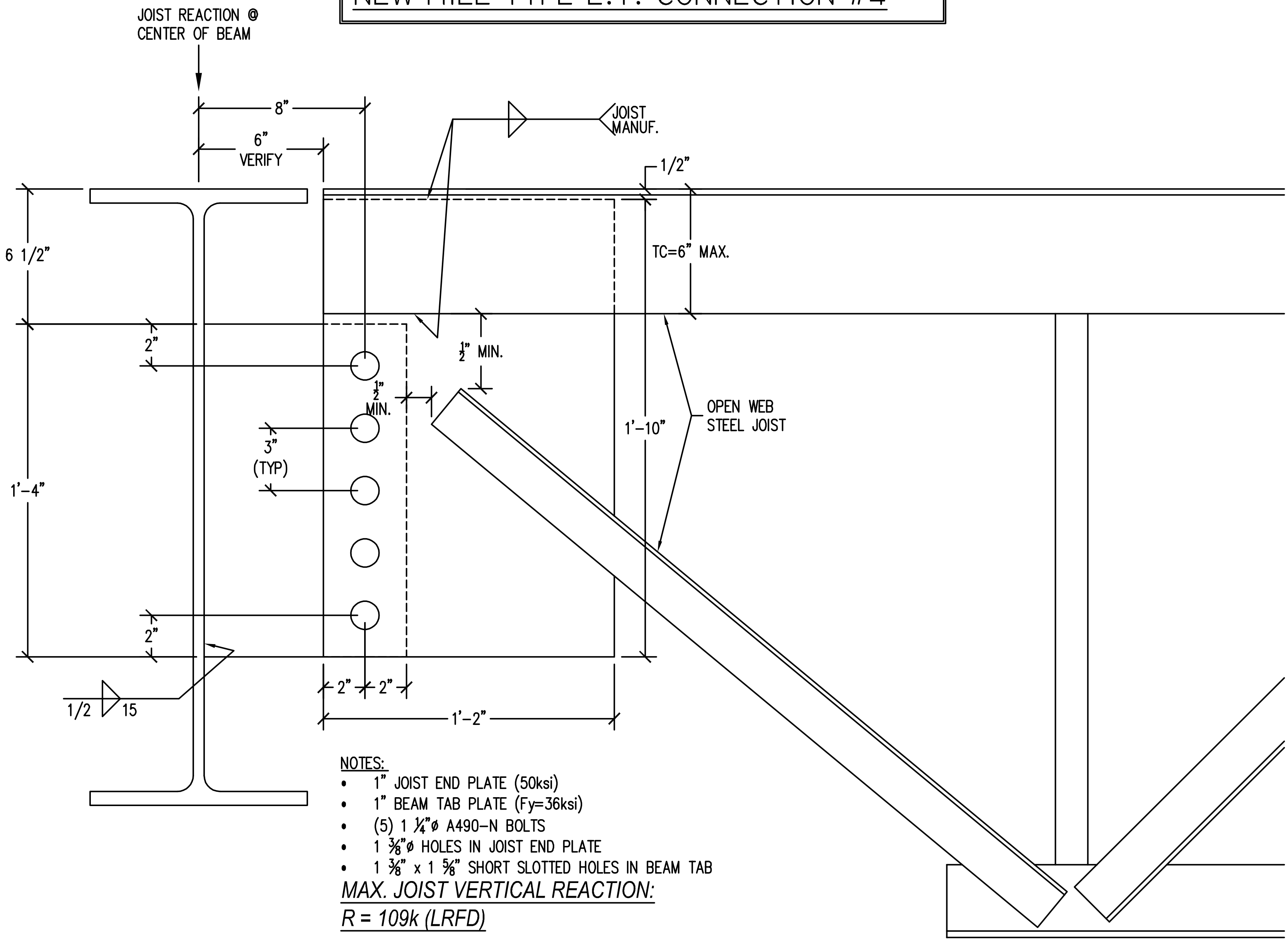
EXTENDED-TAB (ET) CONNECTION

NEW MILL-TYPE E.T. CONNECTION #4



NOTES:

- 1" JOIST END PLATE (50ksi)
- 1" BEAM TAB PLATE (Fy=36ksi)
- (5) 1 1/4"ø A490-N BOLTS
- 1 3/8"ø HOLES IN JOIST END PLATE
- 1 3/8" x 1 5/8" SHORT SLOTTED HOLES IN BEAM TAB

MAX. JOIST VERTICAL REACTION:

R = 109k (LRFD)

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Date: 9/26/2023

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AISC 14TH - p. 10-104
Holes must satisfy AISC J3.2
Horizontal Axial forces (seismic or wind) if present, to be transferred from beam to joist via tie plate

Joist Plate, Fu =	65	ksi	TC Hold Back Distance (H) =	6	in
Joist Tab Plate, Fy =	50	ksi	TC Angle Size =	3.5	in
Joist Plate Width, w=	14	in	Joist Plate Hold-Down from TC =	0.5	in
Joist Plate Thickness, t=	1	in			
Joist Plate Edge Distances, de=	2	in			
e =	8	in	L_{i1} =	6	
Vertical Shear , Vu=	109	k (LRFD)	L_{i2} =	3	
Vertical Ecc. Moment , Mu=	872	k*in (LRFD)	L_{i3} =	0	
Joist Top Chord Axial Force, V_{TC}=	163.5	k (LRFD)...**Assumes 1.5:1 End Web Slope**	L_{i4} =	0	
Bolt Diam. =	1.25	in	L_{i5} =	0	
Bolt Shear Capacity ϕRn =	62.7	k (A490-N)			
# of Bolts, Nb=	5	(Spreadsheet design limitation, max. 10 bolts)			
Spacing of Bolt Group, S=	3	in			
Vert. C.G. of Bolt Group =	6	in			
Fv/0.9 =	75.56	ksi (Table J3.2, A490-N Bolts)			
Short Slotted Hole, Lh =	1.63	in			

Gross Plate Area, Ag =	14	in^2
Effective Plate Area, Ae =	12.63	in^2
Z =	49	in^3 (1/4t*w^2)
S =	32.6667	in^3 (1/6t*w^2)

Stress Ratio Results:	
Bolt Shear (V&M):	**0.99**
Shear Plate Yielding:	**0.50**
Shear Plate Rupture:	**0.29**
Shear Plate Block Shear:	**0.45**
Shear Plate Local Buckling:	**0.94**

Min. Joist TC to Plate Weld:
3 /16th x 13.5 " Fillet Weld

Bolt Shear - Elastic Vector Method: (AISC p. 7-8, 7-9)

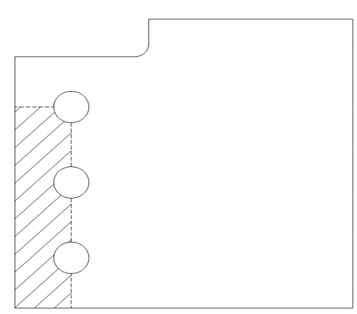
Bolt Group I_p =	90.00	in4/in2
r_{py} =	21.80	k (Vu/# Bolts)
r_{mx} =	58.13	k ($Mu*L_{i1}/I_p$)
Hm =	58.13	k (rmx *Nc) Nc = 1 column of bolts
Ru =	62.09	k $(r_{py}{}^2+r_{mx}{}^2)^{1/2}$
Ru / ϕRn =	**0.99**	**< 1.0 OK**

Shear Plate Yielding:

ϕVn =	420	k (ϕ = 1.0, ϕ*0.6*Fy*Ag)
Horiz. Axial Shear V_{TC}/ϕVn=	**0.39**	**< 1.0 OK**
ϕMn =	1470	k*in (ϕ = 0.9, ϕ*Fy*S)
Mu/ϕMn=	**0.59**	**< 1.0 OK**
Ru / ϕRn =	**0.50**	**< 1.0 OK** $(Vu/\phi Vn)^2+(Mu/\phi Mn)^2$

Shear Plate Rupture: (AISC p.9-6)

Crushed Hole Width, W' =	1.4375	in (plate hole + 1/16" Crushed width)
Net Plastic Modulus, Z_{net} =	39.97	in^3 ($Z - W'*t*d_{hole}$) d_{hole} = 6.28125 in
ϕVn =	369.28	k (ϕ = 0.75, ϕ*.60*Fu*Ae)
ϕMn =	1948.57	k*in (ϕ = 0.75, ϕ*Fu*Z)
Ru / ϕRn =	**0.29**	**< 1.0 OK** $(Vu/\phi Vn)^2+(Mu/\phi Mn)^2$



Shear Plate Block Shear: (AISC J4.3)
Vertical Direction

Gross Area in Shear, Agv=	14.00	in^2 $(t*(d_e+(Nb-1)*S)$
Net Area in Shear, Anv=	6.81	in^2 Agv-(Nb*W')*t
Net Area in Tension, Ant=	1.19	in^2 $(t*(d_e-(Nc-0.5)*Lh)$, Nc = 1 column of bolts
Gross Area, ϕRn =	392.19	k
Net Area, ϕRn =	276.45	k

Note: Use of Lh for determination of Net Plate Area, allows for the slots to be in the joist end plate, rather than the beam tab.

Horizontal Direction

Gross Area in Shear, Agv=	4.00	in^2 $(2*t*d_e)$
Net Area in Shear, Anv=	2.38	in^2 $(2*t*(d_e-(Nc-0.5)*Lh)$, Nc = 1 column of bolts
Net Area in Tension, Ant=	6.25	in^2 (t*((Nb-1)*S-(Nb-1)*W)
Gross Area, ϕRn =	496.25	k
Net Area, ϕRn =	475.72	k

Note: Use of Lh for determination of Net Plate Area, allows for the slots to be in the joist end plate, rather than the beam tab.

ϕRn =	**276.45**	**k Controls**
Ru / ϕRn =	**0.45**	**< 1.0 OK** $(Vu^2+Hm^2)^{1/2}/\phi Rn$

Shear Plate Local Buckling: (AISC p.10-103, p.9-6)

Shear Stress, fv=	11.68	ksi (V_{TC}/Ag)
Critical Stress, Fcr =	31.58	ksi $((\phi*Fy)^2-3*fv^2)^{1/2}$ ϕ=0.75, von Mises Yield
λ =	0.08	AISC Eq. 9-18
Q=	1	AISC Eq. 9-15 through 9-17
Fcr =	50	ksi (Q*Fy) Classic Plate Buckling
von Mises ϕMn =	928.35	k*in (ϕ*Fcr*S) ϕ = 0.9
Classic Plate Buckling ϕMn =	1470.00	k*in (ϕ*Fcr*S) ϕ = 0.9
Govering ϕMn =	**928.35**	**k*in**
Mu / ϕMn =	**0.94**	**< 1.0 OK**

Joist Plate Weld (Angle = 0 deg. & C_1 = 1.00 E70 Electrode):

Length of Plate Weld L_w =	**13.5**	in (w-0.5")
a_y =	0.1	AISC Table 8-4 a_y = (Weld Centroid - TC Centroid) / L_w
k_y =	0.3	AISC Table 8-4 k_y = Weld Spacing / L_w
Cy =	**3.71**	(y-axis weld eccentricity, AISC Table 8-4)
D_{min} =	**3**	/16ths of an inch Fillet Weld Size (min)

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cp

Beam Tab Plate, Fu =	58	ksi		
Beam Tab Plate, Fy =	36	ksi		
Beam Tab Plate Depth, d=	16	in		
Beam Tab Thickness, t=	1	in		
Beam Tab Edge Distances, d_e=	2	in	L_{i1} =	6
e =	8	in	L_{i2} =	3
Vertical Shear , Vu=	109	k (LRFD)	L_{i3} =	0
Vertical Ecc. Moment , Mu=	872	k*in (LRFD)	L_{i4} =	0
Bolt Diam. =	1.25	in	L_{i5} =	0
Bolt Shear Capacity ϕRn =	62.7	k		
# of Bolts, Nb=	5	(Spreadsheet design limitation, max. 10 bolts)		
Spacing of Bolt Group, S=	3	in		
C.G. of Bolt Group =	6	in		
Fv/0.9 =	75.56	ksi (Table J3.2, A490-N Bolts)		
Short Slotted Hole, Lh =	1.63	in		

Ab=	1.23	in^2 (Bolt Area)
C'=	17.15	AISC Eq. 7-21, p. 7-19
Mmax =	1589.97	k*in (Fv/0.9*Ab*C', Eq. 10-4)
Max. Beam Tab Thickness, tmax =	**1.04**	**in** (6*Mmax)/(Fy*d^2) **AISC Eq. 10-3**

Stress Ratio Results:	
Bolt Shear (V&M):	**0.99**
Shear Tab Yielding:	**0.28**
Shear Tab Rupture:	**0.49**
Shear Tab Block Shear:	**0.50**
Shear Tab Local Buckling:	**0.63**
8 /16" Tab Weld:	**0.31**
Plate Stability Acceptable	

Gross Plate Area, Ag =	16	in^2
Effective Plate Area, Ae =	9.13	in^2
Z =	64	in^3 (1/4t*d^2)
S_{net} =	42.67	in^3 (1/6t*d^2)

Bolt Shear - Elastic Vector Method: (AISC p. 7-8, 7-9)

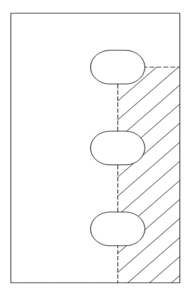
Bolt Group I_p =	90.00	in4/in2
r_{py} =	21.80	k (Vu/# Bolts)
r_{mx} =	58.13	k (Mu*L_{i1}/I_p)
Hm =	58.13	k (rmx *Nc) Nc = 1 column of bolts
Ru =	62.09	k $(r_{py}^2+r_{mx}^2)^{1/2}$
Ru / ϕRn =	**0.99**	**< 1.0 OK**

Shear Tab Yielding:

ϕVn =	345.6	k (ϕ = 1.0, ϕ*0.6*Fy*Ag)
ϕMn =	2073.6	k*in (ϕ = 0.9, ϕ*Fy*Z)
Ru / ϕRn =	**0.28**	**< 1.0 OK** $(Vu/\phi Vn)^2+(Mu/\phi Mn)^2$

Shear Tab Rupture: (AISC p.9-6)

Crushed Hole Width, W' =	1.4375	in (plate hole + 1/16" Crushed width)
Net Plastic Modulus, Z_{net} =	37.61	in^3 (Summation of A*d of net plate section)
ϕVn =	238.16	k (ϕ = 0.75, ϕ*.60*Fu*Ae)
ϕMn =	1635.97	k*in (ϕ = 0.75, ϕ*Fu*Z)
Ru / ϕRn =	**0.49**	**< 1.0 OK** $(Vu/\phi Vn)^2+(Mu/\phi Mn)^2$



Shear Tab Block Shear: (AISC J4.3)
Vertical Direction

Gross Area in Shear, Agv=	14.00	in^2 (t*(d_e+(Nb-1)*S)
Net Area in Shear, Anv=	6.81	in^2 Agv-(Nb*W')*t
Net Area in Tension, Ant=	1.19	in^2 (t*(d_e-(Nc-0.5)*Lh), Nc = 1 column of bolts
Gross Area, ϕRn =	295.68	k
Net Area, ϕRn =	246.68	k

Horizontal Direction

Gross Area in Shear, Agv=	4.00	in^2 (2*t*d_e)
Net Area in Shear, Anv=	2.38	in^2 (2*t*(d_e-(Nc-0.5)*Lh), Nc = 1 column of bolts
Net Area in Tension, Ant=	6.25	in^2 (t*((Nb-1)*S-(Nb-1)*W)
Gross Area, ϕRn =	427.30	k
Net Area, ϕRn =	424.49	k

ϕRn =	**246.68**	**k Controls**
Ru / ϕRn =	**0.50**	**< 1.0 OK** $(Vu^2+Hm^2)^{1/2}/\phi Rn$

Shear Tab Local Buckling: (AISC p.10-103, p.9-6)

Shear Stress, fv=	6.81	ksi (Vu/Ag)
Critical Stress, Fcr =	34.01	ksi $(Fy^2-3*fv^2)^{1/2}$ von Mises Yield AISC p. 10-103
λ =	0.24	AISC Eq. 9-18
Q =	1	AISC Eq. 9-15 through 9-17
Fcr =	36	ksi (Q*Fy) Classic Plate Buckling
von Mises ϕMn =	1959.05	k*in (ϕ*Fcr*Z) ϕ = 0.9
Classic Plate Buckling ϕMn =	1382.40	k*in (ϕ*Fcr*Z) ϕ = 0.9
Govering ϕMn =	**1382.40**	**k*in**
Mu / ϕMn =	**0.63**	**< 1.0 OK**

Shear Tab Weld: (AISC p.10-102, p.9-6)

Min. Weld Thickness t_{wmin} =	0.45	in. $t_{wmin} = (t*Fy*3^{1/2})/(2*F_{EXX})$, F_{EXX} = 70ksi Electrode, AISC Engineering Journal, Vol. 46, 2009
Weld Provided t_w =	0.5	in
Min. Plate Thickness =	0.85	in (AISC Eq. 9-3, 6.19*D/Fu) **GOOD**
ϕRw =	356.33	k (ϕ*0.6*F_{EXX}*0.707*t_w*d*2)
Ru / ϕRn =	**0.31**	**< 1.0 OK**

Shear Tab Stability: (Thorton and Fortney, 2011)
Lateral Torsional Buckling Check:

ϕRn = **1060** k **Acceptable**

$$R_{req'd} \le \phi R_n \text{ (LRFD)}$$

$$R_n = 1500\pi \frac{lt^3}{a^2}$$

l = beam tab plate length (depth)
t = tp = beam tab plate thickness
a = Eccentricity 'e'

Lap Splice Eccentricity Check:

ϕMt,u =	118.3	k*in
Mt,u =	109.0	k*in (R*(t_p+t_j)/2

$$M_{t,u} \le \left[\phi_v\left(0.6F_{yp}\right) - \frac{R_u}{lt_p}\right]\frac{lt_p^2}{2} \text{ (LRFD)}$$

$$\phi_v = 1.0$$

Acceptable