Giza Steel 1801 Park 270 Drive Suite 220 St. Louis, MO 63146 Email: info@gizasteel.com www.gizasteel.com	Job Code:	Giza
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	Sheet No:	1 of 18
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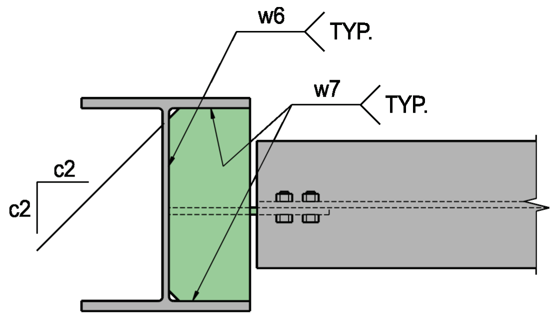
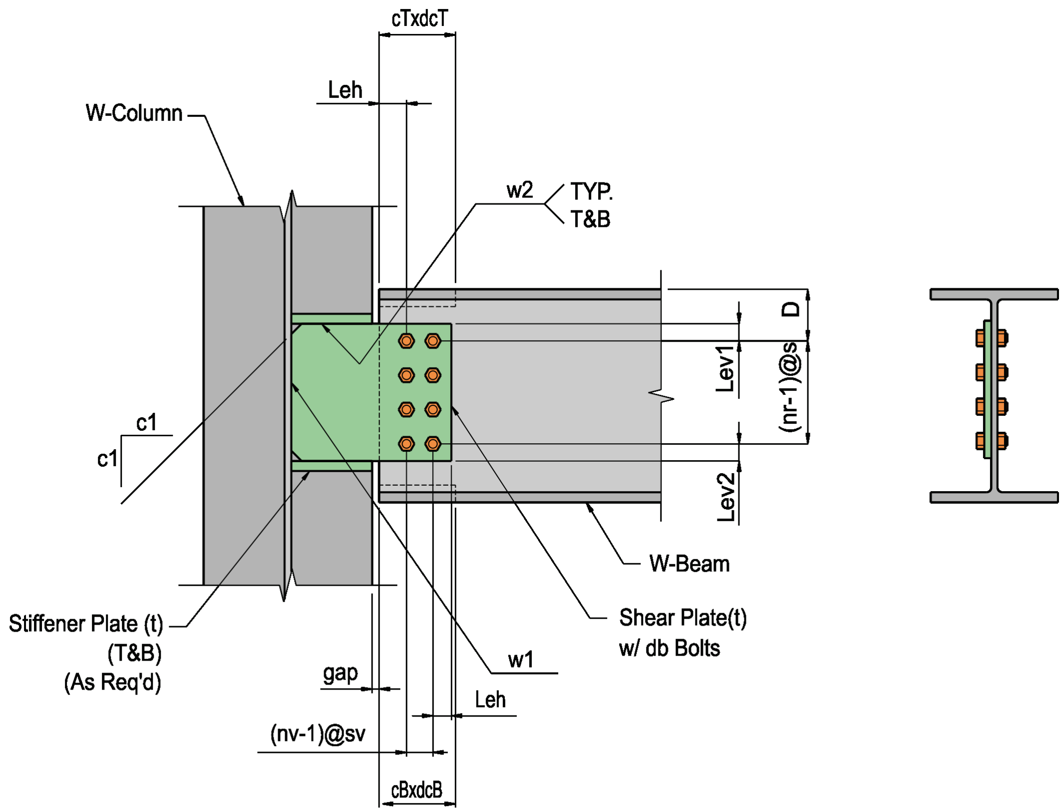




Note: Figure above does not represent actual design. Refer to connection schedule

SHEAR CONNECTION: W BEAM WITH SHEAR PLATE ONE-WAY SHEAR CONNECTION TO W COLUMN WEB

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

COLUMN PROPERTIES: **W14X90 - A992**

Depth,	d = 14 in	*Web Thickness,*	tw = 0.44 in
Flange Width,	bf = 14.5 in	*Flange Thickness,*	tf = 0.71 in
Distance k,	k = 2 in	*Distance k1,*	k1 = 1.438 in
Area,	Ag = 26.5 in^2	*Distance k (Design),*	kdes = 1.31 in
Minimum Yield Stress,	Fy = 50 ksi	*Minimum Tensile Stress,*	Fu = 65 ksi
Modulus of Elasticity,	E = 29000 ksi		

BEAM PROPERTIES: **W16X26 - A992**

Depth,	d = 15.7 in	*Web Thickness,*	tw = 0.25 in
Flange Width,	bf = 5.5 in	*Flange Thickness,*	tf = 0.345 in
Distance k,	k = 1.063 in	*Distance k1,*	k1 = 0.75 in
Area,	Ag = 7.68 in^2	*Distance k(Design),*	kdes = 0.747 in
Minimum Yield Stress,	Fy = 50 ksi	*Minimum Tensile Stress,*	Fu = 65 ksi
Modulus of Elasticity,	E = 29000 ksi		
Top of Steel Elevation,	Elev = 0 ft + 0 in		
Span Length,	L = 30 ft	*Erection Clearance,*	gap = 0.5 in
Slope,	θsl = 0 deg	*Skew,*	θsk = 0 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

BOLTS PROPERTIES: **3/4" ø - A325-N**

For Shear Plate to Beam Web Connection:

Bolt Diameter,	db = 0.75 in		
Bolt Shear Strength,	Λrv = 11.928 kips	*Bolt Tensile Strength,*	Λrn = 19.88 kips
Bolt Type,	Bolt_Type = A325-N	*Connection Type,*	Conn_type= Bearing Type

Number of Bolt Rows,	nr = 4	*Bolt Vertical Spacing,*	s = 3 in
Number of Bolt Column Lines,	nv = 1	*Bolt Horizontal Spacing,*	sv = 0 in
Total Number of Bolts (nr·nv),	nb = 4		
Holes at Beam Web,		*Holes at Shear Plate,*	
Vertical Hole Dimension,	hdv = 0.875 in	*Vertical Hole Dimension,*	hdv = 0.875 in
Horizontal Hole Dimension,	hdh = 0.875 in	*Horizontal Hole Dimension,*	hdh = 1.063 in
Bolt First Down from Top of Beam.	D = 3 in		
Vertical Edge Distance (D-dcT),	Lev = 3 in	*Vertical Edge Distance min(Lev,Lev2),*	Lev = 1.25 in
Horizontal Edge Distance,	Leh = 1.75 in	*Horizontal Edge Distance,*	Leh = 1.25 in

WELDS PROPERTIES :

Minimum Tensile Stress, Fu = 70 ksi

For Shear Plate to Column Web Connection:

Preferred Weld Size (w1), w = 0.25 in

SAFETY AND RESISTANCE FACTORS:

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

Modification Factor,

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

	safety factor	*resistance factor*	*modification factor*
For Member in Bearing/Bolt Bearing(brg),	Ωbrg = 2.00	ϕbrg = 0.75	Λbrg = 0.50
For Block Shear(bs),	Ωbs = 2.00	ϕbs = 0.75	Λbs = 0.50
For Flexural Local Buckling/Flexural Strength(b),	Ωb = 1.67	ϕb = 0.90	Λb = 0.60
For Flexural Rupture(fr),	Ωfr = 2.00	ϕfr = 0.75	Λfr = 0.50
For Member Shear for C, WT, L(v),	Ωv = 1.67	ϕv = 0.90	Λv = 0.60

For Shear Rupture(vr),	Ωvr = 2.00	ϕvr = 0.75	Λvr = 0.50
For Shear Yielding(vy),	Ωvy = 1.50	ϕvy = 1.00	Λvy = 0.67

APPLIED LOADS:

Given End Reaction

Beam:

Shear Load,	V = 10 kips
Adjacent Shear Load (if any),	V_2 = 0 kips

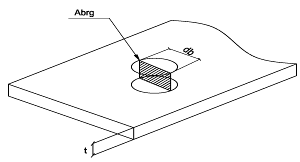
II. CALCULATIONS

A. BEAM WEB CHECK

1. Bolt Capacity

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

Bolt Capacity due to Shear Load



Bearing Area,

$Abrg = db \cdot tw$ $\qquad$ $Abrg = 0.187\ in^2$

Bolt Centerline Distance from Face of Support,

$$ab = \frac{0.5 \cdot (bf - tw) + gap}{\cos(\theta sk)} + Leh + 0.5(nv - 1) \cdot sv$$

$ab = 9.28\ in$

Eccentricity Distance of End Reaction from Bolt Line,

$ebv = ab$ $\qquad$ $ebv = 9.28\ in$

Load Inclination from Vertical,

$\theta = 0\ deg$

Eccentric Load Coefficient,

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

$C = 1.172$

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

$hdh < hdls$

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

$ebv > 0in$

$Fbe = \min\ (Fbe)$ $\qquad$ $Fbe = 12.797\ kips$

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

$hdh < hdls$

$Fbs = \Lambda brg \cdot Fu \cdot \min[1.2 \cdot (s - hdv) \cdot tw\ ,\ 2.4 \cdot Abrg]$

$Fbs = 14.625\ kips$

Number of Area in Consideration,

$n1 = 1$

Bolt Bearing Capacity,

$ebv > 0in$

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

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Rbrg = 13.977 V = 10 kips

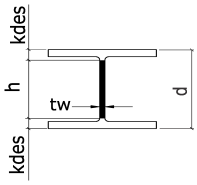
Bolt Capacity > Applied Force, UCV = 0.715, OK

2. Shear Capacity

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

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

$h = d - 2 \cdot kdes$ $h = 14.206$ in



Limiting Depth-thickness Ratio,

$htw = \frac{h}{tw}$ $htw = 56.824$

Clear Distance Between Transverse Stiffeners,

$htw < 260$

$a = 0$ in $a = 0$ in

Web Plate Buckling Coefficient, (G2-6)

$htw < 260$

$kv = 5$ $kv = 5$

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

$$htw \le 1.1 \cdot \left(\frac{kv \cdot E}{Fy}\right)^{0.5}$$

$Cv = 1$ $Cv = 1$

Shear Capacity, (G2-1)

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

$Rv = 70.509$ kips $V = 10$ kips

Shear Capacity of Section > Applied Force, UCV = 0.142, OK

B. BEAM WEB TO SHEAR PLATE CHECK

1. Bolt Shear Capacity

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

Shear Capacity Per Bolt,

$\Lambda rv = 11.928$ kips

Bolt Shear Capacity,

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

$Rb = 13.977$ kips $V = 10$ kips

Bolt Shear Capacity > Applied Force, UCV = 0.715, OK

2. Check for Spacing

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

Shear Plate Thickness,

$t1 = 0.375\ in$

Beam Web Thickness,

$t2 = 0.25\ in$

Vertical Spacing of Bolts,

$s = 3\ in$

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

$smax = min(12in, 24 \cdot min\ (t1, t2))$ $\qquad smax = 6\ in$

Spacing > Min. Spacing & Spacing < Max. Spacing, OK

3. Check for Edge Distance

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

Shear Plate Thickness,

$t1 = 0.375\ in$

Shear Plate Edge Distances,

$Lev1 = 1.25\ in$

$Leh1 = 1.25\ in$

Beam Web Thickness,

$t2 = 0.25\ in$

Beam Web Edge Distances,

$Lev2 = NA$

$Leh2 = 1.75\ in$

Vertical Edge Distance,

$Levcon = \begin{pmatrix} Lev1 \\ Lev2 \end{pmatrix}$ $\qquad Levcon = \begin{pmatrix} 1.25\ in \\ NA \end{pmatrix}$

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

$min(Levcon) = Lev1$

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

$Levmax = 4.5\ in$

Edge Distance ≥ Min. Edge Distance & Edge Distance ≤ Max. Edge Distance, OK

Horizontal Edge Distance,

$$Lehcon = \begin{pmatrix} Leh1 \\ Leh2 \end{pmatrix} \qquad Lehcon = \begin{pmatrix} 1.25\ in \\ 1.75\ in \end{pmatrix}$$

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

$$\min(Lehcon) = Leh1$$

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

$$Lehmax = 4.5\ in$$

Edge Distance ≥ Min. Edge Distance & Edge Distance ≤ Max. Edge Distance, OK

C. SHEAR PLATE CHECK

1. Check for Maximum Thickness

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

Exceptions for nv = 1 and nv = 2,

Shear Plate,

$$t \le \frac{db}{2} + \frac{1}{16}$$

$$Leh \ge 2 \cdot db$$

Beam Web,

$$tw \le \frac{db}{2} + \frac{1}{16}$$

$$Leh \ge 2 \cdot db$$

Check maximum thickness of plate

Coefficient for Eccentrically Loaded Bolts,

(AISC 14th Ed. Manual Part 7, page 7-19)

$$C' = 11.256\ in$$

Area of Bolts,

$$Ab = \frac{\pi \cdot db^2}{4} \qquad Ab = 0.442\ in^2$$

Length of Plate,

$$L = (nr-1) \cdot s + 2 \cdot Lev \qquad L = 11.5\ in$$

Maximum Thickness,

$$tmax = \frac{6 \cdot \left(\frac{Fnv}{0.9} \cdot Ab \cdot C'\right)}{Fy \cdot L^2}$$

$$tmax = 0.376\ in \qquad t = 0.375\ in$$

Plate thickness < Maximum Thickness Permitted, OK

Governing Shear Plate Thickness,

Case = Maximum thickness need be checked

$$t \le tmax$$

$$tg = t \qquad tg = 0.375\ in$$

2. Check for Stiffener Plate Requirement

(AISC 14th Ed. Manual Part 10, pages 10-105 to 10-106)

("On the Need for Stiffeners for the Effect of Lap Eccentricity on Extended Single-Plate Connections", William A. Thornton and Patrick J. Fortney)

Distance of First Bolt Line from the Face of Support,

$$ab1 = \frac{0.5(bf - tw) + gap}{\cos(\theta sk)} + Leh \qquad ab1 = 9.28\ in$$

Lateral Displacement Capacity, (10-6)

$$Rreq = \Lambda b1500 \cdot \pi \cdot \frac{L \cdot tg^3}{ab1^2} \cdot ksi$$

$Rreq = 19.871\ in$ $\qquad V = 10\ kips$

Lateral Displacement Capacity > Applied Force, UCV = 0.503, OK

Check for Requirement of Stiffener Plates,

$$\eta = \frac{Rreq}{V} \qquad \eta = 1.987$$

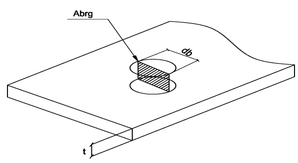
Stiffener Plate is not required. OK

3. Bolt Capacity

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

Bolt Capacity due to Shear Load

Bearing Area,



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

Bolt Centerline Distance from Face of Support,

$$ab = \frac{0.5 \cdot (bf - tw) + gap}{\cos(\theta sk)} + Leh + 0.5(nv - 1) \cdot sv$$

$ab = 9.28\ in$

Eccentricity Distance of End Reaction from Bolt Line,

$ebv = ab$ $\qquad ebv = 9.28\ in$

Load Inclination from Vertical,

$\theta = 0\ deg$

Eccentric Load Coefficient,

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

$C = 1.172$

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

hdh < hdls

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

ebv > 0in

Fbe = min (Fbe) Fbe = 9.38 kips

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

hdh < hdls

$Fbs = \Lambda brg \cdot Fu \cdot min[1.2 \cdot (s - hdv) \cdot tg , 2.4 \cdot Abrg]$

Fbs = 19.575 kips

Number of Area in Consideration,

n1 = n

Bolt Bearing Capacity,

ebv > 0in

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

Rbrg = 10.99 V = 10 kips

Bolt Capacity > Applied Force, UCV = 0.91, OK

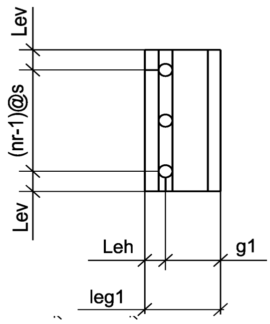
4. Shear Yielding Capacity

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

a. Shear Yielding Capacity due to Shear Load

Length,

$L = (nr - 1) \cdot s + 2 \cdot Lev$ L = 11.5 in



Erection Stability,

(AISC 14th Ed. Manual Part 10, page 10-106)

Length of Connector > One-half of T-Dimension, OK

Number of Areas in Consideration,

n1 = n

Shear Yielding Capacity, (J4-3)

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

Rvy = 62.1 kips V = 10 kips

Shear Yielding Capacity > Applied Force, UCV = 0.161, OK

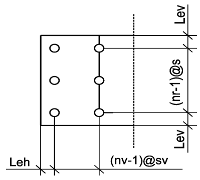
5. Rupture Capacity

(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·hdv)·tg Anv = 3 in²



Number of Areas in Consideration,

n1 = n

Shear Rupture Capacity, (J4-4)

Rvr = Λvr·n1·0.6·Fu·Anv

Rvr = 52.2 kips V = 10 kips

Shear Rupture Capacity > Applied Force, UCV = 0.192, OK

6. Block Shear Capacity

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

a. Block Shear Capacity due to Shear Load

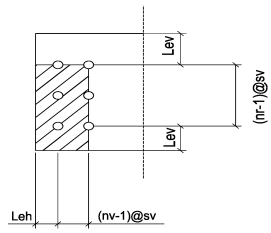
Reduction Factor,

nv = 1

Ubs = 1.0 *(tension stress is uniform)*

Gross Shear Area,

Agv = [(nr - 1·s+Lev]·tg Agv = 3.844 in²



Net Tension Area,

Ant = [Leh + (nv - 1)·sv - (nv - 0.5)·hdh]·tg

Ant = 0.27 in²

Net Shear Area,

Anv = Agv - [(nr - 0.5)·hdv]·tg Anv = 2.695 in²

Number of Areas in Consideration,

n1 = n

Block Shear Capacity,(J4-5)

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

Rbs = 49.329 kips V = 10 kips

Block Shear Capacity > Applied Force, UCV = 0.203, OK

7. Local Buckling Capacity

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

Distance of Bolt Line to Support,

Stiffeners = Required

ab = gap + Leh ab = 9.5 in

Coefficient,

$$\lambda = \frac{L \cdot Fy^{0.5}}{10 \cdot tg \cdot \left[475 + 280 \left(\frac{L}{ab}\right)^2\right]^{0.5}} \cdot \frac{1}{ksi^{0.5}} \qquad \lambda = 0.618$$

$\lambda > 1.41$

$$Q = \frac{1.30}{\lambda^2} \qquad Q = 1$$

Allowable Flexural Local Buckling Stress or Yielding Stress,

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

Gross Plastic Section Modulus,

$$Zx = \left(\frac{tg \cdot L^2}{4}\right) \qquad Zx = 12.398\ in^3$$

Eccentricity,

$$e = ab \qquad e = 9.5\ in$$

Local Buckling Capacity,

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

$Rbc = 28.134\ kips$ $\qquad V = 10\ kips$

Local Buckling Capacity will not control, OK

8. 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,

$mod(nr,2) \leq 0$

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

$Znet = 8.461\ in^3$

Flexural Rupture Capacity,

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

$Rfr = 25.828\ kips$ $\qquad V = 10\ kips$

Flexural Rupture Capacity > Applied Force, UCV = 0.387, OK

9. Flexural Yielding Capacity with Von-Mises Shear Reduction

(AISC 14th Ed. Manual Part 10, page 10-103)

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

Flexural Yielding Capacity,

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

Rfc = 25.618 kips   V = 10 kips

Flexural Yielding Capacity > Applied Force, UCV = 0.39, OK

10. Interaction of Shear Yielding, Shear Buckling, and Flexural Yielding Capacities

(AISC 14th Ed. Manual Part 10, pages 10-104 to 10-105)

From AISC Manual Equation 10-5,

$$\left(\frac{Vr}{Vc}\right)^2 + \left(\frac{Mr}{Mc}\right)^2 \leq 1.0$$

Vr = V   Vr = 10 kips

Mr = Vr·e   Mr = 95 kips·in

Shear Yielding Capacity,

Vc = Λvy·0.6·Fy·L·tg   Vc = 62.1 kips

Flexural Yielding Capacity,

Mc = Λb·Fy·Zx   Mc = 267.272 kips·in

Interaction, (10-5)

$$UCV = \left(\frac{Vr}{Vc}\right)^2 + \left(\frac{Mr}{Mc}\right)^2$$

UCV = 0.152

Interaction < 1.0, UCV = 0.152, OK

D. SHEAR PLATE TO COLUMN WEB CHECK

1. Welds Check

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

Number of Weld Sides,

nws = 2

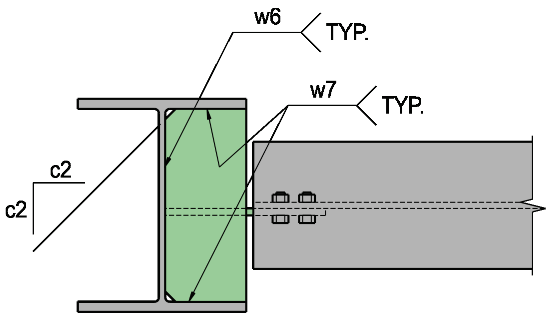
Minimum Weld Size,

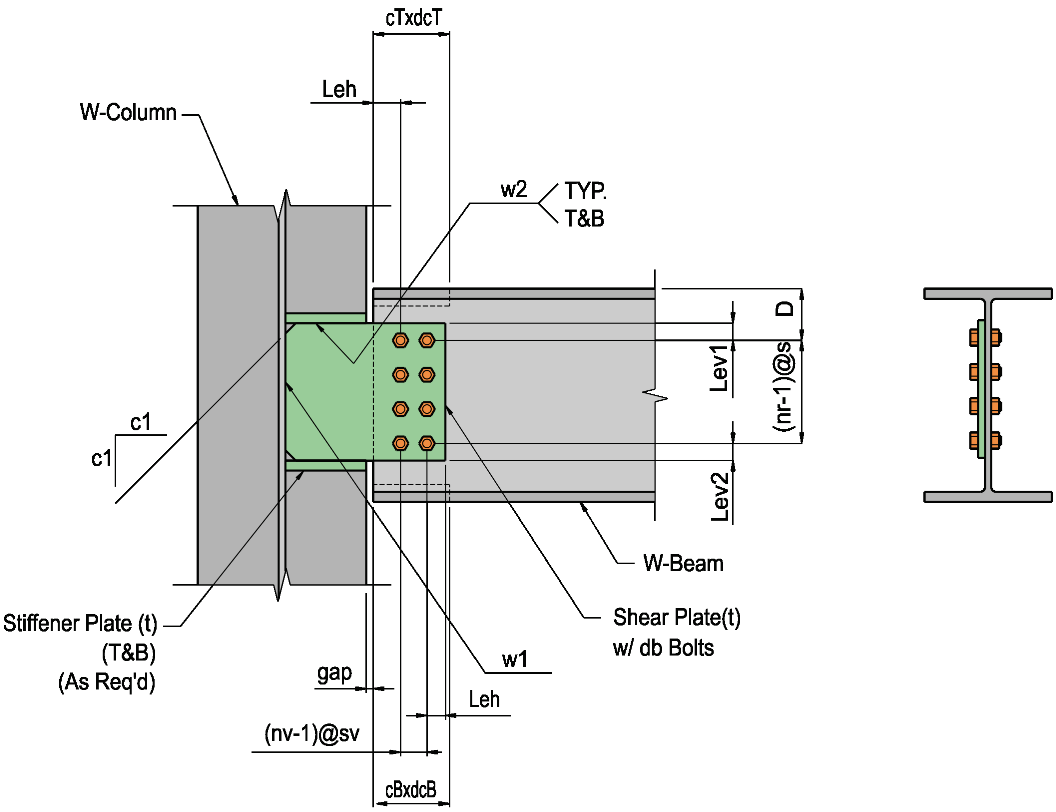
wmin = 0.25 in   w = 0.25 in

Preferred Weld Size = Minimum Weld Size, OK

III. DETAILS

A. SKETCH





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

SHEAR CONNECTION: W BEAM WITH SHEAR PLATE ONE-WAY SHEAR CONNECTION TO W COLUMN WEB

B. CONNECTION SCHEDULE

Column		
Mark	**Size**	**Grade**
C1000	W14X90	A992

Beam						Web	
Mark	**Size**	**Grade**	**gap**	**θsl**	**θsk**	**D**	**Leh**
B1000	W16X26	A992	1/2"	0°	0°	3"	1 3/4"

Cope Dimension				
dcT	**cT**	**dcB**	**cB**	**Cut Flush Case**
0"	0"	0"	0"	NR

Bolts at Beam Web						
db	**Bolt Type**	**Remarks**	**nr**	**s**	**nv**	**sv**
3/4"	A325-N	Short-Slot on Shear Plate ONLY	4	3"	1	0"

Shear Plate			
t	**Grade**	**Lev**	**Leh**
3/8"	A36	1 1/4"	1 1/4"

Remarks as per AISC Equation 10-5
Interaction < 1.0, UCV = 0.152, OK

Governing Limit State of SC Connection			
V	**Connection Capacity**	**UCV**	**Governing Check**
10 kips	10.990 kips	0.910	Bolt Capacity at Shear Plate

Remarks on Connection / Connecting Elements				
For Bolts	**For Connector Thickness**	**For Connector Length**	**For Bolt Spacing**	**For Edge Distance**
OK	OK	OK	OK	OK

Remarks on Connection / Connecting Elements
For Weld on Shear Plate to Column Web
OK

Remarks on Connection / Connecting Elements		
For Weld on Shear Plate to Column Web	**For Weld on Shear Plate to Stiffener**	**For Weld on Stiffener Plate to Column**
OK	NA	NA

Remarks on Beam Web / Column Web	
For Beam Web	**For Column Web**
OK	OK

IV. REFERENCES

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