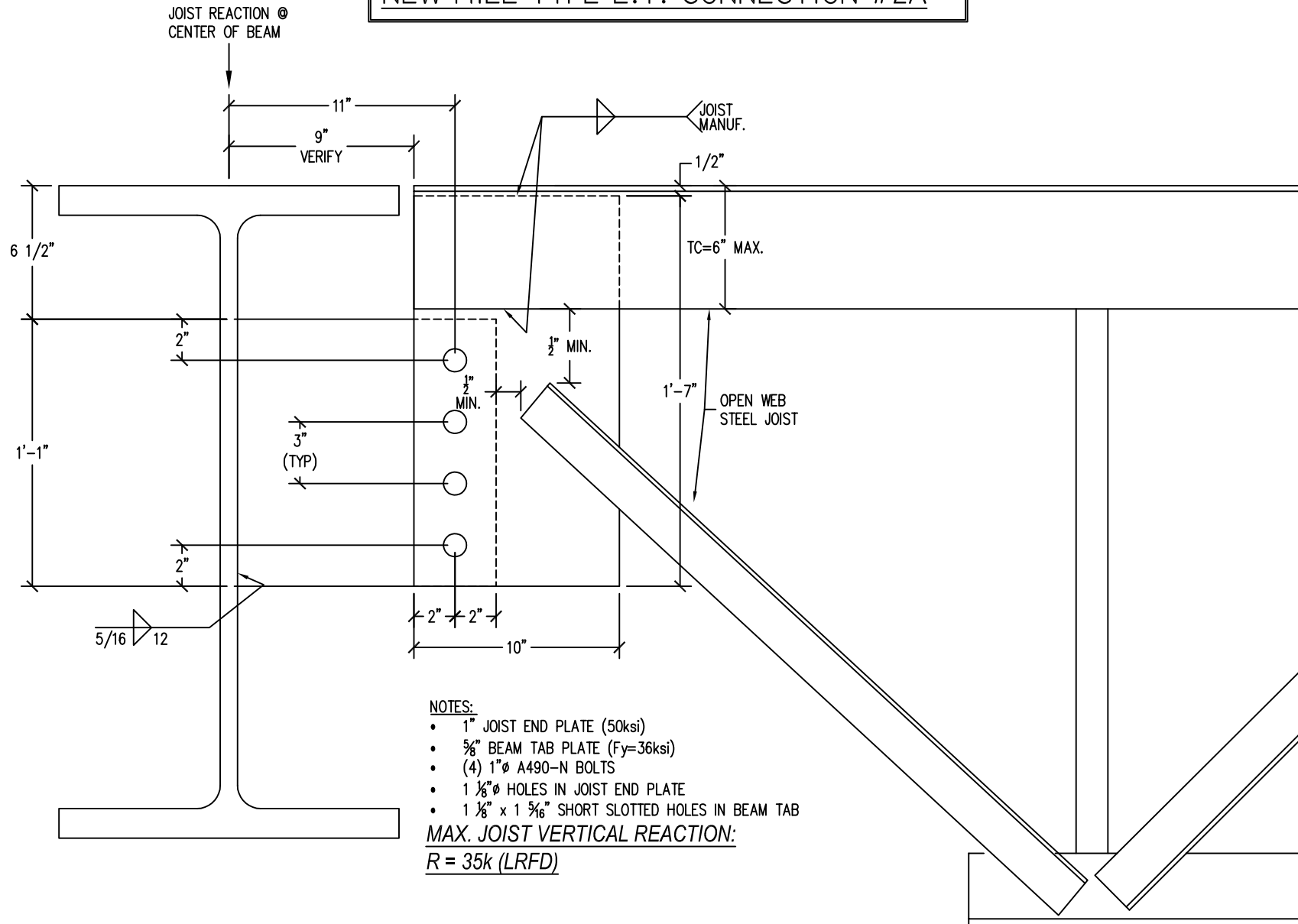


# EXTENDED-TAB (ET) CONNECTION

## NEW MILL-TYPE E.T. CONNECTION #2A



**NEW MILLENNIUM**

A Steel Dynamics Company

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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, $F_u =$	65	ksi	TC Hold Back Distance (H) =	9	in
Joist Tab Plate, $F_y =$	50	ksi	TC Angle Size =	3.5	in
Joist Plate Width, $w =$	10	in	Joist Plate Hold-Down from TC =	0.5	in
Joist Plate Thickness, $t =$	1	in			
Joist Plate Edge Distances, $d_e =$	2	in			
$e =$	11	in	$L_{i1} =$	4.5	
Vertical Shear, $V_u =$	35	k (LRFD)	$L_{i2} =$	1.5	
Vertical Ecc. Moment, $M_u =$	385	k*in (LRFD)	$L_{i3} =$	0	
Joist Top Chord Axial Force, $V_{TC} =$	52.5	k (LRFD)...	$L_{i4} =$	0	
		<b>Assumes 1.5:1 End Web Slope</b>	$L_{i5} =$	0	
Bolt Diam. =	1	in			
Bolt Shear Capacity $\phi R_n =$	40	k (A490-N)			
# of Bolts, $N_b =$	4	(Spreadsheet design limitation, max. 10 bolts)			
Spacing of Bolt Group, $S =$	3	in			
Vert. C.G. of Bolt Group =	4.5	in			
$F_y/0.9 =$	75.56	ksi (Table J3.2, A490-N Bolts)			
Short Slotted Hole, $L_h =$	1.31	in			

Gross Plate Area, $A_g =$	10	in <sup>2</sup>
Effective Plate Area, $A_e =$	8.88	in <sup>2</sup>
$Z =$	25	in <sup>3</sup> (1/4t*w <sup>2</sup> )
$S =$	16.6667	in <sup>3</sup> (1/6t*w <sup>2</sup> )

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

Bolt Group $l_p =$	45.00	in <sup>4</sup> /in <sup>2</sup>
$r_{pv} =$	8.75	k (Vu/# Bolts)
$r_{mx} =$	38.50	k (Mu*L <sub>i1</sub> /l <sub>p</sub> )
$H_m =$	38.50	k (r <sub>mx</sub> *N <sub>c</sub> ) N <sub>c</sub> = 1 column of bolts
$R_u =$	39.48	k (r <sub>pv</sub> <sup>2</sup> +r <sub>mx</sub> <sup>2</sup> ) <sup>1/2</sup>
$R_u / \phi R_n =$	0.99	<1.0 OK

**Shear Plate Yielding:**

$\phi V_n =$	300	k ( $\phi = 1.0, \phi * 0.6 * F_y * A_g$ )
Horiz. Axial Shear $V_{TC} / \phi V_n =$	0.18	<1.0 OK
$\phi M_n =$	750	k*in ( $\phi = 0.9, \phi * F_y * S$ )
$M_u / \phi M_n =$	0.51	<1.0 OK
$R_u / \phi R_n =$	0.29	<1.0 OK (Vu/ $\phi V_n$ ) <sup>2</sup> +(Mu/ $\phi M_n$ ) <sup>2</sup>

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

Crushed Hole Width, $W' =$	1.1875	in (plate hole + 1/16" Crushed width)
Net Plastic Modulus, $Z_{net} =$	19.77	in <sup>3</sup> (Z - W'*t*d <sub>hole</sub> ) d <sub>hole</sub> = 4.40625 in
$\phi V_n =$	259.59	k ( $\phi = 0.75, \phi * .60 * F_u * A_e$ )
$\phi M_n =$	963.67	k*in ( $\phi = 0.75, \phi * F_u * Z$ )
$R_u / \phi R_n =$	0.18	<1.0 OK (Vu/ $\phi V_n$ ) <sup>2</sup> +(Mu/ $\phi M_n$ ) <sup>2</sup>

**Shear Plate Block Shear:** (AISC J4.3)

Vertical Direction		
Gross Area in Shear, $A_{gv} =$	11.00	in <sup>2</sup> (t*(d <sub>e</sub> +(N <sub>b</sub> -1)*S)
Net Area in Shear, $A_{nv} =$	6.25	in <sup>2</sup> A <sub>gv</sub> -(N <sub>b</sub> *W')*t
Net Area in Tension, $A_{nt} =$	1.34	in <sup>2</sup> (t*(d <sub>e</sub> -(N <sub>c</sub> -0.5)*L <sub>h</sub> ), N <sub>c</sub> = 1 column of bolts
Gross Area, $\phi R_n =$	334.84	k
Net Area, $\phi R_n =$	270.16	k

Note: Use of L<sub>h</sub> 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, $A_{gv} =$	4.00	in <sup>2</sup> (2*t*d <sub>e</sub> )
Net Area in Shear, $A_{nv} =$	2.69	in <sup>2</sup> (2*t*(d <sub>e</sub> -(N <sub>c</sub> -0.5)*L <sub>h</sub> ), N <sub>c</sub> = 1 column of bolts
Net Area in Tension, $A_{nt} =$	5.44	in <sup>2</sup> (t*((N <sub>b</sub> -1)*S-(N <sub>b</sub> -1)*W)
Gross Area, $\phi R_n =$	443.44	k
Net Area, $\phi R_n =$	432.05	k

Note: Use of L<sub>h</sub> for determination of Net Plate Area, allows for the slots to be in the joist end plate, rather than the beam tab.

$\phi R_n =$	270.16	k Controls
$R_u / \phi R_n =$	0.19	<1.0 OK (Vu <sup>2</sup> +H <sub>m</sub> <sup>2</sup> ) <sup>1/2</sup> / $\phi R_n$

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

Shear Stress, $f_v =$	5.25	ksi (V <sub>TC</sub> /A <sub>g</sub> )
Critical Stress, $F_{cr} =$	36.38	ksi (( $\phi * F_y$ ) <sup>2</sup> -3*f <sub>v</sub> <sup>2</sup> ) <sup>1/2</sup> $\phi = 0.75$ , von Mises Yield

$\lambda =$	0.08	AISC Eq. 9-18
$Q =$	1	AISC Eq. 9-15 through 9-17
$F_{cr} =$	50	ksi (Q*F <sub>y</sub> ) Classic Plate Buckling

von Mises $\phi M_n =$	545.71	k*in ( $\phi * F_{cr} * S$ ) $\phi = 0.9$
Classic Plate Buckling $\phi M_n =$	750.00	k*in ( $\phi * F_{cr} * S$ ) $\phi = 0.9$
<b>Governing <math>\phi M_n =</math></b>	<b>545.71</b>	<b>k*in</b>

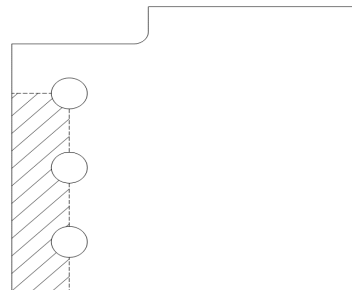
$M_u / \phi M_n =$	0.71	<1.0 OK
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**Joist Plate Weld (Angle = 0 deg. & C<sub>1</sub> = 1.00 E70 Electrode):**

Length of Plate Weld $L_w =$	9.5	in (w-0.5")
$a_y =$	0.2	AISC Table 8-4 $a_y =$ (Weld Centroid - TC Centroid) / L <sub>w</sub>
$k_y =$	0.4	AISC Table 8-4 $k_y =$ Weld Spacing / L <sub>w</sub>
<b>C<sub>y</sub> =</b>	<b>3.47</b>	(y-axis weld eccentricity, AISC Table 8-4)
<b>D<sub>min</sub> =</b>	<b>2</b>	/16ths of an inch Fillet Weld Size (min)

<b>Stress Ratio Results:</b>	
<b>Bolt Shear (V&amp;M):</b>	<b>0.99</b>
<b>Shear Plate Yielding:</b>	<b>0.29</b>
<b>Shear Plate Rupture:</b>	<b>0.18</b>
<b>Shear Plate Block Shear:</b>	<b>0.19</b>
<b>Shear Plate Local Buckling:</b>	<b>0.71</b>

**Min. Joist TC to Plate Weld:**  
**2 /16th x 9.5 " Fillet Weld**



**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  
cp

Beam Tab Plate, Fu =	58	ksi	
Beam Tab Plate, Fy =	36	ksi	
Beam Tab Plate Depth, d =	13	in	
Beam Tab Thickness, t =	0.625	in	
Beam Tab Edge Distances, d <sub>e</sub> =	2	in	
e =	11	in	L <sub>11</sub> = 4.5
Vertical Shear, Vu =	35	k (LRFD)	L <sub>12</sub> = 1.5
Vertical Ecc. Moment, Mu =	385	k*in (LRFD)	L <sub>13</sub> = 0
Bolt Diam. =	1	in	L <sub>14</sub> = 0
Bolt Shear Capacity φRn =	40	k	L <sub>15</sub> = 0
# of Bolts, Nb =	4	(Spreadsheet design limitation, max. 10 bolts)	
Spacing of Bolt Group, S =	3	in	
C.G. of Bolt Group =	4.5	in	
Fv/0.9 =	75.56	ksi (Table J3.2, A490-N Bolts)	
Short Slotted Hole, Lh =	1.31	in	

Ab =	0.79	in <sup>2</sup> (Bolt Area)
C =	11.26	AISC Eq. 7-21, p. 7-19
Mmax =	667.96	k*in (Fv/0.9*Ab*C, Eq. 10-4)
<b>Max. Beam Tab Thickness, tmax =</b>	<b>0.66</b>	<b>in (6*Mmax)/(Fy*d<sup>2</sup>) AISC Eq. 10-3</b>
Gross Plate Area, Ag =	8.125	in <sup>2</sup>
Effective Plate Area, Ae =	5.31	in <sup>2</sup>
Z =	26.4063	in <sup>3</sup> (1/4t*d <sup>2</sup> )
S <sub>net</sub> =	17.60	in <sup>3</sup> (1/6t*d <sup>2</sup> )

<b>Stress Ratio Results:</b>	
Bolt Shear (V&M):	<b>0.99</b>
Shear Tab Yielding:	<b>0.24</b>
Shear Tab Rupture:	<b>0.32</b>
Shear Tab Block Shear:	<b>0.35</b>
Shear Tab Local Buckling:	<b>0.67</b>
5 /16" Tab Weld:	<b>0.19</b>
<b>Plate Stability Acceptable</b>	

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

Bolt Group I <sub>p</sub> =	45.00	in <sup>4</sup> /in <sup>2</sup>
r <sub>py</sub> =	8.75	k (Vu/# Bolts)
r <sub>mx</sub> =	38.50	k (Mu*L <sub>12</sub> /I <sub>p</sub> )
Hm =	38.50	k (rmx*Nc) Nc = 1 column of bolts
Ru =	39.48	k (r <sub>py</sub> <sup>2</sup> +r <sub>mx</sub> <sup>2</sup> ) <sup>1/2</sup>
Ru / φRn =	<b>0.99</b>	<b>&lt;1.0 OK</b>

**Shear Tab Yielding:**

φVn =	175.5	k (φ = 1.0, φ*0.6*Fy*Ag)
φMn =	855.563	k*in (φ = 0.9, φ*Fy*Z)
Ru / φRn =	<b>0.24</b>	<b>&lt;1.0 OK (Vu/φVn)<sup>2</sup>+(Mu/φMn)<sup>2</sup></b>

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

Crushed Hole Width, W' =	1.1875	in (plate hole + 1/16" Crushed width)
Net Plastic Modulus, Z <sub>net</sub> =	17.50	in <sup>3</sup> (Summation of A*d of net plate section)
φVn =	138.66	k (φ = 0.75, φ*.60*Fu*Ae)
φMn =	761.25	k*in (φ = 0.75, φ*Fu*Z)
Ru / φRn =	<b>0.32</b>	<b>&lt;1.0 OK (Vu/φVn)<sup>2</sup>+(Mu/φMn)<sup>2</sup></b>

**Shear Tab Block Shear:** (AISC J4.3)

Vertical Direction

Gross Area in Shear, Agv =	6.88	in <sup>2</sup> (t*(d <sub>e</sub> +(Nb-1)*S)
Net Area in Shear, Anv =	3.91	in <sup>2</sup> Agv-(Nb*W')*t
Net Area in Tension, Ant =	0.84	in <sup>2</sup> (t*(d <sub>e</sub> -(Nc-0.5)*Lh), Nc = 1 column of bolts
Gross Area, φRn =	160.09	k
Net Area, φRn =	150.66	k

Horizontal Direction

Gross Area in Shear, Agv =	2.50	in <sup>2</sup> (2*t*d <sub>e</sub> )
Net Area in Shear, Anv =	1.68	in <sup>2</sup> (2*t*(d <sub>e</sub> -(Nc-0.5)*Lh), Nc = 1 column of bolts
Net Area in Tension, Ant =	3.40	in <sup>2</sup> (t*((Nb-1)*S-(Nb-1)*W)
Gross Area, φRn =	237.61	k
Net Area, φRn =	240.95	k

φRn =	<b>150.66 k Controls</b>
<b>Ru / φRn =</b>	<b>0.35 &lt;1.0 OK (Vu<sup>2</sup>+Hm<sup>2</sup>)<sup>1/2</sup>/φRn</b>

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

Shear Stress, fv =	4.31	ksi (Vu/Ag)
Critical Stress, Fcr =	35.22	ksi (Fy <sup>2</sup> -3*f <sub>v</sub> <sup>2</sup> ) <sup>1/2</sup> von Mises Yield AISC p. 10-103
λ =	0.42	AISC Eq. 9-18
Q =	1	AISC Eq. 9-15 through 9-17
Fcr =	36	ksi (Q*Fy) Classic Plate Buckling
von Mises φMn =	836.99	k*in (φ*Fcr*Z) φ = 0.9
Classic Plate Buckling φMn =	570.38	k*in (φ*Fcr*Z) φ = 0.9
<b>Governing φMn =</b>	<b>570.38 k*in</b>	
<b>Mu / φMn =</b>	<b>0.67 &lt;1.0 OK</b>	

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

Min. Weld Thickness t <sub>wmin</sub> =	0.28	in. t <sub>wmin</sub> = (t*Fy*3 <sup>1/2</sup> )/(2*F <sub>EXX</sub> ), F <sub>EXX</sub> = 70ksi Electrode, AISC Engineering Journal, Vol. 46, 2009
Weld Provided t <sub>w</sub> =	0.3125	in
Min. Plate Thickness =	0.53	in (AISC Eq. 9-3, 6.19*D/Fu) <b>GOOD</b>
φRw =	180.95	k (φ*0.6*F <sub>EXX</sub> *0.707*t <sub>w</sub> *d*2)
<b>Ru / φRn =</b>	<b>0.19 &lt;1.0 OK</b>	

**Shear Tab Stability:** (Thornton and Fortney, 2011)

**Lateral Torsional Buckling Check:**

φRn =	111	k <b>Acceptable</b>
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$R_{req'd} \leq \phi R_n$  (LRFD)

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

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

**Lap Splice Eccentricity Check:**

φMt,u =	43.9	k*in
Mt,u =	28.4	k*in (R*(t <sub>p</sub> +t <sub>j</sub> )/2)

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

φ<sub>v</sub> = 1.0

**Acceptable**

