of Gusset Plate to Beam Connection,

Ruw = max(fmax, 1.25·fave) Ruw = 1.641 kips/in

Shear Strength,

For Beam,

$Rv1 = \Lambda vr \cdot 0.6 \cdot Fu \cdot tf \cdot nws$ Rv1 = 48.847 kips/in

For Gusset Plate,

$Rv2 = \Lambda vr \cdot 0.6 \cdot Fu \cdot t$ $\quad Rv2 = 9.787$ kips/in

Effective Load Angle Factor,

$$\theta = \operatorname{atan}\left(\frac{Vb + \frac{4 \cdot Mb}{Lw}}{Hb}\right) \qquad \theta = 61.027 \text{ deg}$$

$$\mu = 1.0 + 0.50 \cdot \sin(\theta)^{1.5} \qquad \mu = 1.409$$

For Weld,

$Rv3 = \Lambda vw \cdot \mu \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws$

$Rv3 = 62.774$ ksi

Maximum effective weld size,

$$weff = \frac{\min(Rv1, Rv2)}{Rv3} \qquad weff = 0.156 \text{ in}$$

Weld Capacity,

$Rw = \Lambda vw \cdot \mu \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws \cdot \min(weff, w)$

$Rw = 9.787$ kips/in $\quad Ruw = 1.641$ kips/in

Weld Capacity > Applied Force, UCV = 0.168, OK

G.A. GUSSET PLATE 2 CHECK

1. Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Yielding Capacity due to Shear Load

Length,

$L = Lw$ $\quad L = 16.312$ in

Number of Areas in Consideration,

$n1 = n$

Shear Yielding Capacity, (J4-3)

$Rvy = \Lambda vy \cdot n1 \cdot 0.6 \cdot Fy \cdot L \cdot t$

$Rvy = 132.131$ kips $\quad Hb = 12.97$ kips

Shear Yielding Capacity > Applied Force, UCV = 0.098, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, page 16.1-128)

b. Tensile Yielding Capacity due to Axial Load

Length,

$L = Lw$ $\quad L = 16.312$ in

Equivalent Normal Force,

$$Nb = Vb + \frac{4 \cdot Mb}{L} \qquad Nb = 23.425 \text{ kips}$$

Gross Tension Area,

$Ag = L \cdot t$

Tensile Yielding Capacity, (J4-1)

$Rty = \Lambda ty \cdot n \cdot Fy \cdot Ag$

$Rty = 198.197$ kips $\qquad Nb = 23.425$ kips

Tensile Yielding Capacity > Applied Force, UCV = 0.118, OK

Interaction of Yielding Capacities,

$$\left(\frac{Hb}{Rvy}\right)^2 + \left(\frac{Nb}{Rty}\right)^2 \le 1.0$$

$$UCV = \left(\frac{Hb}{Rvy}\right)^2 + \left(\frac{Nb}{Rty}\right)^2 \qquad UCV = 0.024$$

Yielding Capacity > Applied Force, UCV = 0.024, OK

H. BEAM WEB CHECK DUE TO GUSSET PLATE 2 STRESSES

1. Force Acting on Connection

Equivalent Normal Force Acting on the Connection,

$Nb = 23.425$ kips

2. Web Local Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J10.2, page 16.1-134)

Distance of Force to Beam End,

$De = 0.5 \cdot Lw \qquad De = 8.156$ in

Bearing Length,

$N = Lw \qquad N = 16.312$ in

Web Local Yielding Capacity, (J10-2, J10-3)

$De \le d$

$Rwy = \Lambda wy \cdot Fy \cdot tw \cdot (N + 2.5 \cdot kdes)$

$Rwy = 506.309$ kips $\qquad Nb = 23.425$ kips

Web Local Yielding Capacity > Applied Force, UCV = 0.046, OK

3. Web Local Crippling Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J10.3, pages 16.1-134 to 16.1-135)

Bearing Length,

$N = L \qquad N = 16.312$ in

Web Crippling Capacity, (J10-4, J10-5a, J10-5b)

$$Esq = \left(\frac{E \cdot Fy \cdot tf}{tw}\right)^{0.5} \qquad Esq = 1533.288 \text{ ksi}$$

$$N2 = 1 + \left(\frac{4 \cdot N}{d} - 0.2\right) \cdot \left(\frac{tw}{tf}\right)^{1.5}$$ $N2 = 2.38$

$$De < \frac{d}{2} \wedge \frac{N}{d} > 0.2$$

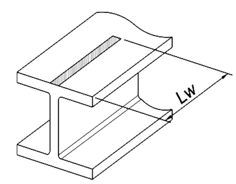
$Rwc = \Lambda cr \cdot 0.4 \cdot tw^2 \cdot N2 \cdot Esq$

$Rwc = 290.361$ kips $Nb = 23.425$ kips

Web Local Crippling Capacity > Applied Force, UCV = 0.081, OK

4. Web Horizontal Shear Capacity

Force Acting on the Beam,



Horizontal Shear Force,

$$Vw = Hb \cdot \left(1 - \frac{tf \cdot bf}{Ag}\right)$$ $Vw = 9.26$ kips

Web Horizontal Shear Capacity,

$Rv = \Lambda vy \cdot 0.6 \cdot Fy \cdot Lw \cdot tw$

$Rv = 252.028$ kips $Vw = 9.26$ kips

Web Horizontal Shear Capacity > Applied Force, UCV = 0.037, OK

I. BEAM WEB TO COLUMN WALL CHECK

1. Forces Acting on Connection

Other Brace Connection Vertical Force at Gusset-to-Beam Interface,

$Vb2 = 9.448$ kips

Other Brace Connection Maximum Axial Load,

$P2 = 25$ kips

Other Brace Connection Angle from Vertical Member,

$\theta 2 = 50$ deg

Other Brace Connection Horizontal Force at Gusset-to-Beam Interface,

$Hb2 = 12.97$ kips

Vertical Force,

$Vbm = V + Vb + Vb2$ $Vbm = 28.895$ kips

Horizontal Force,

$Hbm = TF + |(P \cdot \sin(\theta) - Hb) - (P2 \cdot \sin(\theta 2) - Hb2)|$

$Hbm = 0$ kips

Resultant Force,

$$Rbm = \left(Vbm^2 + Hbm^2\right)^{0.5}$$ $Rbm = 28.895$ kips

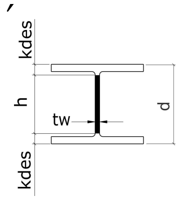
I.A. BEAM WEB CHECK

1. Shear Capacity

(AISC 14th Ed. Specifications, Chapter G, Section G2.1, pages 16.1-67 to 16.1-69)

Clear Distance Between Flanges of Beam Less the Fillet or Corner Radii,

$h = d - 2 \cdot kdes$ $\quad$ $h = 18.72$ in



Limiting Depth-Thickness Ratio,

$htw = \dfrac{h}{tw}$ $\quad$ $htw = 36.35$

Clear Distance Between Transverse Stiffeners,

$htw < 260$ $\quad$ $a = 0$ in

Web Plate Buckling Coefficient, (G2-6)

$htw < 260$ $\quad$ $kv = 5$

Web Shear Coefficient, (G2-3, G2-4, G2-5)

$htw \le 1.1 \cdot \left(\dfrac{kv \cdot E}{Fy}\right)^{0.5}$ $\quad$ $Cv = 1$

Shear Capacity, (G2-1)

$Rv = \Lambda vbm \cdot 0.6 \cdot Fy \cdot d \cdot tw \cdot Cv$

$Rv = 330.63$ kips $\quad$ $Vbm = 28.895$ kips

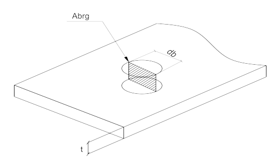
Shear Capacity of Section > Applied Force, UCV = 0.087, OK

2. Bolt Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)

a. Bolt Capacity due to Resultant Load

Bearing Area,



$Abrg = db \cdot tw$ $\quad$ $Abrg = 0.386$ in²

Bolt Centerline Distance from Face of Support,

$ab = Leh + 0.5 \cdot (nv - 1) \cdot sv$

$ab = 2.25$ in

Eccentricity Distannce of End Reaction from Bolt Group Centerline,

$ebv = ebv = ab$ $\quad$ $ebv = 2.25$ in

Bolt Vertical Centerline Distance from Beam Centerline,

$ah = |0.5 \cdot d - [D + 0.5 \cdot (nr - 1) \cdot s]|$ $\quad$ $ah = 1.7$ in

Eccentricity distance of Axial Load from Bolt Group Centerline,

$Yo = ah$ $\quad$ $Yo = 1.7$ in

Load Inclination from Vertical,

$\theta = \operatorname{atan}\left(\dfrac{Hbm}{Vbm}\right)$ $\quad$ $\theta = 0$ deg

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-8)

$C = 4.276$

Available Bearing Strength Using Edge Distance, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbe = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (Lev - 0.5 \cdot hdv) \cdot tw \\ 1.2 \cdot (Leh - 0.5 \cdot hdh) \cdot tw \\ 2.4 \cdot Abrg \end{bmatrix}$$

$Fbe = \min(Fbe1, Fbe2)$ $\quad$ $Fbe = 39.542$ kips

Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)

$hdv < hdls(db)$

$$Fbs = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (s - hdv) \cdot tw \\ 1.2 \cdot (sv - hdh) \cdot tw \\ 2.4 \cdot Abrg \end{bmatrix}$$

$nv > 1$

$Fbs = \min(Fbs_0, Fbs_2)$ $\quad$ $Fbs = 45.191$ kips

Number of Area in Consideration,

$n1 = 1$

Bolt Capacity,

$Rbrg = C \cdot \min(n1 \cdot Fbe, n1 \cdot Fbs, n \cdot \Lambda rv)$

$Rbrg = 34.496$ kips $\quad$ $Rbm = 28.895$ kips

Bolt Capacity > Applied Force, UCV = 0.838, OK

3. Rupture Capacity

(AISC 14th Ed. Specifications, Chapter D, Section D2, pages 16.1-26 to 16.1-27)

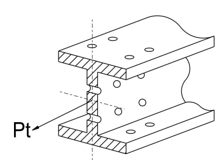
(AISC 14th Ed. Specifications Chapter J, Section J4.1, pages 16.1-128 to 16.1-129)

Length of the Connection,

$Lcon = (nv - 1) \cdot sv$ $\quad$ $Lcon = 0$ in

Net Tension Area,

$Ant = 22.147\ in^2$



Eccentricity of the Connection,

$econ = 1.25$

Reduction Coefficient,

$U = 0.452$

Effective Net Tension Area,

$Ae = U \cdot Ant$

$Ae = 10.003\ in^2$

Tensile Rupture Capacity, (D2-2)

$Rtr = \Lambda tr \cdot Fu \cdot Ae$

Rtr = 487.661 kips   Hbm = 0 kips

Tensile Rupture Capacity > Applied Force, UCV = 0, OK

4. Block Shear Capacity

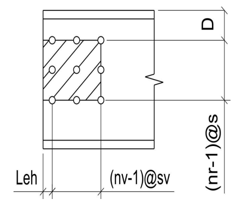
(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

Reduction Factor,

Ubs = 0.5   *(tension stress is non-uniform)*

Gross Shear Area,

Agv = 2·[Leh + (nv − 1)·sv]·tw   Agv = 1.802 in²



Net Tension Area,

Ant = [(nr − 1)·s − (nr − 1)·hdv]·tw   Ant = 4.377 in²

Net Shear Area,

Anv = Agv − 2·[(nv − 0.5)·hdh]·tw   Anv = 1.352 in²

Number of Areas in Consideration,

n1 = 1

Block Shear Capacity, (J4-5)

Rbs = Λbs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)

Rbs = 146.244 kips   Hbm = 0 kips

Block Shear Capacity > Applied Force, UCV = 0, OK

I.B. BEAM WEB TO BEAM SHEAR PLATE CHECK

1. Bolt Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.6, page 16.1-125)

Load Inclination from Vertical,

$\theta = \mathrm{atan}\left(\frac{Hbm}{Vbm}\right)$ θ = 0 deg

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-12)

C = 4.276

Shear Capacity Per Bolt,

Λrv = 8.068 kips

Bolt Shear Capacity,

Rb = n·C·Λrv

Rb = 34.496 kips   Rbm = 28.895 kips

Bolt Shear Capacity > Applied Force, UCV = 0.838, OK

2. Check for Spacing

(AISC 14th Ed. Specifications, Chapter J, Section J3.3 and J3.5, pages 16.1-122 to 16.1-124)

Shear Plate Thickness,

$t1 = 0.375 \text{ in}$

Beam Web Thickness,

$t2 = 0.515 \text{ in}$

a. Vertical Spacing,

Minimum Bolt Spacing,

$s = 3 \text{ in}$

$smin = 2\frac{2}{3} \cdot db$ $\qquad smin = 2 \text{ in}$

Maximum Bolt Spacing,

$smax = \min(12 \cdot in,\ 24 \cdot \min(t1,\ t2))$ $\qquad smax = 9 \text{ in}$

Specified Bolt Spacing is acceptable, OK

3. Check for Edge Distance

(AISC 14th Ed. Specifications, Chapter J, Section J3.4 and J3.5, pages 16.1-122 to 16.1-124)

Shear Plate Edge Distances,

$Lev1 = 1.5 \text{ in}$

$Leh1 = 1.5 \text{ in}$

Beam Web Edge Distances,

$Lev2 = NA$

$Leh2 = NA$

i) <u>Minimum Vertical Edge Distance,</u>

Connection Edge Distance,

$Levcon = \begin{bmatrix} Lev1 \end{bmatrix}$ $\qquad Levcon = \begin{bmatrix} 1.5 \end{bmatrix} \text{ in}$

Minimum Edge Distance,

$Levmin = \begin{bmatrix} Levmin1 \end{bmatrix}$ $\qquad Levmin = \begin{bmatrix} 1 \end{bmatrix} \text{ in}$

Specified Edge Distance is Acceptable, OK

ii) <u>Minimum Horizontal Edge Distance,</u>

Connection Edge Distance,

$Lehcon = \begin{bmatrix} Leh1 \\ Leh2 \end{bmatrix}$ $\qquad Lehcon = \begin{bmatrix} 1.5 \\ 1.75 \end{bmatrix} \text{ in}$

Minimum Edge Distance,

$Lehmin = \begin{bmatrix} Lehmin1 \\ Lehmin2 \end{bmatrix}$ $\qquad Lehmin = \begin{bmatrix} 1.125 \\ 1 \end{bmatrix} \text{ in}$

Specified Edge Distance is Acceptable, OK

iii) Maximum Edge Distance,

Shear Plate Thickness,

$t1 = 0.375\ in$

Beam Web Thickness,

$t2 = 0.515\ in$

Nearest Connection Edge Distance,

$Lemin = \min(Lehcon, Levcon)$

$Lemin = 1.5\ in$

Maximum Edge Distance,

$Lemin = Levcon_0 \lor Lemin = Lehcon_0$

$Lemax = \min(6in, 12 \cdot t1)$

$Lemax = 4.5\ in$

Maximum Edge Distance Requirement is Satisfied, OK

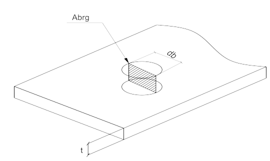
I.C. BEAM SHEAR PLATE CHECK

1. Bolt Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J3.10, pages 16.1-127 to 16.1-128)

a. Bolt Capacity due to Resultant Load

Bearing Area,



$Abrg = db \cdot t$ $\qquad$ $Abrg = 0.281\ in^2$

Bolt Centerline Distance from Face of Support,

$ab = gap + Leh + 0.5 \cdot (nv - 1) \cdot sv$

$ab = 2.25\ in$

Eccentricity Distannce of End Reaction from Bolt Group Centerline,

$ebv = ab$ $\qquad$ $ebv = 2.25\ in$

Bolt Vertical Centerline Distance from Beam Centerline,

$ah = |0.5 \cdot d - [D + 0.5 \cdot (nr - 1) \cdot s]|$ $\qquad$ $ah = 1.7\ in$

Eccentricity distance of Axial Load from Bolt Group Centerline,

$Yo = ah$ $\qquad$ $Yo = 1.7\ in$

Load Inclination from Vertical,

$$\theta = \operatorname{atan}\left(\frac{Hbm}{Vbm}\right) \qquad \theta = 0\ deg$$

Eccentric Load Coefficient,

(AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-8)

$C = 4.276$

Available Bearing Strength Using Edge Distance, (J3-6a, J3-6c)

hdv < hdls(db)

$$Fbe = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (Lev - 0.5 \cdot hdv) \cdot t \\ 1.2 \cdot (Leh - 0.5 \cdot hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

$Fbe = \min(Fbe)$ $Fbe = 18.963$ kips

Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)

hdv < hdls(db)

$$Fbs = \Lambda brg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (s - hdv) \cdot t \\ 1.2 \cdot (sv - hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$$

nv > 1

$Fbs = \min(Fbs_0, Fbs_2)$ $Fbs = 29.362$ kips

Number of Area in Consideration,

n1 = n

Bolt Capacity,

$Rbrg = C \cdot \min(n1 \cdot Fbe, n1 \cdot Fbs, n \cdot \Lambda rv)$

$Rbrg = 34.496$ kips $Rbm = 28.895$ kips

Bolt Capacity > Applied Force, UCV = 0.838, OK

2. Yielding Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Yielding Capacity due to Shear Load

Length,

$L = (nr - 1) \cdot s + 2 \cdot Lev$ $L = 15$ in

Number of Areas in Consideration,

n1 = n

Shear Yielding Capacity, (J4-3)

$Rvy = \Lambda vy \cdot n1 \cdot 0.6 \cdot Fy \cdot L \cdot t$

$Rvy = 121.5$ kips $Vbm = 28.895$ kips

Shear Yielding Capacity > Applied Force, UCV = 0.238, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, page 16.1-128)

b. Tensile Yielding Capacity due to Axial Load

Length,

$L = (nr - 1) \cdot s + 2 \cdot Lev$ $L = 15$ in

Gross Tension Area,

$Ag = L \cdot t$

Number of Areas in Consideration,

$n1 = n$

Tensile Yielding Capacity, (J4-1)

$Rty = \Lambda ty \cdot n1 \cdot Fy \cdot Ag$

$Rty = 182.25$ kips $Hbm = 0$ kips

Tensile Yielding Capacity > Applied Force, UCV = 0, OK

Interaction of Yielding Capacities,

$$\left(\frac{Vbm}{Rvy}\right)^2 + \left(\frac{Hbm}{Rty}\right)^2 \leq 1.0$$

$$UCV = \left(\frac{Vbm}{Rvy}\right)^2 + \left(\frac{Hbm}{Rty}\right)^2$$ $UCV = 0.057$

Yielding Capacity > Applied Force, UCV = 0.057, OK

3. Rupture Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)

a. Shear Rupture Capacity due to Shear Load

Net Shear Area,

$Anv = (L - nr \cdot hdv) \cdot t$

$Anv = 3.984\ in^2$

Number of Areas in Consideration,

$n1 = n$

Shear Rupture Capacity, (J4-4)

$Rvr = \Lambda vr \cdot n1 \cdot 0.6 \cdot Fu \cdot Anv$

$Rvr = 103.992$ kips $Vbm = 28.895$ kips

Shear Rupture Capacity > Applied Force, UCV = 0.278, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.1, pages 16.1-128 to 16.1-129)

b. Tensile Rupture Capacity due to Axial Load

Effective Net Area,

$Ae = \min[(L - nr \cdot hdv) \cdot t,\ 0.85 \cdot L \cdot t]$ $Ae = 3.984\ in^2$

Number of Areas in Consideration,

$n1 = n$

Tensile Rupture Capacity, (J4-2)

$Rtr = \Lambda tr \cdot n1 \cdot Fu \cdot Ae$

$Rtr = 173.32$ kips $Hbm = 0$ kips

Tensile Rupture Capacity > Applied Force, UCV = 0, OK

Interaction of Rupture Capacities,

$$\left(\frac{Vbm}{Rvr}\right)^2 + \left(\frac{Hbm}{Rtr}\right)^2 \le 1.0$$

$$UCV = \left(\frac{Vbm}{Rvr}\right)^2 + \left(\frac{Hbm}{Rtr}\right)^2 \qquad UCV = 0.006$$

Rupture Capacity > Applied Force, UCV = 0.006, OK

4. Block Shear Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

a. Block Shear Capacity due to Shear Load

Reduction Factor,

nv = 1 *(tension stress is uniform)*

Ubs = 1.0

Gross Shear Area,

$Agv = [(nr - 1) \cdot s + Lev] \cdot t$ $\qquad Agv = 5.063$ in²

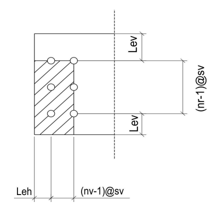
Net Tension Area,

$Ant = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot t$

$Ant = 0.363$ in²

Net Shear Area,

$Anv = Agv - [(nr - 0.5) \cdot hdv] \cdot t$ $\qquad Anv = 3.586$ in²



Number of Areas in Consideration,

n1 = n

Block Shear Capacity, (J4-5)

$Rbss = \Lambda bs \cdot n1 \cdot min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant, 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbss = 97.815$ kips $\qquad Vbm = 28.895$ kips

Block Shear Capacity > Applied Force, UCV = 0.295, OK

(AISC 14th Ed. Specifications, Chapter J, Section J4.3, page 16.1-129)

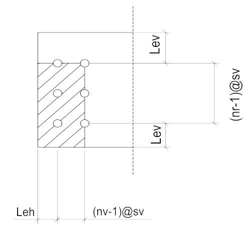
b. Block Shear Capacity due to Axial Load

Pattern 1

Reduction Factor,

$ah \neq 0$ in   *(tension stress is non-uniform)*

$Ubs = 0.5$



Gross Shear Area,

$Agv = [Leh + (nv - 1) \cdot sv] \cdot t$ $Agv = 0.563\ in^2$

Net Tension Area,

$Ant = [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hdv] \cdot t$

$Ant = 3.586\ in^2$

Net Shear Area,

$Anv = Agv - [(nv - 0.5) \cdot hdh] \cdot t$ $Anv = 0.363\ in^2$

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs1 = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbs1 = 87.107$ kips $Hbm = 0$ kips

Block Shear Capacity > Applied Force, UCV = 0, OK

Interaction of Bolt Shear Capacities,

$$\left(\frac{Vbm}{Rbss}\right)^2 + \left(\frac{Hbm}{Rbsa}\right)^2 \leq 1.0$$

$$UCV = \left(\frac{Vbm}{Rbss}\right)^2 + \left(\frac{Hbm}{Rbsa}\right)^2 \qquad UCV = 0.087$$

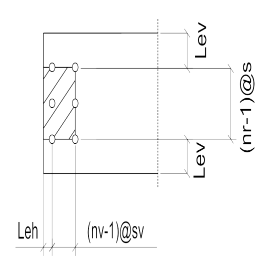
Block Shear Capacity > Applied Force, UCV = 0.087, OK

Pattern 2

Reduction Factor,

$ah \neq 0$ in   *(tension stress is non-uniform)*

$Ubs = 0.5$



Gross Shear Area,

$Agv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot t$ $Agv = 1.125\ in^2$

Net Tension Area,

$Ant = [(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t$

$Ant = 3.187\ in^2$

Net Shear Area,

$Anv = Agv - 2 \cdot [(nv - 0.5) \cdot hdh] \cdot t$ $Anv = 0.727\ in^2$

Number of Areas in Consideration,

$n1 = n$

Block Shear Capacity, (J4-5)

$Rbs2 = \Lambda bs \cdot n1 \cdot \min(0.6 \cdot Fu \cdot Anv + Ubs \cdot Fu \cdot Ant,\ 0.6 \cdot Fy \cdot Agv + Ubs \cdot Fu \cdot Ant)$

$Rbs2 = 87.553$ kips $\qquad Hbm = 0$ kips

Block Shear Capacity > Applied Force, UCV = 0, OK

5. Local Buckling Capacity

(AISC 14th Ed. Manual, Part 9, page 9-9)

Distance of Bolt Line to Support,

$ab = gap + Leh \qquad ab = 2.25$ in

Coefficient,

$$\lambda = \frac{L \cdot Fy^{0.5}}{10 \cdot t \cdot \left[475 + 280 \cdot \left(\frac{L}{ab}\right)^2\right]^{0.5}} \cdot \frac{1}{ksi^{0.5}} \qquad \lambda = 0.211$$

$\lambda \le 0.7$

$Q = 1$

Allowable Flexural Local Buckling Stress or Yielding Stress,

$Fcr = Q \cdot Fy \qquad Fcr = 36$ ksi

Gross Plastic Section Modulus,

$$Zx = \frac{t \cdot L^2}{4} \qquad Zx = 21.094\ in^3$$

Eccentricity,

$e = ab \qquad e = 2.25$ in

Local Buckling Capacity,

$$Rbc = \Lambda b \cdot \frac{Fcr \cdot Zx}{e}$$

$Rbc = 303.75$ kips $\qquad Vbm = 28.895$ kips

Local Buckling Capacity > Applied Force, UCV = 0.095, OK

6. Shear and Flexural Yielding Capacity with Von-Mises Yield Criterion

(AISC 14th Ed., Manual Part 10, page 10-104)

(Muir, Larry and Hewitt,Christopher, "Design of Unstiffened Extended Single Plate Shear Connections", Engineering Journal, 2nd Quarter 2009, page 69)

Shear and Flexural Yielding Interaction Capacity,

$$Rfc = \frac{\Lambda b \cdot Fy \cdot L \cdot t}{\left[2.25 + 16 \cdot \left(\frac{e}{L}\right)^2\right]^{0.5}}$$

$Rfc = 112.81$ kips $\qquad Vbm = 28.895$ kips

Yielding Capacity > Applied Force, UCV = 0.256, OK

7. Flexural Rupture Capacity

(AISC 14th Ed. Manual Part 15, page 15-4)

(AISC 14th Ed. Steel Construction Manual Design Examples, page IIA-104)

Net Plastic Section Modulus,

mod(nr, 2) > 0

$$Znet = \left[\frac{t \cdot L^2}{4} - \frac{t \cdot hdv \cdot s \cdot (nr^2 - 1)}{4} - \frac{t \cdot (hdv)^2}{4}\right]$$

$Znet = 15.116\ in^3$

Flexural Rupture Capacity,

$$Rfr = \frac{\Lambda fr \cdot Fu \cdot Znet}{e}$$

$Rfr = 292.237\ kips$ $\quad$ $Vbm = 28.895\ kips$

Flexural Rupture Capacity > Applied Force, UCV = 0.099, OK

8. Compression Capacity

(AISC 14th Ed. Specifications, Chapter J, Section J4.4, page 16.1-129 to 16.1-130)

Effective Length Factor,

(Commentary on the Specification for Structural Steel Building Table C-A-7.1)

$K = 1.2$

Beam Web Horizontal Edge Distance,

$Le1 = 1.5\ in$

Laterally Unbraced Length,

$Lu = gap + Le1$ $\quad$ $Lu = 2.25\ in$

Gross Area,

$Ag = L \cdot t$ $\quad$ $Ag = 5.625\ in^2$

Radius of Gyration,

$$r = \frac{t}{(12)^{0.5}}$$ $r = 0.108\ in$

Slenderness Ratio,

$$KLr = \frac{K \cdot Lu}{r}$$ $KLr = 24.942$

Elastic Critical Buckling Stress,

$$Fe = \frac{\pi^2 \cdot E}{KLr^2}$$ $Fe = 460.099\ ksi$

Flexural Buckling Stress,

$KLr \leq 25$

$Fcr = Fy$ $\quad$ $Fcr = 36\ ksi$

Number of Areas in Consideration,

$n1 = n$

Compression Capacity,

$Rcb = \Lambda c \cdot n1 \cdot Fcr \cdot Ag$

$Rcb = 182.25$ kips $\qquad$ $Hbm = 0$ kips

Compression Capacity > Applied Force, UCV = 0, OK

I.D. BEAM SHEAR PLATE TO COLUMN WALL CHECK

1. Weld Capacity

(AISC 14th Ed. Specifications, Chapter J, pages 16.1-110 to 16.1-117)

(AISC 14th Ed. Manual, Part 8, pages 8-9 to 8-15)

a. Using Fillet Weld,

Number of Weld Sides,

$nws = 2$

Minimum Weld Size,

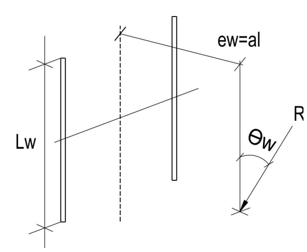
$wmin = 0.187$ in $\qquad$ $w = 0.25$ in

Preferred Weld Size > Minimum Weld Size, OK

Shear Strength,

For Column,

$Rv1 = \Lambda vr \cdot 0.6 \cdot Fu \cdot tw \cdot nws$ $\qquad$ $Rv1 = 30.328$ kips/in



For Shear Plate,

$Rv2 = \Lambda vr \cdot 0.6 \cdot Fu \cdot t$ $\qquad$ $Rv2 = 9.787$ kips/in

For Weld,

$Rv3 = \Lambda vw \cdot 0.6 \cdot Fu \cdot \sin(45deg) \cdot nws$ $\qquad$ $Rv3 = 44.548$ ksi

Maximum Effective Weld Size,

$$weff = \frac{\min(Rv1,\ Rv2)}{Rv3}$$ $\qquad$ $weff = 0.22$ in

Length of Weld,

$Lw = (nr - 1) \cdot s + 2 \cdot Lev$ $\qquad$ $Lw = 15$ in

Eccentric Load Coefficient,

$$al = ew + \frac{Hbm \cdot ah}{Vbm}$$ $\qquad$ $al = 2.25$ in

For Load Not in Plane of Weld Group,

$k = 0$

$$a = \frac{al}{Lw}$$ $\qquad$ $a = 0.15$

Load Inclination from Vertical,

$\theta = \operatorname{atan}\left(\frac{Hbm}{Vbm}\right)$ $\theta = 0$ deg

Electrode Strength Coefficient,

(AISC 14th Ed. Manual Part 8, Table 8-3, page 8-65)

$C1 = 1$ ksi

(AISC 14th Ed. Manual Part 8, Table 8-4)

$Co = 3.666$

Weld Capacity,

$Rew = \Lambda ew \cdot Co \cdot C1 \cdot 16 \cdot Lw \cdot \min(w, weff)$

$Rew = 144.964$ kips $Rbm = 28.895$ kips

Weld Capacity > Applied Force, UCV = 0.199, OK

J. COLUMN WALL CHECK

1. HSS Local Check

a. HSS Punching Shear

(AISC Specification for the Design of Steel Hollow Structural Sections, page 15)

Thickness of Shear Plate,

$t = 0.375$ in

Maximum Normal Stress in the Plate,

$Nmax = \frac{Hbm}{t \cdot L} + \frac{4 \cdot Hbm \cdot ah}{t \cdot L^2}$ $Nmax = 0$ ksi

Maximum Shear Plate Thickness to Avoid Shear Tab Punching Thru Column Wall,

$tPSmax = \frac{1.2 \cdot \Lambda vr \cdot Fu \cdot tw}{\Lambda ty \cdot Nmax}$

$tPSmax = 33698$ in $t = 0.375$ in

Plate thickness < Maximum Plate Thickness, OK

b. HSS Wall Plastification Capacity

(AISC 14th Ed. Specifications, Chapter K, Table K1.2, page 16.1-144)

Axial Load Required,

$PuHSS = Hbm$ $PuHSS = 0$ kips

For Required Axial Strength,

Code = LRFD

$PUR = \frac{P}{Ag \cdot Fy}$ $PUR = 0$

For Required Flexural Strength,

Code = LRFD

$$MUR = \frac{M}{S \cdot Fy} \qquad MUR = 0$$

For Uplift Force (if any),

Code = LRFD

$$ULUR = \frac{PUplift}{Ag \cdot Fy} \qquad ULUR = 0$$

Utilization Ratio,

(CIDECT Design Guide 3 Second Edition, Table 7.1, page 78)

$$n = \begin{bmatrix} -PUR - MUR + ULUR \\ -PUR + MUR + ULUR \\ PUR + MUR + ULUR \\ PUR - MUR + ULUR \end{bmatrix} \quad \begin{matrix} \text{(Axial in Compression, Moment in Compression)} \\ \text{(Axial in Compression, Moment in Tension)} \\ \text{(Axial in Tension, Moment in Tension)} \\ \text{(Axial in Tension, Moment in Compression)} \end{matrix}$$

Coefficient of Chord Stress Functions,

if $n < 0$ then $Cs = 0.20$

if $n \ge 0$ then $Cs = 0.10$

Chord-stress Interaction Parameter,

for $i \in 0..3$

if $n_i < 0$ then $Cs_i = 0.20$

if $n_i \ge 0$ then $Cs_i = 0.10$

for $i \in 0..3$

$x_i = (1 - |n_i|)^{Cs_i}$

$Qf = \min(x_i)$ $\qquad Qf = 1$

Branch Angle from the HSS Chord Face,

$\theta = 90$ deg

Nominal HSS Wall Plastification Capacity,

$$Rn = \frac{\dfrac{Fy \cdot tw^2}{1 - \dfrac{t}{BHSS}} \cdot \left[\dfrac{2L}{BHSS} + 4 \left(1 - \dfrac{t}{BHSS} \right)^{0.5} \cdot Qf \right]}{\sin(\theta)}$$

HSS Wall Plastification Capacity,

$RpHSS = \Lambda pHSS \cdot Rn$

$RpHSS = 146.963$ kips $\qquad HcB = 0$ kips

Wall Plastification Capacity > Applied Force, UCV = 0, OK

c. HSS Flexural Plastification Capacity

(AISC 14th Ed. Specifications Chapter K, Table K3.2, page 16.1-157)

Moment Load Required,

$MuHSS = Hbm \cdot ah$ $\quad$ $MuHSS = 0 \ kips \cdot ft$

Width Ratio,

$\beta = \frac{t}{BHSS}$ $\quad$ $\beta = 0.063$

Load Length Parameter,

$\eta = \frac{L}{BHSS}$ $\quad$ $\eta = 2.5$

Coefficient of Chord Stress Functions,

(CIDECT Design Guide 3 Second Edition, Table 4.1, page 37)

if $n < 0$ then $Cs = 0.60 - 0.5 \cdot \beta$

if $n \geq 0$ then $Cs = 0.10$

Chord-stress Interaction Parameter,

for $i \in 0..3$

if $n_i < 0$ then $Cs_i = 0.60 - 0.5 \cdot \beta$

if $n_i \geq 0$ then $Cs_i = 0.10$

for $i \in 0..3$

$x_i = (1 - |n_i|)^{Cs_i}$

$Qf = \min(x_i)$ $\quad$ $Qf = 1$

Nominal HSS Flexural Plasification Capacity,

$$Mn = Fy \cdot tw^2 \cdot L \cdot \left[\frac{1}{2 \cdot \eta} + \frac{2}{(1 - \beta)^{0.5}} + \frac{\eta}{1 - \beta}\right] \cdot Qf$$

HSS Flexural Plastification Capacity,

$MpHSS = \Lambda pHSS \cdot Mn$ $\quad$ $MpHSS = 95.734 \ kips \cdot ft$

Interaction of Capacities, (K3-13)

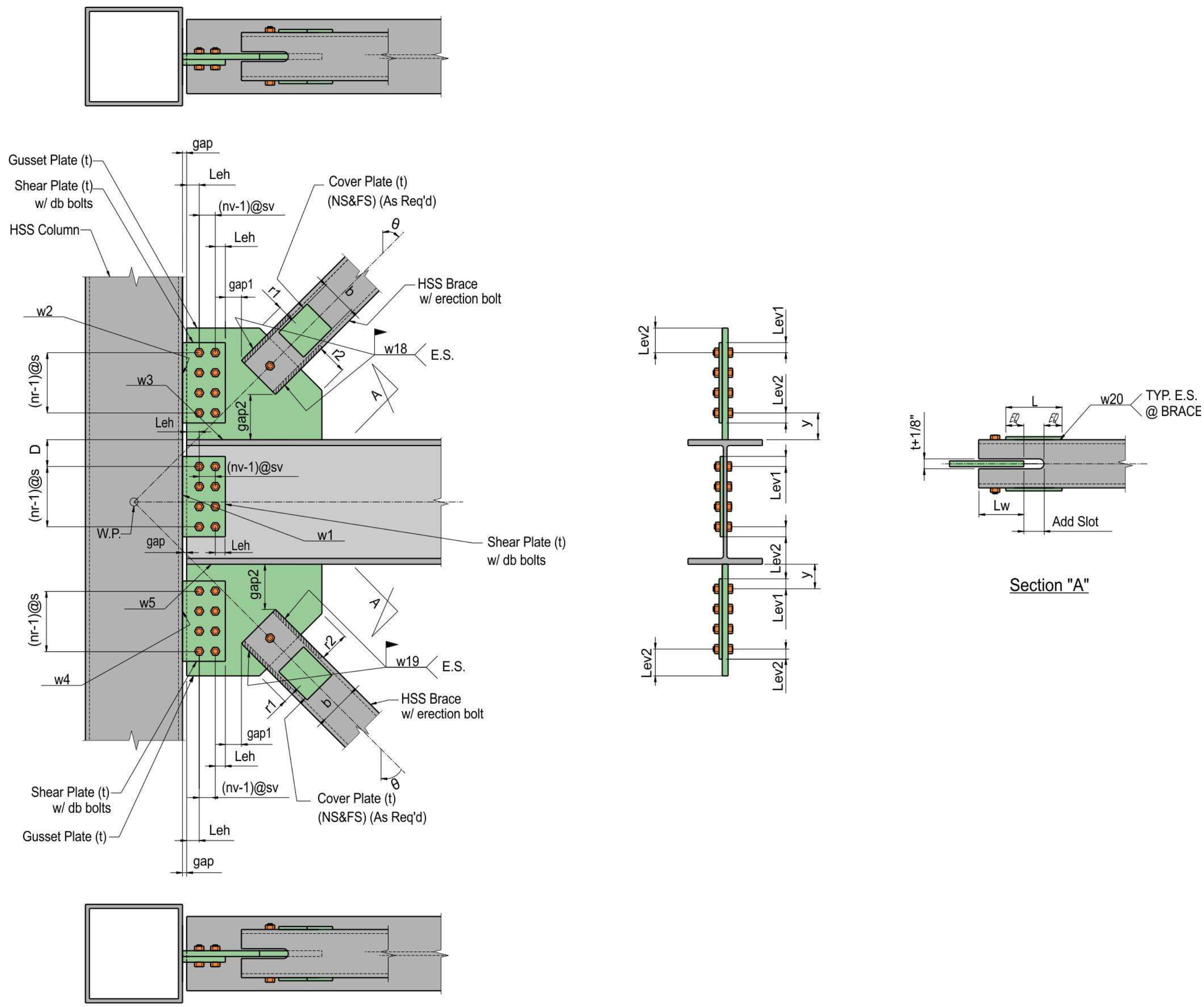
$$\frac{PuHSS}{RpHSS} + \frac{MuHSS}{MpHSS} \leq 1.0$$

$UCV = \frac{PuHSS}{RpHSS} + \frac{MuHSS}{MpHSS}$ $\quad$ $UCV = 0$

Wall Plastification Capacity > Applied Force, UCV = 0, OK

III. DETAILS

A. SKETCH



Note: Figure above does not represent actual design. Refer to connection schedule.

VERTICAL BRACE CONNECTION: HSS K-BRACE (DIRECTLY WELDED TO GUSSET PLATE) WITH SHEAR PLATE TWO-WAY GUSSET PLATE CONNECTION TO W BEAM AND RECTANGULAR HSS COLUMN

B. CONNECTION SCHEDULE

Column		
Mark	**Size**	**Grade**
	HSS14X6X5/8	A500-B

Bolts 1 at Beam Web						
db	**Bolt Type**	**Remarks**	**nr**	**s**	**nv**	**sv**
3/4"	A325-SC-SSLP-CLASS A	Short-Slotted Holes in Shear Plate Only	5	3"	1	0"

Beam						Web	
Mark	**Size**	**Grade**	**gap**	**θsl**	**θsk**	**D**	**Leh**
	W21X83	A992	1/2"	0°	0°	3"	1 3/4"

Beam Shear Plate				Weld
t	**Grade**	**Lev**	**Leh**	**w1**
3/8"	A36	1 1/2"	1 1/2"	1/4"

Beam Loads	
(Shear Load) V	**(Transfer Force) TF**
10 kips	0 kips

Brace 1					Weld	
Mark	**Size**	**Grade**	**θ (±2°)**	**Add Slot**	**w18**	**Lw**
	HSS6X5X3/8	A500-B	50°	2"	1/4"	6"

Brace 2					Weld	
Mark	**Size**	**Grade**	**θ (±2°)**	**Add Slot**	**w19**	**Lw**
	HSS6X5X3/8	A500-B	50°	2"	1/4"	6"

Cover Plate 1				Weld
t	**Grade**	**b**	**L**	**w20**
NR	NR	NR	NR	NR

Cover Plate 2				Weld
t	**Grade**	**b**	**L**	**w21**
NR	NR	NR	NR	NR

Gusset Plate 1				Weld	
t	**Grade**	**y**	**Lev2**	**w3**	**Lw**
3/8"	A36	3"	2 1/2"	1/4"	1'-4 5/16"

Gusset Plate 2				Weld	
t	**Grade**	**y**	**Lev2**	**w5**	**Lw**
3/8"	A36	3"	2 1/2"	1/4"	1'-4 5/16"

Bolts 1 at Gusset Plate						
db	**Bolt Type**	**Remarks**	**nr**	**s**	**nv**	**sv**
3/4"	A325-SC-SSLP-CLASS A	Short-Slotted Holes in Shear Plate Only	4	3"	1	0"

Bolts 2 at Gusset Plate						
db	**Bolt Type**	**Remarks**	**nr**	**s**	**nv**	**sv**
3/4"	A325-SC-SSLP-CLASS A	Short-Slotted Holes in Shear Plate Only	4	3"	1	0"

Gusset Shear Plate				Weld
t	**Grade**	**Lev**	**Leh**	**w2**
3/8"	A36	1 1/2"	1 1/2"	1/4"

Gusset Shear Plate				Weld
t	**Grade**	**Lev**	**Leh**	**w4**
3/8"	A36	1 1/2"	1 1/2"	1/4"

Brace 1 Loads		
(Tension Load) Pt	**(Compression Load) Pc**	**(Maximum Axial Load) P**
25 kips	25 kips	25 kips

Brace 2 Loads		
(Tension Load) Pt	**(Compression Load) Pc**	**(Maximum Axial Load) P**
25 kips	25 kips	25 kips

Width of Whitmore Section Outside Gusset Plate 1
0"

Width of Whitmore Section Outside Gusset Plate 2
0"

Through Plate Requirement		
Gusset Shear Plate 1	**Beam Shear Plate**	**Gusset Shear Plate 2**
NOT REQUIRED	NOT REQUIRED	NOT REQUIRED

Column Loads		
Axial(P)	**Moment(M)**	**Uplift Force (PUplift)**
0 kips	0 kips·ft	0 kips

IV. REFERENCES

Steel Construction Manual (14th Ed.) - LRFD American Institute of Steel Construction, Inc. 2011