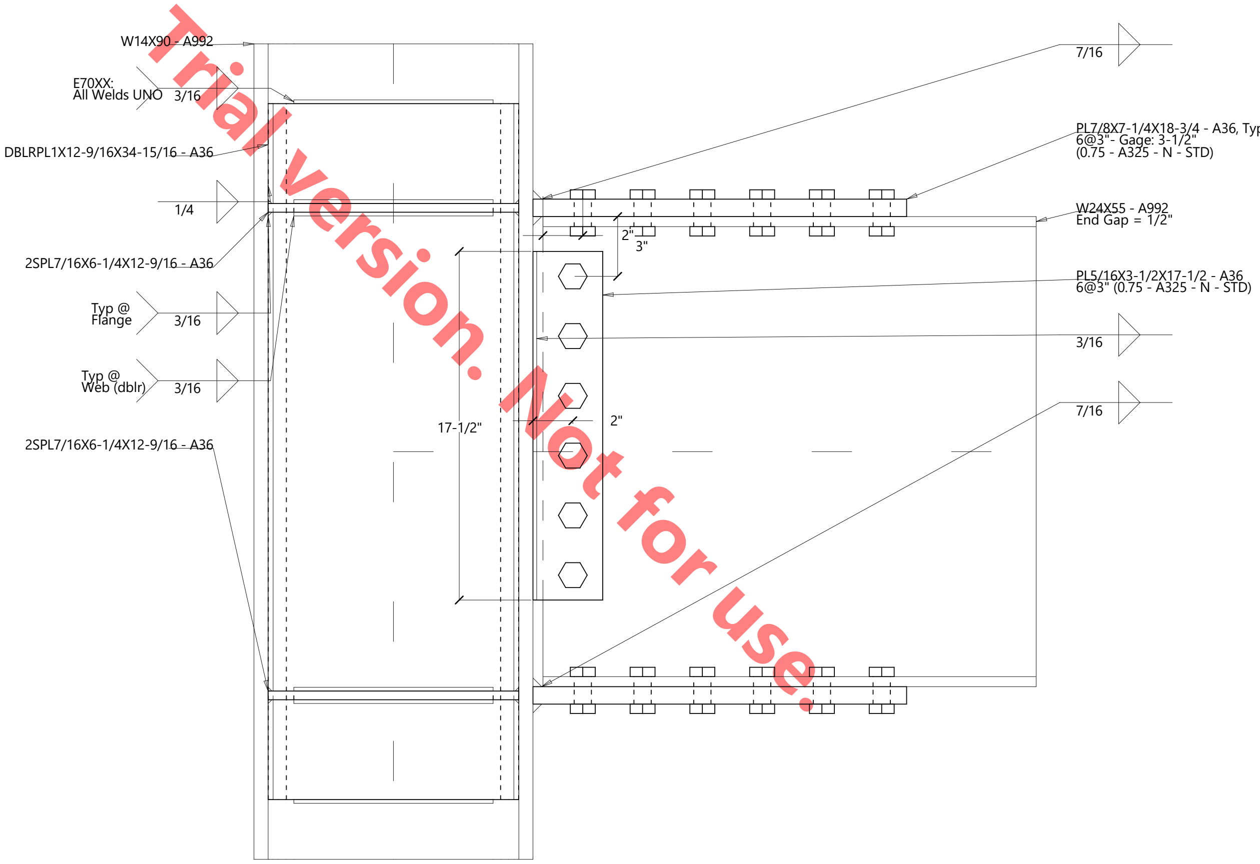
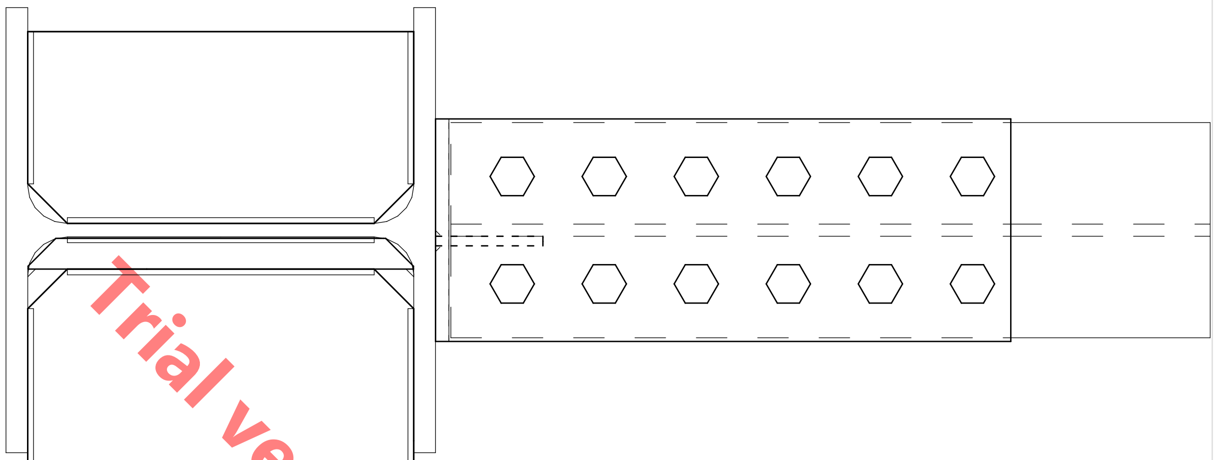
PROJECT NAME	Sample Configuration	**PAGES**	1 / 9
PROJECT NO		**CODE**	AISC13
PROJECT DATE		**METHOD**	ASD
CALC DATE	1/17/2019	**UNITS**	US
CALCULATED BY		**SEISMIC**	No
CHECKED BY		**FILE NAME**	Sample Moment Configuration.dsn
DESCRIPTION			

Front View



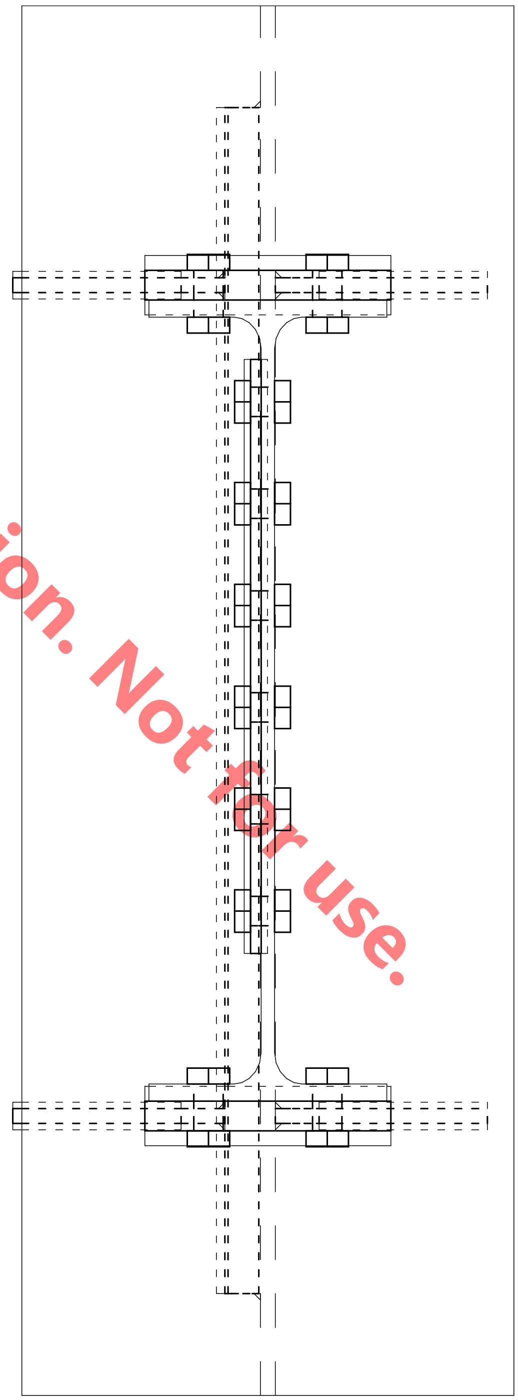
PROJECT NAME	Sample Configuration	**PAGES**	2 / 9
PROJECT NO		**CODE**	AISC13
PROJECT DATE		**METHOD**	ASD
CALC DATE	1/17/2019	**UNITS**	US
CALCULATED BY		**SEISMIC**	No
CHECKED BY		**FILE NAME**	Sample Moment Configuration.dsn
DESCRIPTION			

Top View



PROJECT NAME	Sample Configuration	**PAGES**	3 / 9
PROJECT NO		**CODE**	AISC13
PROJECT DATE		**METHOD**	ASD
CALC DATE	1/17/2019	**UNITS**	US
CALCULATED BY		**SEISMIC**	No
CHECKED BY		**FILE NAME**	Sample Moment Configuration.dsn
DESCRIPTION			

Right Side View



PROJECT NAME	Sample Configuration	**PAGES**	4 / 9
PROJECT NO		**CODE**	AISC13
PROJECT DATE		**METHOD**	ASD
CALC DATE	1/17/2019	**UNITS**	US
CALCULATED BY		**SEISMIC**	No
CHECKED BY		**FILE NAME**	Sample Moment Configuration.dsn
DESCRIPTION			

BASIC DETAILS OVERVIEW

Joint Configuration: Beam to Column Flange

Member: Column
Section: W14X90
Material: A992

Member: Right Side Beam
Section: W24X55
Material: A992

DETAILED CALCULATION REPORT
Beam Connection to Column Flange
Column: W14X90 - A992

Right Side Beam: W24X55 - A992
Moment: 250 k-ft.
Shear: 60 kips
Axial Force (Hc): 0 kips
Axial Force (Ht): 0 kips

All Welds Are E70XX

RIGHT SIDE BEAM

1. RIGHT SIDE BEAM - W24X55 MOMENT CONNECTION

1.a. Moment Connection Using Flange Plate:
Flange Force (Ff):
= P / 2 + M / d
= 0 / 2 + 3000 / 23.6
= 127.1186 kips
Top Plate: 18.75 in. X 7.25 in. X 0.875 in.
Bottom Plate: 18.75 in. X 7.25 in. X 0.875 in.
Plate Material: A36
Bolts on Flange: 12 Bolts - (0.75 - A325 - N - STD) in 2 Lines
Bolt Holes on Plate: 0.8125 in. Lateral X 0.8125 in. Longitudinal
Bolt Holes on Flange: 0.8125 in. Lateral X 0.8125 in. Longitudinal

1.b. Check Beam:
Beam Flange Effective Area:
Afg = tf * bf = 0.505 * 7.01 = 3.540 in^2
Afn = tf * (bf - Nt * (dh + 0.0625)) = 0.505 * (7.01 - (2 * (0.8125 + 0.0625))) = 2.6563 in^2
Fy / Fu ≤ 0.8 ---- Yt = 1
Fu * Afn = 65 * 2.6563 = 172.6595 kips
Yt * Fy * Afg = 1 * 50 * 3.540 = 177.0025 kips
Mn = Fu * Afn * Sx / Afg = 65 * 2.6563 * 114 / 3.540
= 5560.1426 kips/in.
(1 / FS) Mn = (1 / 1.67) * Mn = 277.4522 ≥ 250 k-ft. (OK)

1.c. Check Bolts:
Spacing (s) = 3 ≥ Minimum Spacing = 2 in. (OK)
Edge Distance on Plate Parallel to Beam Axis (el):
= 1.25 ≥ 1.25 in. (OK)
Edge Distance on Plate Transverse to Beam (et):
= 1.875 ≥ 1.25 in. (OK)
Edge Distance on Beam Parallel to Beam Axis (el):
= 2 ≥ 1.25 in. (OK)
Edge Distance Transverse to Beam (et):
= 1.755 ≥ 1 in. (OK)

1.d. Allowable Bolt Shear Strength - Flange Plate
(1 / FS)Rn = nlN * nSP * nB * Fv
= 1 * 1 * 12 * 10.6028
= 127.2345 ≥ 127.1186 kips (OK)

Bolt Bearing on Plate:
Bearing Strength / Bolt / Thickness Using Bolt Edge Distance = Fbe
Edge Dist. = 1.25 in., Hole Size = 0.8125 in.
= (1 / 2.0) * 1.2 * Lc * Fu ≤ (1 / 2.0) * 2.4 * d * Fu = 52.2 kips/in.
= (1 / 2.0) * 1.2 * 0.8437 * 58 = 29.3625 kips/in.

Bearing Strength / Bolt / Thickness Using Bolt Spacing = Fbs
Bolt Spacing = 3 in., Hole Size = 0.8125 in.
= (1 / 2.0) * 1.2 * Lc * Fu ≤ (1 / 2.0) * 2.4 * d * Fu = 52.2 kips/in.
= (1 / 2.0) * 1.2 * 2.1875 * 58 = 76.125 kips/in.
Use: Fbs = 52.2 kips/in.

Allowable Bearing Strength at Bolt Holes = nlN * nL * (Fbe + Fbs * (nB - 1)) * t * ef
= 1 * 2 * (29.3625 + 52.2 * (6 - 1)) * 0.875 * 1
= 508.1343 ≥ 127.1186 kips (OK)

Bolt Bearing on Flange:
Bearing Strength / Bolt / Thickness Using Bolt Edge Distance = Fbe
Edge Dist. = 2 in., Hole Size = 0.8125 in.
= (1 / 2.0) * 1.2 * Lc * Fu ≤ (1 / 2.0) * 2.4 * d * Fu = 58.5 kips/in.
= (1 / 2.0) * 1.2 * 1.5937 * 65 = 62.1562 kips/in.
Use: Fbe = 58.5 kips/in.

Bearing Strength / Bolt / Thickness Using Bolt Spacing = Fbs
Bolt Spacing = 3 in., Hole Size = 0.8125 in.
= (1 / 2.0) * 1.2 * Lc * Fu ≤ (1 / 2.0) * 2.4 * d * Fu = 58.5 kips/in.
= (1 / 2.0) * 1.2 * 2.1875 * 65 = 85.3125 kips/in.
Use: Fbs = 58.5 kips/in.

Allowable Bearing Strength at Bolt Holes = nlN * nL * (Fbe + Fbs * (nB - 1)) * t * ef
= 1 * 2 * (58.5 + 58.5 * (6 - 1)) * 0.505 * 1
= 354.51 ≥ 127.1186 kips (OK)

1.e. Plate Tension Allowable Strength:

1.e.1. Tension Yielding:
(1 / FS) Rn = (1 / 1.67) * Fy * b * t
= (1 / 1.67) * 36 * 7.25 * 0.875
= 136.7514 ≥ 127.1186 kips (OK)

1.e.2. Tension Rupture:
Effective Net Width:
bn1 = b - Max(0.15 * b; nT * (dh + 0.0625))
= 7.25 - Max(0.15 * 7.25; 2 * (0.8125 + 0.0625)) = 5.5 in.
bn2 = 2 * 0.85 * Ws = 2 * 0.85 * 3.625 = 6.1625 in.
bn = Min(bn1, bn2) = Min(5.5, 6.1625) = 5.5 in.
(1 / FS) Rn = (1 / 2.0) * Fu * bn * t
= (1 / 2.0) * 58 * 5.5 * 0.875
= 139.5625 ≥ 127.1186 kips (OK)

1.e.3. Block shear rupture of the Plate:
Agt = Min(g, 2 * e) * t = 3.5 * 0.875
= 3.0625 in^2
Ant = Agt - (dh + 0.0625) * t
= 3.0625 - (0.875) * 0.875
= 2.2968 in^2
Agv = 2 * ((nl - 1) * s + Le) * t
= 2 * ((6 - 1) * 3 + 1.25) * 0.875
= 28.4375 in^2
Anv = Agv - 2 * (nl - 0.5) * (dh + 0.0625) * t
=28.4375 - 2 * (6 - 0.5) * (0.875)*0.875
= 20.0156 in^2
(1 / FS) Rn = (1 / 2.0) * Min((0.6 * Fu * Anv + Ubs * Fu * Ant); (0.6 * Fy * Agv + Ubs * Fu * Ant))
= (1 / 2.0) * Min((0.6 * 58 * 20.0156 + 1 * 58 * 2.2968); (0.6 * 36 * 28.4375 + 1 * 58 * 2.2968))
= 373.7343 ≥ 127.1186 kips (OK)

1.e.4. Block shear rupture of the Beam Flange:
Agt = (bf - g) * t = (7.01 - 3.5)* 0.505
= 1.7725 in^2
Ant = Agt - (nt - 1) * (dh + 0.0625) * t
= 1.7725 - (2 - 1) * (0.875) * 0.505
= 1.3306 in^2
Agv = 2 * ((nl - 1) * s + ef) * t
= 2 * ((6 - 1) * 3 + 2) * 0.505
= 17.17 in^2
Anv = Agv - 2 * (nl - 0.5) * (dh + 0.0625) * t
=17.17 - 2 * (6 - 0.5) * (0.875) * 0.505
= 12.3093 in^2
(1 / FS) Rn = (1 / 2.0) * Min((0.6 * Fu * Anv + Ubs * Fu * Ant); (0.6 * Fy * Agv + Ubs * Fu * Ant))
= (1 / 2.0) * Min((0.6 * 65 * 12.3093 + 1 * 65 * 1.3306); (0.6 * 50 * 17.17 + 1 * 65 * 1.3306))
= 283.2797 ≥ 127.1186 kips (OK)

1.f. Bottom Plate Allowable Compressive Strength:
Unbraced Length (L) = c + ef = 0.5 + 2 = 2.5 in.
Effective Length Factor, K = 0.65
KL / r = k * L / (t / 3.464) = 0.65 * 2.5 / (0.875 / 3.464) = 6.4333
KL / r ≤ 25
Fcr = Fy = 36 ksi
(1 / FS)cPn = (1 / 1.67) * Fcr * Ag = (1 / 1.67) * 36 * 7.25 * 0.875 = 136.7514 ≥ 127.1186 kips (OK)

1.g. Top Plate-to-Support Weld:
Required Fillet Weld Size = Ff / ((1 / 2.0) * 1.5 * 0.4242 * Fexx * b * 2)
= 127.1186 / ((1 / 2.0) * 1.5 * 0.4242 * 70 * 7.25 * 2)
= 0.3936 in. ≤ 0.4375 in. (OK)

1.h. Bottom Plate-to-Support Weld:
Required Fillet Weld Size = Ff / ((1 / 2.0) * 1.5 * 0.4242 * Fexx * b * 2)
= 127.1186 / ((1 / 2.0) * 1.5 * 0.4242 * 70 * 7.25 * 2)
= 0.3936 in. ≤ 0.4375 in. (OK)

Note: Descon does not check the moment versus rotation behavior of the connection.
If your particular application requires this check, you must do it outside the program.

2. RIGHT SIDE BEAM - W24X55 SHEAR CONNECTION

2.a. Shear Connection Using One Plate:
Plate (W x L x T): 17.5 in. X 3.5 in. X 0.3125 in.

Shear Connection Using One Plate:
Max. Thickness = db / 2 + 0.0625
= 0.4375 ≥ Min(0.3125, 0.395) in. (OK)

Plate Material: A36
Beam Setback: 0.5 in.
Bolts: (6) (0.75 - A325 - N - STD)
Bolt Holes on Beam Web: 0.8125 in. Vert. X 0.8125 in. Horiz.
Bolt Holes on Plate: 0.8125 in. Vert. X 0.8125 in. Horiz.
Weld: 0.1875 E70XX - Fillet Welds

Loading:
Vertical Shear (V) = 60 kips
Axial Load (H) = 0 kips
Resultant (R) = (V^2 + H^2)^0.5
= (60^2 + 0^2)^0.5
= 60 kips
Theta = Atan(V / H) = Atan(60 / 0) = 90 degrees

Check Bolt Spacing and Edge Distance:

Spacing (s) = 3 ≥ Minimum Spacing = 2 in. (OK)

Distance to Horiz. Edge of PL (ev):
(If Sheared Edge)
= 1.25 ≥ 1.25 in. (OK)
(If Rolled or Gas Cut Edge)
= 1.25 ≥ 1 in. (OK)

Minimum Distance to Vert. Edge of PL:
= Max(2 * db, 1.25)= 1.5 in.
Distance to Vert. Edge of PL (eh):
= 1.5 ≥ 1.5 in. (OK)

Minimum Distance to End of Beam:
= Max(2 * db, 1.25) = 1.5 in.
Distance to End of Beam (Lh):
= 1.5 ≥ 1.5 in. (OK)

Connection Top Location: (OK)
Connection Bottom Location: (OK)
Connection Depth = 17.5 ≥ T / 2 (OK)

2.b. Bolt Strength:
Load Eccentricity for Bolts (eb):
Bolt Rows = 6 ≤ 9 ... eb = 0

2.c. Allowable Bolt Shear Strength - Single Plate
(1 / FS)Rn = nIN * nSP * nB * Fv
= 1 * 1 * 6 * 10.6028
= 63.6172 ≥ 60 kips (OK)

2.d. Allowable Shear Strength of the Beam:

2.d.1. **Allowable Shear Yield Strength:**
A = dw * tw = 23.6 * 0.395 = 9.322 in^2
Rn = 0.6 * Fy * A * Cv
= 0.6 * 50 * 9.322 * 1
= 279.66 kips
(1 / FS) Rn = (1 / 1.67) * 279.66 = 167.461 kips
= 167.461 ≥ 60 kips (OK)

2.d.2. **Allowable Shear Rupture Strength:**
Anv = (dw - N * (dh + 0.0625)) * tw
= (23.6 - 6 * (0.8125 + 0.0625)) * 0.395
= 7.2482 in^2
Rn = 0.6 * Fu * Anv
= 0.6 * 65 * 7.2482
= 282.6817 kips
(1 / FS) Rn = (1 / 2.0) * 282.6817 = 141.3408 kips
= 141.3408 ≥ 60 kips (OK)

2.d.3. **Allowable Shear Strength of the Plate:**

2.d.4. **Allowable Shear Yield Strength:**
A = dw * tw = 17.5 * 0.3125 = 5.4687 in^2
Rn = 0.6 * Fy * A * Cv
= 0.6 * 36 * 5.4687 * 1
= 118.125 kips
(1 / FS) Rn = (1 / 1.5) * 118.125 = 78.75 kips
(1 / FS)Vn = 78.75 ≥ 60 kips (OK)

2.d.5. **Allowable Shear Rupture Strength:**
Net Area (An) = (L - nL * (dh + 0.0625)) * t
= (17.5 - 6 * 0.875) * 0.3125 = 3.8281 in^2
Shear Rupture Strength = Npl * An * (1 / 2.0) * 0.6 * Fu = 1 * 3.8281 * (1 / 2.0) * 0.6 * 58
= 66.6093 ≥ 60 kips (OK)

2.d.6. **Block Shear Strength of the Plate:**
Gross Area with Tension Resistance (Agt)
= (et + (Nh - 1) * sh) * t
= (1.5 + (1 - 1) * 3) * 0.3125
= 0.4687 in^2

Net Area with Tension Resistance (Ant)
= Agt - (Nh - 0.5) * (dh + 0.0625) * t
= 0.4687 - (1 - 0.5) * (0.8125 + 0.0625) * 0.3125
= 0.332 in^2

Gross Area with Shear Resistance (Agv)
= (L - el) * t = (17.5 - 1.25) * 0.3125 = 5.0781 in^2

Net Area with Shear Resistance (Anv)
= Agv - (Nv - 0.5) * (dv + 0.0625) * t
= 5.0781 - (6 - 0.5) * (0.8125 + 0.0625)* 0.3125
= 3.5742 in^2

(1 / FS) Rn = (1 / 2.0) * Min((0.6 * Fu * Anv + Ubs * Fu * Ant); (0.6 * Fy * Agv + Ubs * Fu * Ant))
= (1 / 2.0) * Min((0.6 * 58 * 3.5742 + 1 * 58 * 0.332); (0.6 * 36 * 5.0781 + 1 * 58 * 0.332))
= 64.4726 ≥ 60 kips (OK)

Bolt Bearing on Plate:
Bearing Strength / Bolt / Thickness Using Bolt Edge Distance = Fbe
Edge Dist. = 1.25 in., Hole Size = 0.8125 in.
= (1 / 2.0) * 1.5 * Lc * Fu ≤ (1 / 2.0) * 3.0 * d * Fu = 65.25 kips/in.
= (1 / 2.0) * 1.5 * 0.8437 * 58 = 36.7031 kips/in.

Bearing Strength / Bolt / Thickness Using Bolt Spacing = Fbs
Bolt Spacing = 3 in., Hole Size = 0.8125 in.
= (1 / 2.0) * 1.5 * Lc * Fu ≤ (1 / 2.0) * 3.0 * d * Fu = 65.25 kips/in.
= (1 / 2.0) * 1.5 * 2.1875 * 58 = 95.1562 kips/in.
Use: Fbs = 65.25 kips/in.

Allowable Bearing Strength at Bolt Holes = nIN * nL * (Fbe + Fbs * (nB - 1)) * t * ef
= 1 * 1 * (36.7031 + 65.25 * (6 - 1)) * 0.3125 * 1
= 113.4228 ≥ 60 kips (OK)

2.e. Bolt Bearing on Beam Web:

Bearing Strength / Bolt / Thickness Using Bolt Spacing = Fbs
Bolt Spacing = 3 in., Hole Size = 0.8125 in.
= (1 / 2.0) * 1.5 * Lc * Fu ≤ (1 / 2.0) * 3.0 * d * Fu = 73.125 kips/in.
= (1 / 2.0) * 1.5 * 2.1875 * 65 = 106.6406 kips/in.
Use: Fbs = 73.125 kips/in.

Allowable Bearing Strength at Bolt Holes = nIN * nL * Fbs * nB * t * ef
= 1 * 1 * 73.125 * 6 * 0.395 * 1
= 173.3062 ≥ 60 kips (OK)

2.e.1. Weld Strength:
Weld Size (w) = 0.1875 ≥ Minimum Weld, 0.1875 in. (OK)

Eccentric Weld
k = 0
a = 0
Theta = 0
(1 / FS) C = 0.9266
Maximum useful weld size for support thickness:
= Fu * t_eff / (0.707 * Fexx)
= 65 * 0.71 / (0.707 * 70)
= 0.9325 ≥ 0.1875 in. (OK)

Maximum useful weld size for plate thickness:
= Fu * tp / (2 * 0.707 * Fexx)
= 58 * 0.3125 / (2 * 0.707 * 70)
= 0.1831 in.
0.1831 << 0.1875 in.
Use 0.1831 in. for strength calculation.

(1 / FS) Rn = 2 * C * C1 * D * L = 2 * 0.9266 * 1 * 2.9298 * 17.5
= 95.0259 ≥ 60 kips (OK)

3. COLUMN WEB SHEAR REINFORCEMENT
Framing System: OMF
Column Axial Force (Pu) = 0 kips
Column Shear Force (Vus) = 0 kips

3.a. Right Side Beam Flange Forces:
PufRight = Mu / dm + Pu / 2
= 3000 / 24.475 + 0 / 2
= 122.574 kips

3.b. Column Panel Zone:
Required Strength (Vu)
= |PufLeft + PufRight - Vus|
= |0 + 122.574 - 0|
= 122.574 kips

3.b.1. Column Web Shear Strength:
Pc = 0.6 * Py = 0.6 * A * Fy = 0.6 * 26.5 * 50 = 795 kips
Pr ≤ 0.4 * Pc
(1 / FS)Rv = (1 / 1.67) * 0.6 * Fy * d * tw
= (1 / 1.67) * 0.6 * 50 * 14 * 0.44
= 110.6586 << 122.574 kips
Doubler Plate Required for Strength

3.b.2. Shear Buckling of Web:
Thickness Required = h * $(Fy^{\wedge 0.5})$ / $(2.24 * E^{\wedge 0.5})$ = 11.38 * $(50^{\wedge 0.5})$ / $(2.24 * 29000^{\wedge 0.5})$
= 0.2109 ≤ 0.44 in.
Doubler Plate Not Required for Shear Buckling
Doubler Plate Thickness = 1 in.

Required for strength = Vudp / (Npl * (1 / 1.67) * 0.6 * Fy * dc)
= 11.9153 / (1 * (1 / 1.67) * 0.6 * 36 * 14)
= 0.0658 ≤ 1 in. (OK)
Required for fillet weld detail = k - tf - re
= 2 - 0.71 - 0.375 = 0.915 ≤ 1 in. (OK)
Required for shear buckling = (d - 2 * k) * $(Fy)^{\wedge 0.5}$ / $(2.24 * E^{\wedge 0.5})$
= (14 - 2 * 1.31) * $(36)^{\wedge 0.5}$/$(2.24 * 29000^{\wedge 0.5})$
= 0.1789 ≤ 1 in. (OK)
Required to transmit stiffener force:
= (Rust1 + Rust2) / ((1 / 1.67) * 0.6 * Fy * 2 * Min(2 * (L - 2 * clip), dc))
= (28.2439 + 0) / ((1 / 1.67) * 0.6 * 36 * 2 * Min(2 * (12.5625 - 2 * 1.29); 14))
= 0.0779 ≤ 1 in. (OK)
Doubler Plate Thickness excluding Fillet Weld Detail:
= 0.1875 in.

4. COLUMN STIFFENERS
Framing System: OMF
Column Axial Force (Pu) = 0 kips
Column Shear Force (Vus) = 0 kips

4.a. Right Side Beam Flange Forces:
PufRight = Mu / dm + Pu / 2
= 3000 / 24.475 + 0 / 2
= 122.574 kips

4.b. Column Stiffeners
Right Side Beam
Local Flange Bending Strength, (1 / FS) Rn
= (1 / 1.67) * 6.25 * (tf^2) * Fy * ct
= (1 / 1.67) * 6.25 * (0.71^2) * 50 * 1
= 94.330 kips

Local Web Yielding Strength, (1 / FS) Rn
= (1 / 1.5) * (ct * 5 * k + t + 2 * w) * tw * Fy
= (1 / 1.5) * (1 *5* 1.31 + 0.875 + 2 * 0.4375) * 0.44 * 50
= 121.7333 kips

4.b.1. **Column Web Crippling:**
N = t + 2 * w = 0.875 + 2 * 0.4375 = 1.75 in.
Ct = 1.0
Nd = 3 * N / d = 3 * 1.75 / 14 = 0.375
(1 / FS) Rn = (1 / 2.0) * 0.8 * ct * (tw^2) * [1 + Nd * (tw / tf)^1.5] * (E * Fy * tf / tw)$^{0.5}$
= (1 / 2.0) * 0.8 * 1 * (0.44^2) * [1 + 0.375 * (0.44 / 0.71)^1.5] * (29000 * 50 * 0.71 / 0.44)$^{0.5}$
= 140.1254 kips

Tension Flange Stiffener Force (TFrc):
Right Side:
RTFrc = Max(RPuf - R(1 / FS) Rn_FlBending; RPuf - R (1 / FS) Rn_WebYielding) ≥ 0
= Max(122.574 - 94.330; 122.574 - 121.7333) = 28.2439 kips
Compression Flange Stiffener Force (CFrc):
Right Side:
RCFrc = Max[(RPuf - R(1 / FS) Rn_WebCrippling), (RPuf - R (1 / FS) Rn_WebYielding)] ≥ 0
= Max[(122.574 - 140.1254), (122.574 - 121.7333)] = 0.8407 kips

TFrc = Max(LTFrc, RTFrc) = Max(0, 28.2439) = 28.2439 kips
CFrc = Max(LCFrc, RCFrc) = Max(0, 0.8407) = 0.8407 kips

TFrc >> 0 or High Seismic Loading
Stiffeners required opposite tension flange
CFrc >> 0 or High Seismic Loading
Stiffeners required opposite compression flange.
Required stiffener area for strength:
Tension and/or compresion:
Ast = Max(TFrc, CFrc) / ((1 / 1.67) * Fy)
= Max(28.2439, 0.8407) / ((1 / 1.67) * 36)
= Max(1.3102, 0.039) in^2
Stiffener Width (bs) = 6.25 ≥ Minimum Width = 2.1966 in. (OK)

Stiffener Length:
L = d - 2 * tf = 14 - 2 * 0.71 = 12.5625 in.
(Using Full Length Stiffeners)
Stiffener thickness required for shear:
= Max([LTFrc + RCFrc], [LCFrc + RTFrc]) / ((1 / 1.67) * 0.6 * Fy * (L - 2 * clip) * 2)
= Max([0 + 0.8407]; [0 + 28.2439]) / ((1 / 1.67) * 0.6 * 36 * (12.5625 - 2 * 1.29) * 2)
= 0.1093 ≤ 0.4375 in. (OK)
Stiffener thickness required for minimum area:
= Ast / (2 * (bs - clip))
= 1.3102 / (2 * (6.25 - 1.29)) = 0.132 ≤ 0.4375 in. (OK)

Minimum Thickness = Max(tm / 2, bs * (Fy / E)$^{0.5}$ / 0.56)
= Max(0.875 / 2, 6.25 * (36 / 29000)$^{0.5}$ / 0.56)
= 0.4375 ≤ 0.4375 in. (OK)

5. DOUBLER PLATE WELDS:
Doubler Plate to Flange Weld = 0.25 in.
Minimum weld size for material thickness = 0.25 ≤ 0.25 in. (OK)
Weld size to develop doubler plate force:
= Max(Vupl / ((1 / 2.0) * 0.707 * 0.6 * Fexx * Column d), teff * 1.4142)
= Max(11.9153 / ((1 / 2.0) * 0.707 * 0.6 * 70 * 14), 0.0658 * 1.4142)
= Max(0.0573, 0.093) in.
= 0.0625 ≤ 0.25 in. (OK)
Doubler Plate to Web Weld:
Minimum weld size = Min(Wmin, t - 0.0625)
= 0.1875 ≤ 0.1875 in. (OK)

6. STIFFENER WELDS:

6.a. **Stiffener to Flange Weld:**
Minimum Weld Size = 0.1875 ≤ 0.1875 in. (OK)

6.b. **Tension Stiffener to Flange Weld:**
w_Req. = Rust / ((1 / FS) * (1.5 * 0.6 * Fexx) * bs - clip) * 2 * 1.4142)
= 28.2439 / ((1 / 2.0) * 1.5 * 0.6 * 70) * (6.25 - 1.29) * 2 * 1.4142)
= 0.0639 ≤ 0.1875 in. (OK)

6.c. **Compression Stiffener to Flange Weld:**
w_Req. = Rust / ((1 / FS) * (1.5 * 0.6 * Fexx) * bs - clip) * 2 * 1.4142)
= 28.2439 / ((1 / 2.0) * 1.5 * 0.6 * 70) * (6.25 - 1.29) * 2 * 1.4142)
= 0.0019 ≤ 0.1875 in. (OK)

6.d. **Stiffener to Panel Zone Weld:**
Stiffener Force (Rust) = Max[(LTRust + RcRust); (RTRust + LCRust)]
= Max[(0 + 0.8407); (28.2439 + 0)] = 28.2439 kips

6.e. **Welds need to develop only the lesser of Rust and the minimum of the following forces:**
Allowable Strength of stiffeners to flange connection:
= (1 / 1.67) * Fy * 2 * (L - clip) * t
= (1 / 1.67) * 36 * 2 * (12.5625 - 1.29) * 0.4375 = 212.625 kips

Allowable Shear Strength of stiffener and web interface area:
= (1 / 1.67) * 0.6 * Fy * (bs - 2 * clip) * 2 * t
= (1 / 1.67) * 0.6 * 36 * (6.25 - 2 * 1.29) * 2 * 0.4375 = 112.9755 kips

Shear yield strength of the panel zone:
= (1 / 1.67) * 0.6 * Fyc * dc * tw (for column web, if applicable)
= (1 / 1.67) * 0.6 * 50 * 14 * 0.44 = 110.6586 kips
= (1 / 1.67) * 0.6 * Fyp * dc * tp (for doubler plate, if applicable)

= (1 / 1.67) * 0.6 * 36 * 14 * 1 = 181.0778 kips

Weld Design Force, Rust_Weld = 28.2439 kips
Minimum Weld Size = 0.1875 ≤ 0.1875 in. (OK)

Required weld size for strength:
= Rust_Weld / (0.8485 * Fexx * (L - 2 * clip))
= 28.2439 / (0.8485 * 70 * (12.5625 - 2 * 1.29))
= 0.0476 ≤ 0.1875 in. (OK)