

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, $w =$	16	in	Joist Plate Hold-Down from TC =	0.5	in
Joist Plate Thickness, $t =$	1	in			
Joist Plate Edge Distances, $d_e =$	2	in			
	11	in			
Vertical Shear, $V_u =$	113	k (LRFD)	$L_{i1} =$	7.5	
Vertical Ecc. Moment, $M_u =$	1243	k*in (LRFD)	$L_{i2} =$	4.5	
Joist Top Chord Axial Force, $V_{TC} =$	169.5	k (LRFD)...	$L_{i3} =$	1.5	
		Assumes 1.5:1 End Web Slope	$L_{i4} =$	0	
Bolt Diam. =	1.25	in	$L_{i5} =$	0	
Bolt Shear Capacity $\phi R_n =$	62.7	k (A490-N)			
# of Bolts, $N_b =$	6	(Spreadsheet design limitation, max. 10 bolts)			
Spacing of Bolt Group, $S =$	3	in			
Vert. C.G. of Bolt Group =	7.5	in			
$F_y/0.9 =$	75.56	ksi (Table J3.2, A490-N Bolts)			
Short Slotted Hole, $L_h =$	1.63	in			

Gross Plate Area, $A_g =$	16	in ²
Effective Plate Area, $A_e =$	14.63	in ²
$Z =$	64	in ³ (1/4t*w ²)
$S =$	42.6667	in ³ (1/6t*w ²)

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

Bolt Group $I_p =$	157.50	in ⁴ /in ²
$r_{py} =$	18.83	k (Vu/# Bolts)
$r_{mx} =$	59.19	k ($M_u * L_{i1} / I_p$)
$H_m =$	59.19	k ($r_{mx} * N_c$) $N_c = 1$ column of bolts
$R_u =$	62.11	k ($r_{py}^2 + r_{mx}^2$) ^{1/2}
$R_u / \phi R_n =$	0.99	<1.0 OK

Shear Plate Yielding:

$\phi V_n =$	480	k ($\phi = 1.0, \phi * 0.6 * F_y * A_g$)
Horiz. Axial Shear $V_{TC} / \phi V_n =$	0.35	<1.0 OK
$\phi M_n =$	1920	k*in ($\phi = 0.9, \phi * F_y * S$)
$M_u / \phi M_n =$	0.65	<1.0 OK
$R_u / \phi R_n =$	0.54	<1.0 OK ($(V_u / \phi V_n)^2 + (M_u / \phi M_n)^2$)

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

Crushed Hole Width, $W' =$	1.4375	in (plate hole + 1/16" Crushed width)
Net Plastic Modulus, $Z_{net} =$	53.53	in ³ ($Z - W' * t * d_{hole}$) $d_{hole} = 7.28125$ in
$\phi V_n =$	427.78	k ($\phi = 0.75, \phi * .60 * F_u * A_e$)
$\phi M_n =$	2609.74	k*in ($\phi = 0.75, \phi * F_u * Z$)
$R_u / \phi R_n =$	0.30	<1.0 OK ($(V_u / \phi V_n)^2 + (M_u / \phi M_n)^2$)

Shear Plate Block Shear: (AISC J4.3)

Vertical Direction		
Gross Area in Shear, $A_{gv} =$	17.00	in ² ($t * (d_e + (N_b - 1) * S)$)
Net Area in Shear, $A_{nv} =$	8.38	in ² $A_{gv} - (N_b * W) * t$
Net Area in Tension, $A_{nt} =$	1.19	in ² ($t * (d_e - (N_c - 0.5) * L_h)$, $N_c = 1$ column of bolts)
Gross Area, $\phi R_n =$	459.69	k
Net Area, $\phi R_n =$	322.16	k

Note: Use of L_h 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 ² ($2 * t * d_e$)
Net Area in Shear, $A_{nv} =$	2.38	in ² ($2 * t * (d_e - (N_c - 0.5) * L_h)$, $N_c = 1$ column of bolts)
Net Area in Tension, $A_{nt} =$	7.81	in ² ($t * ((N_b - 1) * S - (N_b - 1) * W)$)
Gross Area, $\phi R_n =$	597.81	k
Net Area, $\phi R_n =$	577.28	k

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

$\phi R_n =$	322.16	k Controls
$R_u / \phi R_n =$	0.40	<1.0 OK ($(V_u^2 + H_m^2)^{1/2} / \phi R_n$)

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

Shear Stress, $f_v =$	10.59	ksi (V_{TC} / A_g)
Critical Stress, $F_{cr} =$	32.70	ksi ($(\phi * F_y)^2 - 3 * f_v^2$) ^{1/2} $\phi = 0.75$, von Mises Yield

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

von Mises $\phi M_n =$	1255.84	k*in ($\phi * F_{cr} * S$) $\phi = 0.9$
Classic Plate Buckling $\phi M_n =$	1920.00	k*in ($\phi * F_{cr} * S$) $\phi = 0.9$
Governing $\phi M_n =$	1255.84	k*in

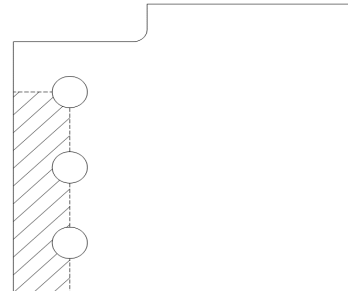
$M_u / \phi M_n =$	0.99	<1.0 OK
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Joist Plate Weld (Angle = 0 deg. & $C_1 = 1.00$ E70 Electrode):

Length of Plate Weld $L_w =$	15.5	in ($w - 0.5$ ")
$a_y =$	0.1	AISC Table 8-4 $a_y = (\text{Weld Centroid} - \text{TC Centroid}) / L_w$
$k_y =$	0.2	AISC Table 8-4 $k_y = \text{Weld Spacing} / L_w$
$C_y =$	3.72	(y-axis weld eccentricity, AISC Table 8-4)
$D_{min} =$	2	/16ths of an inch Fillet Weld Size (min)

Stress Ratio Results:	
Bolt Shear (V&M):	0.99
Shear Plate Yielding:	0.54
Shear Plate Rupture:	0.30
Shear Plate Block Shear:	0.40
Shear Plate Local Buckling:	0.99

Min. Joist TC to Plate Weld:
2 /16th x 15.5 " Fillet Weld



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cp

Beam Tab Plate, Fu =	58	ksi
Beam Tab Plate, Fy =	36	ksi
Beam Tab Plate Depth, d =	19	in
Beam Tab Thickness, t =	1	in
Beam Tab Edge Distances, d _e =	2	in
e =	11	in
Vertical Shear, Vu =	113	k (LRFD)
Vertical Ecc. Moment, Mu =	1243	k*in (LRFD)
Bolt Diam. =	1.25	in
Bolt Shear Capacity φRn =	62.7	k
# of Bolts, Nb =	6	(Spreadsheet design limitation, max. 10 bolts)
Spacing of Bolt Group, S =	3	in
C.G. of Bolt Group =	7.5	in
Fv/0.9 =	75.56	ksi (Table J3.2, A490-N Bolts)
Short Slotted Hole, Lh =	1.63	in

L ₁₁ =	7.5
L ₁₂ =	4.5
L ₁₃ =	1.5
L ₁₄ =	0
L ₁₅ =	0

Ab =	1.23	in ² (Bolt Area)
C =	25.09	AISC Eq. 7-21, p. 7-19
Mmax =	2326.63	k*in (Fv/0.9*Ab*C, Eq. 10-4)
Max. Beam Tab Thickness, tmax =	1.07	in (6*Mmax)/(Fy*d²) AISC Eq. 10-3
Gross Plate Area, Ag =	19	in ²
Effective Plate Area, Ae =	10.75	in ²
Z =	90.25	in ³ (1/4t*d ²)
S _{net} =	60.17	in ³ (1/6t*d ²)

Stress Ratio Results:	
Bolt Shear (V&M):	0.99
Shear Tab Yielding:	0.26
Shear Tab Rupture:	0.47
Shear Tab Block Shear:	0.44
Shear Tab Local Buckling:	0.64
8 /16" Tab Weld:	0.27
Plate Stability Acceptable	

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

Bolt Group I _p =	157.50	in ⁴ /in ²
r _{py} =	18.83	k (Vu/# Bolts)
r _{mx} =	59.19	k (Mu*L ₁₂ /I _p)
Hm =	59.19	k (rmx*Nc) Nc = 1 column of bolts
Ru =	62.11	k (r _{py} ² +r _{mx} ²) ^{1/2}
Ru / φRn =	0.99	<1.0 OK

Shear Tab Yielding:

φVn =	410.4	k (φ = 1.0, φ*0.6*Fy*Ag)
φMn =	2924.1	k*in (φ = 0.9, φ*Fy*Z)
Ru / φRn =	0.26	<1.0 OK (Vu/φVn)²+(Mu/φMn)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} =	51.44	in ³ (Summation of A*d of net plate section)
φVn =	280.58	k (φ = 0.75, φ*.60*Fu*Ae)
φMn =	2237.53	k*in (φ = 0.75, φ*Fu*Z)
Ru / φRn =	0.47	<1.0 OK (Vu/φVn)²+(Mu/φMn)²

Shear Tab Block Shear: (AISC J4.3)

Vertical Direction

Gross Area in Shear, Agv =	17.00	in ² (t*(d _e +(Nb-1)*S)
Net Area in Shear, Anv =	8.38	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 =	344.28	k
Net Area, φRn =	287.46	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 =	7.81	in ² (t*((Nb-1)*S-(Nb-1)*W)
Gross Area, φRn =	517.93	k
Net Area, φRn =	515.11	k

φRn =	287.46 k Controls
Ru / φRn =	0.44 <1.0 OK (Vu²+Hm²)^{1/2}/φRn

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

Shear Stress, fv =	5.95	ksi (Vu/Ag)
Critical Stress, Fcr =	34.49	ksi (Fy ² -3*fv ²) ^{1/2} von Mises Yield AISC p. 10-103
λ =	0.31	AISC Eq. 9-18
Q =	1	AISC Eq. 9-15 through 9-17
Fcr =	36	ksi (Q*Fy) Classic Plate Buckling

von Mises φMn =	2801.83	k*in (φ*Fcr*Z) φ = 0.9
Classic Plate Buckling φMn =	1949.40	k*in (φ*Fcr*Z) φ = 0.9
Governing φMn =	1949.40	k*in

Mu / φMn =	0.64 <1.0 OK
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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 =	423.14	k (φ*0.6*F _{EXX} *0.707*t _w *d*2)

Ru / φRn =	0.27 <1.0 OK
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Shear Tab Stability: (Thornton and Fortney, 2011)

Lateral Torsional Buckling Check:

φRn =	666	k Acceptable
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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 =	148.7	k*in
Mt,u =	113.0	k*in (R*(t _p +t _j)/2)

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

(LRFD)

$\phi_v = 1.0$

Acceptable

