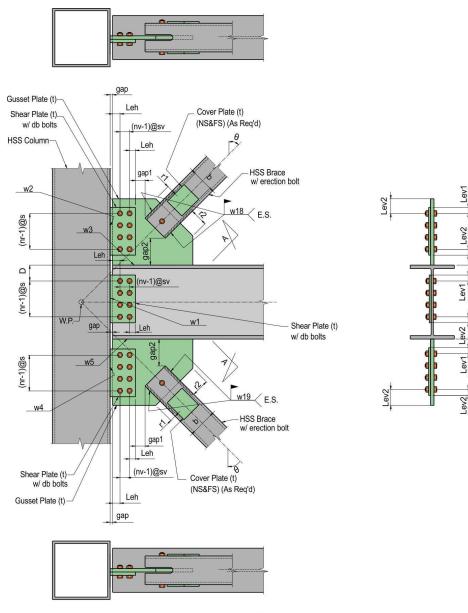
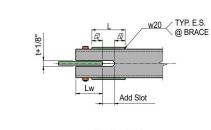


GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com www.gizasteel.com

	Job Code:	YYYY		
	Job Name:	NASCC 2019		
6	Sheet No.:	1 of 78		
	Designed by:	RCM		
	Revision No:	00	Date:	03/28/2019
	Subject:	V1H-C1DD		





>

>

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



Job Code:	YYYY				
Job Name:	NASCC 2019				
Sheet No.:	2 of 78				
Designed by:	RCM				
Revision No:	00	Date:	03/28/2019		
Subject:	V1H-C1DD				

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,	$Ag = 21 in^2$		
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,	$Ag = 24.4 in^2$	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,	$\theta sl = 0 deg$	Skew,	θ sk = 0 deg
		Inclination to the Column,	0in = 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 : H	SS6X5X3/8 - 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

	ZA Steel	Job Code: Job Name:	YYYY NASCC 2019	
A A A A A A A A A A A A A A A A A A A	uite 220 St. Louis, MO 63146	Sheet No.:	3 of 78	
	fo@gizasteel.com	Designed by:	RCM	
GIZA [™] ^{www.g}	izasteel.com	Revision No:	00	Date: 03/28/2019
GIZA		Subject:	V1H-C1DD	
BRACE 2 PROPERTIES : H	SS6X5X3/8 - A500-B			
Height,	H = 6 in	Design Wa Thickness		tw = 0.349 in
Width,	B = 5 in	Wall Thic	ckness,	tnom = 0.375 in
Area,	$Ag = 6.88 in^2$			
Minimum Yield Stress,	Fy = 46 ksi	Minimum 1 Stress,	ensile	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi			
Unbraced Length,	Lu = 16 ft + 3.375 in			
Angle from Vertical Member,	θ = 50 deg	Additiona	al Slot,	Add Slot = 2 in
GUSSET PLATE 1 PROPERT	IES : A36			
Thickness,	t = 0.375 in	Number of	Plates,	n = 1
Minimum Yield Stress,	Fy = 36 ksi	Minimum 1 Stress,	ensile	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi			
Clip,	c = 0 in			
GUSSET PLATE 2 PROPERT	IES : A36			
Thickness,	t = 0.375 in	Number of	Plates,	n = 1
Minimum Yield Stress,	Fy = 36 ksi	Minimum 1 Stress,	ensile	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi			
Clip,	c = 0 in			
GUSSET SHEAR PLATE 1 P	ROPERTIES : A36			
Thickness,	t = 0.375 in	Number of	Plates,	n = 1
Minimum Yield Stress,	Fy = 36 ksi	Minimum 1 Stress,	ensile	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi			
Clip,	c = 0 in			
GUSSET SHEAR PLATE 2 P	ROPERTIES : A36			
Thickness,	t = 0.375 in	Number of	Plates,	n = 1
Minimum Yield Stress,	Fy = 36 ksi	Minimum 1 Stress,	ensile	Fu = 58 ksi
Modulus of Elasticity,	E = 29000 ksi			
Clip,	c = 0 in			

		1			
		Job Code:	YYYY		
A A A A A A A A A A A A A A A A A A A	A Steel	Job Name:	NASCC 2019		
	ite 220 St. Louis, MO 63146	Sheet No.:	4 of 78		
	@gizasteel.com	Designed by:	RCM		
GIZA [™] ^{www.giz}	zasteel.com	Revision No:	00	Date: 03/28/2019	
OTZA		Subject:	V1H-C1DD		
BEAM SHEAR PLATE PROPER	TIES : A36				
Thickness,	t = 0.375 in	Number of	Plates,	n = 1	
Minimum Yield Stress,	Fy = 36 ksi	Minimum T Stress,	ensile	Fu = 58 ksi	
Modulus of Elasticity,	E = 29000 ksi				
Clip,	c = 0 in				
BOLTS PROPERTIES : 3/4"	- ø - A325-SC-SSLP-C	LASS A			
For Gusset Shear Plac	te 1 to Gusset Plate 1	Connectio	n:		
Bolt Diameter,	db = 0.75 in				
Bolt Shear Strength,	Arv = 8.068 kips	Bolt Tens Strength,		Arn = 29.821 kips	
Bolt Type,	Bolt_Type = A325-SC- SSLP-CLASS A	Connectio	n Type,	Conn_type = Slip Critical Type	
Number of Bolt Rows,	nr = 4	Bolt Vert Spacing,	ical	s = 3 in	
Number of Bolt Column Lines,	nv = 1	Bolt Hori Spacing,	zontal	sv = 0 in	
Total Number of Bolts (nr·nv),	nb = 4				
Holes at Gusset Plate	2,	Holes at	Gusset Shear	r Plate,	
Vertical Hole Dimension,	hdv = 0.875 in	Vertical Dimension		hdv = 0.875 in	
Horizontal Hole Dimension,	hdh = 0.875 in	Horizonta Dimension		hdh = 1.063 in	
Vertical Edge Distance (Lev2),	Lev = 2.5 in	Vertical Distance Lev2),	-	Lev = 1.5 in	
Horizontal Edge Distance,	Leh = 1.75 in	Horizonta Distance,	-	Leh = 1.5 in	
BOLTS PROPERTIES : 3/4"	- ø - A325-SC-SSLP-C	LASS A			
For Gusset Shear Play	te 2 to Gusset Plate 2	? Connectio	n:		
Bolt Diameter,	db = 0.75 in				
Bolt Shear Strength,	Arv = 8.068 kips	Bolt Tens Strength,		Λrn = 29.821 kips	
Bolt Type,	Bolt_Type = A325-SC- SSLP-CLASS A	Connectio		Conn_type = Slip Critical Type	
Number of Bolt Rows,	nr = 4	Bolt Vert Spacing,	ical	s = 3 in	
Number of Bolt Column Lines,	nv = 1	Bolt Hori Spacing,	zontal	sv = 0 in	
Total Number of Bolts (nr·nv),	nb = 4				

•		Job Code: YYYY	
G	IZA Steel	Job Name: NASCC 2019	
1801 Park 270 Drive Suite 220 St. Louis, MO 63146		Sheet No.: 5 of 78	
	nfo@gizasteel.com	Designed by: RCM	
GIZA [™] ^{₩₩₩.}	gizasteel.com	Revision No: 00	Date: 03/28/2019
OTZA		Subject: V1H-C1DD	
Holes at Gusset Pla	ite,	Holes at Gusset She	ar 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-C	CLASS A	
For Beam Shear Plat	e to Beam Web Connection	on:	
Bolt Diameter,	db = 0.75 in		
Bolt Shear Strength	a, Arv = 8.068 kips	Bolt Tensile Strength,	Arn = 29.821 kips
Bolt Type,	Bolt_Type = A325-SC- SSLP-CLASS A	Connection Type,	Conn_type = Slip Critical Type
Number of Bolt Rows	s, nr = 5	Bolt Vertical	s = 3 in

Spacing, Number of Bolt nv = 1 Bolt Horizontal sv = 0 in Column Lines, Spacing, Total Number of nb = 5 Bolts (nr · nv), Holes at Beam Web, Holes at Beam Shear Plate, Vertical Hole hdv = 0.875 inVertical Hole hdv = 0.875 inDimension, Dimension, Horizontal Hole hdh = 0.875 inHorizontal Hole hdh = 1.063 inDimension, Dimension, Bolt First Down from D = 3 in Top of Beam, Lev = 1.5 in Vertical Edge Lev = 3 in Vertical Edge Distance min(Lev1, Distance (D - dcT), Lev2), Horizontal Edge Leh = 1.75 inHorizontal Edge Leh = 1.5 in Distance, Distance,

WELDS PROPERTIES : E70xx LH

Minimum Tensile Stress,

Fu = 70 ksi

For Brace 1 to Gusset Plate 1 Connection:

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

For Brace 2 to Gusset Plate 2 Connection:

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

GIZA Steel1801 Park 270 Drive Suite 220 SL Louis, MC 63146 Louis, MC 63146 Louis, MC 63146 Louis, MC 63146 Louis, MC 63146 Louis, MC 63146Dec Colspan="2">Very Louis, MC 63146 Louis, MC 63146 Louis, MC 63146State 1 Column Mall Connection: Prefereed Weld Size (w2), R = 0.25 inFor Gusset Shear Plate 1 to Column Mall Connection: Prefereed Weld Size (w2), R = 0.25 inFor Gusset Plate 1 to Deam Flange Connection: Prefereed Weld Size w = 0.25 inFor Gusset Plate 2 to Column Mall Connection: Prefereed Weld Size w = 0.25 inFor Gusset Plate 2 to Deam Flange Connection: Prefereed Weld Size w = 0.25 inFor Gusset Plate 2 to Deam Flange Connection: Prefereed Weld Size w = 0.25 inGate of Gusset Plate 2 to Deam Flange Connection: Prefereed Weld Size w = 0.25 inGate of Gusset Plate 2 to Deam Flange Connection: Prefereed Weld Size w = 0.25 inGate of Gusset Plate 2 to Deam Flange Connection: Prefereed Weld Size (w1), Safety Factor, Q(LSD)Resistance Factor, $\phi(LSTD)$ Modification Factor, $\lambda = \frac{1}{0}$ (if ASD)A = ϕ (if LRFD) Safety FactorConstant Shear (bs), Gba = 2.00 Borg = 0.75Abs = 0.75Abs = 0.75Abs = 0.75Are = 0.90 Borg = 0.75Are = 0.80 Borg for flow of $\phi = 0.75$ Are = 0.20 Borg for flow of $\phi = 0.75$ Are = 0.20 Borg for flow of $\phi = 1.67$ Borg for flow of $\phi = 0$	Γ				
1801 Park 270 Dive Sufit 220 SL Louis, MO 6314 Email: info@gizastel.com www.gizastel.com preferred.weid.gize www.gizastel.com Preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com www.gizastel.com preferred.weid.gize www.gizastel.com www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize www.gizastel.com preferred.weid.gize pre		10 11 - MAR (1997) - 1927			
Email: Info@gizastel.comhard typeWWW.gizastel.comAutomation to a definition to a defini	A A A A A A A A A A A A A A A A A A A				
Very Num gizzabeel.comPeriation No. 10Date:: 0.7287203Periation No. 10Periation No. 10Peria					
Notice: VinciteColspan="2">Notice: VinciteFor Gusset Shear Plate 1 to Column Wall Connection:Freferred Weld Size (W2), w = 0.25 inFor Gusset Shear Plate 2 to Column Wall Connection:Preferred Weld Size (W4), w = 0.25 inLength of Weld, Iw = 16.312 inPreferred Weld Size w = 0.25 inLength of Weld, Iw = 16.312 in(W3),For Gusset Plate 1 to Beam Flange Connection:Preferred Weld Size w = 0.25 inLength of Weld, Iw = 16.312 in(W3),For Gusset Plate to Column Wall Connection:Preferred Weld Size (w1), w = 0.25 inSafety Factor, B(ASD)Resistance Factor, $\phi(LRFD)$ Modification Factor;Safety factorresistance factor modification factorFor Member in Gubrg = 2.00Abrg = 0.75Abrg =		00			Data: 03/28/2019
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 $w = 0.25$ in Length of Weld, Lw = 16.312 in (W3), For Gusset Plate 2 to Beam Flange Connection: Preferred Weld Size $w = 0.25$ in Length of Weld, Lw = 16.312 in (W5), For Gusset Plate 2 to Beam Flange Connection: Preferred Weld Size $w = 0.25$ in Length of Weld, Lw = 16.312 in (W5), For Beam Shear Plate to Column Wall Connection: Preferred Weld Size (w1), $w = 0.25$ in Safety Factor, $\Omega(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $h = \frac{1}{\alpha} (1f ASD)$ For Shear (ba), (bs = 2.00 ϕ br = 0.75 Abr = 0.75 For Compression (c), Uc = 1.67 ϕ c = 0.90 Ac = 0.90 For Filtel Weld Drw = 2.00 ϕ ww = 0.75 Avr = 0.75 For Compression (c), Uc = 1.67 ϕ c = 0.90 Ac = 0.90 For Filtel Weld Drw = 2.00 ϕ ww = 0.75 Avr = 0.75 For Compression (b), For Flexural Local Cab = 1.67 ϕ b = 0.90 Ab = 0.90 For Filtel Weld Drw = 2.00 ϕ ww = 0.75 Avr = 0.75 For Compression Cab = 1.67 ϕ b = 0.90 Ab = 0.90 For Filter Wall Carve = 2.00 ϕ ww = 0.75 Arr = 0.75 For Flexural Local Cab = 1.67 ϕ b = 0.90 Ab = 0.90 For Flexural Local Cab = 1.67 ϕ b = 0.90 Ab = 0.90 For Flexural Local Cab = 1.67 ϕ b = 0.90 Ab = 0.90 For Flexural Local Cab = 1.67 ϕ b = 0.75 Afr = 0.75 (fr), For Flexural Local Cab = 1.67 ϕ b = 0.90 Ab = 0.90 For Flexural Local Cab = 1.67 ϕ b = 0.90 Ab = 0.90 For Flexural Local Cab = 1.68 ϕ ww = 0.75 Afr = 0.75 (fr), For Flexural Local Cab = 1.69 ϕ ww = 0.75 Afr = 0.75 (fr), For Shear (wp), For Shear Rupture Qr = 2.00 ϕ r = 0.75 Afr = 0.75 (fr), For Shear Rupture Qr = 2.00 ϕ r = 0.75 Afr = 0.75 (fr), For Shear Rupture Qr = 2.00 ϕ r = 0.75 Afr = 0.75 (fr), For Shear Rupture Qr = 2.00 ϕ r = 0.75 Arr = 0.75 (fr), For Shear Rupture Qr = 2.00 ϕ r = 0.75 Arr = 0.75 (fr), For Shear Rupture Qr = 2.00 ϕ r = 0.75 Arr = 0.7	GIZA [™] ^{₩₩₩.gl2}	asteer.com			Date: 03/20/2019
Freferred Weld Size (w2), w = 0.25 in For Gusset Shear Plate 2 to Column Hall Connection: Preferred Weld Size (w4), w = 0.25 in For Gusset Plate 1 to Beam Flange Connection: Preferred Weld Size w = 0.25 in Length of Weld, Lw = 16.312 in (w3), For Gusset Plate 2 to Beam Flange Connection: Preferred Weld Size w = 0.25 in Length of Weld, Lw = 16.312 in (w5), For Gusset Plate 2 to Beam Flange Connection: Preferred Weld Size w = 0.25 in Length of Weld, Lw = 16.312 in (w5), For Ream Shear Plate to Column Wall Connection: Preferred Weld Size (w1), w = 0.25 in SAFEY AND RESISTANCE FACTORS: Safety Factor, $Q(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $\lambda = \frac{1}{\Omega} (if ASD)$ $\lambda = \phi (if LRFD)$ Safety factor resistance factor modification factor For Member in Darg = 2.00 ϕ brg = 0.75 h brg = 0.75 Bearing / Bolt Bearing / Bolt Bearing / Bolt Bearing / Bolt Bearing / Bolt Bearing (brg), For Block Shear (bs), Gbs = 2.00 ϕ ws = 0.75 h vs = 0.73 For Compression (c), Uc = 1.67 ϕ c = 0.90 h c = 0.90 Por Fillet Weld Ω vw = 2.00 ϕ vw = 0.75 h vs = 0.75 Shear (vw), For Flexural Local Ω b = 1.67 ϕ b = 0.90 h c = 0.90 Por Fillet Weld Ω vw = 2.00 ϕ vw = 0.75 h vs = 0.75 For Compression (c), Uc = 1.67 ϕ b = 0.90 h c = 0.90 For Filexural Repture Ω fr = 2.00 ϕ rr = 0.75 h vr = 0.75 For Flexural Repture Ω fr = 2.00 ϕ rr = 0.75 h vr = 0.75 For Flexural Repture Ω vr = 2.00 ϕ vwg = 0.75 h vr = 0.75 For Partial Fen Weld Ω vwp = 2.00 ϕ vwg = 0.75 h vr = 0.75 For Shear Repture Ω vr = 2.00 ϕ vr = 0.75 h vr = 0.75 For Shear Repture Ω vr = 2.00 ϕ vr = 0.75 h vr = 0.75 For Shear Repture Ω vr = 2.00 ϕ vr = 0.75 h vr = 0.75 For Shear Repture Ω vr = 2.00 ϕ vr = 0.75 h vr = 0.75 For Shear Repture Ω vr = 2.00 ϕ vr = 0.75 h vr = 0.75 For Shear Repture Ω vr = 2.00 ϕ vr = 0.75 h vr = 0.75 For Shear Repture Ω vr = 2.00 ϕ vr = 0.75 h vr = 0.75 For Shear Repture Ω vr = 2.00 ϕ vr = 0.75 h vr = 0.75 F			bubjeet.		
For Guesset Shear Plate 2 to Column Mall Connection:Preferred Meld Size (x4), $w = 0.25 \text{ in}$ For Guesset Plate 1 to Beam Flange Connection:Preferred Meld Size $w = 0.25 \text{ in}$ Length of Meld,Lw = 16.312 in(x3),For Guesset Plate 2 to Beam Flange Connection:Lw = 16.312 inPreferred Meld Size $w = 0.25 \text{ in}$ Length of Meld,Lw = 16.312 in(x5),For Beam Shear Plate to Column Mall Connection:Lw = 16.312 inFor Beam Shear Plate to Column Mall Connection:Preferred Meld Size (w1), $w = 0.25 \text{ in}$ Sharetor, Quesson Shear Old Size (w1), $w = 0.25 \text{ in}$ Sharetor, Quesson Shear Old Size (w1), $w = 0.25 \text{ in}$ Sharetor, Quesson Shear Old Size (w1), $w = 0.25 \text{ in}$ Sharetor, Quesson Guesson Shear Old Size (w1), $w = 0.25 \text{ in}$ Sharetor, Quesson Guesson Shear Old Size (w1), $w = 0.25 \text{ in}$ Sharetor, Quesson Guesson Shear Old Size (w1),No of (1f LRFD)Safety factor resistance factor modification factorFor Member in Durg 2.00 $\phi br = 0.75$ Abs = 0.75Nor Compression (c), Qc = 1.67 $\phi c = 0.90$ Ac = 0.90 $hc = 0.90$ For Fielward LocalGue 2.00 $\phi w = 0.75$ Are 0.75 Are 0.75 Are 0.90 Are 0.90 Are 0.90 For Fi	For Gusset Shear Plat	e 1 to Column Wall Co	onnection:		
Preferred Weld Size (w4), $w = 0.25 in$ For Quaset Plate 1 to Beam Flange Connection:Preferred Weld Size $w = 0.25 in$ Length of Meld, Lw = 16.312 in (w3),For Source Plate 2 to Beam Flange Connection:Preferred Weld Size $w = 0.25 in$ Length of Meld, Lw = 16.312 in (w5),For Beam Shear Plate to Column Mall Connection:Preferred Weld Size (w1), $w = 0.25 in$ Share Meld Size (w1), $w = 0.25 in$ Share TRATE To Column Mall Connection:Preferred Weld Size (w1), $w = 0.25 in$ Share TRATE To Column Mall Connection:Share Trate to Column Mall Connection:Share Trate to Column Mall Connection:Share Trate to Column Mall Connection:Matter State (w1), $w = 0.25 in$ Share Trate to Column Mall Connection:Matter State (w1), $w = 0.25 in$ Share Trate to Column Mall Connection:Matter State (w1), $w = 0.25 in$ Share Trate to Column Mall Connection:Share Trate to Column Mall Connection:Matter State (w1), $w = 0.25 in$ Share Trate to Column Mall Connection:Share Trate to Column Mall Connection:Share (w1, GASD)A e (if LRFO)Matter State (bs), Gbs = 2.00& bbs = 0.75& bbs = 0.75For Compression (c), Gc = 1.67& bc = 0.90& c = 0.90<	Preferred Weld Size (w2),	w = 0.25	in	
For Gausset Plate 1 to Beam Flange Connection:Preferred Weld Size $w = 0.25$ inLength of Weld, $Lw = 16.312$ in(w3),For Gusset Plate 2 to Beam Flange Connection:Preferred Weld Size $w = 0.25$ inLength of Weld, $Lw = 16.312$ in(w3),For Beam Shear Plate to Column Wall Connection:Preferred Weld Size $(w1)$, $w = 0.25$ inSAFETY AND RESISTANCE FACTORS:Safety Factor, $\Omega(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $h = \phi(1f LRFD)$ $h = \frac{1}{\alpha}(if ASD)$ $h = \phi(if LRFD)$ Safety factorresistance factorFor Member in Bearing/ Bolt Bearing (brg),Obrg = 2.00For Dick Shear (bs), Obs = 2.00 $\phi bs = 0.75$ Aber = 0.90 $hc = 0.90$ For Ellet Weld Strength (b), $\Omega vw = 2.00$ For Fillet Weld Strength (b), $\Omega vw = 2.00$ For Partial Local Strength (b), $D = 1.67$ For Partial Rupture Strength (b), $\Omega tw = 2.00$ For Partial Pen Weld Strength (b), $\Omega vw = 2.00$ For Partial Pen Weld Strength (b), $\Omega vw = 2.00$ For Partial Pen Weld Strength (b), $\Omega vw = 0.75$ For Shear (vmp), $\Omega vw = 2.00$ For Shear (vmp), $\Omega vw = 2.00$ For Shear (vmp), $\Omega vw = 0.75$ For Shear (vmp), $\Omega vw = 0.75$ For Shear (vmp), $\Omega vw = 1.88$ For Shear (vmp), $\Omega vw = 0.75$ For Shear (vmp), $\Omega vw = 1.00$ For Shear (vmp), $\Omega vw = 1.50$ For Shea	For Gusset Shear Plat	e 2 to Column Wall Co	onnection:		
Preferred Weld Sizew = 0.25 inLength of Weld,Lw = 16.312 in(w3),For Susce Plate 2 to Beam Flange Connection:Preferred Weld Sizew = 0.25 inLength of Weld,Lw = 16.312 in(w5),For Beam Shear Plate to Column Well Connection:Freferred Weld Size(w1),w = 0.25 inSAFETY AND RESISTANCE FACTORS:Safety Factor, $Q(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $h = \phi(If LRFD)$ $h = \frac{1}{\alpha}(If ASD)$ $h = \phi(If LRFD)$ Safety factorresistance factorFor Member in Bearing Bolt Bearing (brg),Gbr = 2.00For Elack Shear (bs), Shear (us),Gbs = 2.00For Fillet Weld Strength (br),Gw = 1.67For Compression (c), Sc = 1.67 $\phi c = 0.90$ For Fillet Weld Strength (b),Gbr = 2.00For Fillet Weld Strength (b),Gfr = 2.00For Fillet Weld Strength (b),Gfr = 2.00For Partial Rupture Strength (b),Gfr = 2.00For Partial Pen Weld Strength (b),Gfr = 0.75For Shear (wp), For Shear Rupture (wp),Gur = 2.00For Shear Rupture (v),Gur = 2.00For Shear Rupture (v),Gur = 2.00For Shear Rupture (v),Gur = 2.00For Shear Strength (b),Gur = 2.00For Shear Strength (b),Gur = 2.00For Shear Strength (b),Gur = 2.00For Shear Rupture (v),Gur = 2.00For Shear Strength (c),Gur = 2.00For Shear Yielding (v),Gur = 2.00<	Preferred Weld Size (w4),	w = 0.25	in	
(w3), For Gusset Plate 2 to Beam Flange Connection: Preferred Weld Size $w = 0.25$ in Length of Weld, Lw = 16.312 in (w5), For Beam Shear Plate to Column Wall Connection: Preferred Weld Size (w1), $w = 0.25$ in SAFETY AND RESISTANCE FACTORS: Safety Factor, $\Omega(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $\Lambda = \frac{1}{\Omega} (1f \ ASD)$ $\Lambda = \phi (1f \ LRFD)$ Safety factor resistance factor modification factor For Member in $\Omega brg = 2.00$ $\phi brg = 0.75$ $\Lambda brg = 0.75$ Bearing (brg), For Block Shear (bs), $\Omega bs = 2.00$ $\phi bs = 0.75$ $\Lambda brg = 0.75$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $\Lambda c = 0.90$ For Fillet Weld $\Omega vw = 2.00$ $\phi vw = 0.75$ $\Lambda vw = 0.75$ Shear (w), For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ Buckling/Flexural Strength (b), For Partial Pen Weld $\Omega vwp = 2.00$ $\phi vw = 0.75$ $\Lambda tr = 0.75$ (fr), For Partial Pen Weld $\Omega vwp = 2.00$ $\phi vw = 0.75$ $\Lambda vw = 0.75$ For Partial Pen Weld $\Omega vwp = 1.88$ $\phi twp = 0.80$ $\Lambda twp = 0.80$ - Tension (twp), For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (r), For Shear Pick $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$	For Gusset Plate 1 to	Beam Flange Connect:	ion:		
Preferred Weld Sizew = 0.25 inLength of Weld,Lw = 16.312 in(w5),For Beam Shear Plate to Column Wall Connection: Preferred Weld Size (W1),w = 0.25 inSAFETY AND RESISTANCE FACTORS:Safety Factor, $Q(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $\lambda = \frac{1}{\alpha} (1f ASD)$ $\Lambda = \phi (1f LRFD)$ $\lambda = \frac{1}{\alpha} (1f ASD)$ $\Lambda = \phi (1f LRFD)$ Safety factorresistance factorFor Member in Bearing (brg), $Qbrg = 2.00$ $\phi brg = 0.75$ For Block Shear (bs), Shear (ww), $Qbrw = 2.00$ $\phi brs = 0.75$ For Fillet Weld Buckling/Flexural Strength (b), $Qtrw = 2.00$ $\phi trw = 0.75$ For Flexural Local Buckling/Flexural Strength (b), $Qtrw = 2.00$ $\phi trw = 0.75$ For Plexural Rupture (fr), $Qtr = 2.00$ $\phi trw = 0.75$ For Shear (ww), $Qtrw = 2.00$ $\phi trw = 0.75$ For Flexural Local Strength (b), $Qtrw = 2.00$ $\phi trw = 0.75$ For Plexural Rupture (fr), $Qtrw = 2.00$ $\phi trw = 0.75$ For Partial Pen Weld - Tension (twp), $Qtrw = 2.00$ $\phi trw = 0.75$ For Shear Rupture (tw), $Qtr = 2.00$ $\phi trw = 0.75$ For Shear Rupture (tw), $Qtr = 2.00$ $\phi trw = 0.75$ For Shear Rupture (tw), $Qtr = 2.00$ $\phi trw = 0.75$ For Shear Rupture (tw), $Qtr = 2.00$ $\phi trw = 0.75$ For Shear Rupture (tw), $Qtr = 2.00$ $\phi trw = 0.75$ For Tension Rupture $Qtr = 2.00$ $\phi trw = 0.75$ For Shear Yupture (tw), Qt		w = 0.25 in	Length of	Weld,	Lw = 16.312 in
(w5), For Beam Shear Plate to Column Wall Connection: Preferred Weld Size (w1), w = 0.25 in SHFETY AND RESISTANCE FACTORS: Safety Factor, $\Omega(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $\lambda = \frac{1}{\Omega} (if ASD)$ $\Lambda = \phi (if LRFD)$ Safety factor resistance factor modification factor For Member in $\Omega brg = 2.00$ $\phi brg = 0.75$ $\Lambda brg = 0.75$ Bearing (brg), For Block Shear (bs), $\Omega bs = 2.00$ $\phi bs = 0.75$ $\Lambda brg = 0.75$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $\Lambda c = 0.90$ For Fillet Weld $\Omega vw = 2.00$ $\phi vw = 0.75$ $\Lambda vw = 0.75$ Shear (vw), For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ For Flexural Rupture $\Omega fr = 2.00$ $\phi fr = 0.75$ $\Lambda fr = 0.75$ For Compression (b), For Flexural Rupture $\Omega fr = 2.00$ $\phi rw = 0.75$ $\Lambda tr = 0.75$ For Compression (b), For Flexural Rupture $\Omega fr = 2.00$ $\phi r = 0.75$ $\Lambda tr = 0.75$ For Shear (vwp), For Partial Pen Weld $\Omega vwp = 2.00$ $\phi vwp = 0.75$ $\Lambda vwp = 0.75$ - shear (vwp), For Partial Pen Weld $\Omega twp = 1.88$ $\phi twp = 0.80$ $\Lambda twp = 0.80$ - Tension (twp), For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Nupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Nupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Nupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Nupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Nupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Tension Rupture $\Omega tr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Tension Rupture $\Omega tr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Tension Rupture $\Omega tr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$	For Gusset Plate 2 to	Beam Flange Connect:	ion:		
Preferred Weld Size (w1), w = 0.25 in SAFETY AND RESISTANCE FACTORS: Safety Factor, Q(ASD) Resistance Factor, $\phi(LRFD)$ Modification Factor, $\lambda = \frac{1}{\alpha}$ (if ASD) $\lambda = \phi$ (if LRFD) Safety factor resistance factor modification factor For Member in $\Omega brg = 2.00$ $\phi brg = 0.75$ $\Lambda brg = 0.75$ Bearing/ Bolt Bearing (brg), For Block Shear (bs), $\Omega bs = 2.00$ $\phi bs = 0.75$ $\Lambda bs = 0.75$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $\Lambda c = 0.90$ For Fillet Weld $\Omega vw = 2.00$ $\phi vw = 0.75$ $\Lambda vw = 0.75$ Shear (vw), For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ Buckling/Flexural Strength (b), For Partial Pen Weld $\Omega vw = 2.00$ $\phi vwp = 0.75$ $\Lambda fr = 0.75$ (fr), For Partial Pen Weld $\Omega vw = 2.00$ $\phi vwp = 0.75$ $\Lambda vwp = 0.75$ For Shear (vwp), For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vwp = 0.75$ For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vwp = 0.75$ For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vwp = 0.75$ For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vwp = 0.75$ For Shear Strength $\Omega vv = 1.88$ $\phi twp = 0.80$ $\Lambda twp = 0.80$ For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Yielding $\Omega vy = 1.50$ $\phi vy = 1.00$ $\Lambda vy = 1.00$ (vy), For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$		w = 0.25 in	Length of	Weld,	Lw = 16.312 in
SAFETY AND RESISTANCE FACTORS:Safety Factor, $\Omega(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $h = \frac{1}{\Omega} (if ASD)$ $h = \phi (if LRFD)$ $h = \frac{1}{\Omega} (if ASD)$ $h = \phi (if LRFD)$ Safety factorresistance factorFor Member in Bearing/ Bolt Bearing (brg), $\Omega brg = 2.00$ $\phi brg = 0.75$ For Block Shear (bs), $\Omega bs = 2.00$ $\phi bs = 0.75$ $\Lambda bs = 0.75$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $\Lambda c = 0.90$ For Filet Weld Schear (ww, 2.00 $\phi vw = 0.75$ $\Lambda vw = 0.75$ For Flexural Local Strength (b), $\Omega b = 1.67$ $\phi b = 0.90$ For Flexural Rupture Strength (b), $\Omega rr = 2.00$ $\phi rr = 0.75$ For Partial Pen Weld $- rension (twp)$, $\Omega vw = 2.00$ $\phi vw p = 0.75$ For Shear (vwp), $\Omega rr = 2.00$ $\phi vw p = 0.75$ For Shear Nupture $\tau reside (rwp)$, $\Omega rr = 2.00$ $\phi vw p = 0.75$ For Shear Nupture $(vwp),$ $\Omega vw = 2.00$ $\phi vw p = 0.75$ For Shear Nupture $(vwp),$ $\Omega vw = 2.00$ $\phi vw p = 0.75$ For Shear Nupture $(vwp),$ $\Omega vw = 2.00$ $\phi vw p = 0.75$ For Shear Nupture $(vwp),$ $\Omega vw = 2.00$ $\phi vw = 0.75$ For Shear Nupture $(vwp),$ $\Omega vw = 1.00$ $\Lambda vw = 0.75$ For Shear Yielding $(vw) = 1.50$ $\Psi vw = 0.75$ $\Lambda vw = 0.75$ For Tension Rupture $(vw) = 0.200$ $\Phi vw = 0.75$ $\Lambda vw = 0.75$ For Tension Rupture $(vwp) = 1.50$ $\Psi vw = 0.75$ $\Lambda vw = 0.75$	For Beam Shear Plate	to Column Wall Conned	ction:		
Safety Factor, $Q(ASD)$ Resistance Factor, $\phi(LRFD)$ Modification Factor, $\Lambda = \frac{1}{\Omega} (if ASD)$ $\Lambda = \phi (if LRFD)$ $\lambda = \frac{1}{\Omega} (if ASD)$ $\Lambda = \phi (if LRFD)$ safety factorresistance factormodification factorFor Member in Bearing (brg), $\Omega brg = 2.00$ $\phi brg = 0.75$ $\Lambda brg = 0.75$ For Block Shear (bs), $\Omega Ds = 2.00$ $\phi bs = 0.75$ $\Lambda bs = 0.75$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $\Lambda c = 0.90$ For Filet Weld $\Omega vw = 2.00$ $\phi vw = 0.75$ $\Lambda vw = 0.75$ For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ For Flexural Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$ For Shear (vw), $G tr = 2.00$ $\phi vwp = 0.75$ $\Lambda vwp = 0.75$ For Shear (vwp), $G tr = 2.00$ $\phi vr = 0.75$ $\Lambda twp = 0.80$ For Shear (vwp), $G tr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ For Shear Rupture $(vr),$ $\Omega vr = 1.50$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ For Shear Yielding $(vr) = 1.50$ $\phi vr = 1.00$ $\Lambda vr = 1.00$ For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$	Preferred Weld Size (w1),	w = 0.25	in	
Modification Factor, $\Lambda = \frac{1}{\Omega} (if ASD)$ $\Lambda = \phi (if LRFD)$ safety factorresistance factorFor Member in Bearing / Bolt Bearing (brg),For Block Shear (bs), $\Omega bs = 2.00$ $\phi brg = 0.75$ For Block Shear (bs), $\Omega bs = 2.00$ $\phi bs = 0.75$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ For Filet Weld Buckling/Flexural Strength (b), $\Omega vw = 2.00$ For Flexural Local Buckling/Flexural Strength (b), $\Omega tr = 2.00$ For Flexural Rupture $\Gamma ension (twp)$, $\Omega tr = 2.00$ For Shear (vwp), $\Omega twp = 2.00$ For Shear (vwp), $\Delta twp = 1.88$ For Shear (vwp), $\Delta twp = 1.88$ For Shear Rupture $\Omega tr = 2.00$ $\phi vr = 0.75$ For Shear Rupture $(vr),$ $\Omega vr = 1.50$ For Shear Yielding $(vr) = 1.50$ $\phi vr = 0.75$ For Tension Rupture $\Omega tr = 2.00$ $\phi vr = 0.75$ For Tension Rupture $\Omega tr = 2.00$ For Tension Rupture $\Omega tr = 0.75$ For Tension Rupture $\Omega tr = 2.00$ For Tension Rupture $\Omega tr $	SAFETY AND RESISTANCE F	ACTORS :			
$ \begin{split} \lambda &= \frac{1}{\Omega} \left(\text{if ASD} \right) & \lambda &= \phi \left(\text{if LRFD} \right) \\ & \text{safety factor} & \text{resistance factor} & \text{modification factor} \\ & \text{For Member in} & \Omega \text{brg} = 2.00 & \phi \text{brg} = 0.75 & \Lambda \text{brg} = 0.75 \\ & \text{Bearing/ Bolt} \\ & \text{Bearing (brg)}, \\ & \text{For Block Shear (bs)}, & \Omega \text{bs} = 2.00 & \phi \text{bs} = 0.75 & \Lambda \text{bs} = 0.75 \\ & \text{For Compression (c)}, & \Omega \text{c} = 1.67 & \phi \text{c} = 0.90 & \Lambda \text{c} = 0.90 \\ & \text{For Fillet Weld} & \Omega \text{vw} = 2.00 & \phi \text{vw} = 0.75 & \Lambda \text{vw} = 0.75 \\ & \text{Shear (vw)}, & \\ & \text{For Flexural Local} & \Omega \text{b} = 1.67 & \phi \text{b} = 0.90 & \Lambda \text{b} = 0.90 \\ & \text{Buckling/Flexural} \\ & \text{Strength (b)}, & \\ & \text{For Flexural Rupture} & \Omega \text{fr} = 2.00 & \phi \text{fr} = 0.75 & \Lambda \text{fr} = 0.75 \\ & \text{for Partial Pen Weld} & \Omega \text{vwp} = 2.00 & \phi \text{vwp} = 0.75 & \Lambda \text{vwp} = 0.75 \\ & \text{- shear (vwp)}, & \\ & \text{For Shear Rupture} & \Omega \text{vr} = 2.00 & \phi \text{vwp} = 0.80 & \Lambda \text{twp} = 0.80 \\ & - \text{Tension (twp)}, & \\ & \text{For Shear Yielding} & \Omega \text{vy} = 1.50 & \phi \text{vy} = 1.00 & \Lambda \text{vy} = 1.00 \\ & \text{(vy)}, & \\ & \text{For Tension Rupture} & \Omega \text{tr} = 2.00 & \phi \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{(vy)}, & \\ & \text{For Tension Rupture} & \Omega \text{tr} = 2.00 & \phi \text{tr} = 0.75 & \Lambda \text{vyp} = 0.80 \\ & \text{Tension Rupture} & \Omega \text{tr} = 2.00 & \phi \text{vy} = 1.00 & \Lambda \text{vy} = 1.00 \\ & \text{(vy)}, & \\ & \text{For Tension Rupture} & \Omega \text{tr} = 2.00 & \phi \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{(vr)}, & \\ & \text{For Shear Yielding} & \Omega \text{vy} = 1.50 & \phi \text{vy} = 1.00 \\ & \text{vy} = 1.00 & \Lambda \text{vy} = 1.00 \\ & \text{vy} = 1.00 & \Lambda \text{vy} = 1.00 \\ & \text{vy} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & \Lambda \text{tr} = 0.75 \\ & \text{tr} = 0.75 & $	Safety Factor, $\Omega(ASD)$		Resistanc	e Factor,	ϕ (LRFD)
$Safety factor resistance factor modification factor$ For Member in $\Omega brg = 2.00$ $\phi brg = 0.75$ $\Lambda brg = 0.75$ Bearing/ Bolt Bearing (brg), For Block Shear (bs), $\Omega bs = 2.00$ $\phi bs = 0.75$ $\Lambda bs = 0.75$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $\Lambda c = 0.90$ For Fillet Weld $\Omega vw = 2.00$ $\phi vw = 0.75$ $\Lambda vw = 0.75$ Shear (vw), For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ Buckling/Flexural Strength (b), For Flexural Rupture $\Omega fr = 2.00$ $\phi fr = 0.75$ $\Lambda fr = 0.75$ (fr), For Partial Pen Weld $\Omega vwp = 2.00$ $\phi vwp = 0.75$ $\Lambda vwp = 0.75$ For Shear (vwp), For Shear (vwp), For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vwp = 0.80$ - Tension (twp), For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Yielding $\Omega vy = 1.50$ $\phi vy = 1.00$ $\Lambda vy = 1.00$ (vy), For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$	Modification Factor,				
For Member in Bearing/ Bolt Bearing (brg), $\Omega brg = 2.00$ $\phi brg = 0.75$ $Abrg = 0.75$ For Block Shear (bs), $\Omega bs = 2.00$ $\phi bs = 0.75$ $Abs = 0.75$ For Block Shear (bs), $\Omega c = 1.67$ $\phi c = 0.90$ $Ac = 0.90$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $Avw = 0.75$ For Fillet Weld $\Omega vw = 2.00$ $\phi vw = 0.75$ $Avw = 0.75$ Shear (vw), $B = 1.67$ $\phi b = 0.90$ $Ab = 0.90$ For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $Ab = 0.90$ Buckling/Flexural Strength (b), $Strength (b)$, $Arr = 0.75$ $Arr = 0.75$ For Partial Rupture $\Omega rwp = 2.00$ $\phi vwp = 0.75$ $Avwp = 0.75$ For Partial Pen Weld $\Omega vwp = 1.88$ $\phi twp = 0.80$ $Atwp = 0.80$ - Tension (twp), $Gvr = 2.00$ $\phi vr = 0.75$ $Avr = 0.75$ For Shear Rupture $\Omega vy = 1.50$ $\phi vy = 1.00$ $Avy = 1.00$ (vy),For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $Atr = 0.75$	$\Lambda = \frac{1}{\Omega} (if ASD)$		$\Lambda = \phi$ (if	LRFD)	
Bearing/ Bolt Bearing (brg),DescriptionDescriptionFor Block Shear (bs), $\Omega bs = 2.00$ $\phi bs = 0.75$ $\Lambda bs = 0.75$ For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $\Lambda c = 0.90$ For Fillet Weld $\Omega vw = 2.00$ $\phi vw = 0.75$ $\Lambda vw = 0.75$ Shear (vw), $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ Buckling/Flexural Strength (b),For Flexural Rupture $\Omega fr = 2.00$ $\phi fr = 0.75$ $\Lambda fr = 0.75$ For Partial Pen Weld $\Omega vwp = 2.00$ $\phi vwp = 0.75$ $\Lambda vwp = 0.75$ For Partial Pen Weld $\Omega twp = 1.88$ $\phi twp = 0.80$ $\Lambda twp = 0.80$ - Tension (twp), $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vz),For Shear Yielding $\Omega vy = 1.50$ $\phi vy = 1.00$ $\Lambda vy = 1.00$ (vy),For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$		safety factor	resistanc	e factor	modification factor
For Compression (c), $\Omega c = 1.67$ $\phi c = 0.90$ $\Lambda c = 0.90$ For Fillet Weld $\Omega vw = 2.00$ $\phi vw = 0.75$ $\Lambda vw = 0.75$ Shear (vw), $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ For Flexural Local $\Omega b = 1.67$ $\phi b = 0.90$ $\Lambda b = 0.90$ Buckling/Flexural $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda fr = 0.75$ For Flexural Rupture $\Omega fr = 2.00$ $\phi vwp = 0.75$ $\Lambda vwp = 0.75$ For Partial Pen Weld $\Omega vwp = 2.00$ $\phi vwp = 0.75$ $\Lambda vwp = 0.75$ For Partial Pen Weld $\Omega twp = 1.88$ $\phi twp = 0.80$ $\Lambda twp = 0.80$ - Tension (twp), $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ For Shear Yielding $\Omega vy = 1.50$ $\phi vy = 1.00$ $\Lambda vy = 1.00$ (vy),For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$	Bearing/ Bolt	Ωbrg = 2.00	φbrg = 0.	75	Abrg = 0.75
For Fillet Weld Shear (vw), $\Omega vw = 2.00$ $\varphi vw = 0.75$ $\Lambda vw = 0.75$ For Flexural Local Buckling/Flexural Strength (b), $\Omega b = 1.67$ $\varphi b = 0.90$ $\Lambda b = 0.90$ For Flexural Rupture (fr), $\Omega fr = 2.00$ $\varphi fr = 0.75$ $\Lambda fr = 0.75$ For Partial Pen Weld - shear (vwp), $\Omega vwp = 2.00$ $\varphi vwp = 0.75$ $\Lambda vwp = 0.75$ For Partial Pen Weld - Tension (twp), $\Omega twp = 1.88$ $\varphi twp = 0.80$ $\Lambda twp = 0.80$ For Shear Rupture (vr), $\Omega vr = 2.00$ $\varphi vr = 0.75$ $\Lambda vr = 0.75$ For Shear Yielding (vy), $\Omega vy = 1.50$ $\varphi vy = 1.00$ $\Lambda vy = 1.00$ For Tension Rupture (vy), $\Omega tr = 2.00$ $\varphi tr = 0.75$ $\Lambda tr = 0.75$	For Block Shear (bs),	$\Omega bs = 2.00$	ϕ bs = 0.7	5	Abs = 0.75
Shear (vw), (vu) (vu) (vu) (vu) (vu) For Flexural Local Buckling/Flexural Strength (b), $(b) = 1.67$ $(b) = 0.90$ $(b) = 0.90$ For Flexural Rupture (fr), (b) , (b) (b) (b) For Flexural Rupture (fr), (b) (b) (b) For Partial Pen Weld - shear (vwp), (c) (c) (c) For Partial Pen Weld - Tension (twp), (c) (c) (c) For Shear Rupture (vr), (c) (c) (c) For Shear Yielding (vy), (c) (c) (c) For Tension Rupture (c) (c) (c) (c) (c) (c) (c) For Tension Rupture (c) (c) (c) (c) (c) (c) (c) (c) For Tension Rupture (c) $(c$	For Compression (c),	$\Omega c = 1.67$	φc = 0.90		Ac = 0.90
For Flexural Local Buckling/Flexural Strength (b), $\Omega b = 1.67$ $\varphi b = 0.90$ $\Lambda b = 0.90$ For Flexural Rupture (fr), $\Omega fr = 2.00$ $\varphi fr = 0.75$ $\Lambda fr = 0.75$ For Partial Pen Weld - shear (vwp), $\Omega vwp = 2.00$ $\varphi vwp = 0.75$ $\Lambda vwp = 0.75$ For Partial Pen Weld - shear (vwp), $\Omega twp = 1.88$ $\varphi twp = 0.80$ $\Lambda twp = 0.80$ For Shear Rupture (vr), $\Omega vr = 2.00$ $\varphi vr = 0.75$ $\Lambda vr = 0.75$ For Shear Rupture (vr), $\Omega vy = 1.50$ $\varphi vy = 1.00$ $\Lambda vy = 1.00$ For Tension Rupture (vy), $\Omega tr = 2.00$ $\varphi tr = 0.75$ $\Lambda tr = 0.75$		$\Omega vw = 2.00$	φvw = 0.7	5	Avw = 0.75
(fr) ,For Partial Pen Weld $\Omega vwp = 2.00$ $\phi vwp = 0.75$ $\Lambda vwp = 0.75$ - shear (vwp),For Partial Pen Weld $\Omega twp = 1.88$ $\phi twp = 0.80$ $\Lambda twp = 0.80$ - Tension (twp), $\nabla vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr),For Shear Yielding $\Omega vy = 1.50$ $\phi vy = 1.00$ $\Lambda vy = 1.00$ For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$	For Flexural Local Buckling/Flexural	Ωb = 1.67	φb = 0.90		Ab = 0.90
- shear (vwp), For Partial Pen Weld $\Omega twp = 1.88$ $\phi twp = 0.80$ $\Lambda twp = 0.80$ - Tension (twp), For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Yielding $\Omega vy = 1.50$ $\phi vy = 1.00$ $\Lambda vy = 1.00$ (vy), For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$	-	$\Omega fr = 2.00$	φfr = 0.7	5	Afr = 0.75
- Tension (twp), For Shear Rupture $\Omega vr = 2.00$ $\phi vr = 0.75$ $\Lambda vr = 0.75$ (vr), For Shear Yielding $\Omega vy = 1.50$ $\phi vy = 1.00$ $\Lambda vy = 1.00$ (vy), For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$		Ωvwp = 2.00	$\varphi vwp = 0$.	75	Avwp = 0.75
(vr) ,For Shear Yielding $\Omega vy = 1.50$ $\phi vy = 1.00$ $\Lambda vy = 1.00$ (vy) ,For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$		Ωtwp = 1.88	$\phi twp = 0$.	80	Atwp = 0.80
(vy), For Tension Rupture $\Omega tr = 2.00$ $\phi tr = 0.75$ $\Lambda tr = 0.75$	_	$\Omega vr = 2.00$	φvr = 0.7	5	Avr = 0.75
	_	Ωvy = 1.50	φvy = 1.0	0	Avy = 1.00
		Ωtr = 2.00	φtr = 0.7	5	Atr = 0.75

1801 Park 270 Drive Sui Email: info GIZA [™] www.giz	A Steel te 220 St. Louis, MO 63146 @gizasteel.com asteel.com	Job Code: Job Name: Sheet No.: Designed by: Revision No: Subject:	YYYY NASCC 2019 7 of 78 RCM 00 VIH-C1DD	Date:	03/28/2019
For Tension Yielding(ty),	$\Omega ty = 1.67$	φty = 0.9	0	Λty =	0.90
For Web Crippling(cr),	$\Omega cr = 2.00$	¢cr = 0.7	5	Acr =	0.75
For Member Shear Yielding for S, M, W, HSS (wy),	Ωwy = 1.50	φwy = 1.0	0	∧wy =	1.00
For Eccentric Weld (ew),	$\Omega ew = 2.00$	¢ew = 0.7	5	∧ew =	0.75
For Rectangular- Square HSS Chord Plastification (pHSS),	ΩpHSS = 1.67	φpHSS = 0	.90	ЛрHSS	= 1.00
APPLIED LOADS:					
Brace 1:					
Given Tension Load,	Pt1 = 25 kips	Given Com Load,	pression	Pc1	= 25 kips
Given Load					
Governing Tension Load,	Pt = 25 kips	Governing Load,	Compressi	on Pc :	= 25 kips
Maximum Axial Load,	P = max(Pt, Pc)	P = 25 ki	ps		
Brace 2:					
Given Tension Load,	Pt1 = 25 kips	Given Com Load,	pression	Pcl	= 25 kips
Given Load					
Governing Tension Load,	Pt = 25 kips	Governing Load,	Compressi	on Pc :	= 25 kips
Maximum Axial Load,	P = max(Pt, Pc)	P = 25 ki	ps		
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				



Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	8 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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,

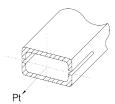
Lcon = Lw

Net Tension Area,

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

Ant = 6.531 in^2

Lcon = 6 in



Eccentricity of the Connection,

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

$$econ = 1.932 \text{ in}$$
Reduction Coefficient,

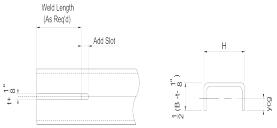
$$U = 1 - \frac{econ}{Lcon}$$

$$U = 0.678$$
Effective Net Tension Area,
Ae = U \cdot Ant
Ae = 4.428 in²
Tensile Rupture Capacity, (D2-2)
Rtr = Atr ·Fu ·Ae

Rtr = 192.627 kips 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 SectionUnstiffened Width,

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



Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	9 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		
Sheet No.: Designed by: Revision No:	9 of 78 RCM 00	Date:	03/28/2019

Limiting Width-to-Thickness Ratio,

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

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

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

ycg = 1.639 in

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

b. Compression Capacity of C-Shape Section

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

Effective Length Factor,

Laterally Unbraced Length,

Lu = Add Slot

Modulus of Elasticity

E = 29000 ksi

Gross Area,

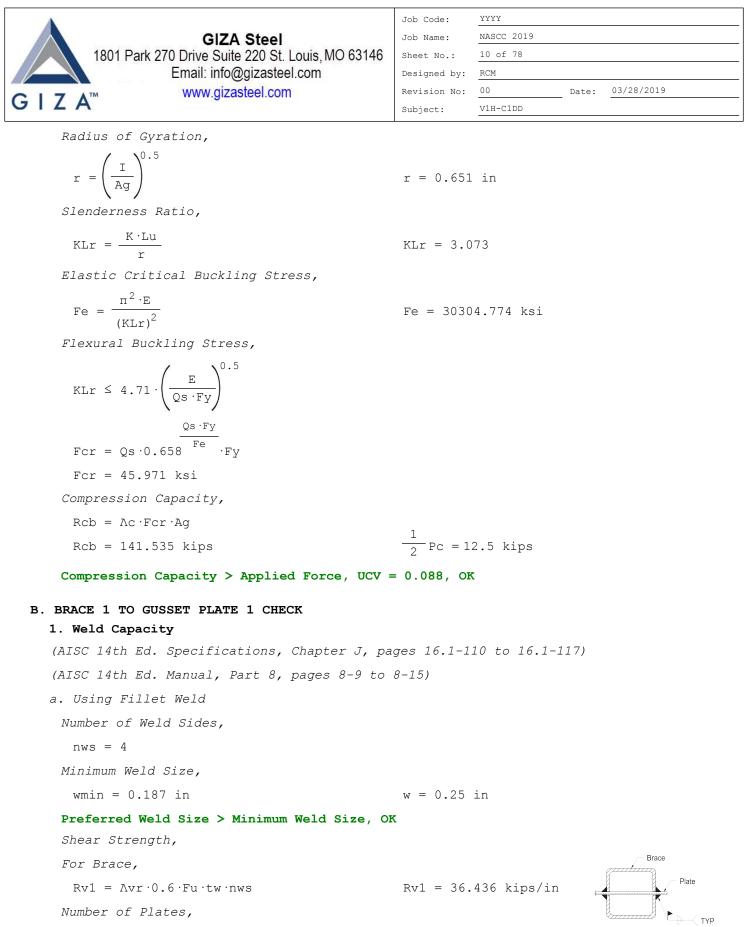
$$A1 = 2 \cdot (b \cdot tw)$$
 $A1 = 1.57 \text{ in}^2$ $A2 = (H - 2 \cdot tw) \cdot tw$ $A2 = 1.85 \text{ in}^2$ $Ag = A1 + A2$ $Ag = 3.421 \text{ in}^2$

Centroid,

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

Moment of Inertia,

	- H	-
B 217		by by



For Gusset Plate,

```
Job Code:
                                                                        YYYY
                             GIZA Steel
                                                            Job Name:
                                                                        NASCC 2019
            1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                            Sheet No.:
                                                                        11 of 78
                        Email: info@gizasteel.com
                                                            Designed by:
                                                                        RCM
                                                                                             03/28/2019
                          www.gizasteel.com
                                                            Revision No:
                                                                        00
                                                                                      Date:
G I Z A<sup>™</sup>
                                                                        V1H-C1DD
                                                            Subject:
         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 = _____, Rv2)
                                                            weff = 0.22 in
                        Rv3
       Length of Weld,
         Lw = 6 in
       Weld Capacity,
         Rw = \Lambda vw \cdot 0.6 \cdot Fu \cdot sin(45 deg) \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,
         bwhoq = 0 in
       Available Width of Whitmore Section in Gusset Plate,
         bwh = bwh1 - 2 \cdot bwhoq
                                                            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 = 12.928 in
         b = bwh
       Gross Tension Area,
         Aq = 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
```

GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com www.gizasteel.com	Job Code:	<u>үүүү</u>		
	Job Name:	NASCC 2019		
	Sheet No.:	12 of 78		
	Designed by: Revision No:	RCM 00 Date: 03/28/2019		
GIZA [™] www.gizasteel.com	Subject:	V1H-C1DD		
(AISC 14th Ed. Specifications, Chapter J, Se	ction J4.4	, page 16.1-129 to 16.1-130)		
Effective Length Factor,				
(Commentary on the Specification for Struc	ctural Stee	el Building Table C-A-7.1)		
K = 0.65				
Laterally Unbraced Length,				
Lu = Lwh	Lu = 5.12	5 in		
Gross Area,				
$Ag = bwh \cdot t$	Ag = 4.84	8 in²		
Radius of Gyration,				
$r = \frac{t}{(12)^{0.5}}$	r = 0.108	in		
Slenderness Ratio,				
$KLr = \frac{K \cdot Lu}{r}$	KLr = 30.	773		
Elastic Critical Buckling Stress,				
$Fe = \frac{\pi^2 \cdot E}{KLr^2}$	Fe = 302.	249 ksi		
Flexural Buckling Stress,				
KLr > 25				
$KLr \leq 4.71 \cdot \left(\frac{E}{Fy}\right)^{0.5}$				
$Fcr = 0.658 \frac{Fy}{Fe} \cdot Fy$				
For = 0.658 ^{re} Fy Number of Areas in Consideration,	Fcr = 34.2	249 KSI		
n1 = n				
Compression Capacity,				
Rcb = Ac ·nl ·Fcr ·Ag				
Rcb = 149.439 kips	Pc = 25 k	-		
Compression Capacity > Applied Force, UCV 4. Block Shear Capacity	= 0.167, 0	ĸ		
(AISC 14th Ed. Specifications, Chapter J, Se	ction J4.3	, page 16.1-129)		
Reduction Factor,		,		
Ubs = 1.0	(tension	stress is uniform)		
Gross Shear Area,	(
$Agv = 2 \cdot Lw \cdot t$	Aqv = 4.5	j in²		
	119V - 1.J			
Net Tension Area,	∿∽ + - 0 0	NE in 2		
Ant = $H \cdot t$	Ant = 2.2	25 IN-		

GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com WWW.gizasteel.com	Job Code: Job Name: Sheet No.: Designed by: Revision No: Subject:	YYYY NASCC 2019 13 of 78 RCM 00 Date: 03/28/2019 V1H-C1DD
Net Shear Area,		
Anv = Agv	Anv = 4.5	in²
Number of Areas in Consideration,		
n1 = n		

Block Shear Capacity, (J4-5)

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

Rbs = 170.775 kips

Pt = 25 kips

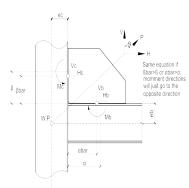
Block Shear Capacity > Applied Force, UCV = 0.146, OK

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,

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

 $\alpha bar = 8.656 in$
 $\alpha = (\beta bar + eb) \cdot tan(\theta) - ec$
 $\alpha = 14.69 in$
 $r = \frac{P}{\left[(\alpha + ec)^2 + (\beta + eb)^2\right]^{0.5}}$

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

	7) 0)	үүүү
	Job Code:	
GIZA Steel	Job Name:	NASCC 2019
1801 Park 270 Drive Suite 220 St. Louis, MO 63146	Sheet No.:	14 of 78
Email: info@gizasteel.com	Designed by:	RCM
GIZA [™] www.gizasteel.com	Revision No:	00 Date: 03/28/2019
GIZA [™]	Subject:	V1H-C1DD
Horizontal Side,	Vertical	Side,

Horizontal Side,

- Hb = $\alpha \cdot r$
- Hb = 12.97 kips
- $Vb = eb \cdot r$
- Vb = 9.448 kips
- $Mb = |Vb \cdot (\alpha \alpha bar)|$
- $Mb = 57.003 \text{ kips} \cdot \text{in}$

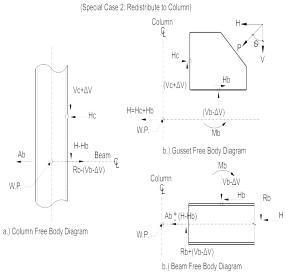
 $Hc = ec \cdot r$ Hc = 6.181 kips

- $Vc = \beta \cdot r$
- Vc = 6.622 kips

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

Mc = 0 kips \cdot in

Redistribution of Forces,



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 $Mb = 57.003 \text{ kips} \cdot \text{in}$ Gusset-to-Column Connection, $VC = |VC + \Delta V|$ HC = |HC| $Mc = |Hc \cdot (\beta - \beta bar)|$ Vc = 6.622 kips Hc = 6.181 kips $Mc = 0 \text{ kips} \cdot \text{in}$

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



Job Code:	ҮҮҮҮ		
Job Name:	NASCC 2019		
Sheet No.:	15 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

Moment Force,

 $Mc = 0 \text{ kips} \cdot \text{in}$

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

Rc = 9.058 kips

E.A. GUSSET PLATE 1 CHECK

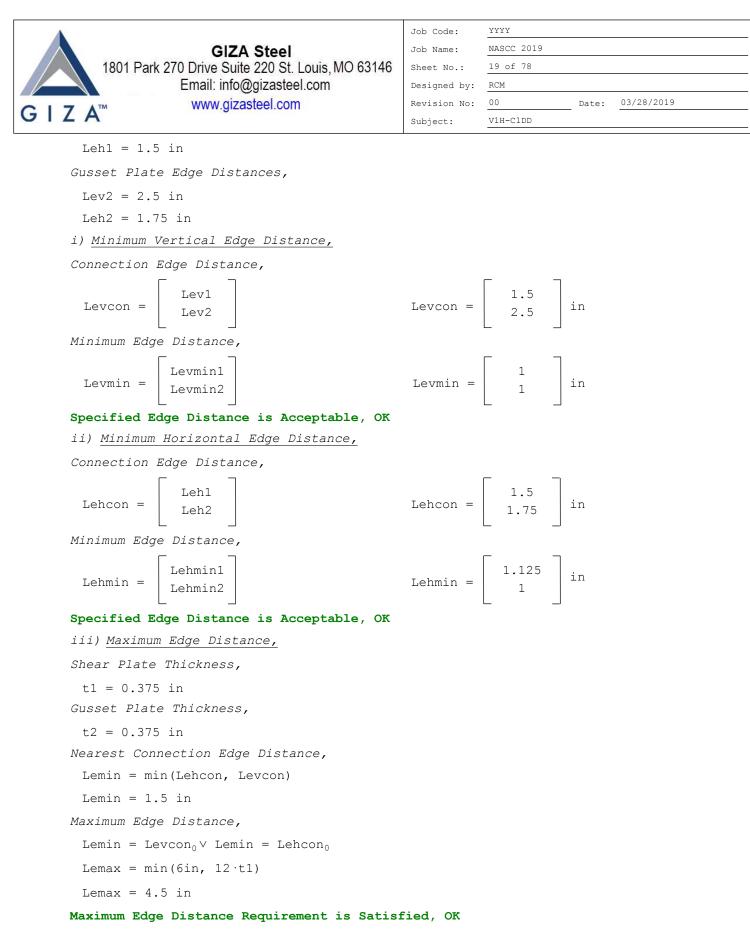
1. Bolt Capacity

```
(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 = 0.281 in^{2}
   Abrg = db \cdot t
 Bolt Centerline Distance from Face of Support,
   ab = qap + Leh + 0.5 \cdot (nv - 1) \cdot sv
   ab = 2.25 in
 Eccentricity Distance of End Reaction from Bolt Group Centerline,
   ebv = ab
                                                   ebv = 2.25 in
 Bolt Vertical Centerline Distance from Beam Centerline,
   ah = |\beta - \beta bar|
                                                   ah = 0 in
 Eccentricity distance of Axial Load from Bolt Group Centerline,
   Yo = ah
                                                   Yo = 0 in
 Load Inclination from Vertical,
   \theta = \operatorname{atan}\left(\frac{\operatorname{Hc}}{\operatorname{Vc}}\right)
                                                   \theta = 43.025 \text{ 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)
                      1.2 · (Lev - 0.5 · hdv) ·t
                     1.2 · (Leh - 0.5 · hdh) ·t
   Fbe = \Lambda brq \cdot Fu \cdot
                              2.4 ·Abrg
                                                   Fbe = 20.798 kips
   Fbe = min(Fbe)
```

```
Job Code:
                                                                YYYY
                       GIZA Steel
                                                    Job Name:
                                                                NASCC 2019
       1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                    Sheet No.:
                                                                16 of 78
                  Email: info@gizasteel.com
                                                    Designed by:
                                                               RCM
                                                                                    03/28/2019
                    www.gizasteel.com
                                                    Revision No:
                                                               00
                                                                             Date:
7 A<sup>™</sup>
                                                    Subject:
                                                                V1H-C1DD
   Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)
     hdv < hdls(db)
                           1.2 · (s - hdv) ·t
                           1.2 · (sv - hdh) ·t
     Fbs = Abrq · Fu ·
                                2.4 ·Abrg
     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,
   Aqv = [(nr - 1) \cdot s + Lev] \cdot t
                                                   Aqv = 4.312 in^{2}
 Net Tension Area,
   Ant = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot t
   Ant = 0.492 \text{ in}^2
 Net Shear Area,
   Anv = Aqv - [(nr - 0.5) \cdot hdv] \cdot t
                                          Anv = 3.164 in<sup>2</sup>
                                                                                              (nr-1)@sv
 Number of Areas in Consideration,
   n1 = n
                                                                                      (nv-1)@s
 Block Shear Capacity, (J4-5)
   Rbss = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Aqv + Ubs·Fu·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
```

```
Job Code:
                                                                  YYYY
                       GIZA Steel
                                                      Job Name:
                                                                  NASCC 2019
      1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                      Sheet No.:
                                                                  17 of 78
                  Email: info@gizasteel.com
                                                      Designed by:
                                                                  RCM
                                                                                       03/28/2019
                    www.gizasteel.com
                                                                  00
                                                      Revision No:
                                                                                Date:
ZAM
                                                      Subject:
                                                                  V1H-C1DD
 Reduction Factor,
   ah = 0 in
                                                       (tension stress is uniform)
   Ubs = 1.0
                                                                                                Lev
 Gross Shear Area,
                                                                                                      (nr-1)@sv
   Agv = [Leh + (nv - 1) \cdot sv] \cdot t
                                                      Aqv = 0.656 in^{2}
 Net Tension Area,
   Ant = [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hdv] \cdot t
                                                                                     Leh (nv-1)@sv
   Ant = 3.164 \text{ in}^2
 Net Shear Area,
   Anv = Aqv - [(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 = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)
   Rbs1 = 148.268 kips
                                                      Hc = 6.181 \text{ kips}
 Block Shear Capacity > Applied Force, UCV = 0.042, OK
Pattern 2
 Reduction Factor,
   ah = 0 in
                                                       (tension stress is uniform)
   Ubs = 1.0
                                                                                                   Le<
 Gross Shear Area,
                                                                                                      (nr-1)@s
   Aqv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot t
                                                    Aqv = 1.312 in^{2}
                                                                                                   >0
 Net Tension Area,
   Ant = [(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t
                                                                                        (nv-1)@sv
                                                                                     Leh
   Ant = 2.391 \text{ in}^2
 Net Shear Area,
   Anv = Aqv - 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 = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)
   Rbs2 = 125.255 kips
                                                      Hc = 6.181 \text{ kips}
 Block Shear Capacity > Applied Force, UCV = 0.049, OK
 Governing Block Shear Capacity,
   Rbs = min(Rbs1, Rbs2)
```

A	Job Code:	ҮҮҮҮ
GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com	Job Name:	NASCC 2019
		18 of 78
Email: info@gizasteel.com	Designed by: Revision No:	RCM 00 Date: 03/28/2019
GIZA [™] www.gizasteel.com	Subject:	V1H-C1DD
Rbs = 125.255 kips	Hc = 6.18	21 king
Block Shear Capacity > Applied Force, UC		-
BIOCK Shear Capacity > Applied Force, OC	v = 0.049, Or	X
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,		
$\theta = \operatorname{atan}\left(\frac{\operatorname{Hc}}{\operatorname{Vc}}\right)$	$\theta = 43.02$	25 deg
Eccentric Load Coefficient,		
(AISC 14th Ed. Manual Part 7, Instanta 7-12)	neous Center	of Rotation Method, pages 7-6 to
C = 3.148		
Shear Capacity Per Bolt,		
Arv = 8.068 kips		
Bolt Shear Capacity,		
$Rb = n \cdot C \cdot \Lambda rv$		
Rb = 25.399 kips	Rc = 9.05	58 kins
Bolt Shear Capacity > Applied Force, U		-
	cv = 0.357, 0	ĸ
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,		
$t_2 = 0.375$ in		
a. Vertical Spacing,		
Minimum Bolt Spacing,		
s = 3 in 2		
$smin = 2 \frac{2}{3} \cdot db$	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, 1–124)	Section J3.4	and J3.5, pages 16.1-122 to 16.
Shear Plate Edge Distances,		
Lev1 = 1.5 in		



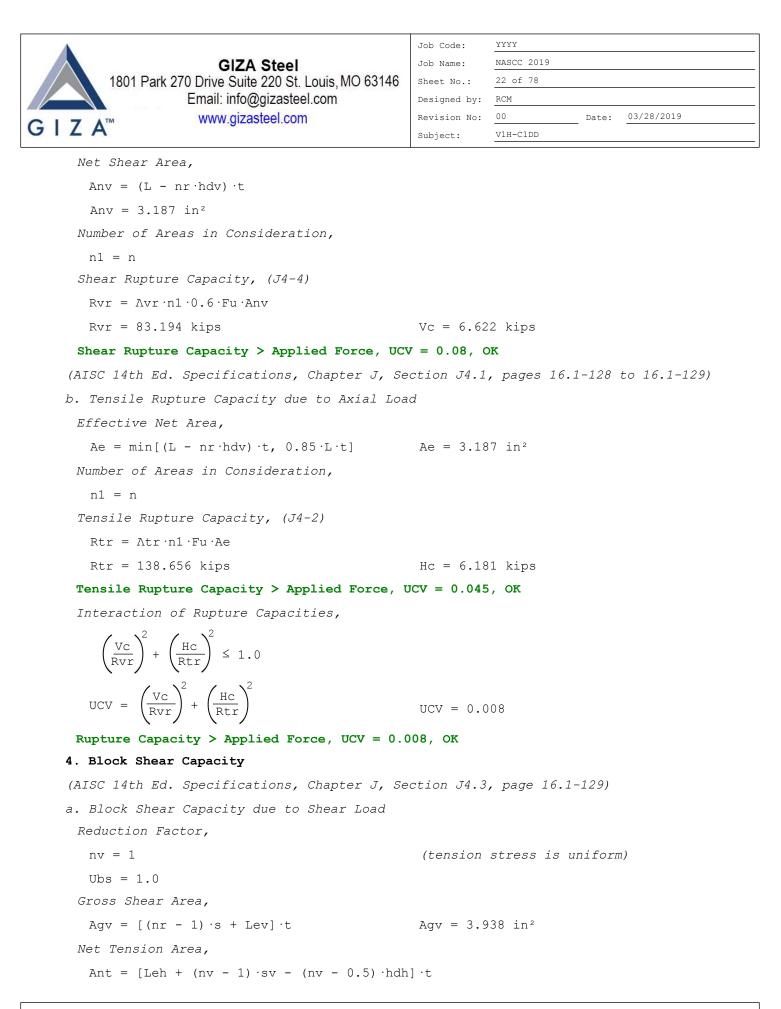
E.C. GUSSET SHEAR PLATE 1 CHECK

1. Bolt Capacity

GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com www.gizasteel.com	Job Code: Job Name: Sheet No.: Designed by: Revision No:	YYYY NASCC 2019 20 of 78 RCM 00 Date: 03/28/2019 V1H-C1DD
(AISC 14th Ed. Specifications, Chapter J,	Subject: Section J3.	
a. Bolt Capacity due to Resultant Load		
Bearing Area,		Abg
$Abrg = db \cdot t$	Abrg = 0 .	281 in ²
Bolt Centerline Distance from Face of Sup	2	
$ab = gap + Leh + 0.5 \cdot (nv - 1) \cdot sv$,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,,	
ab = 2.25 in		
Eccentricity Distannce of End Reaction fr	com Bolt Gr	oup Centerline,
ebv = ab	ebv = 2.2	
Bolt Vertical Centerline Distance from Be	eam Centerl	ine,
$ah = \beta - \beta bar $	ah = 0 in	
Eccentricity distance of Axial Load from	Bolt Group	Centerline,
Yo = ah	Yo = 0 in	
Load Inclination from Vertical,		
$\theta = \operatorname{atan}\left(\frac{\operatorname{Hc}}{\operatorname{Vc}}\right)$ Eccentric Load Coefficient,	θ = 43.02	5 deg
(AISC 14th Ed. Manual Part 7, Instantanec 7-8)	ous Center	of Rotation Method, pages 7-6 to
C = 3.148		
Available Bearing Strength Using Edge Dis hdv < hdls(db)	stance, (J3	-6a, J3-6c)
Fbe = Λ brg · Fu · $\begin{bmatrix} 1.2 \cdot (\text{Lev} - 0.5 \cdot \text{hdv}) \cdot t \\ 1.2 \cdot (\text{Leh} - 0.5 \cdot \text{hdh}) \cdot t \\ 2.4 \cdot \Lambda$ brg		
Fbe = min(Fbe)	Fbe = $18.$	963 kips
Available Bearing Strength Using Bolt Spa	acing, (J3-	6a, J3-6c)
hdv < hdls(db)		
Fbs = $Abrg \cdot Fu \cdot \begin{bmatrix} 1.2 \cdot (s - hdv) \cdot t \\ 1.2 \cdot (sv - hdh) \cdot t \\ 2.4 \cdot Abrg \end{bmatrix}$		
nv > 1		
$Fbs = min(Fbs_0, Fbs_2)$	Fbs = 29.	362 kips
Number of Area in Consideration,		
n1 = n		
Bolt Capacity,		
Rbrg = C·min(nl·Fbe, nl·Fbs, n·Arv)		



a. Shear Rupture Capacity due to Shear Load



```
Job Code:
                                                                           YYYY
                               GIZA Steel
                                                               Job Name:
                                                                           NASCC 2019
             1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                               Sheet No.:
                                                                            23 of 78
                         Email: info@gizasteel.com
                                                               Designed by:
                                                                           RCM
                                                                           00
                                                                                                 03/28/2019
                           www.gizasteel.com
                                                               Revision No:
                                                                                          Date:
G I Z A<sup>™</sup>
                                                                            V1H-C1DD
                                                               Subject:
         Ant = 0.363 \text{ in}^2
       Net Shear Area,
                                                                                                        ev.
         Anv = Agv - [(nr - 0.5) \cdot hdv] \cdot t
                                                              Anv = 2.789 in^2
                                                                                                             (nr-1)@sv
       Number of Areas in Consideration,
         n1 = n
                                                                                                    (nv-1)@sv
       Block Shear Capacity, (J4-5)
         Rbss = Abs·nl·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)
                                                               Vc = 6.622 kips
         Rbss = 79.59 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
                                                               (tension stress is uniform)
         Ubs = 1.0
       Gross Shear Area,
                                                                                                                 (nr-1)@sv
         Aqv = [Leh + (nv - 1) \cdot sv] \cdot t
                                                             Aqv = 0.563 in^{2}
       Net Tension Area,
         Ant = [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hdv] \cdot t
                                                                                               Leh (nv-1)@sv
         Ant = 2.789 \text{ in}^2
       Net Shear Area,
         Anv = Aqv - [(nv - 0.5) \cdot hdh] \cdot t
                                                             Anv = 0.363 in^{2}
       Number of Areas in Consideration,
         n1 = n
       Block Shear Capacity, (J4-5)
         Rbsa = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)
         Rbsa = 130.437 kips
                                                               Hc = 6.181 \text{ kips}
       Block Shear Capacity > Applied Force, UCV = 0.047, OK
        Interaction of Bolt Shear Capacities,
         \left(\frac{\text{Vc}}{\text{Rbss}}\right)^2 + \left(\frac{\text{Hc}}{\text{Rbsa}}\right)^2 \le 1.0
         UCV = \left(\frac{Vc}{Rbss}\right)^2 + \left(\frac{Hc}{Rbsa}\right)^2
                                                               UCV = 0.009
       Block Shear Capacity > Applied Force, UCV = 0.009, OK
      Pattern 2
```

GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146	Job Code: YYYY Job Name: NASCC 2019
	Job Name: NASCC 2019 Sheet No.: 24 of 78
Email: info@gizasteel.com	Designed by: RCM
www.gizated.com	Revision No: 00 Date: 03/28/2019
GIZA [™] www.gizasteet.com	Subject: VIH-CIDD
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 = $[(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t$	Leh (nv-1)@sv
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 = Abs n1 min(0.6 Fu Anv + Ubs Fu Ant	, 0.6 ·Fy ·Agv + Ubs ·Fu ·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,	
$\lambda = \frac{L \cdot Fy^{0.5}}{10 \cdot t \cdot \left[475 + 280 \cdot \left(\frac{L}{ab}\right)^2\right]^{0.5}} \cdot \frac{1}{ksi^{0.5}}$ $\lambda \le 0.7$	$\lambda = 0.209$

Q = 1

Allowable Flexural Local Buckling Stress or Yielding Stress,

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

Gross Plastic Section Modulus,

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

$$Zx = 13.5 \text{ in}^3$$
Eccentricity,



Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	25 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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,

$$mod(nr, 2) = 0$$

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

$$Znet = 9.563 in^{3}$$

$$Flexural Rupture Capacity,$$

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

$$Rfr = 184.875 kips \qquad Vc = 6.622 kips$$

$$Flexural Rupture Capacity > Applied Force, UCV = 0.036, OK$$
8. Compression Capacity
(AISC 14th Ed. Specifications, Chapter J, Section J4.4, page 16.1-129 to 16.1-1.
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,

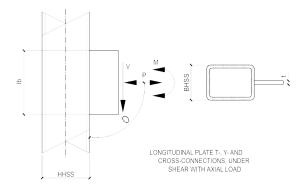
-130)

GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com	Job Code:	<u>YYYY</u>		
	Job Name: Sheet No.: Designed by:	NASCC 2019 26 of 78		
		20 01 /8 RCM		
unuu gizantaal aam	Revision No:	00 Date: 03/28/2019		
GIZA [™] www.gizasteer.com	Subject:	V1H-C1DD		
Lu = gap + Le1	Lu = 2.25	in		
Gross Area,				
Ag = L·t	Ag = 4.5	in²		
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	Fcr = 36	ksi		
Number of Areas in Consideration,				
n1 = n				
Compression Capacity,				
Rcb = Ac ·n1 ·Fcr ·Ag				
Rcb = 145.8 kips	Hc = 6.18	1 kips		
Compression Capacity > Applied Force, UCV	= 0.042, 0	к		
E.D. GUSSET SHEAR PLATE 1 TO COLUMN WALL CHEC	x			
1. Weld Capacity				
(AISC 14th Ed. Specifications, Chapter J, pa	ges 16 . 1-1.	10 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, OF	τ			
Length of Weld,				
$Lw = (nr - 1) \cdot s + 2 \cdot Lev$	Lw = 12 i	n		
Total Force Per Unit Length on Welds of She	ear Plate t	o Column Connection,		
$Ruw = \left[\left(\frac{Vc}{Lw} \right)^2 + \left(\frac{Hc}{Lw} \right)^2 \right]^{0.5}$	Ruw = 0.7	55 kips/in		

	Job Code:	ҮҮҮҮ		
GIZA Steel	Job Name:	NASCC 2019		
1801 Park 270 Drive Suite 220 St. Louis, MO 63146	Sheet No.:	27 of 78		
Email: info@gizasteel.com	Designed by:	RCM		
GIZA [™] www.gizasteel.com	Revision No:	00	Date:	03/28/2019
GILA	Subject:	V1H-C1DD		
Shear Strength,				
For Column,				
Rv1 = Avr · 0.6 · Fu · tw · nws	Rv1 = 30.	328 kips/in		
For Shear Plate,				
$Rv2 = Avr \cdot 0.6 \cdot Fu \cdot t$	Rv2 = 9.7	87 kips/in		
Effective Load Angle Factor,				
$\theta = \operatorname{atan}\left(\frac{\operatorname{Hc}}{\operatorname{Vc}}\right)$	θ = 43.02	5 deg		
$\mu = 1.0 + 0.50 \cdot \sin(\theta)^{1.5}$	μ = 1.282			
For Weld,				
Rv3 = Avw·µ·0.6·Fu·sin(45deg) ·nws				
Rv3 = 57.102 ksi				
Maximum effective weld size,				
weff = $\frac{\min(Rv1, Rv2)}{Rv3}$	weff = 0 .	171 in		
Weld Capacity,				
$Rw = \Lambda vw \cdot \mu \cdot 0.6 \cdot Fu \cdot sin(45 deg) \cdot nws \cdot min(websilon)$	ff, w)			
Rw = 9.787 kips/in	Ruw = 0.7	55 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
```

```
θ ≥ 30deg
```

Angle of Plate Load to the HSS Column Face,

```
\thetain = 90 deg
```

Connection is applicable, OK



Job Code:	ҮҮҮҮ		
Job Name:	NASCC 2019		
Sheet No.:	28 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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 ≤ 52 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,

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

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

t = 0.375 in

Fy = 46 ksi

Plate thickness < Maximum Plate Thickness, OK

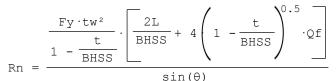
b. HSS Wall Plastification Capacity

```
Job Code:
                                                                      YYYY
                            GIZA Steel
                                                           Job Name:
                                                                      NASCC 2019
            1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                          Sheet No.:
                                                                      29 of 78
                        Email: info@gizasteel.com
                                                           Designed by:
                                                                      RCM
                                                                                          03/28/2019
                         www.gizasteel.com
                                                          Revision No:
                                                                      00
                                                                                    Date:
G I Z A<sup>™</sup>
                                                           Subject:
                                                                      V1H-C1DD
        (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}{Aq \cdot Fv}
                                                           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)
                  -PUR - MUR + ULUR
                                                       (Axial in Compression, Moment in Compression)
                -PUR + MUR + ULUR
PUR + MUR + ULUR
                                                       (Axial in Compression, Moment in Tension)
          n =
                                                      (Axial in Tension, Moment in Tension)
                 PUR - MUR + ULUR
                                                       (Axial in Tension, Moment in Compression)
         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∈ 0..3
            if n_i < 0 then Cs_i = 0.20
            if n_i \ge 0 then Cs_i = 0.10
           for ie 0..3
            x_i = (1 - |n_i|)^{Cs_i}
          Qf = min(x_i)
                                                           Of = 1
         Branch Angle from the HSS Chord Face,
           \theta = 88.429 \text{ deg}
```

1801 F	GIZA Steel Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com
G I Z A [™]	www.gizasteel.com

Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	30 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

Nominal HSS Wall Plastification Capacity,



HSS Wall Plastification Capacity,

RpHSS = ApHSS ·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,

 $Mb = 57.003 \text{ kips} \cdot \text{in}$

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,

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



Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	31 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

Average Force on Welds Per Unit Length,

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

$$fave = 1.242 \text{ kips/in}$$

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

Ruw = max(fmax, 1.25 fave)Ruw = 1.641 kips/inShear Strength,Ruw = 1.641 kips/inFor Beam,Rv1 = Avr
$$\cdot 0.6 \cdot Fu \cdot tf \cdot nws$$
Rv1 = 48.847 kips/inFor Gusset Plate,Rv2 = 9.787 kips/inEffective Load Angle Factor,Rv2 = 9.787 kips/in $\theta = atan \left(\frac{Vb + \frac{4 \cdot Mb}{Lw}}{Hb} \right)$ $\theta = 61.027 deg$ $\mu = 1.0 + 0.50 \cdot sin(\theta)^{1.5}$ $\mu = 1.409$

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

Weld Capacity,

```
Rw = Λvw·μ·0.6·Fu·sin(45deg) ·nws·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
```

	1				
	Job Code:	<u>YYYY</u>			
GIZA Steel	Job Name:	NASCC 2019			
1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com	Sheet No.: Designed by:	32 of 78 RCM			
unus sizesteel com	Revision No:	00 Date: 03/28/2019			
GIZA [™] www.gizasteei.com	Subject:	V1H-C1DD			
Shear Yielding Capacity, (J4-3)	1				
Rvy = Avy·n1·0.6·Fy·L·t					
Rvy = 132.131 kips	Hb = 12.9	7 kips			
Shear Yielding Capacity > Applied Force, U		-			
(AISC 14th Ed. Specifications, Chapter J, Se					
b. Tensile Yielding Capacity due to Axial Lo	ad				
Length,					
L = Lw	L = 16.31	2 in			
Equivalent Normal Force,					
$Nb = Vb + \frac{4 \cdot Mb}{L}$	Nb = 23.4	25 kips			
Gross Tension Area,					
$Ag = L \cdot t$					
Tensile Yielding Capacity, (J4-1)					
Rty = Aty ·n ·Fy ·Ag					
Rty = 198.197 kips	Nb = 23.4	25 kips			
Tensile Yielding Capacity > Applied Force,	UCV = 0.11	.8, ок			
Interaction of Yielding Capacities,					
$\left(\frac{Hb}{Rvy}\right)^2 + \left(\frac{Nb}{Rty}\right)^2 \le 1.0$					
$UCV = \left(\frac{Hb}{Rvy}\right)^2 + \left(\frac{Nb}{Rty}\right)^2$	UCV = 0.0	24			
Yielding Capacity > Applied Force, $UCV = 0$.024, OK				
H. BEAM WEB CHECK DUE TO GUSSET PLATE 1 STRES	SES				
1. Force Acting on Connection					
Equivalent Normal Force Acting on the Conn	ection,				
Nb = 23.425 kips					
2. Web Local Yielding Capacity					
(AISC 14th Ed. Specifications, Chapter J, Se	ction J10.	2, page 16.1-134)			
Distance of Force to Beam End,					
$De = 0.5 \cdot Lw$	De = 8.15	6 in			
Bearing Length,					
N = Lw	N = 16.31	2 in			
Web Local Yielding Capacity, (J10-2, J10-3)					
$De \leq d$					



Job Code:	ҮҮҮҮ		
Job Name:	NASCC 2019		
Sheet No.:	33 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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

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)

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

$$Esq = 1533.288 \text{ ksi}$$

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

$$N2 = 2.38$$

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

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

$$Rwc = 290.361 \text{ kips}$$

$$Nb = 23.425 \text{ kips}$$

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

4. Web Horizontal Shear Capacity

Force Acting on the Beam,

Horizontal Shear Force,

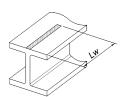
$$Vw = Hb \cdot \left(1 - \frac{tf \cdot bf}{Ag}\right)$$
 $Vw = 9.26$ kips

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





Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	34 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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

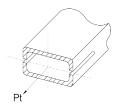
Net Tension Area,

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

Ant = 6.531 in^2

 $Ae = 4.428 in^{2}$

Lcon = 6 in



Eccentricity of the Connection,

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

Reduction Coefficient,

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

Effective Net Tension Area,

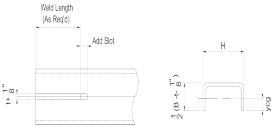
Ae = U·Ant

Tensile Rupture Capacity, (D2-2)

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

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)

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



Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	35 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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

$$\frac{b}{tw} = 6.447$$

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,

$$A1 = 2 \cdot (b \cdot tw)$$
 $A1 = 1.57 \text{ in}^2$ $A2 = (H - 2 \cdot tw) \cdot tw$ $A2 = 1.85 \text{ in}^2$ $Ag = A1 + A2$ $Ag = 3.421 \text{ in}^2$

Centroid,

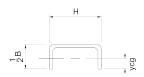
$$ycg = \frac{A1 \cdot \left(\frac{1}{2} \cdot b\right) + A2 \cdot \left(b - \frac{1}{2} \cdot tw\right)}{Ag} \qquad ycg = 1.639 \text{ in}$$

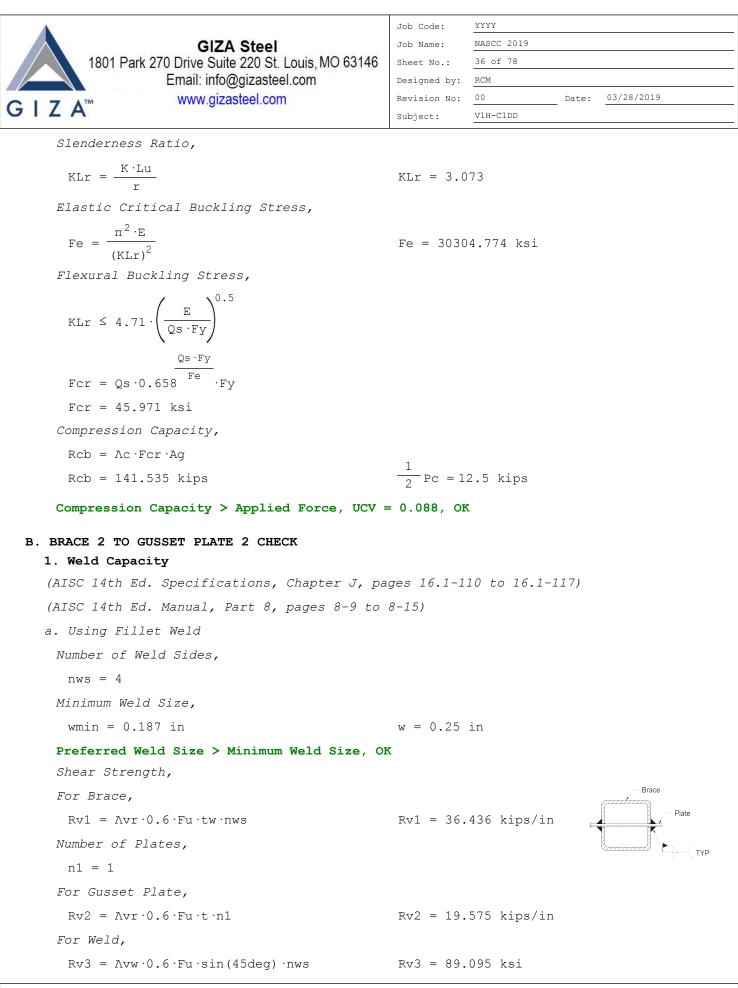
Moment of Inertia,

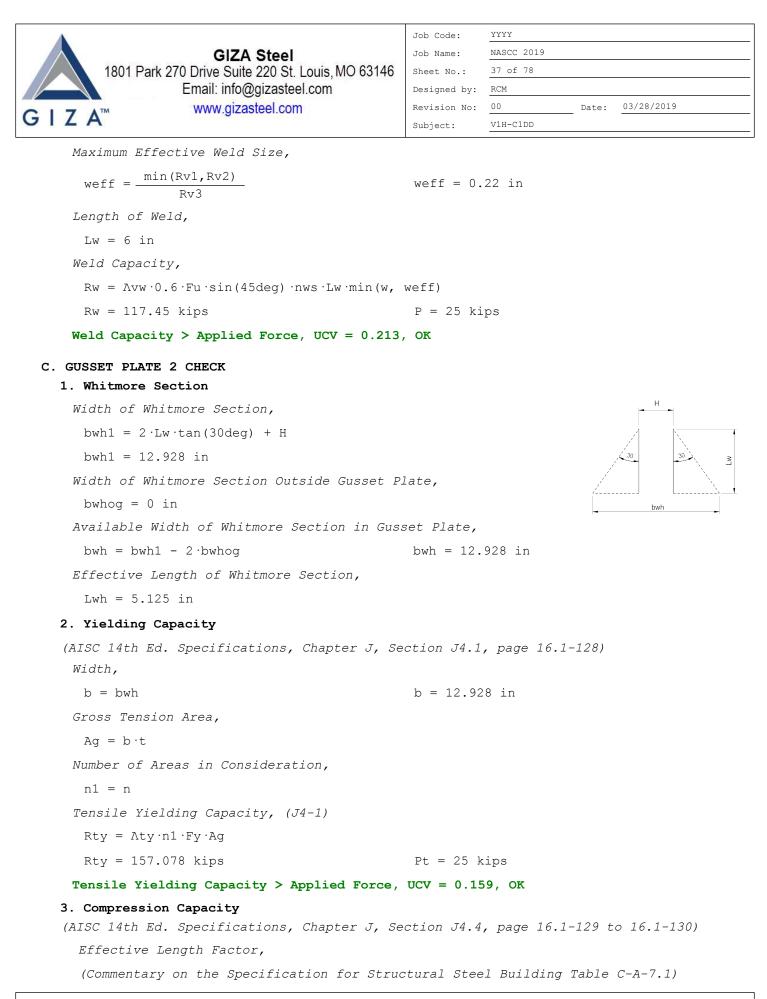
I = I1 + I2

Radius of Gyration,

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







GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146	Job Code:	<u>YYYY</u>		
	Job Name: Sheet No.:	NASCC 2019 38 of 78		
Email: info@gizasteel.com	Designed by:	RCM		
GIZA [™] www.gizasteel.com	Revision No:	00 Date: 03/28/2019		
GILA	Subject:	V1H-C1DD		
K = 0.65				
Laterally Unbraced Length,				
Lu = Lwh	Lu = 5.12	5 in		
Gross Area,				
$Ag = bwh \cdot t$	Ag = 4.84	8 in ²		
Radius of Gyration,				
$r = \frac{t}{(12)^{0.5}}$	r = 0.108	in		
Slenderness Ratio,				
$KLr = \frac{K \cdot Lu}{r}$	KLr = 30.	773		
Elastic Critical Buckling Stress,				
$Fe = \frac{\pi^2 \cdot E}{KLr^2}$	Fe = 302.	249 ksi		
Flexural Buckling Stress,				
KLr > 25				
$KLr \leq 4.71 \cdot \left(\frac{E}{FY}\right)^{0.5}$				
$Fcr = 0.658 \frac{Fy}{Fe} \cdot Fy$	Fcr = 34.2	249 ksi		
Number of Areas in Consideration,				
n1 = n				
Compression Capacity,				
Rcb = Ac·nl·Fcr·Ag				
Rcb = 149.439 kips	Pc = 25 k	ips		
Compression Capacity > Applied Force, UCV	= 0.167, 0	K		
4. Block Shear Capacity				
(AISC 14th Ed. Specifications, Chapter J, Se	ction J4.3	, page 16.1-129)		
Reduction Factor,				
Ubs = 1.0	(tension	stress is uniform)		
Gross Shear Area,				
$Agv = 2 \cdot Lw \cdot t$	Agv = 4.5	jin²		
Net Tension Area,		*		
Ant = $H \cdot t$	Ant = 2.2	25 in ²		
Net Shear Area,		<i>'</i>		
Anv = Agv	Anv = 4.5	in²		
Number of Areas in Consideration,				



Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	39 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

n1 = n

Block Shear Capacity, (J4-5)

Rbs = Abs ·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

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

D. GUSSET PLATE 2 FORCE DISTRIBUTION

1. Gusset Plate Edge Forces

Connecting Face of HSS,

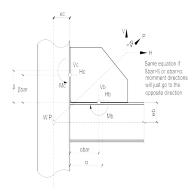
Uniform Force Method

BHSS = B

BHSS = 6 in

HHSS = H

HHSS = 14 in



Beam,Column,
$$eb = 0.5 \cdot d$$
 $ec = 0.5HHSS$ Horizontal Side,Vertical Side, $\alpha bar = 0.5 \cdot Lw + gap$ $\beta bar = 0.5 \cdot (nr - 1) \cdot s + y$ $\alpha bar = 8.656$ in $\beta bar = 7.5$ in $\alpha = (\beta bar + eb) \cdot tan(\theta) - ec$ $\beta = \beta bar$ $\alpha = 14.69$ in $\beta = 7.5$ in $r = \frac{P}{\left[(\alpha + ec)^2 + (\beta + eb)^2\right]^{0.5}}$ $r = 0.883$ kips/inHorizontal Side,Vertical Side,Hb = $\alpha \cdot r$ Hc = ec $\cdot r$ Hb = 12.97 kipsHc = 6.181 kips

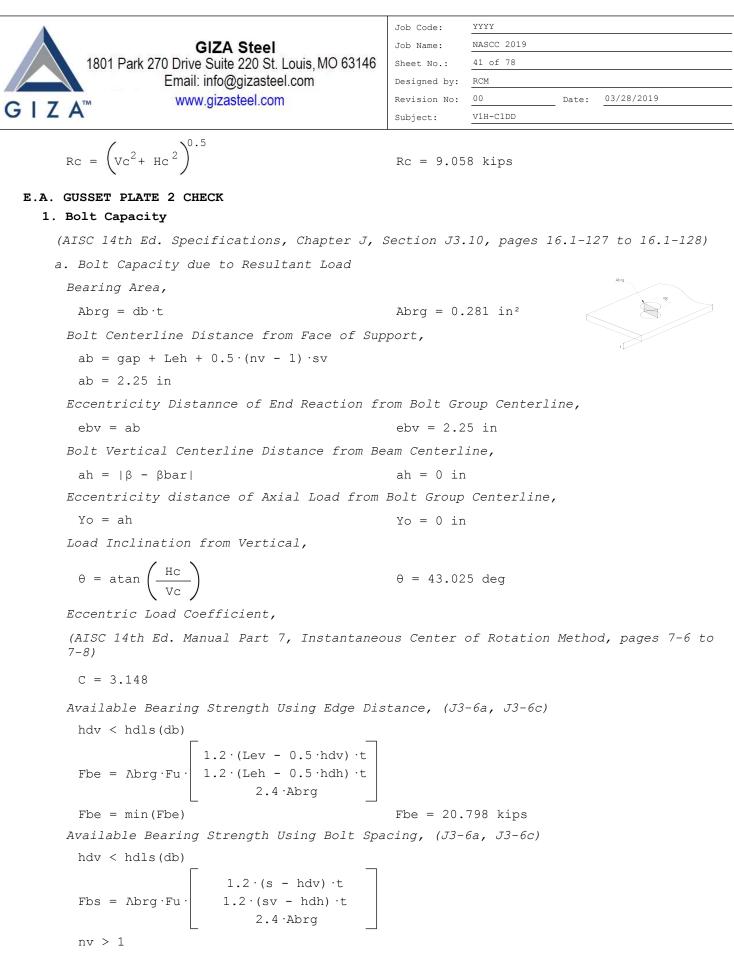
1801 Park 270 Drive Email IZA™ Vb = eb · r Vb = 9.448 kips	GIZA Steel e Suite 220 St. Louis, MO 63146 : info@gizasteel.com w.gizasteel.com	Job Code: Job Name: Sheet No.: Designed by: Revision No: Subject: $Vc = \beta \cdot c$ $Vc = 6 \cdot c$	622 kips	Date:03/28/2019
$Mb = Vb \cdot (\alpha - \alpha) $			c·(β – βba	r)
Mb = 57.003 kip Redistribution of		Mc = 0	kips ·in	
	Ab W.P. W.P. Ab W.P. Kb-(Vb-ΔV) Ca a.) Column Free Body Diagram	Hc (Vc+ΔV) b.) Gusset Free Body Diagram Ab ⁺ (H-Hb) b.) Beam Free Body Diagram	Hb Rb	
Shear Transfer,				
$\Delta V = 0$ kips	and the second			
ΔV = 0 kips Gusset-to-Beam Co				
$\Delta V = 0$ kips Gusset-to-Beam Co $Vb = Vb - \Delta V$	Hb = Hb	king		$Mb = \Delta V \cdot \alpha bar + Mb $
$\Delta V = 0$ kips Gusset-to-Beam Co $Vb = Vb - \Delta V$ Vb = 9.448 kips	Hb = Hb Hb = 12.97	kips		$Mb = \Delta V \cdot \alpha bar + Mb $ $Mb = 57.003 \text{ kips} \cdot \text{in}$
$\Delta V = 0$ kips Gusset-to-Beam Co $Vb = Vb - \Delta V$	Hb = Hb $Hb = 12.97$ Connection,	kips]	

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,



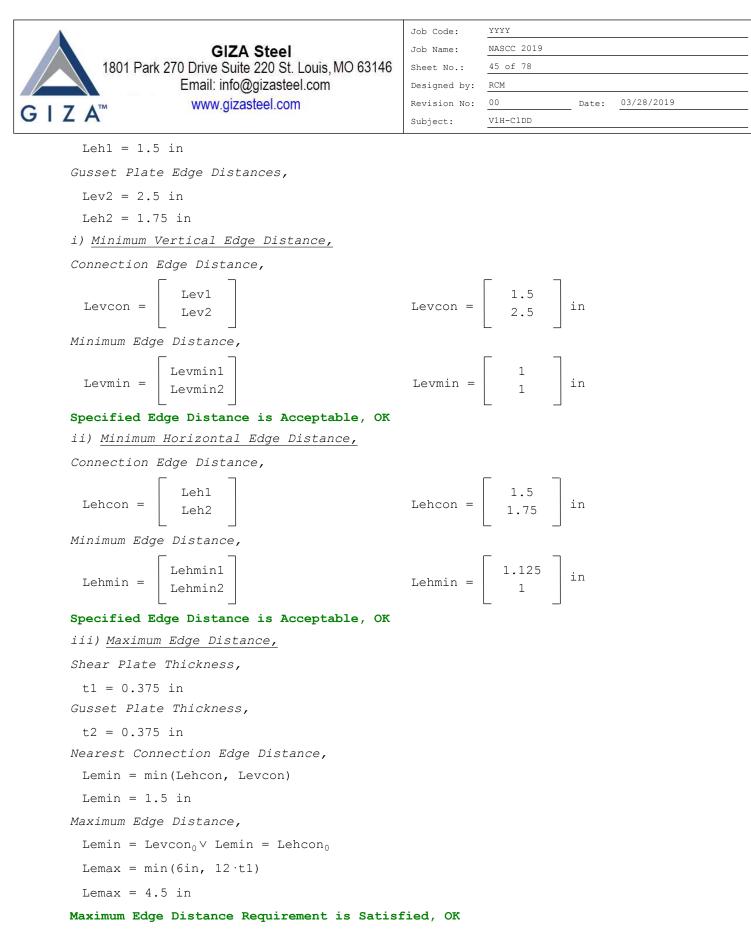
Fbs	=	min	(Fbs ₀ ,	Fbs_2)

Fbs = 29.362 kips

```
Job Code:
                                                                      YYYY
                             GIZA Steel
                                                           Job Name:
                                                                      NASCC 2019
            1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                           Sheet No.:
                                                                       42 of 78
                        Email: info@gizasteel.com
                                                           Designed by:
                                                                      RCM
                                                                                           03/28/2019
                         www.gizasteel.com
                                                                      00
                                                           Revision No:
                                                                                    Date:
G I Z A<sup>™</sup>
                                                           Subject:
                                                                       V1H-C1DD
         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 = 4.312 in^{2}
         Agv = [(nr - 1) \cdot s + Lev] \cdot t
       Net Tension Area,
         Ant = [Leh + (nv - 1) \cdot sv - (nv - 0.5) \cdot hdh] \cdot t
         Ant = 0.492 \text{ in}^2
       Net Shear Area,
                                                                                                 Lev
         Anv = Agv - [(nr - 0.5) \cdot hdv] \cdot t
                                                         Anv = 3.164 in<sup>2</sup>
                                                                                                      (nr-1)@sv
       Number of Areas in Consideration,
         n1 = n
                                                                                             (nv-1)@sv
       Block Shear Capacity, (J4-5)
         Rbss = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·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
```

```
Job Code:
                                                                  YYYY
                       GIZA Steel
                                                      Job Name:
                                                                  NASCC 2019
      1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                      Sheet No.:
                                                                  43 of 78
                  Email: info@gizasteel.com
                                                      Designed by:
                                                                  RCM
                                                                                       03/28/2019
                    www.gizasteel.com
                                                                  00
                                                      Revision No:
                                                                                Date:
ZAM
                                                      Subject:
                                                                  V1H-C1DD
 Reduction Factor,
   ah = 0 in
                                                       (tension stress is uniform)
   Ubs = 1.0
                                                                                                Lev
 Gross Shear Area,
                                                                                                      (nr-1)@sv
   Agv = [Leh + (nv - 1) \cdot sv] \cdot t
                                                      Aqv = 0.656 in^{2}
 Net Tension Area,
   Ant = [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hdv] \cdot t
                                                                                     Leh (nv-1)@sv
   Ant = 3.164 \text{ in}^2
 Net Shear Area,
   Anv = Aqv - [(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 = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)
   Rbs1 = 148.268 kips
                                                      Hc = 6.181 \text{ kips}
 Block Shear Capacity > Applied Force, UCV = 0.042, OK
Pattern 2
 Reduction Factor,
   ah = 0 in
                                                       (tension stress is uniform)
   Ubs = 1.0
                                                                                                   Le<
 Gross Shear Area,
                                                                                                      (nr-1)@s
   Aqv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot t
                                                    Aqv = 1.312 in^{2}
                                                                                                   >0
 Net Tension Area,
   Ant = [(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t
                                                                                        (nv-1)@sv
                                                                                     Leh
   Ant = 2.391 \text{ in}^2
 Net Shear Area,
   Anv = Aqv - 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 = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)
   Rbs2 = 125.255 kips
                                                      Hc = 6.181 \text{ kips}
 Block Shear Capacity > Applied Force, UCV = 0.049, OK
 Governing Block Shear Capacity,
   Rbs = min(Rbs1, Rbs2)
```

	Job Code: YYYY
GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com www.gizasteel.com	Job Name: NASCC 2019
	NITE/WS-
	Designed by: RCM Revision No: 00 Date: 03/28/2019
GIZA [™] www.gizasteel.com	Subject: V1H-C1DD
Rbs = 125.255 kips	Hc = 6.181 kips
-	-
Block Shear Capacity > Applied Force, U	zv = 0.049, 0K
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,	
$\theta = \operatorname{atan}\left(\frac{\operatorname{Hc}}{\operatorname{Vc}}\right)$	θ = 43.025 deg
Eccentric Load Coefficient,	
(AISC 14th Ed. Manual Part 7, Instanta 7-12)	neous Center of Rotation Method, pages 7-6 to
C = 3.148	
Shear Capacity Per Bolt,	
Arv = 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, U	_
	CV = 0.357, OK
	Section J3.3 and J3.5, pages 16.1-122 to 16.
1-124) Shear Plate Thickness,	
t1 = 0.375 in	
Gusset Plate Thickness,	
$t_2 = 0.375$ in	
a. Vertical Spacing,	
Minimum Bolt Spacing,	
s = 3 in 2	
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, O	ĸ
3. Check for Edge Distance	
(AISC 14th Ed. Specifications, Chapter J, 1-124)	Section J3.4 and J3.5, pages 16.1-122 to 16.
Shear Plate Edge Distances,	
Lev1 = 1.5 in	



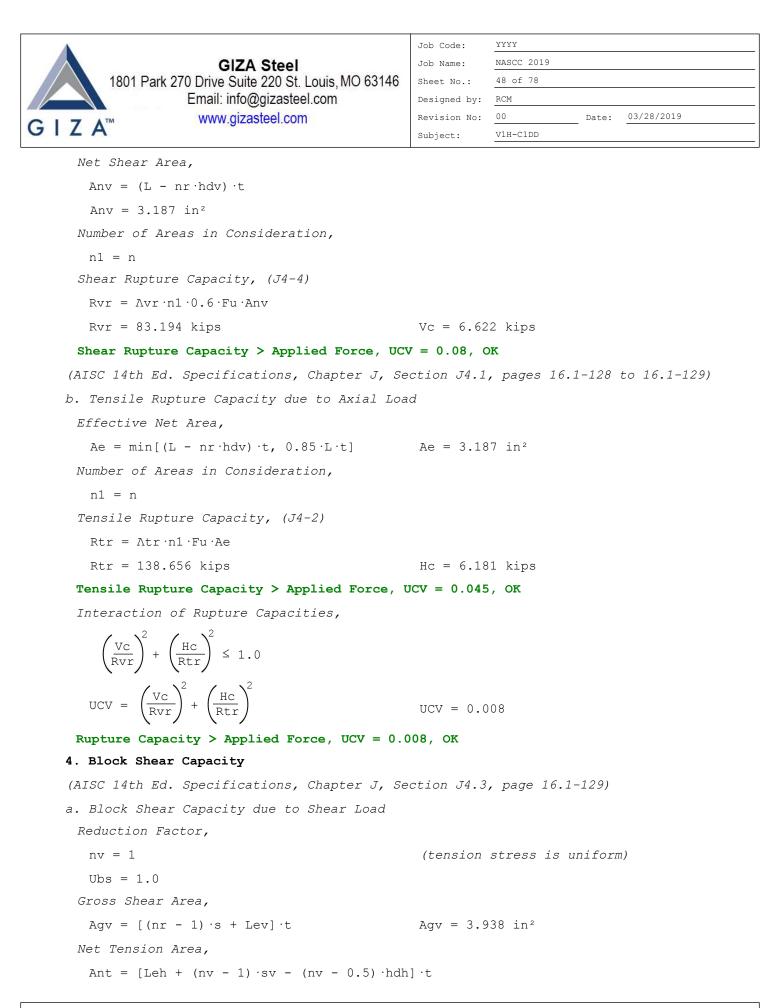
E.C. GUSSET SHEAR PLATE 2 CHECK

1. Bolt Capacity

Job Code: YYYY **GIZA Steel** Job Name: NASCC 2019 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Sheet No.: 46 of 78 Email: info@gizasteel.com Designed by: RCM 03/28/2019 www.gizasteel.com Revision No: 00 Date: Z ATM Subject: V1H-C1DD (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, $Abrg = db \cdot t$ $Abrg = 0.281 in^{2}$ Bolt Centerline Distance from Face of Support, $ab = gap + Leh + 0.5 \cdot (nv - 1) \cdot sv$ ab = 2.25 inEccentricity Distannce of End Reaction from Bolt Group Centerline, ebv = abebv = 2.25 inBolt Vertical Centerline Distance from Beam Centerline, $ah = |\beta - \beta bar|$ ah = 0 inEccentricity distance of Axial Load from Bolt Group Centerline, Yo = ahYo = 0 in Load Inclination from Vertical, $\theta = \operatorname{atan}\left(\frac{\operatorname{Hc}}{\operatorname{Vc}}\right)$ $\theta = 43.025 \, \text{deg}$ Eccentric Load Coefficient, (AISC 14th Ed. Manual Part 7, Instantaneous Center of Rotation Method, pages 7-6 to 7-8) C = 3.148Available Bearing Strength Using Edge Distance, (J3-6a, J3-6c) hdv < hdls(db)1.2 · (Lev - 0.5 · hdv) ·t 1.2 · (Leh - 0.5 · hdh) ·t $Fbe = \Lambda brg \cdot Fu \cdot$ 2.4 ·Abrq Fbe = min(Fbe)Fbe = 18.963 kips Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c) hdv < hdls(db)1.2 · (s - hdv) ·t 1.2 · (sv - hdh) ·t Fbs = Abrg · Fu · 2.4 ·Abrq nv > 1 Fbs = 29.362 kips $Fbs = min(Fbs_0, Fbs_2)$ Number of Area in Consideration, n1 = nBolt Capacity, Rbrg = $C \cdot \min(n1 \cdot Fbe, n1 \cdot Fbs, n \cdot \Lambda rv)$



a. Shear Rupture Capacity due to Shear Load



```
Job Code:
                                                                           YYYY
                               GIZA Steel
                                                               Job Name:
                                                                           NASCC 2019
             1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                               Sheet No.:
                                                                            49 of 78
                         Email: info@gizasteel.com
                                                               Designed by:
                                                                           RCM
                                                                           00
                                                                                                 03/28/2019
                           www.gizasteel.com
                                                               Revision No:
                                                                                          Date:
G I Z A<sup>™</sup>
                                                                            V1H-C1DD
                                                               Subject:
         Ant = 0.363 \text{ in}^2
       Net Shear Area,
                                                                                                        ev.
         Anv = Agv - [(nr - 0.5) \cdot hdv] \cdot t
                                                              Anv = 2.789 in^2
                                                                                                             (nr-1)@sv
       Number of Areas in Consideration,
         n1 = n
                                                                                                    (nv-1)@sv
       Block Shear Capacity, (J4-5)
         Rbss = Abs·nl·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)
                                                               Vc = 6.622 kips
         Rbss = 79.59 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
                                                               (tension stress is uniform)
         Ubs = 1.0
       Gross Shear Area,
                                                                                                                 (nr-1)@sv
         Aqv = [Leh + (nv - 1) \cdot sv] \cdot t
                                                             Aqv = 0.563 in^{2}
       Net Tension Area,
         Ant = [(nr - 1) \cdot s + Lev - (nr - 0.5) \cdot hdv] \cdot t
                                                                                               Leh (nv-1)@sv
         Ant = 2.789 \text{ in}^2
       Net Shear Area,
         Anv = Aqv - [(nv - 0.5) \cdot hdh] \cdot t
                                                             Anv = 0.363 in^{2}
       Number of Areas in Consideration,
         n1 = n
       Block Shear Capacity, (J4-5)
         Rbsa = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant, 0.6·Fy·Agv + Ubs·Fu·Ant)
         Rbsa = 130.437 kips
                                                               Hc = 6.181 \text{ kips}
       Block Shear Capacity > Applied Force, UCV = 0.047, OK
        Interaction of Bolt Shear Capacities,
         \left(\frac{\text{Vc}}{\text{Rbss}}\right)^2 + \left(\frac{\text{Hc}}{\text{Rbsa}}\right)^2 \leq 1.0
         UCV = \left(\frac{Vc}{Rbss}\right)^2 + \left(\frac{Hc}{Rbsa}\right)^2
                                                               UCV = 0.009
       Block Shear Capacity > Applied Force, UCV = 0.009, OK
      Pattern 2
```

GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com	Job Code: YYYY Job Name: NASCC 2019
	Job Name: NASCC 2019 Sheet No.: 50 of 78
	Designed by: RCM
www.gizastaal.com	Revision No: 00 Date: 03/28/2019
GIZA [™]	Subject: VIH-C1DD
Reduction Factor,	
ah = 0 in	(tension stress is uniform)
Ubs = 1.0	õ S
Gross Shear Area,	v → → → − 0 → 0
$Agv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot t$	Agv = 1.125 in ²
Net Tension Area,	
Ant = $[(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot t$	Leh (nv-1)@sv
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 = Abs n1 min(0.6 Fu Anv + Ubs Fu Ant	, 0.6 ·Fy ·Agv + Ubs ·Fu ·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,	
$\lambda = \frac{L \cdot Fy^{0.5}}{10 \cdot t \cdot \left[475 + 280 \cdot \left(\frac{L}{ab}\right)^2\right]^{0.5}} \cdot \frac{1}{k \text{si}^{0.5}}$	$\lambda = 0.209$
$\lambda \leq 0.7$	

$$Q = 1$$

Allowable Flexural Local Buckling Stress or Yielding Stress,

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

Gross Plastic Section Modulus,

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

$$Zx = 13.5 \text{ in}^3$$
Eccentricity,

e = ab e = 2.25 in



Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	51 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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,

$$mod(nr, 2) = 0$$

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

$$Znet = 9.563 \text{ in}^{3}$$
Flexural Rupture Capacity,

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

$$Rfr = 184.875 \text{ kips} \qquad Vc = 6.622 \text{ kips}$$
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,

GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com	Job Code:	<u>YYYY</u>		
	Job Name:	NASCC 2019 52 of 78		
	Sheet No.: Designed by:	RCM		
unuu gizantaal aam	Revision No:	00 Date: 03/28/2019		
GIZA [™] www.gizasteer.com	Subject:	V1H-C1DD		
Lu = gap + Le1	Lu = 2.25	in		
Gross Area,				
Ag = L·t	Ag = 4.5	in²		
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	Fcr = 36	ksi		
Number of Areas in Consideration,				
n1 = n				
Compression Capacity,				
Rcb = Ac ·n1 ·Fcr ·Ag				
Rcb = 145.8 kips	Hc = 6.181 kips			
Compression Capacity > Applied Force, UCV	= 0.042, 0	к		
E.D. GUSSET SHEAR PLATE 2 TO COLUMN WALL CHEC	ĸ			
1. Weld Capacity				
(AISC 14th Ed. Specifications, Chapter J, pa	ges 16.1-1.	10 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, OF	x			
Length of Weld,				
$Lw = (nr - 1) \cdot s + 2 \cdot Lev$	Lw = 12 i	n		
Total Force Per Unit Length on Welds of She	ear Plate t	o Column Connection,		
$Ruw = \left[\left(\frac{Vc}{Lw} \right)^2 + \left(\frac{Hc}{Lw} \right)^2 \right]^{0.5}$	Ruw = 0.7	55 kips/in		

A		Job Code:	YYYY		
	GIZA Steel	Job Name:	NASCC 2019		
1801 Park 2	70 Drive Suite 220 St. Louis, MO 63146	Sheet No.:	53 of 78		
	Email: info@gizasteel.com	Designed by:	RCM		
G I Z A [™]	www.gizasteel.com	Revision No:	00	Date:	03/28/2019
		Subject:	V1H-C1DD		
Shear Strengt	h,				
For Column,					
Rv1 = Avr·	0.6 ·Fu ·tw ·nws	Rv1 = 30.	328 kips/in		
For Shear P	late,				
Rv2 = Avr·	0.6 ·Fu ·t	Rv2 = 9.7	87 kips/in		
Effective Lo	oad Angle Factor,				
$\theta = \operatorname{atan}\left(-\right)$	Hc Vc	θ = 43.02	5 deg		
$\mu = 1.0 +$	0.50 ·sin(θ) ^{1.5}	μ = 1.282			
For Weld,					
Rv3 = Avw·	µ·0.6·Fu·sin(45deg) ·nws				
Rv3 = 57.1	02 ksi				
Maximum effe	ective weld size,				
weff = <u>min</u>	Rv1, Rv2) Rv3	weff = 0 .	171 in		
Weld Capacit	ty,				
Rw = Avw·µ	·0.6·Fu·sin(45deg) ·nws·min(wet	ff, w)			
Rw = 9.787	kips/in	Ruw = 0.7	55 kips/in		
Weld Capacit	ty > Applied Force, UCV = 0.07	7, OK			
F. COLUMN WALL CH	IECK				
1. HSS Local Ch	neck				
a. HSS Punchi	ng Shear				
(AISC Specifi	cation for the Design of Steel	l Hollow St	ructural Sec	ctions	s, page 15)
Thickness of	Shear Plate,				
t = 0.375 ir	L				
Maximum Norma	l Stress in the Plate,				
$Nmax = \frac{Hc}{t \cdot L}$		Nmax = 1.	373 ksi		
Maximum Shear	Plate Thickness to Avoid Shea	ar Tab Punc	ching Thru Co	olumn	Wall,
$tPSmax = \frac{1.2}{2}$	2·Avr·Fu·tw				
tPSmax = 24.	.535 in	t = 0.375	in		
Plate thickne	ss < Maximum Plate Thickness,	OK			
	Plastification Capacity				
	. Specifications, Chapter K, D	Table K1.2,	page 16.1-1	144)	
Axial Load H					
PuHSS = Hb	m	PuHSS = 0	kips		

Job Code: YYYY **GIZA Steel** Job Name: NASCC 2019 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Sheet No.: 54 of 78 Email: info@gizasteel.com Designed by: RCM 03/28/2019 www.gizasteel.com Revision No: 00 Date: G I Z A[™] Subject: V1H-C1DD For Required Axial Strength, Code = LRFD $PUR = \frac{P}{Aq \cdot Fy}$ PUR = 0For Required Flexural Strength, Code = LRFD $MUR = \frac{M}{S \cdot Fy}$ MUR = 0For Uplift Force (if any), Code = LRFD $ULUR = \frac{PUplift}{Aq \cdot Fv}$ ULUR = 0Utilization Ratio, (CIDECT Design Guide 3 Second Edition, Table 7.1, page 78) -PUR - MUR + ULUR (Axial in Compression, Moment in Compression) -PUR + MUR + ULUR PUR + MUR + ULUR PUR - MUR + ULUR (Axial in Compression, Moment in Tension) n = (Axial in Tension, Moment in Tension) (Axial in Tension, Moment in Compression) Coefficient of Chord Stress Functions, if n < 0 then Cs = 0.20if $n \ge 0$ then Cs = 0.10Chord-stress Interaction Parameter, for ie 0..3 if $n_i < 0$ then $Cs_i = 0.20$ if $n_i \ge 0$ then $Cs_i = 0.10$ for ie 0..3 $x_{i} = (1 - |n_{i}|)^{Cs_{i}}$ $Qf = min(x_i)$ Of = 1Branch Angle from the HSS Chord Face, $\theta = 88.429 \text{ deg}$ Nominal HSS Wall Plastification Capacity, $\frac{2L}{BHSS} + 4$ $1 - \frac{t}{BHSS}$ ____. ·Qf BHSS Rn = $sin(\theta)$ HSS Wall Plastification Capacity, RpHSS = ApHSS ·Rn

	Job Code:	<u>YYYY</u>		
GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146	Job Name: Sheet No.:	NASCC 2019 55 of 78		
Email: info@gizasteel.com	Designed by:	RCM		
www.gizactool.com	Revision No:	00 Date: 03/28/2019		
GIZA [™] www.gizasteel.com	Subject:	V1H-C1DD		
RpHSS = 130.449 kips	HcB = 6.1	81 kips		
Wall Plastification Capacity > Applied Fo	orce, UCV =	0.047, ок		
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 = 16.0	46 kips		
2. Weld Capacity				
(AISC 14th Ed. Specifications, Chapter J, pa	ages 16.1-1	10 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, O				
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(\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} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{4 \cdot Mb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{Hb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{Hb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{Hb}{Lw^2} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{Lw} + \frac{Hb}{Lw} + \frac{Hb}{Lw} \right)^2 \right]^{0.5} + \left[\left(\frac{Hb}{L$	$\left(\frac{Hb}{Lw}\right)^2 + \left(\frac{Vb}{Lw}\right)^2$	$-\frac{4 \cdot Mb}{Lw^2} \bigg]^2 \bigg]^{0.5}$		
fave = 1.242 kips/in		—		
Total Force Per Unit Length on Welds of Gu	sset Plate	to Beam Connection,		
Ruw = max(fmax, 1.25 · fave)	Ruw = 1.6	41 kips/in		
Shear Strength,				
For Beam,				
Rv1 = Avr·0.6·Fu·tf·nws	Rv1 = 48.	847 kips/in		
For Gusset Plate,				

```
Job Code:
                                                                                  YYYY
                                 GIZA Steel
                                                                    Job Name:
                                                                                  NASCC 2019
              1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                                    Sheet No.:
                                                                                  56 of 78
                           Email: info@gizasteel.com
                                                                    Designed by: RCM
                                                                                                         03/28/2019
                             www.gizasteel.com
                                                                    Revision No: 00
                                                                                                  Date:
G I Z A<sup>™</sup>
                                                                                  V1H-C1DD
                                                                    Subject:
            Rv2 = \Lambda vr \cdot 0.6 \cdot Fu \cdot t
                                                                    Rv2 = 9.787 kips/in
          Effective Load Angle Factor,
            \theta = \operatorname{atan}\left(\frac{\operatorname{Vb} + \frac{4 \cdot \operatorname{Mb}}{\operatorname{Lw}}}{\operatorname{Hb}}\right)
                                                                    \theta = 61.027 \text{ deg}
            \mu = 1.0 + 0.50 \cdot \sin(\theta)^{1.5}
                                                                    \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}
                                                                  weff = 0.156 in
          Weld Capacity,
            Rw = \Lambda vw \cdot \mu \cdot 0.6 \cdot Fu \cdot sin(45 deg) \cdot nws \cdot min(weff, w)
            Rw = 9.787 \text{ kips/in}
                                                                    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 = 16.312 in
          L = Lw
        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
                                                                    Hb = 12.97 \text{ 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
                                                                    L = 16.312 in
        Equivalent Normal Force,
          Nb = Vb + \frac{4 \cdot Mb}{T}
                                                                    Nb = 23.425 \text{ kips}
        Gross Tension Area,
```



GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com

Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	57 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

$Ag = L \cdot t$

Tensile Yielding Capacity, (J4-1)

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

Rty = 198.197 kips

Nb = 23.425 kips

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

Interaction of Yielding Capacities,

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

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 = $0.5 \cdot Lw$ De = 8.156 in Bearing Length, N = Lw N = 16.312 in Web Local Yielding Capacity, (J10-2, J10-3) De $\leq d$ 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)

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



GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com

YYYY		
NASCC 2019		
58 of 78		
RCM		
00	Date:	03/28/2019
V1H-C1DD		
	NASCC 2019 58 of 78 RCM 00	NASCC 2019 58 of 78 RCM 00 Date:

$$N2 = 1 + \left(\frac{4 \cdot N}{d} - 0.2\right) \cdot \left(\frac{tw}{tf}\right)$$
$$De < \frac{d}{2} \wedge \frac{N}{d} > 0.2$$

N2 = 2.38

 $Rwc = Acr \cdot 0.4 \cdot tw^2 \cdot N2 \cdot Esq$

Rwc = 290.361 kips

Nb = 23.425 kips

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

1.5

4. Web Horizontal Shear Capacity

Force Acting on the Beam,

Horizontal Shear Force,

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

Web Horizontal Shear Capacity,

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

Rv = 252.028 kips

Vw = 9.26 kips

Vw = 9.26 kips

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

I. BEAM WEB TO COLUMN WALL CHECK

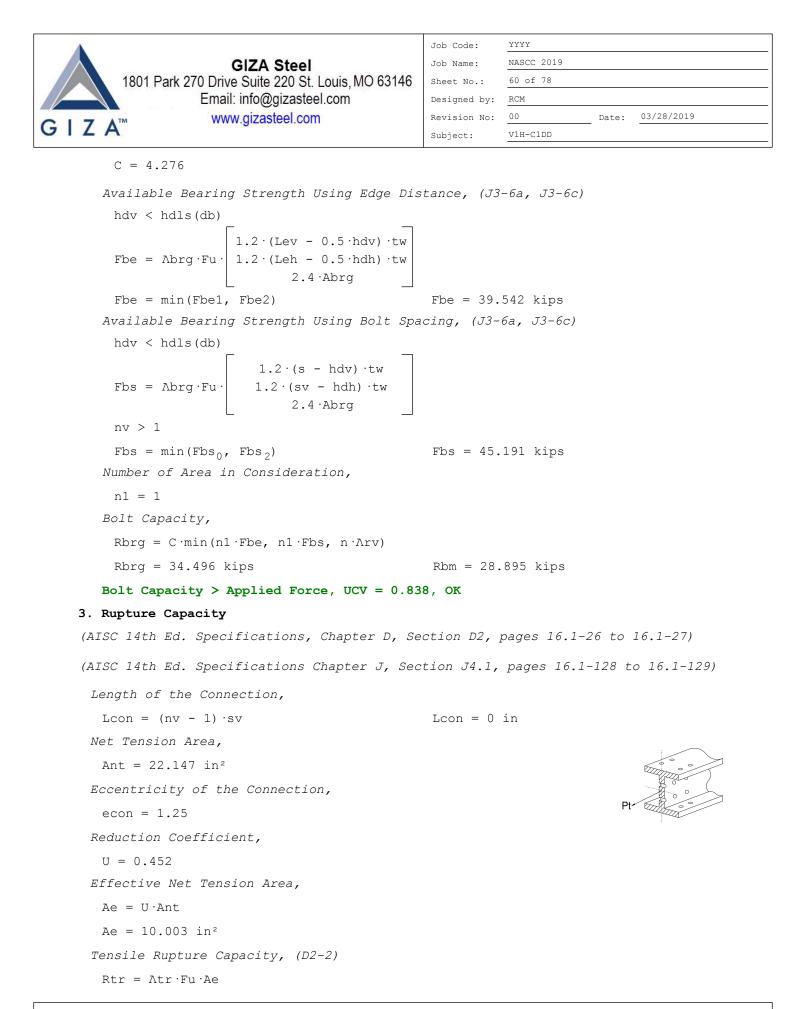
1. Forces Acting on Connection

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, $\theta 2 = 50 \text{ deg}$ Other Brace Connection Horizontal Force at Gusset-to-Beam Interface, Hb2 = 12.97 kipsVertical Force, Vbm = V + Vb + Vb2Vbm = 28.895 kips Horizontal Force, $Hbm = TF + |(P \cdot sin(\theta) - Hb) - (P2 \cdot sin(\theta 2) - Hb2)|$ Hbm = 0 kipsResultant Force, $Rbm = \left(Vbm^2 + Hbm^2\right)^{0.5}$ Rbm = 28.895 kips

I.A. BEAM WEB CHECK

1. Shear Capacity

	Job Code:	ҮҮҮҮ
GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com	Job Name:	NASCC 2019
	Sheet No.:	59 of 78
	Designed by:	RCM
www.gizastool.com	Revision No:	00 Date: 03/28/2019
GIZA [™]	Subject:	V1H-C1DD
(AISC 14th Ed. Specifications, Chapter G, Se	ction G2.1	, pages 16.1-67 to 16.1-69)
Clear Distance Between Flanges of Beam Les:	s the Fille	
$h = d - 2 \cdot kdes$	h = 18.72	in y
Limiting Depth-Thickness Ratio,		د
$htw = \frac{h}{tw}$	htw = 36.	35 s
Clear Distance Between Transverse Stiffene	rs,	
htw < 260	a = 0 in	
Web Plate Buckling Coefficient, (G2-6)		
htw < 260	kv = 5	
Web Shear Coefficient, $(G2-3, G2-4, G2-5)$		
$htw \leq 1.1 \cdot \left(\frac{kv \cdot E}{Fy}\right)^{0.5}$	Cv = 1	
Shear Capacity, (G2-1)		
Rv = Avbm · 0.6 · Fy · d · tw · Cv		
Rv = 330.63 kips	Vbm = 28.	895 kips
Shear Capacity of Section > Applied Force,	UCV = 0.08	7, ок
2. Bolt Capacity		
(AISC 14th Ed. Specifications, Chapter J, S	Section J3.	10, pages 16.1-127 to 16.1-128)
a. Bolt Capacity due to Resultant Load		
Bearing Area,		Abg
Abrg = db ·tw	Abrg = 0 .	386 in ²
Bolt Centerline Distance from Face of Sup	port,	
$ab = Leh + 0.5 \cdot (nv - 1) \cdot sv$		
ab = 2.25 in		
Eccentricity Distannce of End Reaction fr	om Bolt Gr	oup Centerline,
ebv = ebv = ab	ebv = 2.2	
Bolt Vertical Centerline Distance from Be		
$ah = 0.5 \cdot d - [D + 0.5 \cdot (nr - 1) \cdot s] $		
Eccentricity distance of Axial Load from		
Yo = ah		
	Yo = 1.7	1n
Load Inclination from Vertical,		
$\theta = \operatorname{atan}\left(\frac{\operatorname{Hbm}}{\operatorname{Vbm}}\right)$	$\theta = 0 \deg$	
Eccentric Load Coefficient,		
(AISC 14th Ed. Manual Part 7, Instantanec 7-8)	ous Center	of Rotation Method, pages 7-6 to



	1			
A	Job Code:	<u> </u>		
GIZA Steel	Job Name: Sheet No.:	NASCC 2019		
1801 Park 270 Drive Suite 220 St. Louis, MO 63146		61 of 78		
Email: info@gizasteel.com	Designed by: Revision No:	RCM 00 Date: 03/28/2019		
GIZA [™] www.gizasteel.com	Subject:	00 Date: 03/28/2019 V1H-C1DD		
	_			
Rtr = 487.661 kips	Hbm = 0 k	-		
Tensile Rupture Capacity > Applied Force, N	JCV = 0, OK			
4. Block Shear Capacity				
(AISC 14th Ed. Specifications, Chapter J, Se	ction J4.3	, page 16.1-129)		
Reduction Factor,				
Ubs = 0.5	(tension	stress is non-uniform)		
Gross Shear Area,				
$Agv = 2 \cdot [Leh + (nv - 1) \cdot sv] \cdot tw$	Agv = 1.8	02 in ²		
Net Tension Area,				
Ant = $[(nr - 1) \cdot s - (nr - 1) \cdot hdv] \cdot tw$	Ant = 4.3			
Net Shear Area,		Leh		
Anv = Agv - $2 \cdot [(nv - 0.5) \cdot hdh] \cdot tw$	Anv = 1.3	52 in²		
-				
Number of Areas in Consideration, n1 = 1				
Block Shear Capacity, (J4-5)				
Rbs = Abs·n1·min(0.6·Fu·Anv + Ubs·Fu·Ant,	0.6 ·Fy ·Ag	v + Ubs·Fu·Ant)		
Rbs = 146.244 kips	Hbm = 0 k	ips		
Block Shear Capacity > Applied Force, UCV =	= 0, ОК			
I.B. BEAM WEB TO BEAM SHEAR PLATE CHECK				
1. Bolt Shear Capacity				
(AISC 14th Ed. Specifications, Chapter J, S	Section J3.	6, page 16.1-125)		
Load Inclination from Vertical,				
$\theta = atap (Hbm)$	$\theta = 0 \deg$	r		
$\theta = \operatorname{atan}\left(\frac{\operatorname{Hbm}}{\operatorname{Vbm}}\right)$	0 0 409			
Eccentric Load Coefficient,				
(AISC 14th Ed. Manual Part 7, Instantanec 7-12)	us Center	of Rotation Method, pages 7-6 to		
C = 4.276				
Shear Capacity Per Bolt,				
Arv = 8.068 kips				
Bolt Shear Capacity,				
$Rb = n \cdot C \cdot \Lambda rv$				
Rb = 34.496 kips	Rbm = 28.	895 kips		
-		-		
Bolt Shear Capacity > Applied Force, UCV	- 0.030, 0			
2. Check for Spacing				
(AISC 14th Ed. Specifications, Chapter J, Se	ction J3.3	and J3.5, pages 16.1-122 to 16.		

Description:

1-124)

Г	
	Job Code: YYYY
GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO	Job Name: NASCC 2019 63146 Sheet No.: 62 of 78
Email: info@gizasteel.com	Designed by: RCM
GIZA [™] www.gizasteel.com	Revision No: 00 Date: 03/28/2019
GIZA	Subject: <u>V1H-C1DD</u>
Shear Plate Thickness,	
t1 = 0.375 in	
Beam Web Thickness,	
t2 = 0.515 in	
a. Vertical Spacing,	
Minimum Bolt Spacing,	
s = 3 in	
smin = $2 \frac{2}{3}$ ·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,	ОК
3. Check for Edge Distance	
(AISC 14th Ed. Specifications, Chapter 1-124)	J, Section J3.4 and J3.5, pages 16.1-122 to 16.
Shear Plate Edge Distances,	
Lev1 = 1.5 in	
Leh1 = 1.5 in	
Beam Web Edge Distances,	
Lev2 = NA	
Leh2 = NA	
i) Minimum Vertical Edge Distance,	
Connection Edge Distance,	
Levcon = $\begin{bmatrix} Lev1 \end{bmatrix}$	Levcon = $\begin{bmatrix} 1.5 \end{bmatrix}$ in
Minimum Edge Distance,	
Levmin = [Levmin1]	Levmin = $\begin{bmatrix} 1 \end{bmatrix}$ in
Specified Edge Distance is Acceptable	, ОК
ii) <u>Minimum Horizontal Edge Distance</u> ,	
Connection Edge Distance,	
Lehl	
Lehcon = Leh1 Leh2	Lehcon = $\begin{bmatrix} 1.5\\ 1.75 \end{bmatrix}$ in
Minimum Edge Distance,	
Lehmin1	Lehmin = $\begin{bmatrix} 1.125 \\ 1 \end{bmatrix}$ in
Lehmin = Lehmin1 Lehmin2	Lehmin = 1 1 ¹¹¹

Specified Edge Distance is Acceptable, OK



Job Code:	ҮҮҮҮ		
Job Name:	NASCC 2019		
Sheet No.:	63 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

iii) <u>Maximum Edge Distance</u>,

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 = $Levcon_0 \vee Lemin = Lehcon_0$

Lemax = min(6in, 12·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
                                                    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 Distance of End Reaction from Bolt Group Centerline,
   ebv = ab
                                                    ebv = 2.25 in
 Bolt Vertical Centerline Distance from Beam Centerline,
   ah = |0.5 \cdot d - [D + 0.5 \cdot (nr - 1) \cdot s]|
                                                    ah = 1.7 in
 Eccentricity distance of Axial Load from Bolt Group Centerline,
   Yo = ah
                                                    Yo = 1.7 in
 Load Inclination from Vertical,
   \theta = \operatorname{atan}\left(\frac{\operatorname{Hbm}}{\operatorname{Vbm}}\right)
                                                    \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
```

```
Job Code:
                                                                   YYYY
                         GIZA Steel
                                                       Job Name:
                                                                   NASCC 2019
         1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                       Sheet No.:
                                                                   64 of 78
                    Email: info@gizasteel.com
                                                       Designed by:
                                                                   RCM
                                                                                       03/28/2019
                      www.gizasteel.com
                                                       Revision No:
                                                                   00
                                                                                 Date:
IZA<sup>™</sup>
                                                       Subject:
                                                                   V1H-C1DD
      Available Bearing Strength Using Edge Distance, (J3-6a, J3-6c)
       hdv < hdls(db)
                          1.2 · (Lev - 0.5 · hdv) ·t
                         1.2 · (Leh - 0.5 · hdh) ·t
       Fbe = \Lambda brq \cdot Fu \cdot
                                  2.4 ·Abrg
                                                       Fbe = 18.963 kips
       Fbe = min(Fbe)
      Available Bearing Strength Using Bolt Spacing, (J3-6a, J3-6c)
       hdv < hdls(db)
                              1.2 · (s - hdv) ·t
                             1.2 · (sv - hdh) ·t
       Fbs = Abrq · Fu
                                  2.4 ·Abrg
       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 = 15 in
     L = (nr - 1) \cdot s + 2 \cdot Lev
    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,
     Aq = L \cdot t
    Number of Areas in Consideration,
```



GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com www.gizasteel.com

Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	65 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

n1 = n

Tensile Yielding Capacity, (J4-1)

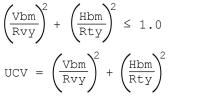
 $Rty = \Lambda ty \cdot n1 \cdot Fy \cdot Ag$

Rty = 182.25 kips

Hbm = 0 kips

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

Interaction of Yielding Capacities,

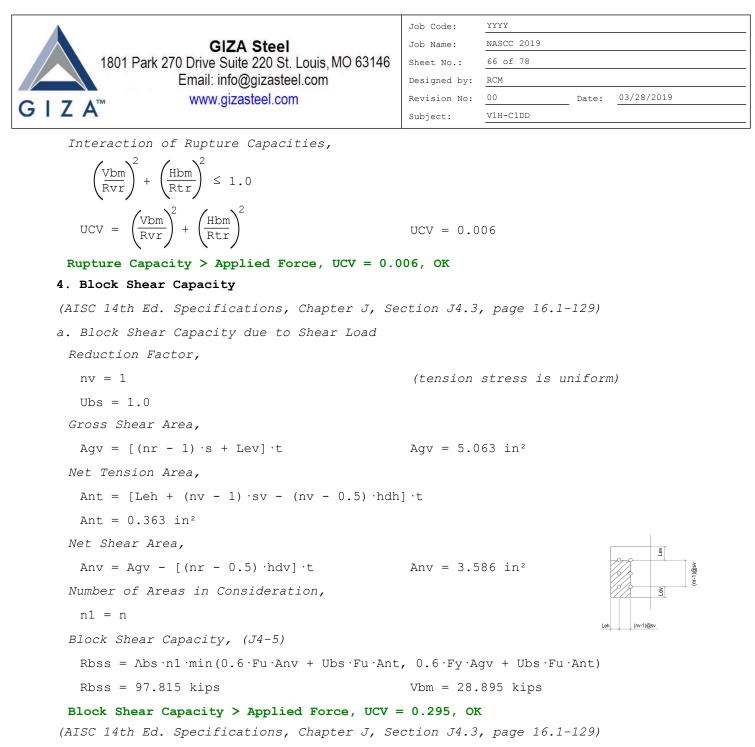


UCV = 0.057

Yielding Capacity > Applied Force, UCV = 0.057, OK

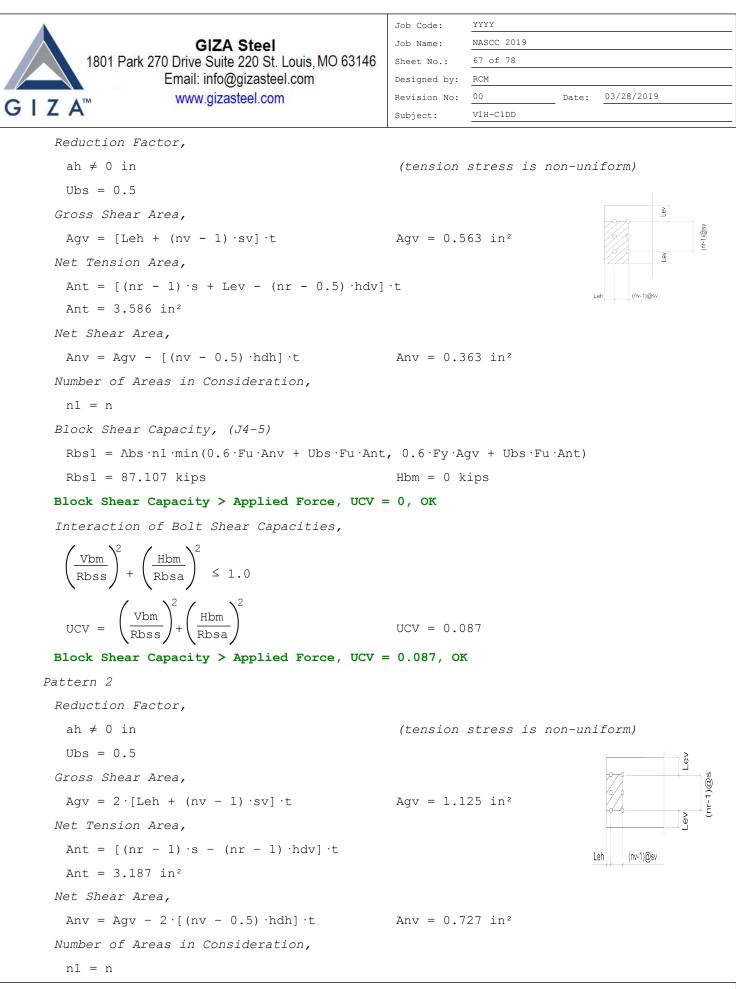
3. Rupture Capacity

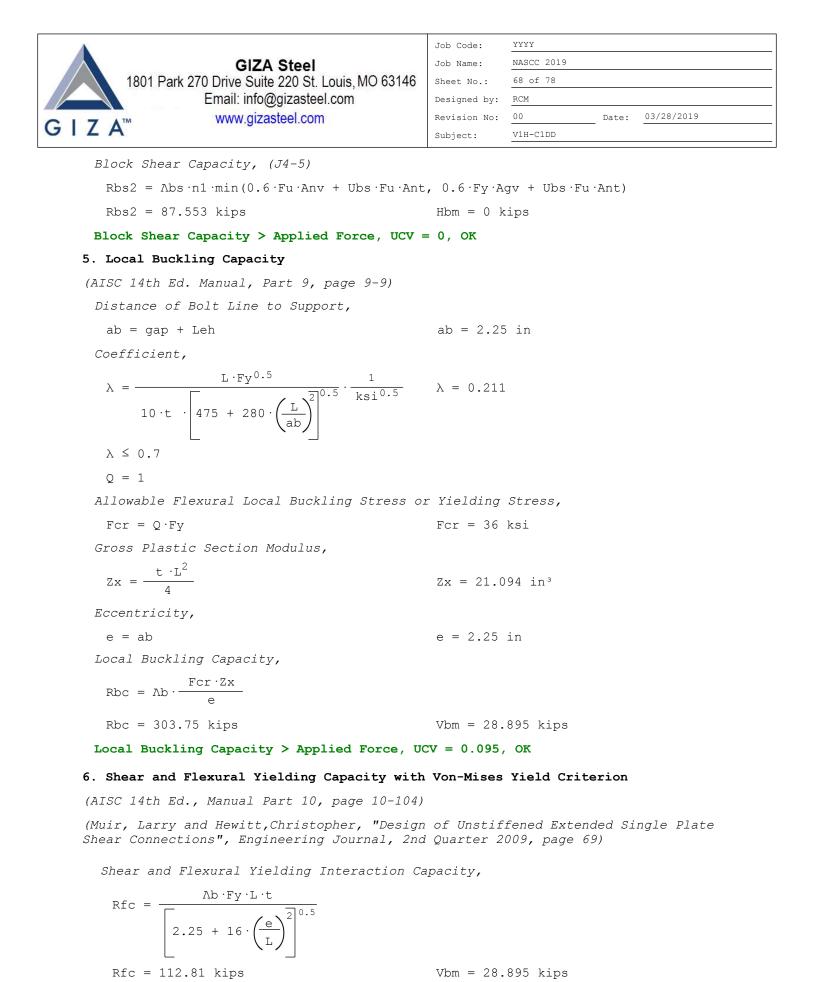
```
(AISC 14th Ed. Specifications, Chapter J, Section J4.2, page 16.1-129)
a. Shear Rupture Capacity due to Shear Load
 Net Shear Area,
   Anv = (L - nr \cdot hdv) \cdot t
   Anv = 3.984 in^{2}
 Number of Areas in Consideration,
   n1 = n
  Shear Rupture Capacity, (J4-4)
   Rvr = Avr ·n1 ·0.6 ·Fu ·Anv
   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] \qquad Ae = 3.984 \text{ in}^2
 Number of Areas in Consideration,
   n1 = n
 Tensile Rupture Capacity, (J4-2)
   Rtr = \Lambda tr \cdot n1 \cdot Fu \cdot Ae
                                                   Hbm = 0 kips
   Rtr = 173.32 kips
 Tensile Rupture Capacity > Applied Force, UCV = 0, OK
```



```
b. Block Shear Capacity due to Axial Load
```

Pattern 1







Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	69 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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

Vbm = 28.895 kips

Lu = 2.25 in

Fcr = 36 ksi

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 + Lel

Gross Area,

 $Ag = L \cdot t \qquad Ag = 5.625 \text{ in}^2$

Radius of Gyration,

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

Slenderness Ratio,

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

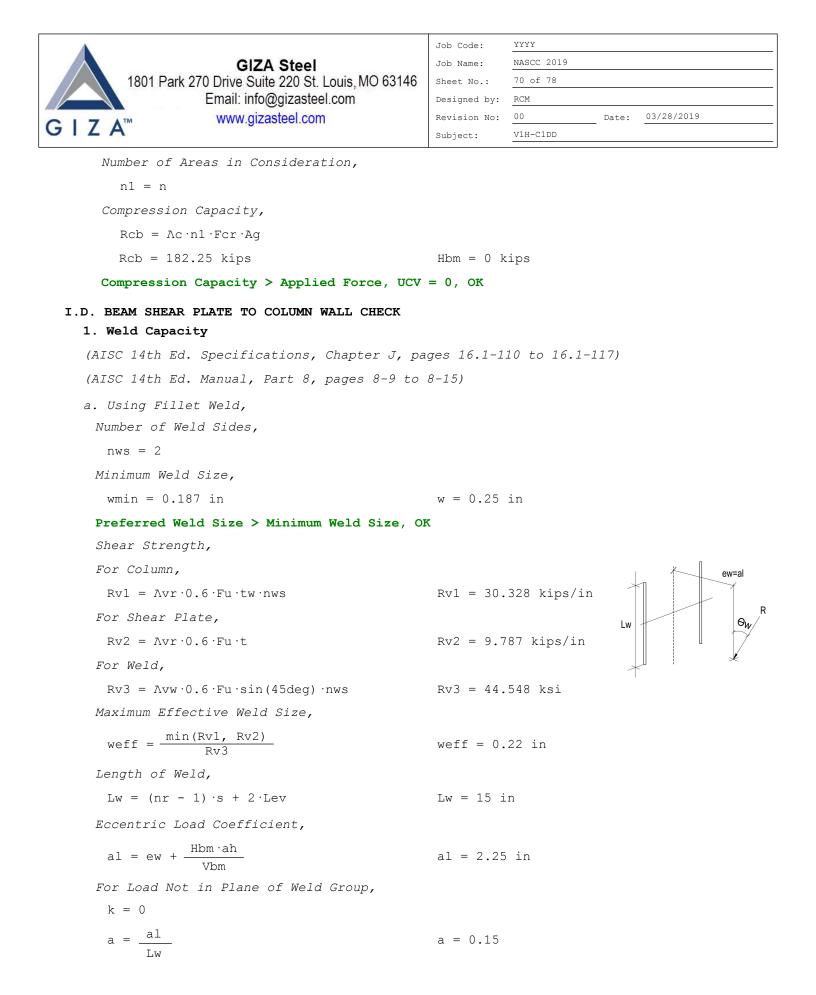
Elastic Critical Buckling Stress,

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

Flexural Buckling Stress,

$$KLr \leq 25$$

Fcr = Fy



		Job Code:	ҮҮҮҮ		
	GIZA Steel	Job Name:	NASCC 2019		
	1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com	Sheet No.:	71 of 78 RCM		
	www.gizastool.com	Designed by: Revision No:	00	Date: 03	3/28/2019
GIZ	A	Subject:	V1H-C1DD		
7	oad Inclination from Vertical,				
-	$\left(H_{hm} \right)$				
	$\theta = \operatorname{atan}\left(\frac{\operatorname{Hom}}{\operatorname{Vbm}}\right)$	$\theta = 0 \deg$	ſ		
E	lectrode Strength Coefficient,				
	AISC 14th Ed. Manual Part 8, Table 8-3, pa	age 8-65)			
	C1 = 1 ksi				
	AISC 14th Ed. Manual Part 8, Table 8-4)				
	Co = 3.666				
Þ	Neld Capacity,				
	Rew = Aew ·Co ·C1 ·16 ·Lw ·min(w, weff)				
	Rew = 144.964 kips	Rbm = 28.	895 kips		
V	<pre>Neld Capacity > Applied Force, UCV = 0.199,</pre>	OK			
	DLUMN WALL CHECK HSS Local Check				
ĉ	. HSS Punching Shear				
	AISC Specification for the Design of Steel	l Hollow St	cructural Se	ctions,	page 15)
1	hickness of Shear Plate,				
	t = 0.375 in				
Γ	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		
Ν	laximum Shear Plate Thickness to Avoid Shea	ar Tab Punc	ching Thru C	olumn Wa	11,
	$tPSmax = \frac{1.2 \cdot Avr \cdot Fu \cdot tw}{Aty \cdot Nmax}$				
	tPSmax = 33698 in	t = 0.375	5 in		
E	Plate thickness < Maximum Plate Thickness,	OK			
k	. HSS Wall Plastification Capacity				
	AISC 14th Ed. Specifications, Chapter K, D	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			

```
Job Code:
                                                                          YYYY
                              GIZA Steel
                                                              Job Name:
                                                                          NASCC 2019
             1801 Park 270 Drive Suite 220 St. Louis, MO 63146
                                                              Sheet No.:
                                                                          72 of 78
                         Email: info@gizasteel.com
                                                              Designed by:
                                                                          RCM
                                                                                               03/28/2019
                           www.gizasteel.com
                                                                          00
                                                              Revision No:
                                                                                         Date:
G I Z A<sup>™</sup>
                                                              Subject:
                                                                          V1H-C1DD
         For Required Flexural Strength,
           Code = LRFD
           MUR = \frac{M}{S \cdot Fy}
                                                              MUR = 0
         For Uplift Force (if any),
           Code = LRFD
           ULUR = \frac{PUplift}{Aq \cdot Fv}
                                                              ULUR = 0
         Utilization Ratio,
          (CIDECT Design Guide 3 Second Edition, Table 7.1, page 78)
                 -PUR - MUR + ULUR
-PUR + MUR + ULUR
PUR + MUR + ULUR
PUR - MUR + ULUR
                                                          (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 \ge 0 then Cs = 0.10
         Chord-stress Interaction Parameter,
           for ie 0..3
             if n_i < 0 then Cs_i = 0.20
             if n_i \ge 0 then Cs_i = 0.10
           for ie 0..3
             x_{i} = (1 - |n_{i}|)^{Cs_{i}}
                                                              Of = 1
           Qf = min(x_i)
         Branch Angle from the HSS Chord Face,
           \theta = 90 \text{ deg}
         Nominal HSS Wall Plastification Capacity,
                   \frac{Fy \cdot tw^2}{t} \cdot \frac{2L}{BHSS} + 4 \left( 1 - \frac{t}{BHSS} \right)
                       BHSS
           Rn =
                                    sin(\theta)
         HSS Wall Plastification Capacity,
           RpHSS = ApHSS ·Rn
           RpHSS = 146.963 kips
                                                              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)
```

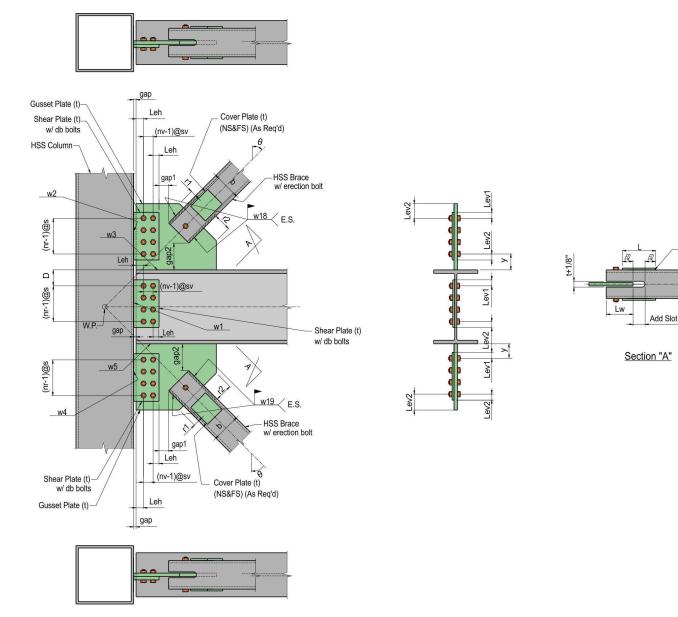
	Job Code:	<u>YYYY</u>		
GIZA Steel	Job Name:	NASCC 2019		
1801 Park 270 Drive Suite 220 St. Louis, MO 63146	Sheet No.:	73 of 78		
Email: info@gizasteel.com	Designed by:	RCM		
GIZA [™] www.gizasteel.com	Revision No:	00 Date: 03/28/2019		
OTZA	Subject:	V1H-C1DD		
Moment Load Required,				
MuHSS = Hbm·ah	MuHSS = 0	kips·ft		
Width Ratio,				
$\beta = \frac{t}{BHSS}$	$\beta = 0.063$			
Load Length Parameter,				
$\eta = \frac{L}{BHSS}$	η = 2.5			
Coefficient of Chord Stress Functions,				
(CIDECT Design Guide 3 Second Edition, Tabl	le 4.1, pag	re 37)		
if n < 0 then Cs = 0.60 - 0.5 $\cdot\beta$				
if $n \ge 0$ then Cs = 0.10				
Chord-stress Interaction Parameter,				
for ie 0 3				
if $n_i < 0$ then $Cs_i = 0.60 - 0.5 \cdot \beta$				
if $n_i \ge 0$ then $Cs_i = 0.10$				
for ie 0 3				
$x_{i} = (1 - n_{i})^{Cs_{i}}$				
$Qf = min(x_i)$	Qf = 1			
Nominal HSS Flexural Plasification Capacity	7,			
$Mn = Fy \cdot tw^{2} \cdot L \cdot \left[\frac{1}{2 \cdot \eta} + \frac{2}{(1 - \beta)^{0.5}} + \frac{\eta}{1 - \beta}\right] \cdot Q$	£			
HSS Flexural Plastification Capacity,				
MpHSS = ApHSS ·Mn	MpHSS = 9	5.734 kips ft		
Interaction of Capacities, (K3-13)				
$\frac{PuHSS}{RpHSS} + \frac{MuHSS}{MpHSS} \le 1.0$				
$UCV = \frac{PuHSS}{RpHSS} + \frac{MuHSS}{MpHSS}$	UCV = 0			
Wall Plastification Capacity > Applied Ford	e, UCV = 0	, ок		



	Job Code:	ҮҮҮҮ		
	Job Name:	NASCC 2019		
16	Sheet No.:	74 of 78		
3353	Designed by:	RCM		
	Revision No:	00	Date:	03/28/2019
	Subject:	V1H-C1DD		

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

w20 TYP. E.S.

@ BRACE



Job Code:	ҮҮҮҮ		
Job Name:	NASCC 2019		
Sheet No.:	75 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

B. CONNECTION SCHEDULE

Column			
Mark Size Grade			
	HSS14X6X5/8	А500-в	

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						Ŵe	b
Mark	Size	Grade	gap	θsl	θsk	D	Leh
	W21X83	A992	1/2"	0°	0 °	3"	1 3/4"

	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				We	ld	
Mark	Size	Grade	θ (±2°)	Add Slot	w18	Lw
	HSS6X5X3/8	А500-В	50°	2"	1/4"	6"

Brace 2					We	ld
Mark	Size	Grade	θ (±2°)	Add Slot	w19	Lw
	HSS6X5X3/8	А500-В	50°	2"	1/4"	6"



GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com www.gizasteel.com

ҮҮҮҮ		
NASCC 2019		
76 of 78		
RCM		
00	Date:	03/28/2019
V1H-C1DD		
	NASCC 2019 76 of 78 RCM 00	NASCC 2019 76 of 78 RCM 00 Date:

	Weld			
t	Grade	b	L	w20
NR	NR	NR	NR	NR

	Weld			
t	Grade	b	L	w21
NR	NR	NR	NR	NR

Gusset Plate 1				We	ld
t	Grade	У	Lev2	w3	Lw
3/8"	A36	3"	2 1/2"	1/4"	1'-4 5/16"

Gusset Plate 2				We	ld
t	Grade	У	Lev2	w 5	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"		Short-Slotted Holes in Shear Plate Only		3"	1	0"

Gusset Shear Plate				Weld
t	Grade	Lev	Leh	w2
3/8"	A36	1 1/2"	1 1/2"	1/4"



GIZA Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com www.gizasteel.com

Job Code:	YYYY		
Job Name:	NASCC 2019		
Sheet No.:	77 of 78		
Designed by:	RCM		
Revision No:	00	Date:	03/28/2019
Subject:	V1H-C1DD		

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		

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	



o Code:	ҮҮҮҮ		
o Name:	NASCC 2019		
eet No.:	78 of 78		
signed by:	RCM		
vision No:	00	Date:	03/28/2019
oject:	V1H-C1DD		

IV. REFERENCES

Steel Construction Manual (14th Ed.) - LRFD American Institute of Steel Construction, Inc. 2011