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Beam Connection to Column Flange

Column: W14X132 - A992
Left Side Beam: W24X117 - A992
Moment: 776 k-ft
Shear: 185.4 kips
Axial Force: 0 kips
Right Side Beam: W24X62 - A992
Moment: 390 k-ft
Shear: 98.6 kips
Axial Force: 0 kips

********** All Welds Are E70XX **********

Right Side Beam - W24X62
Moment Connection With Directly Welded Flanges:

Weld Strength
Flange Force, Ff:
= P/2 + M/(d-tf)
= 0/2) + 4680/ (23.74 - 0.59)
= 202.2 kips

Full Penetration Weld Design Strength = 0.9 *Fy*b*t
= 0.9 * 50 * 7.04 * 0.59
= 186.9 < 202.2 kips (~~NG~~) OK, CJP

Right Side Beam
Shear Connection Using Clip Angle(s):

Clip Angles: 2L4X4X5/16 X 11.5 in.
Angle Material: A36

Support Side Connection: 8 Bolts 3/4"Ø A325-N -SSLN

Bolt Holes on Support: 0.8125 in. Vert. X 0.8125 in. Horiz.
Effective Thickness of Support Material: 1.03 in.
Bolt Holes on Angles: 0.8125 in. Vert. X 1 in. Horiz.

Beam Side Connection: 3/16 E70XX Fillet Welds

Beam Web Thickness: 0.43 in.
Beam Web Height: 20.75 in.
Beam Setback: 0.375 in.

Loading:
Vertical Shear, V = 98.6 kips
Axial Load, H = 0 kips
Resultant, R = (V^2 + H^2)^0.5 = ((98.6)^2 + (0)^2)^0.5 = 98.6 kips

Check Clearances:
Beam Web Clear Height = 20.75 ≥ 11.5 in. (OK)

Support Side Bolts
Spacing, s = 3 ≥ Minimum Spacing = 2 in. (OK)

Distance to Horizontal Edge, ev:
= 1.25 ≥ 1.25 in. (OK)

Distance to Vertical Edge, eh:
= 1.465 ≥ 1.125 in. (OK)

Gage on OSL:
Angle Gage = 2.535 ≥ 1.5625 in. (OK)
Column Gage = 5.5 in.

Design Shear Strength of Bolts:
= 2*n*(Ørn) = 2 * 4 * 17.892 = 143.1 ≥ 98.6 kips (OK)

Bolt Bearing on Angle(s):
Bearing Strength/Bolt/Thickness Using Bolt Edge Distance = Fbe
Edge Dist. = 1.25 in. , Hole Size = 0.8125 in.
= 0.75 * 1.2 * Lc * Fu ≤ 0.75 * 2.4 * d * Fu = 78.3 kips/in.
= 0.75 * 1.2 * 0.8438 * 58 = 44.044 kips/in.
Bearing Strength/Bolt/Thickness Using Bolt Spacing = Fbs
Bolt Spacing = 3 in. , Hole Size = 0.8125 in.
= 0.75 * 1.2 * Lc * Fu ≤ 0.75 * 2.4 * d * Fu = 78.3 kips/in.
= 0.75 * 1.2 * 2.1875 * 58 = 114.2 kips/in.
Use: Fbs = 78.3 kips/in.
Bearing Strength = 2*(Fbe + Fbs*(n-1))*t
= 2*(44.044 + 78.3 * (4 - 1)) * 0.3125
= 174.3 ≥ 98.6 kips (OK)

Bolt Bearing on Support:

Bearing Strength/Bolt/Thickness Using Bolt Spacing = Fbs
Bolt Spacing = 3 in. , Hole Size = 0.8125 in.
= 0.75 * 1.2 * Lc * Fu ≤ 0.75 * 2.4 * d * Fu = 87.75 kips/in.
= 0.75 * 1.2 * 2.1875 * 65 = 128. kips/in.
Use: Fbs = 87.75 kips/in.
Bearing Strength = 2*Fbs*n*t
= 2*87.75 * 4 * 1.03
= 723 ≥ 98.6 kips (OK)

Beam Side Weld: 3/16 E70XX

Angle Thickness = 0.3125 in.
Beam Web Thickness = 0.43 in.

Minimum Weld = 3/16 in.
Weld Size = 3/16 ≥ 3/16 in. (OK)

Maximum Weld = 1/4 in.
Weld Size = 3/16 ≤ 1/4 in. (OK)

k = 0.3152
a = 0.2869
L = 11.5 in.
Theta = 0 Degrees
ØC = 2.1751
C1 = 1

Weld Strength (Before Beam Web Check):
= 2*C*L*C1*D
= 2*2.1751 * 11.5 * 1 * 3
= 150 kips

Reduction Factor for Beam Web Thickness, Rtw:
= 0.707*fu*tw/((D/16)*Fexx)
= 0.707 * 65 * 0.43/((3/16)*70)
= 1.5056 ≥ 1 No Reduction

Weld Design Strength = 150 kips

Weld Capacity = 150 ≥ 98.6 kips (OK)

Design Shear Strength of Beam Web

Block Shear:

Shear Strength on Net Area, ØRn
= d*tw*0.75 *0.6*Fu
= 23.74 * 0.43 * 0.75 *0.6 * 65
= 298.6 kips

Design Shear Yield Strength:

Rn = 0.6*Fy*A
= 0.6 * 50 * 10.208
= 306.2 kips

ØRn = 1*306.2 = 306.2 kips

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Design Shear Rupture Strength:

Rn = 0.6*Fu*Anv
= 0.6 * 65 * 10.208
= 398.1 kips

ØRn = 0.75*398.1 = 298.6 kips

Beam Shear Strength = Min(ØRn_rupture , ØRn_yield , ØRn_block_shear) = 298.6 ≥ 98.6 kips (OK)
= 298.6 ≥ 98.6 kips (OK)

Design Shear Strength of Angle(s):

Shear Yielding Design Strength:

Gross Area, Ag = L*t = 11.5 * 0.3125 = 3.5938 in^2

ØRn = 2*1 *0.6*Ag*Fy = 2 * 1 *0.6 * 3.5938 * 36 = 155.3 ≥ 98.6 kips (OK)

Shear Rupture Design Strength:

Net Area on Osl, An:
= (L-n*(dh+ .0625))*t = (11.5 - 4 (* 0.8125+ .0625))* 0.3125 = 0 in^2

An = 2.5 in^2

ØRn = 2*0.75 *0.6*An*Fu = 2* 0.75 *0.6 * 2.5 * 58 = 130.5 ≥ 98.6 kips (OK)

Block Shear Strength of Supportside Leg of One Angle:

Gross Length with Tension resistance, Lgt = Lh = 1.465 in.

Net Length with Tension resistance, Lnt
= Lgt - (dh+ .0625)/2 = 1.465 - 1.0625 / 2 = 0.9338 in.

Gross Length with Shear resistance, Lgv
= (n - 1)*s + Lv
= (4 - 1) * 3 + 1.25 = 10.25 in.

Net Length with Shear resistance, Lnv
= Lgv - (n - 0.5)*(dv+ .0625)
= 10.25 - (4 - 0.5)*0.875)
= 7.1875 in.

ØRn = 0.75 *(0.6* Min[Fu * Lnv ; Fy*Lgv] + Fu*Lnt)*t
= 0.75 *(0.6* Min(58 * 7.1875 ; 36 * 10.25) + 1 * 58 * 0.9338) * 0.3125
= 64.584 ≥ 49.3 kips (OK)

Left Side Beam - W24X117
Moment Connection With Directly Welded Flanges:

Weld Strength
Flange Force, Ff:
= P/2 + M/(d-tf)
= 0/2) + 9312/ (24.26 - 0.85)
= 397.8 kips

Full Penetration Weld Design Strength = 0.9 *Fy*b*t
= 0.9 * 50 * 12.8 * 0.85
= 489.6 ≥ 397.8 kips (OK)

Left Side Beam
Shear Connection Using Clip Angle(s):

Clip Angles: 2L4X4X5/16 X 17.5 in.

Angle Material: A36

Support Side Connection: 12 Bolts 3/4"Ø A325-N -SSLN

Bolt Holes on Support: 0.8125 in. Vert. X 0.8125 in. Horiz.
Effective Thickness of Support Material: 1.03 in.
Bolt Holes on Angles: 0.8125 in. Vert. X 1 in. Horiz.

Beam Side Connection: 3/16 E70XX Fillet Welds

Beam Web Thickness: 0.55 in.
Beam Web Height: 20.75 in.
Beam Setback: 0.375 in.

Loading:
Vertical Shear, V = 185.4 kips
Axial Load, H = 0 kips
Resultant, R = (V^2 + H^2)^0.5 = ((185.4)^2 + (0)^2)^0.5 = 185.4 kips

Check Clearances:
Beam Web Clear Height = 20.75 ≥ 17.5 in. (OK)

Support Side Bolts
Spacing, s = 3 ≥ Minimum Spacing = 2 in. (OK)

Distance to Horizontal Edge, ev:
= 1.25 ≥ 1.25 in. (OK)

Distance to Vertical Edge, eh:
= 1.525 ≥ 1.125 in. (OK)

Gage on OSL:
Angle Gage = 2.475 ≥ 1.5625 in. (OK)
Column Gage = 5.5 in.

Design Shear Strength of Bolts:
= 2*n*(Ørn) = 2 * 6 * 17.892 = 214.7 ≥ 185.4 kips (OK)

Bolt Bearing on Angle(s):
Bearing Strength/Bolt/Thickness Using Bolt Edge Distance = Fbe
Edge Dist. = 1.25 in. , Hole Size = 0.8125 in.
= 0.75 * 1.2 * Lc * Fu ≤ 0.75 * 2.4 * d * Fu = 78.3 kips/in.
= 0.75 * 1.2 * 0.8438 * 58 = 44.044 kips/in.
Bearing Strength/Bolt/Thickness Using Bolt Spacing = Fbs
Bolt Spacing = 3 in. , Hole Size = 0.8125 in.
= 0.75 * 1.2 * Lc * Fu ≤ 0.75 * 2.4 * d * Fu = 78.3 kips/in.
= 0.75 * 1.2 * 2.1875 * 58 = 114.2 kips/in.
Use: Fbs = 78.3 kips/in.
Bearing Strength = 2*(Fbe + Fbs*(n-1))*t
= 2*(44.044 + 78.3 * (6 - 1)) * 0.3125
= 272.2 ≥ 185.4 kips (OK)

Bolt Bearing on Support:

Bearing Strength/Bolt/Thickness Using Bolt Spacing = Fbs
Bolt Spacing = 3 in. , Hole Size = 0.8125 in.
= 0.75 * 1.2 * Lc * Fu ≤ 0.75 * 2.4 * d * Fu = 87.75 kips/in.
= 0.75 * 1.2 * 2.1875 * 65 = 128. kips/in.
Use: Fbs = 87.75 kips/in.
Bearing Strength = 2*Fbs*n*t
= 2*87.75 * 6 * 1.03
= 1085 ≥ 185.4 kips (OK)

Beam Side Weld: 3/16 E70XX

Angle Thickness = 0.3125 in.
Beam Web Thickness = 0.55 in.

Minimum Weld = 3/16 in.
Weld Size = 3/16 ≥ 3/16 in. (OK)

Maximum Weld = 1/4 in.
Weld Size = 3/16 ≤ 1/4 in. (OK)

k = 0.2071
a = 0.1982
L = 17.5 in.
Theta = 0 Degrees
ØC = 2.0076
C1 = 1

Weld Strength (Before Beam Web Check):
= 2*C*L*C1*D
= 2*2.0076 * 17.5 * 1 * 3
= 210.8 kips

Reduction Factor for Beam Web Thickness, Rtw:
= 0.707*fu*tw/((D/16)*Fexx)
= 0.707 * 65 * 0.55/((3/16)*70)
= 1.9257 $\geq$ 1 No Reduction

Weld Design Strength = 210.8 kips

Weld Capacity = 210.8 $\geq$ 185.4 kips (OK)

Design Shear Strength of Beam Web

Block Shear:

Shear Strength on Net Area, ØRn
= d*tw*0.75 *0.6*Fu
= 24.26 * 0.55 * 0.75 *0.6 * 65
= 390.3 kips

Design Shear Yield Strength:

Rn = 0.6*Fy*A
= 0.6 * 50 * 13.343
= 400.3 kips

ØRn = 1*400.3 = 400.3 kips

Design Shear Rupture Strength:

Rn = 0.6*Fu*Anv
= 0.6 * 65 * 13.343
= 520.4 kips

ØRn = 0.75*520.4 = 390.3 kips

Beam Shear Strength = Min(ØRn_rupture , ØRn_yield , ØRn_block_shear) = 390.3 $\geq$ 185.4 kips (OK)
= 390.3 $\geq$ 185.4 kips (OK)

Design Shear Strength of Angle(s):

Shear Yielding Design Strength:

Gross Area, Ag = L*t = 17.5 * 0.3125 = 5.4688 in^2

ØRn = 2*1 *0.6*Ag*Fy = 2 * 1 *0.6 * 5.4688 * 36 = 236.3 $\geq$ 185.4 kips (OK)

Shear Rupture Design Strength:

Net Area on Osl, An:
= (L-n*(dh+ .0625))*t = (17.5 - 6 (* 0.8125+ .0625))* 0.3125 = 0 in^2

An = 3.8281 in^2

ØRn = 2*0.75 *0.6*An*Fu = 2* 0.75 *0.6 * 3.8281 * 58 = 199.8 $\geq$ 185.4 kips (OK)

Block Shear Strength of Supportside Leg of One Angle:

Gross Length with Tension resistance, Lgt = Lh = 1.525 in.

Net Length with Tension resistance, Lnt
= Lgt - (dh+ .0625)/2 = 1.525 - 1.0625 / 2 = 0.9938 in.

Gross Length with Shear resistance, Lgv
= (n - 1)*s + Lv
= (6 - 1) * 3 + 1.25 = 16.25 in.

Net Length with Shear resistance, Lnv
= Lgv - (n - 0.5)*(dv+ .0625)
= 16.25 - (6 - 0.5)*0.875)
= 11.438 in.

ØRn = 0.75 *(0.6* Min[Fu * Lnv ; Fy*Lgv] + Fu*Lnt)*t
= 0.75 *(0.6* Min(58 * 11.438 ; 36 * 16.25) + 1 * 58 * 0.9938) * 0.3125
= 95.774 $\geq$ 92.7 kips (OK)

Column Web Shear Reinforcement

Framing System: OMF

Column Axial Force, Pu = 0 kips
Column Shear Force, Vus = 0 kips

Right Side Beam Flange Forces:

PufRight = Mu / dm + Pu / 2
= 4680/23.15 + 0/2
= 202.2 kips

Left Side Beam Flange Forces:

PufLeft = Mu / dm + Pu / 2
= 9312/23.41 + 0/2
= 397.8 kips

Column Panel Zone:

Required Strength, Vu
= | PufLeft + PufRight - Vus |
= | 397.8 + 202.2 - 0 |
= 599.9 kips
Use Vu = 0 kips (User Specified)

Column Web Shear Strength:

Pc = Py = A * Fy = 38.8 * 50 = 1940 kips

Pr $\leq$ 0.4*Pc
ØRv = 0.9 *0.6*Fy*d*tw
= 0.9 * 0.6 * 50 * 14.66 * 0.645
= 255.3 $\geq$ 0 kips
(Doubler Plate Not Required for Strength)

Shear Buckling of Web:

Thickness Required = h*(Fy^0.5)/(2.24*E^0.5) = 11.4 * (50^0.5)/(2.24*(290.x10^2)^0.5)
= 0.2113 $\leq$ 0.645 in.
(Doubler Plate Not Required for Shear Buckling)

Column Stiffeners

Framing System: OMF

Column Axial Force, Pu = 0 kips
Column Shear Force, Vus = 0 kips

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Right Side Beam Flange Forces:

PufRight = Mu / dm + Pu / 2
= 4680/23.15 + 0/2
= 202.2 kips

Left Side Beam Flange Forces:

PufLeft = Mu / dm + Pu / 2
= 9312/23.41 + 0/2
= 397.8 kips

Column Stiffeners

Right Side Beam

Local Flange Bending Strength,ØRn

= 0.9 *6.25*(tf^2)*Fy*ct
= 0.9 *6.25*(1.03^2)*50*1
= 298.4 kips

Local Web Yielding Strength,ØRn
= 1*(ct*5*k + t)*tw*Fy
= 1*(1*5* 1.63 + 0.59)* 0.645 * 50
= 281.9 kips

Column Web Crippling:

N = tf = 0.59 in.

Ct = 1.0
Nd = 3*N/d = 3 * 0.59/14.66 = 0.1207

ØRn = 0.75*0.8*ct*(tw^2)*[1+Nd*(tw/tf)^1.5]*(E*Fy*tf/tw)^0.5
= 0.75*0.8*1*(0.645^2)*[1+0.1207*(0.645/1.03)^1.5]*(290.x10^2 * 50*1.03/0.645)^0.5
= 402.6 kips

Compression Buckling of the Web:

ØRn = 0.9 *24*ct * tw^3 * (E*Fy)^0.5 /(d - 2*k)
= 0.9 * 24 * 1 * 0.645^3 * (290.x10^2 * 50)^0.5 / (14.66 - 2 * 1.63)
= 612.2 kips

Left Side Beam

Local Flange Bending Strength,ØRn

= 0.9 *6.25*(tf^2)*Fy*ct
= 0.9 *6.25*(1.03^2)*50*1
= 298.4 kips

Local Web Yielding Strength,ØRn

= 1*(ct*5*k + t)*tw*Fy
= 1*(1*5* 1.63 + 0.85)* 0.645 * 50
= 290.3 kips

Column Web Crippling:

N = tf = 0.85 in.

Ct = 1.0
Nd = 3*N/d = 3 * 0.85/14.66 = 0.1739

ØRn = 0.75*0.8*ct*(tw^2)*[1+Nd*(tw/tf)^1.5]*(E*Fy*tf/tw)^0.5
= 0.75*0.8*1*(0.645^2)*[1+0.1739*(290.x10^2 * 0.645/1.03)^1.5]*(50*1.03/0.645)^0.5
= 412.6 kips

Compression Buckling of the Web:

ØRn = 0.9 *24*ct * tw^3 * (E*Fy)^0.5 /(d - 2*k)
= 0.9 * 24 * 1 * 0.645^3 * (290.x10^2 * 50)^0.5 / (14.66 - 2 * 1.63)
= 612.2 kips

Tension Flange Stiffener Force, TFrc:

Left Side:

LTFrc = Max(LPuf - LØRn_FlBending; LPuf - LØRn_WebYielding) ≥ 0
= max(397.8 - 298.4; 397.8 - 290.3) = 107.5 kips

Right Side:

RTFrc = Max(RPuf - RØRn_FlBending; RPuf - RØRn_WebYielding) ≥ 0
= max(202.2 - 298.4; 202.2 - 281.9) = 0 kips

Compression Flange Stiffener Force, CFrc:

Left Side:

LCFrc = Max[(LPuf-LØRn_WebCrippling); (LPuf-LØRn_WebYielding); (LPuf-LØRn_WebBuckling)] ≥ 0
= max[(397.8 - 412.6); (397.8 - 290.3); (397.8 - 612.2)] = 107.5 kips

Right Side:

RCFrc = Max[(RPuf-RØRn_WebCrippling); (RPuf-RØRn_WebYielding); (RPuf-RØRn_WebBuckling)] ≥ 0
= max[(202.2 - 402.6); (202.2 - 281.9); (202.2 - 612.2)] = (-79.705) kips

TFrc = Max(LTFrc, RTFrc) = Max(107.5; 0) = 107.5 kips
CFrc = Max(LCFrc, RCFrc) = Max(107.5;(-79.705)) = 107.5 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) / (0.9 * Fy)
= max(107.5; 107.5) / (0.9 * 50)
=max(2.3895; 2.3895) in^2
Stiffener Width, bs = 4 ≥ Minimum Width = 3.9442in. (OK)

Stiffener Length:
L = d-2*tf = 14.66 - 2 * 1.03 = 12.563 in.
(Using Full Length Stiffeners)

Stiffener thickness required for shear:
= Max([LTFrc + RCFrc];[LCFrc + RTFrc])/(0.9 *0.6*Fy*(L - 2*clip)*2)
= Max([107.5+(-79.705)]; [107.5+0])/(0.9 *0.6* 50 * (12.563 - 2.565) * 2)
= 0.1992 ≤ 0.5in. (OK)

Stiffener thickness required for minimum area:
= Ast/(2*(bs-clip))
= 2.3895/(2*(4 - 1.2825)) = 0.4397 ≤ 0.5in. (OK)

Minimum Thickness = Max(tm/2; bs*(Fy/E)^0.5/0.56)

= Max(0.85/2; 4 * (50 / 290.x10^2)^0.5 / 0.56)
= 0.425 ≤ 0.5in. (OK)

Stiffener Welds:

Stiffener to Flange Weld:

Minimum Weld Size = 0.1875 ≤ 0.3125 in. (OK)
Tension Stiffener to Flange Weld:

w_Req.= 0.943*Fy*t/Fexx
= 0.943 * 50 * 0.5 / 70 0.4397
= ~~0.3368 >~~ 0.3125 in. (~~NG~~) OK
0.296 <

Compression Stiffener to Flange Weld:

w_Req.= 0.524 * Rust/((bs - clip)*Fexx)
w_Req.= 0.524 * 107.5/((4 - 1.2825) * 70)
= 0.2962 ≤ 0.3125 in. (OK)

Stiffener to Panel Zone Weld:

Stiffener Force, Rust = Max[(LTRust + RcRust); (RTRust + LCRust)]
= Max[(107.5 + (-79.705)); (0 + 107.5)] = 107.5 kips

Welds need to develop only the lesser of Rust
and the minimum of the following forces:

Design Strength of stiffeners to flange connection,
= 0.9 * Fy * 4 * (bs - clip) * t
= 0.9 * 50 * 4 * (4 - 1.2825) * 0.5 = 244.6 kips

Shear Strength of stiffener and web interface area,
= 0.9 * 0.6 * Fy * (bs - 2*clip) * 2 * t
= 0.9 * 0.6 * 50 * (4 - 2*1.2825) * 2 * 0.5 = 269.9 kips

Shear yield strength of the panel zone,
= 0.9 * 0.6 * Fyc * dc * tw (for column web, if applicable)
= 0.9 * 0.6 * 50 * 14.66 * 0.645 = 255.3 kips

= 0.9 * 0.6 * Fyp * dc * tp (for doubler plate, if applicable)
= 0.9 * 0.6 * 50 * 14.66 * 0 = 0 kips

Weld Design Force, Rust_Weld = 107.5 kips

Minimum Weld Size = 0.1875 ≤ 0.3125 in. (OK)
Required weld size for strength,
= Rust_Weld /(1.2728 * Fexx * (L - 2*clip))
= 107.5 / (1.2728 * 70 * (12.563 - 2*1.2825))
= 0.1207 ≤ 0.3125 in. (OK)

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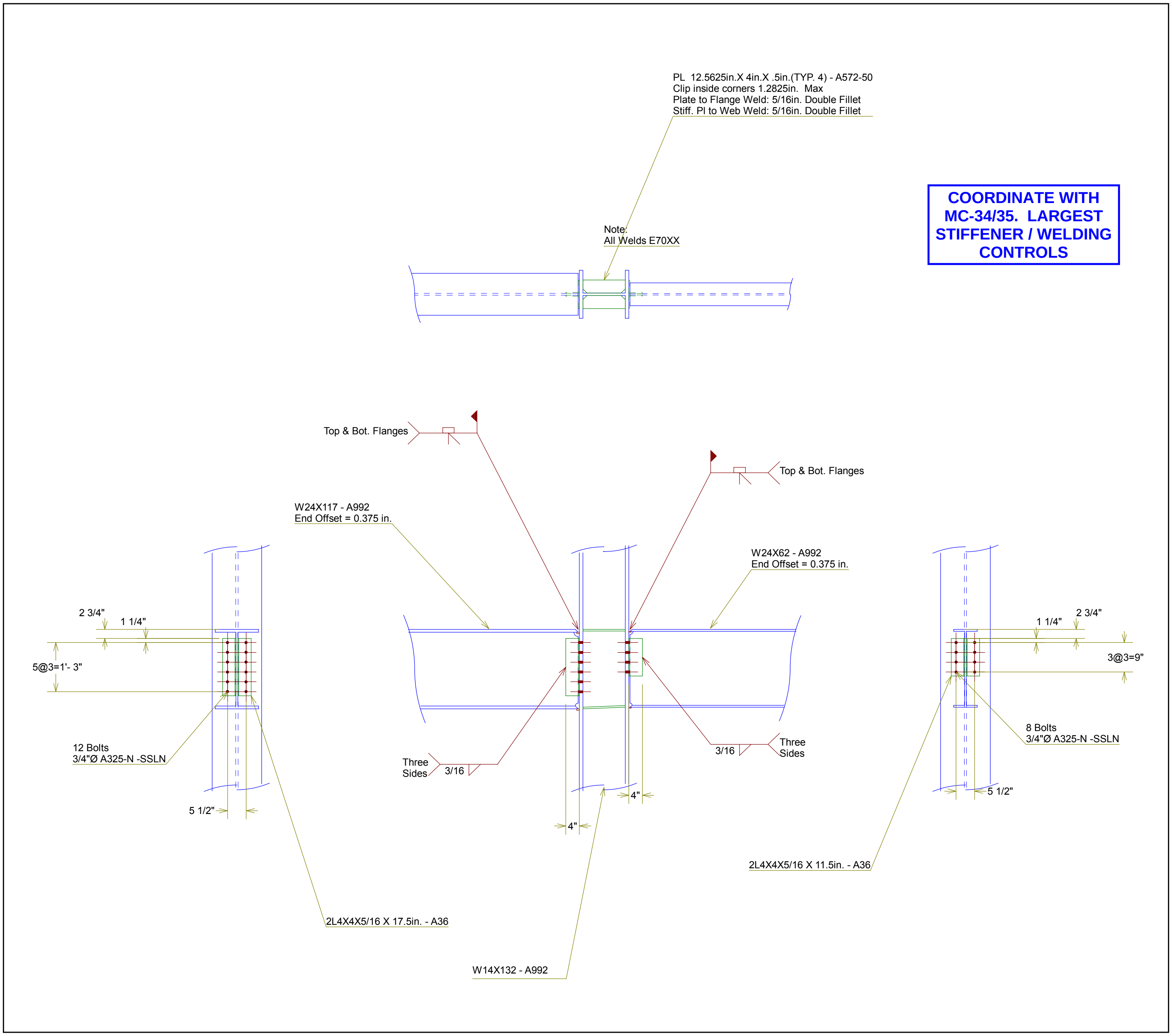

Scale: 1/4" = 1'