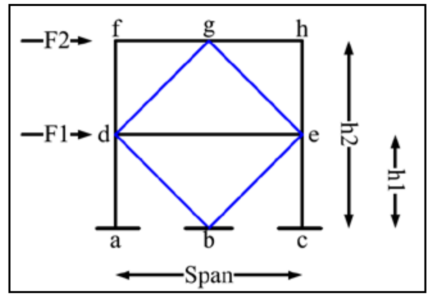
Appendix H

Specimen TCBF-B-1 to TCBF-B-4 Design Calculation Sheets

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
General		Page	1



Building height =	2	stories
Typical floor height =	9	ft
$F_{1, max}$ =	300	kip
$F_{2, max}$ =	600	kip
SR =	4	-
ratio =	0.8	-

Calculation Initialize

Items	values	units		Items	values	units
F_1 =	240	kip		V_1 =	720	kip
F_2 =	480	kip		V_2 =	480	kip
h_1 =	9	ft				
h_2 =	18	ft				
span =	20	ft	(beam span)			
h =	9	ft	(typical floor height)			
M_{base} =	13500	kip-ft				
P_{column} =	675	kip				
L_{brace} =	13.45	ft	(work point to work point)			
E_s =	29000	ksi				

Notes

: input value

Materials

Members	Material Type	Fy (ksi)	Fu (ksi)	Ry	Rt	
Columns	ASTM A992	50	65	1.1	1.1	(Ref: Table I-6-1)
Beams	ASTM A992	50	65	1.1	1.1	
Braces	ASTM A500B	46	58	1.4	1.3	(HSS-Square)
Plates 1	ASTM A36	36	58	1.3	1.2	
Bolts	A490	130	150	-	-	
Welds	E70XX	-	70	-	-	
Plates 2	ASTM A572 Gr.50	50	65	1.1	1.2	

Load Combinations

Per ASCE-7-2005

Basic Reference Codes

AISC Specification for Structural Steel Buildings (March 9, 2005)
AISC Seismic Provisions for Structural Steel Buildings (March 9, 2005)

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
2F-Brace		Page	2

P_u =	131.72	kip	(compression)				
L_{brace} =	8.1	ft					
k =	1.0	-					
Try section	HSS5x5x5/16		(HSS-Square)				
A_s =	5.26	in^2		I_x =	19.00	in^4	
Z_x =	9.16	in^3		I_y =	19.00	in^4	
b =	5.00	in		h =	5.00	in	
t_{nom} =	0.31	in		t_{des} =	0.291	in	
r_x =	1.90	in					
r_y =	1.90	in					
F_y (brace) =	46	ksi		E_s =	29000	ksi	
kL/r =	51.05	-		Limit =	100.43	OK	$\boxed{Kl/r \leq 4\sqrt{E/F_y}}$
F_e =	109.81	ksi		0.44 F_y =	20.24	ksi	
ϕ =	0.90	-					
ϕP_n =	182.74	kip	(compression)		Check	OK	

Check Compactness Seismically (AISC Seismic Provisions 2005, Sec 8.2b)

λ_{ps} =	16.07	-		b/t =	14.20	OK	
							(Table I-8-1)
				h/t =	14.20	OK	

ϕ =	0.90	-				
ϕP_n =	217.76	kip	(tension)		Check	OK

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
1F-Brace		Page	3

P_u =	249.37	kip	(compression)				
L_{brace} =	9	ft					
k =	1.0	-					
Try section	HSS6x6x3/8		(HSS-Square)				
A_s =	7.58	in2		I_x =	39.50	in4	
Z_x =	15.80	in3		I_y =	39.50	in4	
b =	6.00	in		h =	6.00	in	
t_{nom} =	0.38	in		t_{des} =	0.349	in	
r_x =	2.28	in					
r_y =	2.28	in					
F_y (brace) =	46	ksi		E_s =	29000	ksi	
kL/r =	47.37	-		Limit =	100.43	OK	$\boxed{Kl/r \le 4\sqrt{E/F_y}}$
F_e =	127.55	ksi		0.44 F_y =	20.24	ksi	
ϕ =	0.90	-					
ϕP_n =	269.85	kip	(compression)		Check	OK	

Check Compactness Seismically (AISC Seismic Provisions 2005, Sec 8.2b)

λ_{ps} =	16.07	-	b/t =	14.20	OK	
						(Table I-8-1)
			h/t =	14.20	OK	

ϕ =	0.90	-				
ϕP_n =	313.81	kip	(tension)		Check	OK

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
2F	Brace to Gusset Plate Connection	Page	4
Brace	HSS5x5x5/16		

$R_yF_yA_g$ =	338.74	kip	(T_u)							
F_uA_g =	305.08	kip	(P_u)	T_u/P_u =	1.11		-			
R_yF_y =	64.4	ksi								
R_tF_u =	75.4	ksi								
U =	0.9	-								
ϕ_t =	0.75	(tensile rupture in net section)								
A_n/A_g =	1.27	(Net section reinforcement required!)								
ϕ_t =	0.90	(tensile yield in gross section)								
t_{gusset} =	0.75	in	(estimated)			F_y =	50	ksi		
t_g =	0.75	in	(use)				(gusset plate)			
						F_u =	65	ksi		
A_{cut} =	0.51	in^2								
A_{net} =	4.75	in^2								
A_e =	5.99	in^2	(Reinforcement required!)							

Reinforcement Plates

l =	12	in	B =	5	in	H =	5	in
x_bar =	1.875	in						
U =	0.84	-	$A_{e, req}$ =	5.99	in^2	$A_{net, req}$ =	7.10	in^2
A_{reinf} =	1.17	in^2	(both sides)					
b_{reinf} =	2	in						
t_{req} =	0.59	in	t_{use} =	0.625	in	L_{plate} =	14	in
$F_{y, plate}$ =	50	ksi	$R_yF_yA_g$ =	68.75	kip			
L_{weld} =	6	in	weld =	5	x 1/16 in	(fillet)		
ϕR_n =	83.51	kip	OK					

Brace Block Shear

t_{brace} =	0.291	in	
L_{req} =	11.15	in	OK
L_{use} =	12	in	

Brace to Gusset Plate Weld

L_{weld} =	12	in	
weld =	6	x 1/16 in	(fillet)
ϕ_b =	0.75	-	

F_{exx} =	70	ksi		
F_w =	42	ksi		
$\phi_b R_n$ =	400.87	kip	OK	

Gusset Plate Block Shear				
A_{gv} =	18	in^2		
A_{nt} =	4.31	in^2		
U_{bs} =	1	-		
ϕ =	0.75	-		
ϕR_n =	615.23	kip	OK	

Whitmore Effective Width				
$L_{whitmore}$ =	20.59	in	(theoretical width)	
ϕ =	0.90	-		
ϕR_n =	694.86	kip	OK	(check gross yield)

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
1F	Brace to Gusset Plate Connection	Page	5
Brace	HSS6x6x3/8		

$R_yF_yA_g$ =	488.152	kip	(T_u)								
F_uA_g =	439.64	kip	(P_u)	T_u/P_u =	1.11		-				
R_yF_y =	64.4	ksi									
R_tF_u =	75.4	ksi									
U =	0.9	-									
ϕ_t =	0.75	(tensile rupture in net section)									
A_n/A_g =	1.27	(Net section reinforcement required!)									
ϕ_t =	0.90	(tensile yield in gross section)									
t_{gusset} =	0.90	in	(estimated)			F_y =	50	ksi			
t_g =	0.75	in	(use)				(gusset plate)				
						F_u =	65	ksi			
A_{cut} =	0.61	in^2									
A_{net} =	6.97	in^2									
A_e =	8.63	in^2	(reinforcement required)								

Reinforcement Plates

l =	14	in	B =	6	in	H =	6	in
x_bar =	2.25	in						
U =	0.84	-	$A_{e,\,req}$ =	8.63	in^2	$A_{net,\,req}$ =	10.29	in^2
A_{reinf} =	1.66	in^2	(both sides)					
b_{reinf} =	3	in						
t_{req} =	0.55	in	t_{use} =	0.625	in	L_{plate} =	16	in
$F_{y,\,plate}$ =	50	ksi	$R_yF_yA_g$ =	103.13	kip			
L_{weld} =	7	in	weld =	6	x 1/16 in	(fillet)		
ϕR_n =	116.92	kip	OK					

Brace Block Shear

t_{brace} =	0.349	in	
L_{req} =	13.40	in	OK
L_{use} =	14	in	

Brace to Gusset Plate Weld

L_{weld} =	14	in	
weld =	7	x 1/16 in	(fillet)
ϕ_b =	0.75	-	

F_{exx} =	70	ksi		
F_w =	42	ksi		
$\phi_b R_n$ =	545.63	kip	OK	

Gusset Plate Block Shear				
A_{gv} =	21	in^2		
A_{nt} =	5.16	in^2		
U_{bs} =	1	-		
ϕ =	0.75	-		
ϕR_n =	723.87	kip	OK	

Whitmore Effective Width				
$L_{whitmore}$ =	23.90	in	(theoretical width)	
ϕ =	0.90	-		
ϕR_n =	806.55	kip	OK	(check gross yield)

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
2F	Roof Beam Design Check	Page	6

$R_yF_yA_g$ =	338.74	kip		θ =	0.73	(rad)	42.0	(deg)
0.3 P_n =	60.91	kip		$\sin(\theta)$ =	0.67			
				$\cos(\theta)$ =	0.74			
V =	185.86	kip						
H =	297.06	kip						
P_u =	600.00	kip	(conservatively)					
M_u =	100.97	kip-ft	(revised from structural analysis)					

Try	w24x117					
A_g =	34.4	in^2	b_f =	12.8	in	
I_x =	3540	in^4	t_f =	0.85	in	
I_y =	297	in^4	d =	24.3	in	
r_x =	10.1	in	t_w =	0.55	in	
r_y =	2.94	in	F_y =	50	ksi	
λ_{p1} =	9.15		b/t =	7.53	Compact	
λ_{p2} =	90.55		h/tw =	41.09	Compact	
L_p =	10.38	ft	Z_x =	327	in^3	
c =	1	-	J =	6.72	in^4	
C_w =	40800	in^6	h_o =	23.45	in	
S_x =	291	in^3	r_{ts} =	3.46	in	
L_r =	29.90	ft	Brace PT=	2	-	
L_b =	10	ft	C_b =	1.0	(Conservatively)	
M_p =	1362.5	kip-ft	F_{cr} =	248.50	ksi	
ϕ_b =	0.90	-	M_n =	1362.50	kip-ft	(Need Check)
ϕ_bM_n =	1226.25	kip-ft				
kl/r =	40.82	-	k =	1.0	-	
F_e =	171.79	ksi	0.44 F_y =	22	ksi	
ϕ_c =	0.90	-				
ϕ_cP_n =	1370.46	kip				
P_u/ϕ_cP_n =	0.44	use (H1-1a)				
Check	0.51	OK				

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
1F	Lower Beam Design Check	Page	7

$R_yF_yA_g$ =	488.15	kip	(1F)					
$R_yF_yA_g$ =	338.74	kip	(2F)	θ =	0.73	(rad)	42.0	(deg)
0.3 P_n =	89.95	kip	(1F)	$\sin(\theta)$ =	0.67			
0.3 P_n =	60.91	kip	(2F)	$\cos(\theta)$ =	0.74			
V =	285.81	kip						
H =	317.56	kip						
P_u =	317.56	kip	(conservatively)					
M_u =	170.85	kip-ft	(revised from structural analysis)					
Try	w24x68							
A_g =	20.1	in^2		b_f =	8.97	in		
I_x =	1830	in^4		t_f =	0.585	in		
I_y =	70.4	in^4		d =	23.7	in		
r_x =	9.55	in		t_w =	0.415	in		
r_y =	1.87	in		F_y =	50	ksi		
λ_{p1} =	9.15			b/t =	7.67	Compact		
λ_{p2} =	90.55			h/tw =	54.29	Compact		
L_p =	6.61	ft		Z_x =	177	in^3		
c =	1	-		J =	1.87	in^4		
C_w =	9430	in^6		h_o =	23.12	in		
S_x =	154	in^3		r_{ts} =	2.30	in		
L_r =	18.74	ft		Brace PT=	2	-		
L_b =	10	ft		C_b =	1.0	(Conservatively)		
M_p =	737.5	kip-ft		F_{cr} =	110.86	ksi		
ϕ_b =	0.90	-		M_n =	656.87	kip-ft	OK	
ϕ_bM_n =	591.18	kip-ft						
kl/r =	64.17	-		k =	1.0	-		
F_e =	69.50	ksi		0.44 F_y =	22	ksi		
ϕ_c =	0.90	-						
ϕ_cP_n =	669.33	kip						
P_u/ϕ_cP_n =	0.47	use (H1-1a)						
Check	0.73	OK						

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
2F	Column Design Check	Page	8

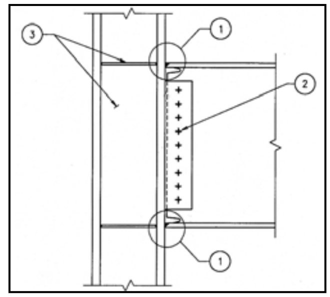
P_u =	42.58	kip	(revised from structural analysis)					
M_u =	126.56	kip-ft	(revised from structural analysis)					
L_{column} =	9	ft						
Try	w12x96							
A_g =	28.2	in^2		b_f =	12.2	in		
I_x =	833	in^4		t_f =	0.9	in		
I_y =	270	in^4		d =	12.7	in		
r_x =	5.44	in		t_w =	0.55	in		
r_y =	3.09	in		F_y =	50	ksi		
λ_{p1} =	7.22	-		b/t =	6.78	Compact		
λ_{p2} =	71.71	-		h/tw =	19.82	Compact		
L_p =	10.91	ft		Z_x =	147	in^3		
c =	1	-		J =	6.85	in^4		
C_w =	9410	in^6		h_o =	11.80	in		
S_x =	131	in^3		r_{ts} =	3.49	in		
L_r =	40.86	ft		Brace PT=	0	-		
L_b =	9	ft		C_b =	1.0	(Conservatively)		
M_p =	612.5	kip-ft		F_{cr} =	344.49	ksi		
ϕ_b =	0.90	-		M_n =	612.50	kip-ft		(Need Check)
$\phi_b M_n$ =	551.25	kip-ft		C_a =	0.03	-		
kl/r =	34.95	-		k =	1.0	-		
F_e =	234.28	ksi		0.44 F_y =	22	ksi		
ϕ_c =	0.90	-						
$\phi_c P_n$ =	1160.56	kip						
$P_u/\phi_c P_n$ =	0.04	use (H1-1b)						
Check	0.25	OK						

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
1F	Column Design Check	Page	9

P_u =	484.19	kip	(revised from structural analysis)				
M_u =	274.18	kip-ft	(revised from structural analysis)				
L_{column} =	9	ft					
Try	w12x96						
A_g =	28.2	in^2		b_f =	12.2	in	
I_x =	833	in^4		t_f =	0.9	in	
I_y =	270	in^4		d =	12.7	in	
r_x =	5.44	in		t_w =	0.55	in	
r_y =	3.09	in		F_y =	50	ksi	
λ_{p1} =	7.22	-		b/t =	6.78	Compact	
λ_{p2} =	52.56	-		h/tw =	19.82	Compact	
L_p =	10.91	ft		Z_x =	147	in^3	
c =	1	-		J =	6.85	in^4	
C_w =	9410	in^6		h_o =	11.80	in	
S_x =	131	in^3		r_{ts} =	3.49	in	
L_r =	40.86	ft		Brace PT=	0	-	
L_b =	9	ft		C_b =	1.0	(Conservatively)	
M_p =	612.5	kip-ft		F_{cr} =	344.49	ksi	
ϕ_b =	0.90	-		M_n =	612.50	kip-ft	(Need Check)
$\phi_b M_n$ =	551.25	kip-ft		C_a =	0.38	-	
kl/r =	34.95	-		k =	1.0	-	
F_e =	234.28	ksi		0.44 F_y =	22	ksi	
ϕ_c =	0.90	-					
$\phi_c P_n$ =	1160.56	kip					
$P_u/\phi_c P_n$ =	0.42	use (H1-1a)					
Check	0.86	OK					

Check Column Web Shear Stress			
M_p =	7350	kip-in	
L =	96.15	in	
V =	152.89	kip	
A_s =	6.99	in^2	A_s = d*tw
S_v =	21.89	ksi	
$S_{v, yield}$ =	29.00	ksi	Elastic

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
2F	Beam-Column Connection	Page	10



Type Bolted (WUF-B)

$H =$ 148.53 kip

$V =$ 92.93 kip

$M =$ 100.97 kip-ft (revised from structural analysis)

$R_u =$ 175.21 kip

Try $d_b =$ 0.88 in — $F_u =$ 150 ksi

$A_b =$ 0.60 in^2 — $F_{nv} =$ 75 ksi (threads excluded)

$N_b =$ 6 bolts (in one row)

$R_n =$ 270.59 kip (bolt shear) — $L_{c\_ex} =$ 1.5 in

$\phi_b =$ 0.75 - — $L_{c\_in} =$ 3 in

$\phi_b R_n =$ 202.94 kip OK

$L_{c1} =$ 1.03 in (edge clear distance) — $R_{n1} =$ 46.41 kip

$L_{c2} =$ 2.06 in (clear distance) — $R_{n2} =$ 236.25 kip

$t =$ 0.50 in (shear tab thickness)

$R_n =$ 1227.66 kip (combined bolt bearing)

$\phi_b =$ 0.75 -

$\phi_b R_n =$ 920.74 kip OK

$L_{tab} =$ 18 in — $A_{s,\ tab} =$ 9 in^2 — $R_n =$ 524.79 kip

$w_{tab} =$ 4.5 in — $F_{y,\ tab} =$ 50 ksi — $P_{nt} =$ 450.00 kip

$F_{v,\ tab} =$ 30.0 ksi — $P_{nv} =$ 270.00 kip

OK

Weld Fillet (shear tab)

$F_{exx} =$ 70 ksi — $R_n =$ 334.06 kip

$Fw =$ 42 ksi — $\phi_b =$ 0.75 -

$w =$ 5 x 1/16 inch — $\phi_b R_n =$ 250.54 kip OK

$L_{weld} =$ 18 in

side = 2 sides

Weld CJP (top, bottom flanges)

$b_f =$ 12.8 in — $F_{y,\ bm} =$ 50 ksi (base metal)

$t_f =$ 0.85 in — $M_n =$ 1063.07 kip-ft OK

$d =$ 24.3 in

$t_w =$ 0.55 in

Check	Shear tab length, OK		

Check Block Shear

Beam
w24x117

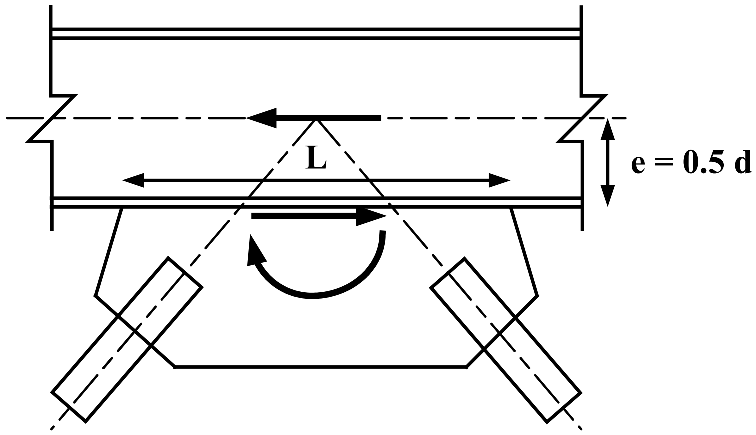
A_{gv} =	9.9	in^2	
A_{gt} =	1.925	in^2	
A_{nv} =	6.6	in^2	
A_{nt} =	1.65	in^2	
U_{bs} =	0.5	-	
ϕ =	0.75	-	
F_y =	50	ksi	
F_u =	65	ksi	
ϕR_n =	233.27	kip	OK

Shear Tab

A_{gv} =	8.25	in^2	
A_{gt} =	1.5	in^2	
A_{nv} =	5.5	in^2	
A_{nt} =	1.25	in^2	
U_{bs} =	0.5	-	
ϕ =	0.75	-	
F_y =	50	ksi	
F_u =	65	ksi	
ϕR_n =	191.34	kip	OK

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
2F	Braces to Beam Connection	Page	11

Braces	HSS5x5x5/16			
T =	338.74	kip	sin(θ) =	0.669
C =	312.69	kip	cos(θ) =	0.743
e =	12.15	in		
Shear =	484.21	kip		
Tension =	169.56	kip		
Moment =	490.26	kip-ft		
t_{gusset} =	0.75	in		
L =	60	in		
s_V =	10.76	ksi		
s_A =	3.77	ksi		
s_M =	13.07	ksi		
ϕ =	0.9	-		
$F_{y,\ gusset}$ =	50	ksi		
Ratio =	0.56	OK		



$L_{whitmore}$ =	20.59	in	L_{min} =	57.78	in	(geometry limit)	OK
L_v =	16	in	$L_{v,\ min}$ =	15.30	in	(geometry limit)	OK
w_{up} =	10.29	in	A_v =	12	in^2		
w_{low} =	20.92	in	P_u =	226.61	kip		
$Whitm_{eff}$ =	20.59	in	ϕR_n=	324	kip	OK	
ϕR_n =	694.86	kip	OK				

Gusset Plate to Beam Flange

Weld	Fillet						
F_{exx} =	70	ksi					
F_w =	42	ksi			s_V =	11.41	ksi
w =	8	x	1/16	inch	s_M =	13.87	ksi
L_{weld} =	60	in			s_A =	4.00	ksi
side =	2	sides			f_{peak} =	21.20	ksi
t_{eff} =	0.354	in			f_{avg} =	18.15	ksi
ϕ =	0.75	-			f_r =	22.68	ksi
Ratio =	0.77	OK			Ratio =	0.72	OK

Check Beam Web

width =	60	in	Beam	w24x117

R_u =	196.10	kip	d =	24.3	in	t_w =	0.55	in
N =	30	in	t_f =	0.85	in	$F_{y,\,web}$ =	50	ksi
ϕ =	0.75	-	R_n =	1060.63	kip	ϕR_n =	795.47	kip
k_{des} =	1.35	in				(web crippling)		OK
ϕ =	1.00	-	R_n =	1010.63	kip	ϕR_n =	1010.63	kip
						(web local yielding)		OK

Check Gusset Plate Buckling

L_{gb} =	17.68	in	kL/r =	98.0	-	L_c =	14.13	in
k =	1.2	-	F_e =	29.82	ksi	L_{c1} =	11.38	in
r =	0.217	in	0.44 F_y =	22	ksi	L_{c2} =	16.94	in
A_g =	15.44	in^2	R_n =	382.72	kip	L_{max} =	24.06	in
ϕ =	0.9	-	ϕR_n =	344.45	kip	L_{tip} =	21.88	in
					OK	L_{ave} =	17.68	in

Free Edge Buckling

L_e =	15.63	in	
L_e/t_g =	20.83	-	
Limit =	18.06	-	Edge stiffener required!

Lateral Stability of Beam

M_r =	1498.75	kip-ft	Z =	327	in^3	L_b =	10	ft
C_d =	1	-	h_o =	23.45	in	L_{pd} =	17.05	ft
P_{br} =	15.34	kip	β_{br} =	85.22	kip/in	n =	1	OK
(Nodal)			(Nodal)			C_b =	1	-

M_{br} =	17.985	kip-ft	
P_{br} =	9.20	kip	(torsional)
β_T =	28842	kip-in/rad	β_{sec} not included
β_{br} =	52.45	kip/in	(torsional)
Δ =	0.18	in	

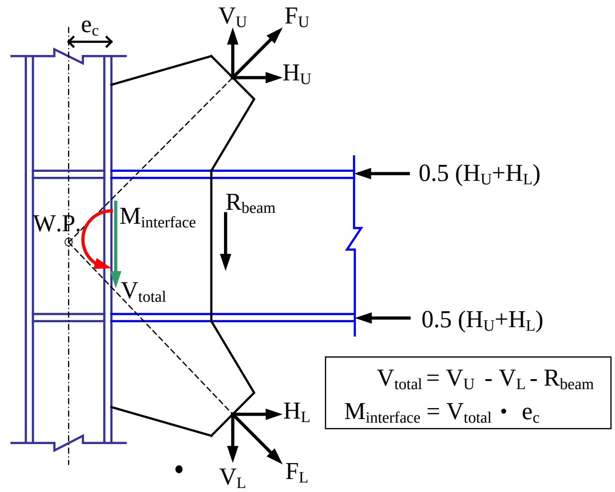
Kicker

L3x2x3/8

A_g =	1.73	in^2	
L =	25	in	
k_{axial} =	2006.8	kip/in	
k =	1419	kip/in	OK

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
1F	Beam-Column-Gusset Connections	Page	12

Big Gusset Plate for Upper Floor Bracing and Lower Floor Bracing



Sway to Right

F_{U2R} =	338.74	kip	$\cos(\theta_U)$ =	0.743
F_{L2R} =	461.74	kip	$\cos(\theta_L)$ =	0.743

Sway to Left

F_{U2L} =	312.69	kip				
F_{L2L} =	488.15	kip				
Beam	w24x68					
d =	23.7	in				
$L_{c,\,min}$ =	56.77	in				
Column	w12x96					
e_c =	6.35	in				
R_{beam} =	22.51	kip	(downward)			
$L_{b,\,min}$ =	15.99	in				
$L_{cu,\,min}$ =	15.30	in				
$L_{cl,\,min}$ =	17.76	in				
t_g =	0.75	in				
L_{cu} =	18	in	(use)	$F_{y,\,gusset}$ =	50	ksi
L_{cl} =	18	in	(use)			
L_c =	59.7	in	(use)			
L_b =	23	in	(use)			

Sway to the Right

V_{U2R} =	226.61	kip	(upward)	s_V =	11.46	ksi	
H_{U2R} =	251.79	kip	(rightward)	s_M =	7.31	ksi	
V_{L2R} =	308.89	kip	(upward)	s_A =	2.04	ksi	
H_{L2R} =	343.21	kip	(leftward)	Ratio =	0.49	-	OK
V_{total} =	512.98	kip	(upward)				
M =	271.45	kip-ft	(counter-clockwise)				
				Column-Side			
V_{cu} =	154.67	kip	(downward)	s_A =	5.11	ksi	
f_1 =	5.48	kip/in	(lefttward)	s_V =	11.46	ksi	
f_2 =	2.18	kip/in	(leftward)	s_M =	2.20	ksi	
H_{cu} =	68.95	kip	(leftward)	Ratio =	0.47	-	OK
H_{bu} =	182.84	kip	(leftward)	Beam-Side			
V_{bu} =	71.94	kip	(downward)	s_A =	4.17	ksi	
M_{cu} =	7.44	kip-ft	(counter-clockwise)	s_V =	10.60	ksi	

M_{bu} =	-118.93	kip-ft	(clockwise)	s_M =	-21.58	ksi	
				Ratio =	0.56	-	OK

V_{cl} =	154.67	kip	(downward)	s_A =	5.11	ksi	
f_1 =	5.48	kip/in	(rightward)	s_V =	11.46	ksi	
f_3 =	2.18	kip/in	(rightward)	s_M =	2.20	ksi	
H_{cl} =	68.95	kip	(rightward)	Ratio =	0.47	-	OK
H_{bl} =	274.26	kip	(rightward)				
V_{bl} =	154.22	kip	(downward)	s_A =	8.94	ksi	
M_{bl} =	86.82	kip-ft	(clockwise)	s_V =	15.90	ksi	
M_{cl} =	7.44	kip-ft	(counter-clockwise)	s_M =	15.76	ksi	
				Ratio =	0.82	-	OK

V_{mid} =	156.39	kip	(downward)	s_A =	0.00	ksi	
M_{mid} =	10.02	kip-ft	(counter-clockwise)	s_V =	8.80	ksi	
H_{mid} =	0.00	kip	(leftward)	s_M =	1.71	ksi	
				Ratio =	0.34	-	OK

Weld Size

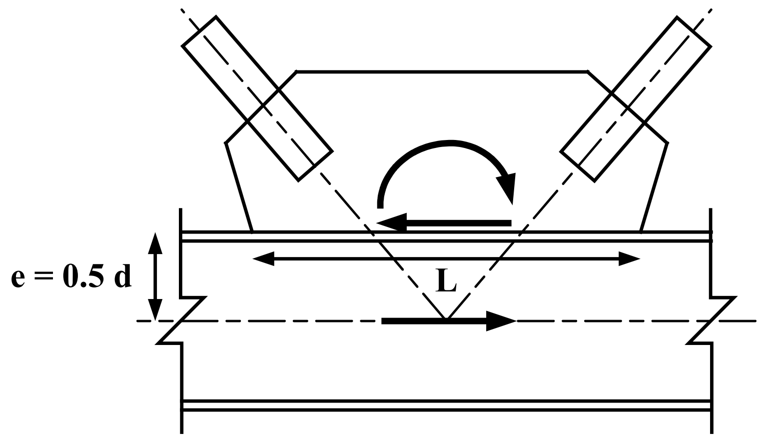
f_v =	8.59	kip/in	
f_a =	4.98	kip/in	(averaged)
f_b =	5.48	kip/in	
f_{peak} =	13.54	kip/in	
f_{avg} =	11.07	kip/in	
f_r =	13.84	kip/in	13.84342
D >=	4.97	x 1/16	(weld size)
Use	6	x 1/16	(weld size)

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
1F	Lower Beam to Gusset Plate Splice	Page	13

Web	Fillet welds with web plates							
Flange	CJP weld		(T & B)					
P_u =	317.56	kip						
t_f =	0.59	in						
b =	8.97	in						
A_s =	5.25	in2						
$2*A_s*F_y$ =	524.75	kip	OK					
R_{beam} =	22.51	kip	(Gravity)					
L_{tab} =	20.375	in			t =	0.5	in	
w_{tab} =	8	in						
Weld	Fillet	(shear tab)						
F_{exx} =	70	ksi			R_n =	315.96	kip	
F_w =	42	ksi			ϕ_b =	0.75	-	
w =	6	x 1/16	inch		$\phi_b R_n$ =	236.97	kip	OK
L_{weld} =	28.375	in						
side =	1	sides						
Shim Plate								
L =	20	in			t =	0.168	in	(shim as required)
w =	4	in						
Weld	Fillet	(shear tab)						
F_{exx} =	70	ksi			R_n =	139.26	kip	
F_w =	42	ksi			ϕ_b =	0.75	-	
w =	2.68	x 1/16	inch		$\phi_b R_n$ =	104.45	kip	OK
L_{weld} =	28	in	(3 sides)					
side =	1	sides						

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
1F	Braces to Floor Beam Connection	Page	14

Braces		HSS6x6x3/8		
T =	488.15	kip	$\sin(\theta)$ =	0.669
C =	461.74	kip	$\cos(\theta)$ =	0.743
e =	0	in		
Shear =	706.05	kip		
Tension =	242.31	kip		
Moment =	0.00	kip-ft		
t_{gusset} =	0.75	in		
L =	46	in		
s_V =	20.47	ksi		
s_M =	0.00	ksi		
ϕ =	0.9	-		
$F_{y,\ gusset}$ =	50	ksi		
Ratio =	0.80	OK		



whitmo =	23.90	in	L_{min} =	35.72	in	(geometry limit)	OK
L_v =	19	in	$L_{v,\ min}$ =	17.76	in	(geometry limit)	OK
w_{up} =	11.95	in	A_v =	14.25	in2		
w_{low} =	14.12	in	P_u =	326.56	kip		
$whit_{eff}$ =	23.90	in	ϕR_n=	384.75	kip		
ϕR_n =	806.55	kip	OK		OK		

Weld	Fillet	(Gusset to beam flange)					
F_{exx} =	70	ksi					
F_w =	42	ksi					
w =	11	x	1/16	inch	s_V =	15.79	ksi
L_{weld} =	46	in			s_M =	0.00	ksi
side =	2	sides			s_A =	5.42	ksi
t_{eff} =	0.486	in					
ϕ =	0.75	-					
Ratio =	0.89	OK					

Check Beam Web

width =	46	in	Beam	w30x391			
R_u =	0.00	kip	d =	33.2	in	t_w =	1.36 in

N =	23	in	t_f =	2.44	in	$F_{y,\,web}$ =	50	ksi
ϕ =	0.75	-	R_n =	4450.60	kip	ϕR_n=	3337.95	kip
k_{des} =	3.23	in				(web crippling)		OK
ϕ =	1.00	-	R_n =	2662.20	kip	ϕR_n=	2662.20	kip
						(web local yielding)		OK

Check Gusset Plate Buckling

L_{gb} =	14.84	in	kL/r =	82.2	-	L_c =	15.88	in
k =	1.2	-	F_e =	42.32	ksi	L_{c1} =	18.06	in
r =	0.217	in	$0.44\ F_y$ =	22	ksi	L_{c2} =	12.56	in
A_g =	17.92	in2	R_n =	546.54	kip	L_{max} =	18.56	in
ϕ =	0.9	-	ϕR_n=	491.88	kip	L_{tip} =	9.13	in
					OK	L_{ave} =	14.84	in

Free Edge Buckling

L_e =	17.88	in	
L_e/t_g =	23.83	-	
Limit =	18.06	-	Edge stiffener required!

Lateral Stability of Beam

M_r =	6645.83	kip-ft	Z =	1450	in3	L_b =	10	ft
C_d =	1	-	h_o =	30.76	in	L_{pd} =	17.05	ft
P_{br} =	51.85	kip	β_{br} =	288.07	kip/in	n =	1	OK
(Nodal)			(Nodal)			C_b =	1	-
M_{br} =	79.75	kip-ft				I_y =	1550	in4
P_{br} =	31.11	kip	(torsional)					
β_T =	108666	kip-in/rad	βsec not included					
β_{br} =	114.85	kip/in	(torsional)					
Δ =	0.27	in						

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 16, 2008
1F	Column Base Plate Design Check	Page	15
Column	w12x96		

Z_x =	147	in3		L =	96.15	in
F_y =	50	ksi		V_{Mp} =	152.89	kip
M_p =	7350	kip-in				
P_u =	484.19	kip		d =	12.7	in
M_u =	3290.12	kip-in		b_f =	12.2	in
N =	31.25	in		$f_{p,\ max}$ =	36	ksi
B =	28	in				
e =	6.80	in		q_{max} =	1008	kip/in
e_{cr} =	15.38	in	(Small Moment)			
Y =	17.66	in				
q =	27.42	kip/in	OK			
m =	9.59	in				
f_p =	0.98	ksi				
$t_{p,\ req}$ =	2.01	in	eq 3.3.14a (LRFD)			
use =	2.00	in				

All-thread-rods

Type	ASTM A193 B7		
d_{bolt} =	1.125	in	
F_u =	125	ksi	
F_y =	105	ksi	
F_{nt} =	93.75	ksi	
F_{nv} =	50	ksi	
A_b =	0.99	in^2	
ϕ =	0.75	-	
ϕR_n =	69.89	kip	(tension)
ϕR_n =	37.28	kip	(shear)
F_{PT} =	86.98	kip	(minimum required pretension)
V_u =	152.89	kip	(very conservative assumption)
M_u =	7350	kip-in	(very conservative assumption)
P_u =	600	kip	(very conservative assumption)
μ =	0.35	-	(class A surface)
SF =	2	-	(safety factor for not having enough bolt pretension force)

N_V =	10	bolts	(for friction shear)
N_M =	17	bolts	(for bending)
N_T =	9	bolts	(for uplifting)
$N_{req, total}$ =	35	bolts	
use =	34	bolts	

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 8, 2009
2F	Stub Beam	Page	16

$F1 =$	300	kip		$d =$	24.3	in			
$F2 =$	600	kip		$t_w =$	0.55	in			
$L_{stub}=$	19	in		$b =$	12.8	in			
Beam	w24x117			$t_f =$	0.85	in			

Column Dimension List

Column	w12x96					
$A_g =$	28.2	in^2		$b_f =$	12.2	in
$I_x =$	833	in^4		$t_f =$	0.9	in
$I_y =$	270	in^4		$d =$	12.7	in
$r_x =$	5.44	in		$t_w =$	0.55	in
$r_y =$	3.09	in		$F_y =$	50	ksi
$k_{des} =$	1.5	in		$E_s =$	29000	ksi

Column Web Local Yielding

$N =$	24.00	in	
$R_n =$	866.25	kip	
$\phi =$	1.00	-	
$\phi R_n =$	866.25	kip	OK

Column Web Crippling

$R_n =$	1382.366	kip	
$\phi =$	0.75	-	
$\phi R_n =$	1036.77	kip	OK

Column Flange Local Bending

$R_n =$	253.13	kip	$A_{web} =$	12.43	in^2	$A_s =$	34.19	in^2
$\phi =$	0.90	-	$A_{flange} =$	21.76	in^2			
$\phi R_n =$	227.81	kip	$F1_{flange}=$	190.93	kip			
			$F2_{flange}=$	381.87	kip	Continue Plate Required!		

Stub Beam Gross Yielding

$A_{s\,(beam)} =$	34.4	in^2	
$P_y =$	1720	kip	OK

Title	TCBF-B-1 Specimen Design Calculation Sheet	Date	August 8, 2009
1F	Stub Beam	Page	17

F_1 =	300	kip		d =	23.7	in		
F_2 =	600	kip		t_w =	0.415	in		
L_{stub}=	19	in		b =	8.97	in		
Beam	w24x68			t_f =	0.585	in		

Column Dimension List

Column	w12x96							
A_g =	28.2	in^2		b_f =	12.2	in		
I_x =	833	in^4		t_f =	0.9	in		
I_y =	270	in^4		d =	12.7	in		
r_x =	5.44	in		t_w =	0.55	in		
r_y =	3.09	in		F_y =	50	ksi		
k_{des} =	1.5	in		E_s =	29000	ksi		

Column Web Local Yielding

N =	24.00	in	
R_n =	866.25	kip	
ϕ =	1.00	-	
ϕR_n =	866.25	kip	OK

Column Web Crippling

R_n =	1382.366	kip	
ϕ =	0.75	-	
ϕR_n =	1036.77	kip	OK

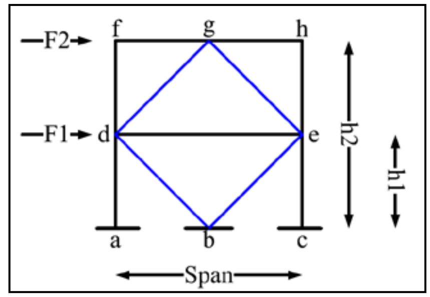
Column Flange Local Bending

R_n =	253.13	kip	A_{web} =	9.35	in^2	A_s =	19.84	in^2
ϕ =	0.90	-	A_{flange} =	10.49	in^2			
ϕR_n =	227.81	kip	$F1_{flange}$=	158.65	kip			
			$F2_{flange}$=	317.31	kip		OK	

Stub Beam Gross Yielding

$A_{s\,(beam)}$ =	20.1	in^2	
P_y =	1005	kip	OK

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
General		Page	1



Building height =	2	stories
Typical floor height =	9	ft
$F_{1,\,max}$ =	300	kip
$F_{2,\,max}$ =	600	kip
SR =	4	-
ratio =	0.8	-

Calculation Initialize

Items	values	units		Items	values	units
F_1 =	240	kip		V_1 =	720	kip
F_2 =	480	kip		V_2 =	480	kip
h_1 =	9	ft				
h_2 =	18	ft				
span =	20	ft	(beam span)			
h =	9	ft	(typical floor height)			
M_{base} =	13500	kip-ft				
P_{column} =	675	kip				
L_{brace} =	13.45	ft	(work point to work point)			
E_s =	29000	ksi				

Notes

: input value

Materials

Members	Material Type	Fy (ksi)	Fu (ksi)	Ry	Rt	(Ref: Table I-6-1)
Columns	ASTM A992	50	65	1.1	1.1	
Beams	ASTM A992	50	65	1.1	1.1	
Braces	ASTM A500B	42	58	1.4	1.3	(HSS-Round)
Plates 1	ASTM A36	36	58	1.3	1.2	
Bolts	A490	130	150	-	-	
Welds	E70XX	-	70	-	-	
Plates 2	ASTM A572 Gr.50	50	65	1.1	1.2	

Load Combinations

Per ASCE-7-2005

Basic Reference Codes

AISC Specification for Structural Steel Buildings (March 9, 2005)

AISC Seismic Provisions for Structural Steel Buildings (March 9, 2005)

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
2F-Brace		Page	2

P_u =	144.00	kip	(compression)				
L_{brace} =	8.1	ft					
k =	1.0	-					
Try section	HSS5x.500		(HSS-Square)				
A_s =	6.62	in^2		I_x =	17.20	in^4	
Z_x =	9.60	in^3		I_y =	17.20	in^4	
OD =	5.00	in		OD =	5.00	in	
t_{nom} =	0.50	in		t_{des} =	0.465	in	
r_x =	1.61	in					
r_y =	1.61	in					
F_y (brace) =	42	ksi		E_s =	29000	ksi	
kL/r =	60.25	-		Limit =	105.11	OK	$\boxed{Kl/r \leq 4\sqrt{E/F_y}}$
F_e =	78.85	ksi		0.44 F_y =	18.48	ksi	
ϕ =	0.90	-					
ϕP_n =	200.23	kip	(compression)		Check	OK	

Check Compactness Seismically (AISC Seismic Provisions 2005, Sec 8.2b)

λ_{ps} =	16.82	-	b/t =	0.00	OK	
						(Table I-8-1)
			h/t =	0.00	OK	

ϕ =	0.90	-				
ϕP_n =	250.24	kip	(tension)		Check	OK

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F-Brace		Page	3

P_u =	252.07	kip	(compression)				
L_{brace} =	9	ft					
k =	1.0	-					
Try section	HSS6x.500		(HSS-Square)				
A_s =	8.09	in2		I_x =	31.20	in4	
Z_x =	14.30	in3		I_y =	31.20	in4	
OD =	6.00	in		OD =	6.00	in	
t_{nom} =	0.50	in		t_{des} =	0.465	in	
r_x =	1.96	in					
r_y =	1.96	in					
F_y (brace) =	42	ksi		E_s =	29000	ksi	
kL/r =	55.10	-		Limit =	105.11	OK	$\boxed{Kl/r \le 4\sqrt{E/F_y}}$
F_e =	94.26	ksi		0.44 F_y =	18.48	ksi	
ϕ =	0.90	-					
ϕP_n =	253.77	kip	(compression)		Check	OK	

Check Compactness Seismically (AISC Seismic Provisions 2005, Sec 8.2b)

λ_{ps} =	16.82	-	b/t =	0.00	OK	
						(Table I-8-1)
			h/t =	0.00	OK	

ϕ =	0.90	-				
ϕP_n =	305.80	kip	(tension)		Check	OK

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
2F	Brace to Gusset Plate Connection	Page	4
Brace	HSS5x.500		

$R_yF_yA_g$ =	389.26	kip	(T_u)				
F_uA_g =	383.96	kip	(P_u)	T_u/P_u =	1.01	-	
R_yF_y =	58.8	ksi					
R_tF_u =	75.4	ksi					
U =	0.9	-					
ϕ_t =	0.75	(tensile rupture in net section)					
A_n/A_g =	1.16	(Net section reinforcement required!)					
ϕ_t =	0.90	(tensile yield in gross section)					
t_{gusset} =	0.87	in	(estimated)		F_y =	50	ksi
t_g =	0.75	in	(use)			(gusset plate)	
					F_u =	65	ksi
A_{cut} =	0.81	in^2					
A_{net} =	5.81	in^2					
A_e =	6.88	in^2	(Reinforcement required!)				

Reinforcement Plates

l =	12	in	Section	HSS6x.500					
x_bar =	1.909859	in	OD =	6	in		ID =	5	in
U =	0.84	-	$A_{e,\ req}$ =	6.88	in^2		$A_{net,\ req}$ =	8.19	in^2
A_{reinf} =	1.19	in^2	(both sides)						
b_{reinf} =	2.38	in	c_{reinf} =	2.32	in				
t_{req} =	0.50	in	b_{use} =	2.5	in		L_{plate} =	14	in
$F_{y,\ plate}$ =	42	ksi	$R_yF_yA_g$ =	73.50	kip				
L_{weld} =	6	in	weld =	5	x 1/16 in	(fillet)			
ϕR_n =	83.51	kip	OK						

Brace Block Shear

t_{brace} =	0.465	in	
L_{req} =	8.02	in	OK
L_{use} =	12	in	

Brace to Gusset Plate Weld

L_{weld} =	12	in	
weld =	6	x 1/16 in	(fillet)
ϕ_b =	0.75	-	

F_{exx} =	70	ksi		
F_w =	42	ksi		
$\phi_b R_n$ =	400.87	kip	OK	

Gusset Plate Block Shear

A_{gv} =	18	in^2		
A_{nt} =	4.31	in^2		
U_{bs} =	1	-		
ϕ =	0.75	-		
ϕR_n =	615.23	kip	OK	

Whitmore Effective Width

$L_{whitmore}$ =	21.59	in	(theoretical width)	
ϕ =	0.90	-		
ϕR_n =	728.61	kip	OK	(check gross yield)

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F	Brace to Gusset Plate Connection	Page	5
Brace	HSS6x.500		

$R_yF_yA_g$ =	475.692	kip	(T_u)							
F_uA_g =	469.22	kip	(P_u)		T_u/P_u =	1.01		-		
R_yF_y =	58.8	ksi								
R_tF_u =	75.4	ksi								
U =	0.9	-								
ϕ_t =	0.75	(tensile rupture in net section)								
A_n/A_g =	1.16	(Net section reinforcement required!)								
ϕ_t =	0.90	(tensile yield in gross section)								
t_{gusset} =	0.88	in	(estimated)			F_y =	50	ksi		
t_g =	0.75	in	(use)				(gusset plate)			
						F_u =	65	ksi		
A_{cut} =	0.81	in^2								
A_{net} =	7.28	in^2								
A_e =	8.41	in^2	(reinforcement required)							

Reinforcement Plates

l =	14	in	Section	HSS7x.500				
x_bar =	2.23	in	OD =	7	in	ID =	6	in
U =	0.84	-	$A_{e,\ req}$ =	8.41	in^2	$A_{net,\ req}$ =	10.00	in^2
A_{reinf} =	1.36	in^2	(both sides)					
b_{reinf} =	2.73	in	c_{reinf} =	2.66	in			
t_{req} =	0.50	in	t_{use} =	2.75	in	L_{plate} =	16	in
$F_{y,\ plate}$ =	42	ksi	$R_yF_yA_g$ =	80.85	kip			
L_{weld} =	7	in	weld =	5	x 1/16 in	(fillet)		
ϕR_n =	97.43	kip	OK					

Brace Block Shear

t_{brace} =	0.465	in	
L_{req} =	9.80	in	OK
L_{use} =	14	in	

Brace to Gusset Plate Weld

L_{weld} =	14	in	
weld =	7	x 1/16 in	(fillet)
ϕ_b =	0.75	-	

F_{exx} =	70	ksi		
F_w =	42	ksi		
$\phi_b R_n$ =	545.63	kip	OK	
Gusset Plate Block Shear				
A_{gv} =	21	in^2		
A_{nt} =	5.16	in^2		
U_{bs} =	1	-		
ϕ =	0.75	-		
ϕR_n =	723.87	kip	OK	
Whitmore Effective Width				
$L_{whitmore}$ =	24.90	in	(theoretical width)	
ϕ =	0.90	-		
ϕR_n =	840.30	kip	OK	(check gross yield)

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
2F	Roof Beam Design Check	Page	6

$R_yF_yA_g$ =	389.26	kip		θ =	0.73	(rad)	42.0	(deg)
0.3 P_n =	66.74	kip		$\sin(\theta)$ =	0.67			
				$\cos(\theta)$ =	0.74			
V =	215.75	kip						
H =	338.94	kip						
P_u =	600.00	kip	(conservatively)					
M_u =	88.57	kip-ft	(revised from structural analysis)					
Try	w24x117							
A_g =	34.4	in^2		b_f =	12.8	in		
I_x =	3540	in^4		t_f =	0.85	in		
I_y =	297	in^4		d =	24.3	in		
r_x =	10.1	in		t_w =	0.55	in		
r_y =	2.94	in		F_y =	50	ksi		
λ_{p1} =	9.15			b/t =	7.53	Compact		
λ_{p2} =	90.55			h/tw =	41.09	Compact		
L_p =	10.38	ft		Z_x =	327	in^3		
c =	1	-		J =	6.72	in^4		
C_w =	40800	in^6		h_o =	23.45	in		
S_x =	291	in^3		r_{ts} =	3.46	in		
L_r =	29.90	ft		Brace PT=	2	-		
L_b =	10	ft		C_b =	1.0	(Conservatively)		
M_p =	1362.5	kip-ft		F_{cr} =	248.50	ksi		
ϕ_b =	0.90	-		M_n =	1362.50	kip-ft	(Need Check)	
ϕ_bM_n =	1226.25	kip-ft						
kl/r =	40.82	-		k =	1.0	-		
F_e =	171.79	ksi		0.44 F_y =	22	ksi		
ϕ_c =	0.90	-						
ϕ_cP_n =	1370.46	kip						
P_u/ϕ_cP_n =	0.44	use (H1-1a)						
Check	0.50	OK						

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F	Lower Beam Design Check	Page	7

$R_yF_yA_g$ =	475.69	kip	(1F)					
$R_yF_yA_g$ =	389.26	kip	(2F)	θ =	0.73	(rad)	42.0	(deg)
0.3 P_n =	84.59	kip	(1F)	$\sin(\theta)$ =	0.67			
0.3 P_n =	66.74	kip	(2F)	$\cos(\theta)$ =	0.74			
V =	273.57	kip						
H =	303.97	kip						
P_u =	303.97	kip	(conservatively)					
M_u =	159.45	kip-ft	(revised from structural analysis)					
Try	w24x68							
A_g =	20.1	in^2		b_f =	8.97	in		
I_x =	1830	in^4		t_f =	0.585	in		
I_y =	70.4	in^4		d =	23.7	in		
r_x =	9.55	in		t_w =	0.415	in		
r_y =	1.87	in		F_y =	50	ksi		
λ_{p1} =	9.15			b/t =	7.67	Compact		
λ_{p2} =	90.55			h/tw =	54.29	Compact		
L_p =	6.61	ft		Z_x =	177	in^3		
c =	1	-		J =	1.87	in^4		
C_w =	9430	in^6		h_o =	23.12	in		
S_x =	154	in^3		r_{ts} =	2.30	in		
L_r =	18.74	ft		Brace PT=	2	-		
L_b =	10	ft		C_b =	1.0	(Conservatively)		
M_p =	737.5	kip-ft		F_{cr} =	110.86	ksi		
ϕ_b =	0.90	-		M_n =	656.87	kip-ft	OK	
ϕ_bM_n =	591.18	kip-ft						
kl/r =	64.17	-		k =	1.0	-		
F_e =	69.50	ksi		0.44 F_y =	22	ksi		
ϕ_c =	0.90	-						
ϕ_cP_n =	669.33	kip						
P_u/ϕ_cP_n =	0.45	use (H1-1a)						
Check	0.69	OK						

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
2F	Column Design Check	Page	8

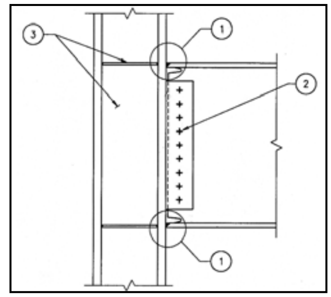
P_u =	43.94	kip	(revised from structural analysis)					
M_u =	110.65	kip-ft	(revised from structural analysis)					
L_{column} =	9	ft						
Try	w12x96							
A_g =	28.2	in^2		b_f =	12.2	in		
I_x =	833	in^4		t_f =	0.9	in		
I_y =	270	in^4		d =	12.7	in		
r_x =	5.44	in		t_w =	0.55	in		
r_y =	3.09	in		F_y =	50	ksi		
λ_{p1} =	7.22	-		b/t =	6.78	Compact		
λ_{p2} =	71.59	-		h/tw =	19.82	Compact		
L_p =	10.91	ft		Z_x =	147	in^3		
c =	1	-		J =	6.85	in^4		
C_w =	9410	in^6		h_o =	11.80	in		
S_x =	131	in^3		r_{ts} =	3.49	in		
L_r =	40.86	ft		Brace PT=	0	-		
L_b =	9	ft		C_b =	1.0	(Conservatively)		
M_p =	612.5	kip-ft		F_{cr} =	344.49	ksi		
ϕ_b =	0.90	-		M_n =	612.50	kip-ft		(Need Check)
$\phi_b M_n$ =	551.25	kip-ft		C_a =	0.03	-		
kl/r =	34.95	-		k =	1.0	-		
F_e =	234.28	ksi		$0.44\ F_y$ =	22	ksi		
ϕ_c =	0.90	-						
$\phi_c P_n$ =	1160.56	kip						
$P_u/\phi_c P_n$ =	0.04	use (H1-1b)						
Check	0.22	OK						

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F	Column Design Check	Page	9

P_u =	493.82	kip	(revised from structural analysis)				
M_u =	263.05	kip-ft	(revised from structural analysis)				
L_{column} =	9	ft					
Try	w12x96						
A_g =	28.2	in^2		b_f =	12.2	in	
I_x =	833	in^4		t_f =	0.9	in	
I_y =	270	in^4		d =	12.7	in	
r_x =	5.44	in		t_w =	0.55	in	
r_y =	3.09	in		F_y =	50	ksi	
λ_{p1} =	7.22	-		b/t =	6.78	Compact	
λ_{p2} =	52.35	-		h/tw =	19.82	Compact	
L_p =	10.91	ft		Z_x =	147	in^3	
c =	1	-		J =	6.85	in^4	
C_w =	9410	in^6		h_o =	11.80	in	
S_x =	131	in^3		r_{ts} =	3.49	in	
L_r =	40.86	ft		Brace PT=	0	-	
L_b =	9	ft		C_b =	1.0	(Conservatively)	
M_p =	612.5	kip-ft		F_{cr} =	344.49	ksi	
ϕ_b =	0.90	-		M_n =	612.50	kip-ft	(Need Check)
$\phi_b M_n$ =	551.25	kip-ft		C_a =	0.39	-	
kl/r =	34.95	-		k =	1.0	-	
F_e =	234.28	ksi		0.44 F_y =	22	ksi	
ϕ_c =	0.90	-					
$\phi_c P_n$ =	1160.56	kip					
$P_u/\phi_c P_n$ =	0.43	use (H1-1a)					
Check	0.85	OK					

Check Column Web Shear Stress			
M_p =	7350	kip-in	
L =	96.15	in	
V =	152.89	kip	
A_s =	6.99	in^2	$A_s = d*tw$
S_v =	21.89	ksi	
$S_{v,\ yield}$ =	29.00	ksi	Elastic

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
2F	Beam-Column Connection	Page	10



Type Bolted (WUF-B)

$H =$ 169.47 kip

$V =$ 107.88 kip

$M =$ 88.57 kip-ft (revised from structural analysis)

$R_u =$ 200.89 kip

Try $d_b =$ 0.88 in $F_u =$ 150 ksi

$A_b =$ 0.60 in^2 $F_{nv} =$ 75 ksi (threads excluded)

$N_b =$ 6 bolts (in one row)

$R_n =$ 270.59 kip (bolt shear) $L_{c\_ex} =$ 1.5 in

$\phi_b =$ 0.75 - $L_{c\_in} =$ 3 in

$\phi_b R_n =$ 202.94 kip OK

$L_{c1} =$ 1.03 in (edge clear distance) $R_{n1} =$ 46.41 kip

$L_{c2} =$ 2.06 in (clear distance) $R_{n2} =$ 236.25 kip

$t =$ 0.50 in (shear tab thickness)

$R_n =$ 1227.66 kip (combined bolt bearing)

$\phi_b =$ 0.75 -

$\phi_b R_n =$ 920.74 kip OK

$L_{tab} =$ 18 in $A_{s,\,tab} =$ 9 in^2 $R_n =$ 524.79 kip

$w_{tab} =$ 4.5 in $F_{y,\,tab} =$ 50 ksi $P_{nt} =$ 450.00 kip

$F_{v,\,tab} =$ 30.0 ksi $P_{nv} =$ 270.00 kip

OK

Weld Fillet (shear tab)

$F_{exx} =$ 70 ksi $R_n =$ 334.06 kip

$Fw =$ 42 ksi $\phi_b =$ 0.75 -

$w =$ 5 x 1/16 inch $\phi_b R_n =$ 250.54 kip OK

$L_{weld} =$ 18 in

side = 2 sides

Weld CJP (top, bottom flanges)

$b_f =$ 12.8 in $F_{y,\,bm} =$ 50 ksi (base metal)

$t_f =$ 0.85 in $M_n =$ 1063.07 kip-ft OK

$d =$ 24.3 in

$t_w =$ 0.55 in

Check Shear tab length, OK

Check Block Shear

Beam
w24x117

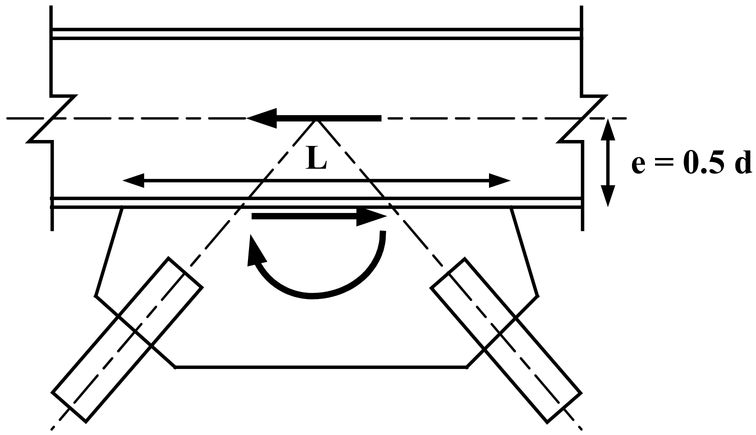
A_{gv} =	9.9	in^2	
A_{gt} =	1.925	in^2	
A_{nv} =	6.6	in^2	
A_{nt} =	1.65	in^2	
U_{bs} =	0.5	-	
ϕ =	0.75	-	
F_y =	50	ksi	
F_u =	65	ksi	
ϕR_n =	233.27	kip	OK

Shear Tab

A_{gv} =	8.25	in^2	
A_{gt} =	1.5	in^2	
A_{nv} =	5.5	in^2	
A_{nt} =	1.25	in^2	
U_{bs} =	0.5	-	
ϕ =	0.75	-	
F_y =	50	ksi	
F_u =	65	ksi	
ϕR_n =	191.34	kip	NG!

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
2F	Braces to Beam Connection	Page	11

Braces		HSS5x.500		
T =	389.26	kip	$\sin(\theta)$ =	0.669
C =	342.61	kip	$\cos(\theta)$ =	0.743
e =	12.15	in		
Shear =	543.99	kip		
Tension =	197.89	kip		
Moment =	550.79	kip-ft		
t_{gusset} =	0.75	in		
L =	60	in		
s_V =	12.09	ksi		
s_A =	4.40	ksi		
s_M =	14.69	ksi		
ϕ =	0.9	-		
$F_{y,\ gusset}$ =	50	ksi		
Ratio =	0.63	OK		



$L_{whitmore}$ =	21.59	in	L_{min} =	59.27	in	(geometry limit)	OK
L_v =	16	in	$L_{v,\ min}$ =	16.05	in	(geometry limit)	NG!

w_{up} =	10.79	in	A_v =	12	in^2	
w_{low} =	20.92	in	P_u =	260.40	kip	
$Whitm_{eff}$ =	21.59	in	ϕR_n=	324	kip	OK
ϕR_n =	728.61	kip	OK			

Gusset Plate to Beam Flange

Weld	Fillet						
F_{exx} =	70	ksi					
F_w =	42	ksi			s_V =	12.82	ksi
w =	8	x 1/16	inch		s_M =	15.58	ksi
L_{weld} =	60	in			s_A =	4.67	ksi
side =	2	sides			f_{peak} =	23.97	ksi
t_{eff} =	0.354	in			f_{avg} =	20.40	ksi
ϕ =	0.75	-			f_r =	25.50	ksi
Ratio =	0.86	OK			Ratio =	0.81	OK

Check Beam Web

width =	60	in	Beam	w24x117

R_u =	220.32	kip	d =	24.3	in	t_w =	0.55	in
N =	30	in	t_f =	0.85	in	$F_{y,\,web}$ =	50	ksi
ϕ =	0.75	-	R_n =	1060.63	kip	ϕR_n =	795.47	kip
k_{des} =	1.35	in				(web crippling)		OK
ϕ =	1.00	-	R_n =	1010.63	kip	ϕR_n =	1010.63	kip
						(web local yielding)		OK

Check Gusset Plate Buckling

L_{gb} =	17.68	in	kL/r =	98.0	-	L_c =	14.13	in
k =	1.2	-	F_e =	29.82	ksi	L_{c1} =	11.38	in
r =	0.217	in	0.44 F_y =	22	ksi	L_{c2} =	16.94	in
A_g =	16.19	in^2	R_n =	401.31	kip	L_{max} =	24.06	in
ϕ =	0.9	-	ϕR_n =	361.18	kip	L_{tip} =	21.88	in
					OK	L_{ave} =	17.68	in

Free Edge Buckling

L_e =	15.63	in	
L_e/t_g =	20.83	-	
Limit =	18.06	-	Edge stiffener required!

Lateral Stability of Beam

M_r =	1498.75	kip-ft	Z =	327	in^3	L_b =	10	ft
C_d =	1	-	h_o =	23.45	in	L_{pd} =	17.05	ft
P_{br} =	15.34	kip	β_{br} =	85.22	kip/in	n =	1	OK
(Nodal)			(Nodal)			C_b =	1	-

M_{br} =	17.985	kip-ft	
P_{br} =	9.20	kip	(torsional)
β_T =	28842	kip-in/rad	β_{sec} not included
β_{br} =	52.45	kip/in	(torsional)
Δ =	0.18	in	

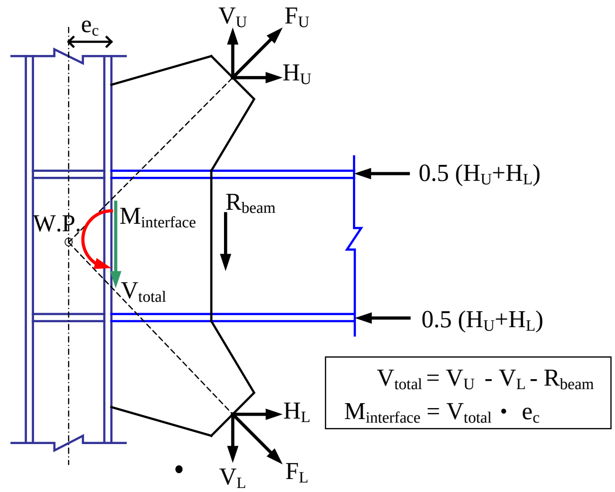
Kicker

L3x2x3/8

A_g =	1.73	in^2	
L =	25	in	
k_{axial} =	2006.8	kip/in	
k =	1419	kip/in	OK

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F	Beam-Column-Gusset Connections	Page	12

Big Gusset Plate for Upper Floor Bracing and Lower Floor Bracing



Sway to Right

F_{U2R} =	389.26	kip	$\cos(\theta_U)$ =	0.743
F_{L2R} =	434.24	kip	$\cos(\theta_L)$ =	0.743

Sway to Left

F_{U2L} =	342.61	kip				
F_{L2L} =	475.69	kip				
Beam	w24x68					
d =	23.7	in				
$L_{c,\ min}$ =	58.25	in				
Column	w12x96					
e_c =	6.35	in				
R_{beam} =	21.32	kip	(downward)			
$L_{b,\ min}$ =	16.66	in				
$L_{cu,\ min}$ =	16.05	in				
$L_{cl,\ min}$ =	18.51	in				
t_g =	0.75	in				
L_{cu} =	18	in	(use)	$F_{y,\ gusset}$ =	50	ksi
L_{cl} =	18	in	(use)			
L_c =	59.7	in	(use)			
L_b =	23	in	(use)			

Sway to the Right

V_{U2R} =	260.40	kip	(upward)	s_V =	11.83	ksi	
H_{U2R} =	289.33	kip	(rightward)	s_M =	7.55	ksi	
V_{L2R} =	290.49	kip	(upward)	s_A =	0.75	ksi	
H_{L2R} =	322.77	kip	(leftward)	Ratio =	0.49	-	OK
V_{total} =	529.57	kip	(upward)				
M =	280.23	kip-ft	(counter-clockwise)				
				Column-Side			
V_{cu} =	159.67	kip	(downward)	s_A =	5.27	ksi	
f_1 =	5.66	kip/in	(lefttward)	s_V =	11.83	ksi	
f_2 =	2.25	kip/in	(leftward)	s_M =	2.28	ksi	
H_{cu} =	71.18	kip	(leftward)	Ratio =	0.49	-	OK
H_{bu} =	218.16	kip	(leftward)	Beam-Side			
V_{bu} =	100.73	kip	(downward)	s_A =	5.84	ksi	
M_{cu} =	7.68	kip-ft	(counter-clockwise)	s_V =	12.65	ksi	

M_{bu} =	-112.45	kip-ft	(clockwise)	s_M =	-20.41	ksi	
				Ratio =	0.58	-	OK

V_{cl} =	159.67	kip	(downward)	s_A =	5.27	ksi	
f_1 =	5.66	kip/in	(rightward)	s_V =	11.83	ksi	
f_3 =	2.25	kip/in	(rightward)	s_M =	2.28	ksi	
H_{cl} =	71.18	kip	(rightward)	Ratio =	0.49	-	OK
H_{bl} =	251.59	kip	(rightward)				
V_{bl} =	130.82	kip	(downward)	s_A =	7.58	ksi	
M_{bl} =	100.71	kip-ft	(clockwise)	s_V =	14.58	ksi	
M_{cl} =	7.68	kip-ft	(counter-clockwise)	s_M =	18.28	ksi	
				Ratio =	0.80	-	OK

V_{mid} =	161.44	kip	(downward)	s_A =	0.00	ksi	
M_{mid} =	10.34	kip-ft	(counter-clockwise)	s_V =	9.08	ksi	
H_{mid} =	0.00	kip	(leftward)	s_M =	1.77	ksi	
				Ratio =	0.35	-	OK

Weld Size

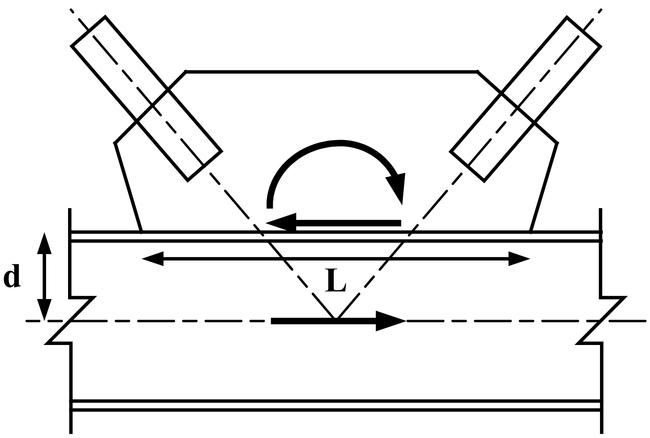
f_v =	8.87	kip/in	
f_a =	5.13	kip/in	(averaged)
f_b =	5.66	kip/in	
f_{peak} =	13.97	kip/in	
f_{avg} =	11.43	kip/in	
f_r =	14.28	kip/in	14.283
D >=	5.13	x 1/16	(weld size)
Use	6	x 1/16	(weld size)

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F	Lower Beam to Gusset Plate Splice	Page	13

Web	Fillet welds with web plates							
Flange	CJP weld		(T & B)					
P_u =	303.97	kip						
t_f =	0.59	in						
b =	8.97	in						
A_s =	5.25	in2						
$2*A_s*F_y$ =	524.75	kip	OK					
R_{beam} =	21.32	kip	(Gravity)					
L_{tab} =	20.375	in			t =	0.5	in	
w_{tab} =	8	in						
Weld	Fillet	(shear tab)						
F_{exx} =	70	ksi			R_n =	315.96	kip	
F_w =	42	ksi			ϕ_b =	0.75	-	
w =	6	x 1/16	inch		$\phi_b R_n$ =	236.97	kip	OK
L_{weld} =	28.375	in						
side =	1	sides						
Shim Plate								
L =	20	in			t =	0.168	in	(shim as required)
w =	4	in						
Weld	Fillet	(shear tab)						
F_{exx} =	70	ksi			R_n =	139.26	kip	
F_w =	42	ksi			ϕ_b =	0.75	-	
w =	2.68	x 1/16	inch		$\phi_b R_n$ =	104.45	kip	OK
L_{weld} =	28	in	(3 sides)					
side =	1	sides						

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F	Braces to Floor Beam Connection	Page	14

Braces		HSS6x.500		
T =	475.69	kip	$\sin(\theta)$ =	0.669
C =	434.24	kip	$\cos(\theta)$ =	0.743
e =	0	in		
Shear =	676.34	kip		
Tension =	239.00	kip		
Moment =	0.00	kip-ft		
t_{gusset} =	0.75	in		
L =	46	in		
s_V =	19.60	ksi		
s_M =	0.00	ksi		
ϕ =	0.9	-		
$F_{y,\ gusset}$ =	50	ksi		
Ratio =	0.77	OK		



whitmo =	24.90	in	L_{min} =	37.22	in	(geometry limit)	OK
L_v =	19	in	$L_{v,\ min}$ =	18.51	in	(geometry limit)	OK

w_{up} =	12.45	in	A_v =	14.25	in2
w_{low} =	14.12	in	P_u =	318.22	kip
$whit_{eff}$ =	24.90	in	ϕR_n=	384.75	kip
ϕR_n =	840.30	kip	OK		OK

Weld	Fillet	(Gusset to beam flange)					
F_{exx} =	70	ksi					
F_w =	42	ksi					
w =	11	x	1/16	inch	s_V =	15.12	ksi
L_{weld} =	46	in			s_M =	0.00	ksi
side =	2	sides			s_A =	5.34	ksi
t_{eff} =	0.486	in					
ϕ =	0.75	-					
Ratio =	0.85	OK					

Check Beam Web

width =	46	in	Beam	w30x391			
R_u =	0.00	kip	d =	33.2	in	t_w =	1.36 in

$N =$	23	in	$t_f =$	2.44	in	$F_{y,\ web} =$	50	ksi
$\phi =$	0.75	-	$R_n =$	4450.60	kip	$\phi R_n =$	3337.95	kip
$k_{des} =$	3.23	in				(web crippling)		OK
$\phi =$	1.00	-	$R_n =$	2662.20	kip	$\phi R_n =$	2662.20	kip
						(web local yielding)		OK

Check Gusset Plate Buckling

$L_{gb} =$	14.84	in	$kL/r =$	82.2	-	$L_c =$	15.88	in
$k =$	1.2	-	$F_e =$	42.32	ksi	$L_{c1} =$	18.06	in
$r =$	0.217	in	$0.44\ F_y =$	22	ksi	$L_{c2} =$	12.56	in
$A_g =$	18.67	in2	$R_n =$	569.41	kip	$L_{max} =$	18.56	in
$\phi =$	0.9	-	$\phi R_n =$	512.47	kip	$L_{tip} =$	9.13	in
					OK	$L_{ave} =$	14.84	in

Free Edge Buckling

$L_e =$	17.88	in	
$L_e/t_g =$	23.83	-	
Limit =	18.06	-	Edge stiffener required!

Lateral Stability of Beam

$M_r =$	6645.83	kip-ft	$Z =$	1450	in3	$L_b =$	10	ft
$C_d =$	1	-	$h_o =$	30.76	in	$L_{pd} =$	17.05	ft
$P_{br} =$	51.85	kip	$\beta_{br} =$	288.07	kip/in	$n =$	1	OK
(Nodal)			(Nodal)			$C_b =$	1	-
$M_{br} =$	79.75	kip-ft				$I_y =$	1550	in4
$P_{br} =$	31.11	kip	(torsional)					
$\beta_T =$	108666	kip-in/rad	βsec not included					
$\beta_{br} =$	114.85	kip/in	(torsional)					
$\Delta =$	0.27	in						

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F	Column Base Plate Design Check	Page	15
Column	w12x96		

Z_x =	147	in3		L =	96.15	in
F_y =	50	ksi		V_{Mp} =	152.89	kip
M_p =	7350	kip-in				
P_u =	493.82	kip		d =	12.7	in
M_u =	3156.6	kip-in		b_f =	12.2	in
N =	31.25	in		$f_{p,\,max}$ =	36	ksi
B =	28	in				
e =	6.39	in		q_{max} =	1008	kip/in
e_{cr} =	15.38	in	(Small Moment)			
Y =	18.47	in				
q =	26.74	kip/in	OK			
m =	9.59	in				
f_p =	0.96	ksi				
$t_{p,\,req}$ =	1.99	in	eq 3.3.14a (LRFD)			
use =	2.00	in				

All-thread-rods

Type	ASTM A193 B7		
d_{bolt} =	1.125	in	
F_u =	125	ksi	
F_y =	105	ksi	
F_{nt} =	93.75	ksi	
F_{nv} =	50	ksi	
A_b =	0.99	in^2	
ϕ =	0.75	-	
ϕR_n =	69.89	kip	(tension)
ϕR_n =	37.28	kip	(shear)
F_{PT} =	86.98	kip	(minimum required pretension)
V_u =	152.89	kip	(very conservative assumption)
M_u =	7350	kip-in	(very conservative assumption)
P_u =	600	kip	(very conservative assumption)
μ =	0.35	-	(class A surface)
SF =	2	-	(safety factor for not having enough bolt pretension force)

N_V =	10	bolts	(for friction shear)
N_M =	17	bolts	(for bending)
N_T =	9	bolts	(for uplifting)
$N_{req,\ total}$ =	35	bolts	
use =	34	bolts	

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
2F	Stub Beam	Page	16

F1 =	300	kip		d =	24.3	in
F2 =	600	kip		t_w =	0.55	in
L_{stub}=	19	in		b =	12.8	in
Beam	w24x117			t_f =	0.85	in

Column Dimension List

Column	w12x96					
A_g =	28.2	in^2		b_f =	12.2	in
I_x =	833	in^4		t_f =	0.9	in
I_y =	270	in^4		d =	12.7	in
r_x =	5.44	in		t_w =	0.55	in
r_y =	3.09	in		F_y =	50	ksi
k_{des} =	1.5	in		E_s =	29000	ksi

Column Web Local Yielding

N =	24.00	in	
R_n =	866.25	kip	
ϕ =	1.00	-	
ϕR_n =	866.25	kip	OK

Column Web Crippling

R_n =	1382.366	kip	
ϕ =	0.75	-	
ϕR_n =	1036.77	kip	OK

Column Flange Local Bending

R_n =	253.13	kip	A_{web} =	12.43	in^2	A_s =	34.19	in^2
ϕ =	0.90	-	A_{flange} =	21.76	in^2			
ϕR_n =	227.81	kip	$F1_{flange}$=	190.93	kip			
			$F2_{flange}$=	381.87	kip	Continue Plate Required!		

Stub Beam Gross Yielding

$A_{s\,(beam)}$ =	34.4	in^2	
P_y =	1720	kip	OK

Title	TCBF-B-2 Specimen Design Calculation Sheet	Date	May 27, 2009
1F	Stub Beam	Page	17

F_1 =	300	kip	d =	23.7	in
F_2 =	600	kip	t_w =	0.415	in
L_{stub}=	19	in	b =	8.97	in
Beam	w24x68		t_f =	0.585	in

Column Dimension List

Column	w12x96				
A_g =	28.2	in^2	b_f =	12.2	in
I_x =	833	in^4	t_f =	0.9	in
I_y =	270	in^4	d =	12.7	in
r_x =	5.44	in	t_w =	0.55	in
r_y =	3.09	in	F_y =	50	ksi
k_{des} =	1.5	in	E_s =	29000	ksi

Column Web Local Yielding

N =	24.00	in	
R_n =	866.25	kip	
ϕ =	1.00	-	
ϕR_n =	866.25	kip	OK

Column Web Crippling

R_n =	1382.366	kip	
ϕ =	0.75	-	
ϕR_n =	1036.77	kip	OK

Column Flange Local Bending

R_n =	253.13	kip	A_{web} =	9.35	in^2	A_s =	19.84	in^2
ϕ =	0.90	-	A_{flange} =	10.49	in^2			
ϕR_n =	227.81	kip	$F1_{flange}$=	158.65	kip			
			$F2_{flange}$=	317.31	kip		OK	

Stub Beam Gross Yielding

$A_{s\,(beam)}$ =	20.1	in^2	
P_y =	1005	kip	OK

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
General		Page	1

Building height = 2 stories

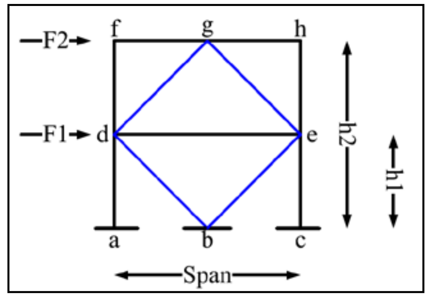
Typical floor height = 9 ft

$F_{1,\,max}$ = 300 kip

$F_{2,\,max}$ = 600 kip

SR = 4 -

ratio = 0.8 -



Calculation Initialize

Items	values	units		Items	values	units
F_1 =	240	kip		V_1 =	720	kip
F_2 =	480	kip		V_2 =	480	kip
h_1 =	9	ft				
h_2 =	18	ft				
span =	20	ft	(beam span)			
h =	9	ft	(typical floor height)			
M_{base} =	13500	kip-ft				
P_{column} =	675	kip				
L_{brace} =	13.45	ft	(work point to work point)			
E_s =	29000	ksi				

Notes

: input value

Materials

Members	Material Type	Fy (ksi)	Fu (ksi)	Ry	Rt	(Ref: Table I-6-1)
Columns	ASTM A992	50	65	1.1	1.1	
Beams	ASTM A992	50	65	1.1	1.1	
Braces	ASTM A992	50	65	1.1	1.1	(Wide Flange)
Plates 1	ASTM A36	36	58	1.3	1.2	
Bolts	A490	130	150	-	-	
Welds	E70XX	-	70	-	-	
Plates 2	ASTM A572 Gr.50	50	65	1.1	1.2	

Load Combinations

Per ASCE-7-2005

Basic Reference Codes

AISC Specification for Structural Steel Buildings (March 9, 2005)

AISC Seismic Provisions for Structural Steel Buildings (March 9, 2005)

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
2F-Brace		Page	2

P_u =	139.67	kip	(compression)				
L_{brace} =	8.1	ft					
k =	1.0	-					
Try section	W8x21		(HSS-Square)				
A_s =	6.16	in^2		I_x =	75.30	in^4	
Z_x =	20.40	in^3		I_y =	75.30	in^4	
b_f =	5.27	in		t_f =	0.40	in	
d =	8.28	in		t_w =	0.250	in	
r_x =	3.49	in					
r_y =	1.26	in					
F_y (brace) =	50	ksi		E_s =	29000	ksi	
kL/r =	76.98	-		Limit =	96.33	OK	$\boxed{Kl/r \leq 4\sqrt{E/F_y}}$
F_e =	48.29	ksi		0.44 F_y =	22	ksi	
ϕ =	0.90	-					
ϕP_n =	179.72	kip	(compression)		Check	OK	

Check Compactness Seismically (AISC Seismic Provisions 2005, Sec 8.2b)

λ_{ps} =	7.22	-	b_f / 2t =	6.59	OK	
Ca =	0.50					(Table I-8-1)
λ_{ps} =	35.88	-	h / t_w =	27.50	OK	

ϕ =	0.90	-			
ϕP_n =	277.20	kip	(tension)	Check	OK

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F-Brace		Page	3

P_u =	254.24	kip	(compression)				
L_{brace} =	9	ft					
k =	1.0	-					
Try section	W8x28		(HSS-Square)				
A_s =	8.24	in2		I_x =	98.00	in4	
Z_x =	27.20	in3		I_y =	98.00	in4	
b =	6.54	in		t_f =	0.47	in	
d =	8.06	in		t_w =	0.285	in	
r_x =	3.45	in					
r_y =	1.62	in					
F_y (brace) =	50	ksi		E_s =	29000	ksi	
kL/r =	66.67	-		Limit =	96.33	OK	$\boxed{Kl/r \le 4\sqrt{E/F_y}}$
F_e =	64.40	ksi		0.44 F_y =	22	ksi	
ϕ =	0.90	-					
ϕP_n =	267.92	kip	(compression)		Check	OK	

Check Compactness Seismically (AISC Seismic Provisions 2005, Sec 8.2b)

λ_{ps} =	7.22	-	b_f / 2t =	7.03	OK	
Ca =	0.69					(Table I-8-1)
λ_{ps} =	35.88	-	h / t_w =	22.30	OK	

ϕ =	0.90	-			
ϕP_n =	370.80	kip	(tension)	Check	OK

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
2F	Brace to Gusset Plate Connection	Page	4
Brace	W8x21		

$R_yF_yA_g$ =	338.80	kip	(T_u)					
F_uA_g =	400.40	kip	(P_u)	T_u/P_u =	0.85	-		
R_yF_y =	55	ksi						
R_tF_u =	71.5	ksi						
U =	0.9	-						
ϕ_t =	0.75	(tensile rupture in net section)						
A_n/A_g =	1.14	(Net section reinforcement required!)						
ϕ_t =	0.90	(tensile yield in gross section)						
t_{gusset} =	0.71	in	(estimated)			F_y =	50	ksi
t_g =	0.75	in	(use)				(gusset plate)	
						F_u =	65	ksi
A_{cut} =	2.57	in^2						
A_{net} =	3.59	in^2						
A_e =	6.32	in^2	(Reinforcement required!)					

Reinforcement Plates

l =	12	in	d =	8.28	in			
U =	1.00	-	$A_{e,\ req}$ =	6.32	in^2	$A_{net,\ req}$ =	6.32	in^2
A_{reinf} =	1.36	in^2	(both sides)					
b_{reinf} =	4	in						
t_{req} =	0.47	in	t_{use} =	0.5	in	L_{plate} =	14	in
$F_{y,\ plate}$ =	36	ksi	$R_yF_yA_g$ =	93.60	kip			
L_{weld} =	6	in	weld =	6	x 1/16 in	(fillet)		
ϕR_n =	100.22	kip	OK					

Brace Block Shear

t_{brace} =	0.4	in	
L_{req} =	7.24	in	OK
L_{use} =	12	in	

Brace to Gusset Plate Weld

L_{weld} =	12	in	
weld =	6	x 1/16 in	(fillet)
ϕ_b =	0.75	-	

F_{exx} =	70	ksi	
F_w =	42	ksi	
$\phi_b R_n$ =	400.87	kip	OK

Gusset Plate Block Shear

A_{gv} =	18	in^2	
A_{nt} =	6.77	in^2	
U_{bs} =	1	-	
ϕ =	0.75	-	
ϕR_n =	735.16	kip	OK

Whitmore Effective Width

$L_{whitmore}$ =	23.87	in	(theoretical width)	
ϕ =	0.90	-		
ϕR_n =	805.56	kip	OK	(check gross yield)

Brace Web Block Shear

A_{gv} =	3	in^2	
A_{nt} =	1.19	in^2	
U_{bs} =	1	-	
ϕ =	0.75	-	
ϕR_n =	125.39	kip	OK

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F	Brace to Gusset Plate Connection	Page	5
Brace	W8x28		

$R_yF_yA_g$ =	453.2	kip	(T_u)					
F_uA_g =	535.6	kip	(P_u)	T_u/P_u =	0.85	-		
R_yF_y =	55	ksi						
R_tF_u =	71.5	ksi						
U =	0.9	-						
ϕ_t =	0.75	(tensile rupture in net section)						
A_n/A_g =	1.14	(Net section reinforcement required!)						
ϕ_t =	0.90	(tensile yield in gross section)						
t_{gusset} =	0.77	in	(estimated)		F_y =	50	ksi	
t_g =	0.75	in	(use)			(gusset plate)		
					F_u =	65	ksi	
A_{cut} =	2.85	in^2						
A_{net} =	5.39	in^2						
A_e =	8.45	in^2	(reinforcement required)					

Reinforcement Plates

l =	14	in	d =	8.06	in			
U =	1.00	-	$A_{e,\ req}$ =	8.45	in^2	$A_{net,\ req}$ =	8.45	in^2
A_{reinf} =	1.53	in^2	(both sides)					
b_{reinf} =	4	in						
t_{req} =	0.53	in	t_{use} =	0.625	in	L_{plate} =	16	in
$F_{y,\ plate}$ =	36	ksi	$R_yF_yA_g$ =	99.00	kip			
L_{weld} =	7	in	weld =	6	x 1/16 in	(fillet)		
ϕR_n =	116.92	kip	OK					

Brace Block Shear

t_{brace} =	0.465	in	
L_{req} =	8.33	in	OK
L_{use} =	14	in	

Brace to Gusset Plate Weld

L_{weld} =	14	in	
weld =	7	x 1/16 in	(fillet)
ϕ_b =	0.75	-	

F_{exx} =	70	ksi		
F_w =	42	ksi		
$\phi_b R_n$ =	545.63	kip	OK	
Gusset Plate Block Shear				
A_{gv} =	21	in^2		
A_{nt} =	6.70	in^2		
U_{bs} =	1	-		
ϕ =	0.75	-		
ϕR_n =	799.19	kip	OK	
Whitmore Effective Width				
$L_{whitmore}$ =	25.96	in	(theoretical width)	
ϕ =	0.90	-		
ϕR_n =	876.08	kip	OK	(check gross yield)
Brace Web Block Shear				
A_{gv} =	3.99	in^2		
A_{nt} =	1.35	in^2		
U_{bs} =	1	-		
ϕ =	0.75	-		
ϕR_n =	155.77	kip	OK	

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
2F	Roof Beam Design Check	Page	6

$R_yF_yA_g$ =	338.80	kip		θ =	0.73	(rad)	42.0	(deg)
0.3 P_n =	59.91	kip		$\sin(\theta)$ =	0.67			
				$\cos(\theta)$ =	0.74			
V =	186.57	kip						
H =	296.36	kip						
P_u =	600.00	kip	(conservatively)					
M_u =	92.46	kip-ft	(revised from structural analysis)					

Try	w24x117					
A_g =	34.4	in^2	b_f =	12.8	in	
I_x =	3540	in^4	t_f =	0.85	in	
I_y =	297	in^4	d =	24.3	in	
r_x =	10.1	in	t_w =	0.55	in	
r_y =	2.94	in	F_y =	50	ksi	
λ_{p1} =	9.15		b/t =	7.53	Compact	
λ_{p2} =	90.55		h/tw =	41.09	Compact	
L_p =	10.38	ft	Z_x =	327	in^3	
c =	1	-	J =	6.72	in^4	
C_w =	40800	in^6	h_o =	23.45	in	
S_x =	291	in^3	r_{ts} =	3.46	in	
L_r =	29.90	ft	Brace PT=	2	-	
L_b =	10	ft	C_b =	1.0	(Conservatively)	
M_p =	1362.5	kip-ft	F_{cr} =	248.50	ksi	
ϕ_b =	0.90	-	M_n =	1362.50	kip-ft	(Need Check)
$\phi_b M_n$ =	1226.25	kip-ft				
kl/r =	40.82	-	k =	1.0	-	
F_e =	171.79	ksi	0.44 F_y =	22	ksi	
ϕ_c =	0.90	-				
$\phi_c P_n$ =	1370.46	kip				
$P_u/\phi_c P_n$ =	0.44	use (H1-1a)				
Check	0.50	OK				

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F	Lower Beam Design Check	Page	7

$R_yF_yA_g$ =	453.20	kip	(1F)						
$R_yF_yA_g$ =	338.80	kip	(2F)	θ =	0.73	(rad)	42.0	(deg)	
0.3 P_n =	89.31	kip	(1F)	sin(θ) =	0.67				
0.3 P_n =	59.91	kip	(2F)	cos(θ) =	0.74				
V =	263.10	kip							
H =	292.33	kip							
P_u =	300.00	kip	(conservatively)						
M_u =	160.72	kip-ft	(revised from structural analysis)						
Try	w24x68								
A_g =	20.1	in^2		b_f =	8.97	in			
I_x =	1830	in^4		t_f =	0.585	in			
I_y =	70.4	in^4		d =	23.7	in			
r_x =	9.55	in		t_w =	0.415	in			
r_y =	1.87	in		F_y =	50	ksi			
λ_{p1} =	9.15			b/t =	7.67	Compact			
λ_{p2} =	90.55			h/tw =	54.29	Compact			
L_p =	6.61	ft		Z_x =	177	in^3			
c =	1	-		J =	1.87	in^4			
C_w =	9430	in^6		h_o =	23.12	in			
S_x =	154	in^3		r_{ts} =	2.30	in			
L_r =	18.74	ft		Brace PT=	2	-			
L_b =	10	ft		C_b =	1.0	(Conservatively)			
M_p =	737.5	kip-ft		F_{cr} =	110.86	ksi			
ϕ_b =	0.90	-		M_n =	656.87	kip-ft		OK	
ϕ_bM_n =	591.18	kip-ft							
kl/r =	64.17	-		k =	1.0	-			
F_e =	69.50	ksi		0.44 F_y =	22	ksi			
ϕ_c =	0.90	-							
ϕ_cP_n =	669.33	kip							
P_u/ϕ_cP_n =	0.45	use (H1-1a)							
Check	0.69	OK							

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
2F	Column Design Check	Page	8

P_u =	43.55	kip	(revised from structural analysis)			
M_u =	115.54	kip-ft	(revised from structural analysis)			
L_{column} =	9	ft				
Try	w12x96					
A_g =	28.2	in^2	b_f =	12.2	in	
I_x =	833	in^4	t_f =	0.9	in	
I_y =	270	in^4	d =	12.7	in	
r_x =	5.44	in	t_w =	0.55	in	
r_y =	3.09	in	F_y =	50	ksi	
λ_{p1} =	7.22	-	b/t =	6.78	Compact	
λ_{p2} =	71.62	-	h/tw =	19.82	Compact	
L_p =	10.91	ft	Z_x =	147	in^3	
c =	1	-	J =	6.85	in^4	
C_w =	9410	in^6	h_o =	11.80	in	
S_x =	131	in^3	r_{ts} =	3.49	in	
L_r =	40.86	ft	Brace PT=	0	-	
L_b =	9	ft	C_b =	1.0	(Conservatively)	
M_p =	612.5	kip-ft	F_{cr} =	344.49	ksi	
ϕ_b =	0.90	-	M_n =	612.50	kip-ft	(Need Check)
$\phi_b M_n$ =	551.25	kip-ft	C_a =	0.03	-	
kl/r =	34.95	-	k =	1.0	-	
F_e =	234.28	ksi	0.44 F_y =	22	ksi	
ϕ_c =	0.90	-				
$\phi_c P_n$ =	1160.56	kip				
$P_u/\phi_c P_n$ =	0.04	use (H1-1b)				
Check	0.23	OK				

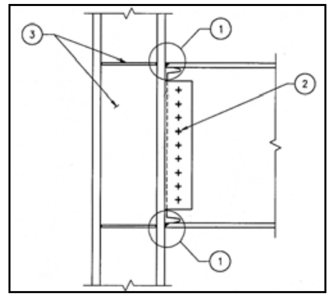
Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F	Column Design Check	Page	9

P_u =	492.31	kip	(revised from structural analysis)				
M_u =	259.53	kip-ft	(revised from structural analysis)				
L_{column} =	9	ft					
Try	w12x96						
A_g =	28.2	in^2		b_f =	12.2	in	
I_x =	833	in^4		t_f =	0.9	in	
I_y =	270	in^4		d =	12.7	in	
r_x =	5.44	in		t_w =	0.55	in	
r_y =	3.09	in		F_y =	50	ksi	
λ_{p1} =	7.22	-		b/t =	6.78	Compact	
λ_{p2} =	52.38	-		h/tw =	19.82	Compact	
L_p =	10.91	ft		Z_x =	147	in^3	
c =	1	-		J =	6.85	in^4	
C_w =	9410	in^6		h_o =	11.80	in	
S_x =	131	in^3		r_{ts} =	3.49	in	
L_r =	40.86	ft		Brace PT=	0	-	
L_b =	9	ft		C_b =	1.0	(Conservatively)	
M_p =	612.5	kip-ft		F_{cr} =	344.49	ksi	
ϕ_b =	0.90	-		M_n =	612.50	kip-ft	(Need Check)
$\phi_b M_n$ =	551.25	kip-ft		C_a =	0.39	-	
kl/r =	34.95	-		k =	1.0	-	
F_e =	234.28	ksi		0.44 F_y =	22	ksi	
ϕ_c =	0.90	-					
$\phi_c P_n$ =	1160.56	kip					
$P_u/\phi_c P_n$ =	0.42	use (H1-1a)					
Check	0.84	OK					

Check Column Web Shear Stress

$M_p =$	7350	kip-in	
$L =$	96.15	in	
$V =$	152.89	kip	
$A_s =$	6.99	in^2	$A_s = d*tw$
$S_v =$	21.89	ksi	
$S_{v,\ yield} =$	29.00	ksi	Elastic

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
2F	Beam-Column Connection	Page	10



Type	Bolted	(WUF-B)						
$H =$	148.18	kip						
$V =$	93.29	kip						
$M =$	92.46	kip-ft	(revised from structural analysis)					
$R_u =$	175.10	kip						
Try $d_b =$	0.88	in	$F_u =$	150	ksi			
$A_b =$	0.60	in^2	$F_{nv} =$	75	ksi	(threads excluded)		
$N_b =$	6	bolts	(in one row)					
$R_n =$	270.59	kip	(bolt shear)		$L_{c\_ex} =$	1.5	in	
$\phi_b =$	0.75	-			$L_{c\_in} =$	3	in	
$\phi_b R_n =$	202.94	kip	OK					
$L_{c1} =$	1.03	in	(edge clear distance)		$R_{n1} =$	46.41	kip	
$L_{c2} =$	2.06	in	(clear distance)		$R_{n2} =$	236.25	kip	
$t =$	0.50	in	(shear tab thickness)					
$R_n =$	1227.66	kip	(combined bolt bearing)					
$\phi_b =$	0.75	-						
$\phi_b R_n =$	920.74	kip	OK					
$L_{tab} =$	18	in	$A_{s,\ tab} =$	9	in^2	$R_n =$	524.79	kip
$w_{tab} =$	4.5	in	$F_{y,\ tab} =$	50	ksi	$P_{nt} =$	450.00	kip
			$F_{v,\ tab} =$	30.0	ksi	$P_{nv} =$	270.00	kip
							OK	
Weld	Fillet	(shear tab)						
$F_{exx} =$	70	ksi		$R_n =$	334.06	kip		
$Fw =$	42	ksi		$\phi_b =$	0.75	-		
$w =$	5	x 1/16 inch		$\phi_b R_n =$	250.54	kip	OK	
$L_{weld} =$	18	in						
side =	2	sides						
Weld	CJP	(top, bottom flanges)						
$b_f =$	12.8	in		$F_{y,\ bm} =$	50	ksi	(base metal)	
$t_f =$	0.85	in		$M_n =$	1063.07	kip-ft	OK	
$d =$	24.3	in						
$t_w =$	0.55	in						

Check	Shear tab length, OK		

Check Block Shear

Beam
w24x117

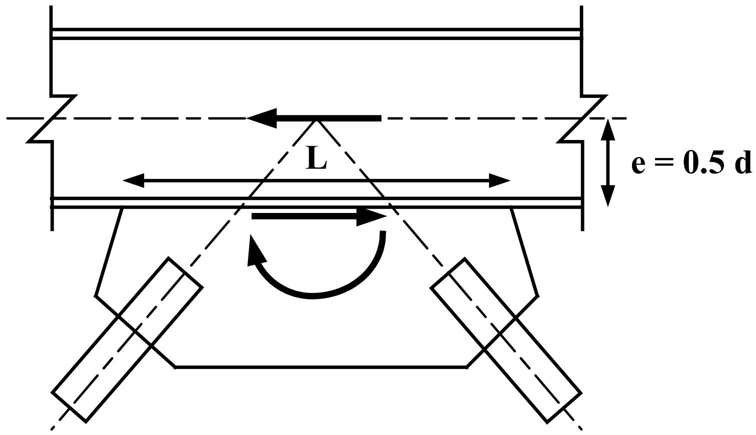
A_{gv} =	9.9	in^2	
A_{gt} =	1.925	in^2	
A_{nv} =	6.6	in^2	
A_{nt} =	1.65	in^2	
U_{bs} =	0.5	-	
ϕ =	0.75	-	
F_y =	50	ksi	
F_u =	65	ksi	
ϕR_n =	233.27	kip	OK

Shear Tab

A_{gv} =	8.25	in^2	
A_{gt} =	1.5	in^2	
A_{nv} =	5.5	in^2	
A_{nt} =	1.25	in^2	
U_{bs} =	0.5	-	
ϕ =	0.75	-	
F_y =	50	ksi	
F_u =	65	ksi	
ϕR_n =	191.34	kip	OK

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
2F	Braces to Beam Connection	Page	11

Braces		W8x21					
T =	338.80	kip	$\sin(\theta)$ =	0.669			
C =	241.62	kip	$\cos(\theta)$ =	0.743			
e =	12.15	in					
Shear =	431.42	kip					
Tension =	182.56	kip					
Moment =	436.81	kip-ft					
t_{gusset} =	0.75	in					
L =	60	in					
s_V =	9.59	ksi					
s_A =	4.06	ksi					
s_M =	11.65	ksi					
ϕ =	0.9	-					
$F_{y,\ gusset}$ =	50	ksi					
Ratio =	0.51	OK					
$L_{whitmore}$ =	23.87	in	L_{min} =	62.68	in	(geometry limit)	NG
L_v =	16	in	$L_{v,\ min}$ =	17.74	in	(geometry limit)	NG!
w_{up} =	11.93	in	A_v =	12	in^2		
w_{low} =	20.92	in	P_u =	226.65	kip		
$Whitm_{eff}$ =	23.87	in	ϕR_n=	324	kip	OK	
ϕR_n =	805.56	kip	OK				



Gusset Plate to Beam Flange

Weld	Fillet						
F_{exx} =	70	ksi					
F_w =	42	ksi			s_V =	10.17	ksi
w =	8	x	1/16	inch	s_M =	12.36	ksi
L_{weld} =	60	in			s_A =	4.30	ksi
side =	2	sides			f_{peak} =	19.52	ksi
t_{eff} =	0.354	in			f_{avg} =	16.25	ksi
ϕ =	0.75	-			f_r =	20.31	ksi
Ratio =	0.68	OK			Ratio =	0.64	OK

Check Beam Web

width =	60	in	Beam	w24x117

R_u =	174.73	kip	d =	24.3	in	t_w =	0.55	in
N =	30	in	t_f =	0.85	in	$F_{y, web}$ =	50	ksi
ϕ =	0.75	-	R_n =	1060.63	kip	ϕR_n =	795.47	kip
k_{des} =	1.35	in				(web crippling)		OK
ϕ =	1.00	-	R_n =	1010.63	kip	ϕR_n =	1010.63	kip
						(web local yielding)		OK

Check Gusset Plate Buckling

L_{gb} =	17.68	in	kL/r =	98.0	-	L_c =	14.13	in
k =	1.2	-	F_e =	29.82	ksi	L_{c1} =	11.38	in
r =	0.217	in	0.44 F_y =	22	ksi	L_{c2} =	16.94	in
A_g =	17.90	in^2	R_n =	443.70	kip	L_{max} =	24.06	in
ϕ =	0.9	-	ϕR_n =	399.33	kip	L_{tip} =	21.88	in
					OK	L_{ave} =	17.68	in

Free Edge Buckling

L_e =	15.63	in	
L_e/t_g =	20.83	-	
Limit =	18.06	-	Edge stiffener required!

Lateral Stability of Beam

M_r =	1498.75	kip-ft	Z =	327	in^3	L_b =	10	ft
C_d =	1	-	h_o =	23.45	in	L_{pd} =	17.05	ft
P_{br} =	15.34	kip	β_{br} =	85.22	kip/in	n =	1	OK
(Nodal)			(Nodal)			C_b =	1	-

M_{br} =	17.985	kip-ft	
P_{br} =	9.20	kip	(torsional)
β_T =	28842	kip-in/rad	β_{sec} not included
β_{br} =	52.45	kip/in	(torsional)
Δ =	0.18	in	

Kicker

L3x2x3/8

A_g =	1.73	in^2	
L =	25	in	
k_{axial} =	2006.8	kip/in	
k =	1419	kip/in	OK

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F	Beam-Column-Gusset Connections	Page	12

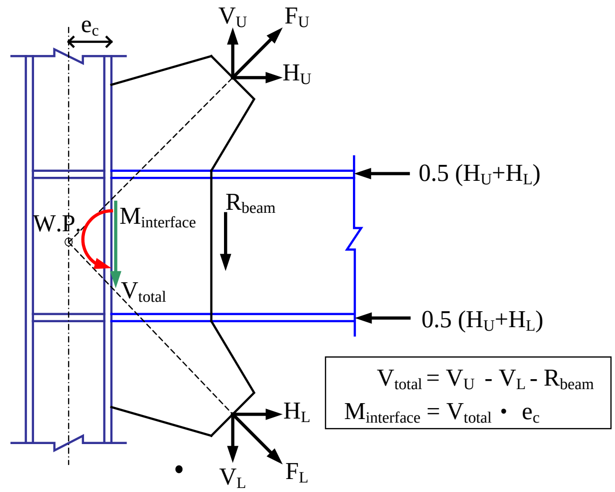
Big Gusset Plate for Upper Floor Bracing and Lower Floor Bracing

Sway to Right

F_{U2R} =	338.80	kip	$\cos(\theta_U)$ =	0.743
F_{L2R} =	360.20	kip	$\cos(\theta_L)$ =	0.743

Sway to Left

F_{U2L} =	241.62	kip				
F_{L2L} =	453.20	kip				
Beam	w24x68					
d =	23.7	in				
$L_{c,\,min}$ =	60.74	in				
Column	w12x96					
e_c =	6.35	in				
R_{beam} =	21.45	kip	(downward)			
$L_{b,\,min}$ =	17.36	in				
$L_{cu,\,min}$ =	17.74	in				
$L_{cl,\,min}$ =	19.29	in				
t_g =	0.75	in				
L_{cu} =	18	in	(use)	$F_{y,\,gusset}$ =	50	ksi
L_{cl} =	18	in	(use)			
L_c =	59.7	in	(use)			
L_b =	23	in	(use)			



Sway to the Right

V_{U2R} =	226.65	kip	(upward)	s_V =	9.96	ksi	
H_{U2R} =	251.83	kip	(rightward)	s_M =	6.36	ksi	
V_{L2R} =	240.96	kip	(upward)	s_A =	0.36	ksi	
H_{L2R} =	267.73	kip	(leftward)	Ratio =	0.41	-	OK
V_{total} =	446.16	kip	(upward)				
M =	236.09	kip-ft	(counter-clockwise)				
				Column-Side			
V_{cu} =	134.52	kip	(downward)	s_A =	4.44	ksi	
f_1 =	4.77	kip/in	(lefttward)	s_V =	9.96	ksi	
f_2 =	1.89	kip/in	(leftward)	s_M =	1.92	ksi	
H_{cu} =	59.96	kip	(leftward)	Ratio =	0.41	-	OK
H_{bu} =	191.86	kip	(leftward)	Beam-Side			
V_{bu} =	92.13	kip	(downward)	s_A =	5.34	ksi	
M_{cu} =	6.47	kip-ft	(counter-clockwise)	s_V =	11.12	ksi	

M_{bu} =	-91.90	kip-ft	(clockwise)	s_M =	-16.68	ksi	
				Ratio =	0.50	-	OK

V_{cl} =	134.52	kip	(downward)	s_A =	4.44	ksi	
f_1 =	4.77	kip/in	(rightward)	s_V =	9.96	ksi	
f_3 =	1.89	kip/in	(rightward)	s_M =	1.92	ksi	
H_{cl} =	59.96	kip	(rightward)	Ratio =	0.41	-	OK
H_{bl} =	207.77	kip	(rightward)				
V_{bl} =	106.44	kip	(downward)	s_A =	6.17	ksi	
M_{bl} =	86.32	kip-ft	(clockwise)	s_V =	12.04	ksi	
M_{cl} =	6.47	kip-ft	(counter-clockwise)	s_M =	15.66	ksi	
				Ratio =	0.67	-	OK

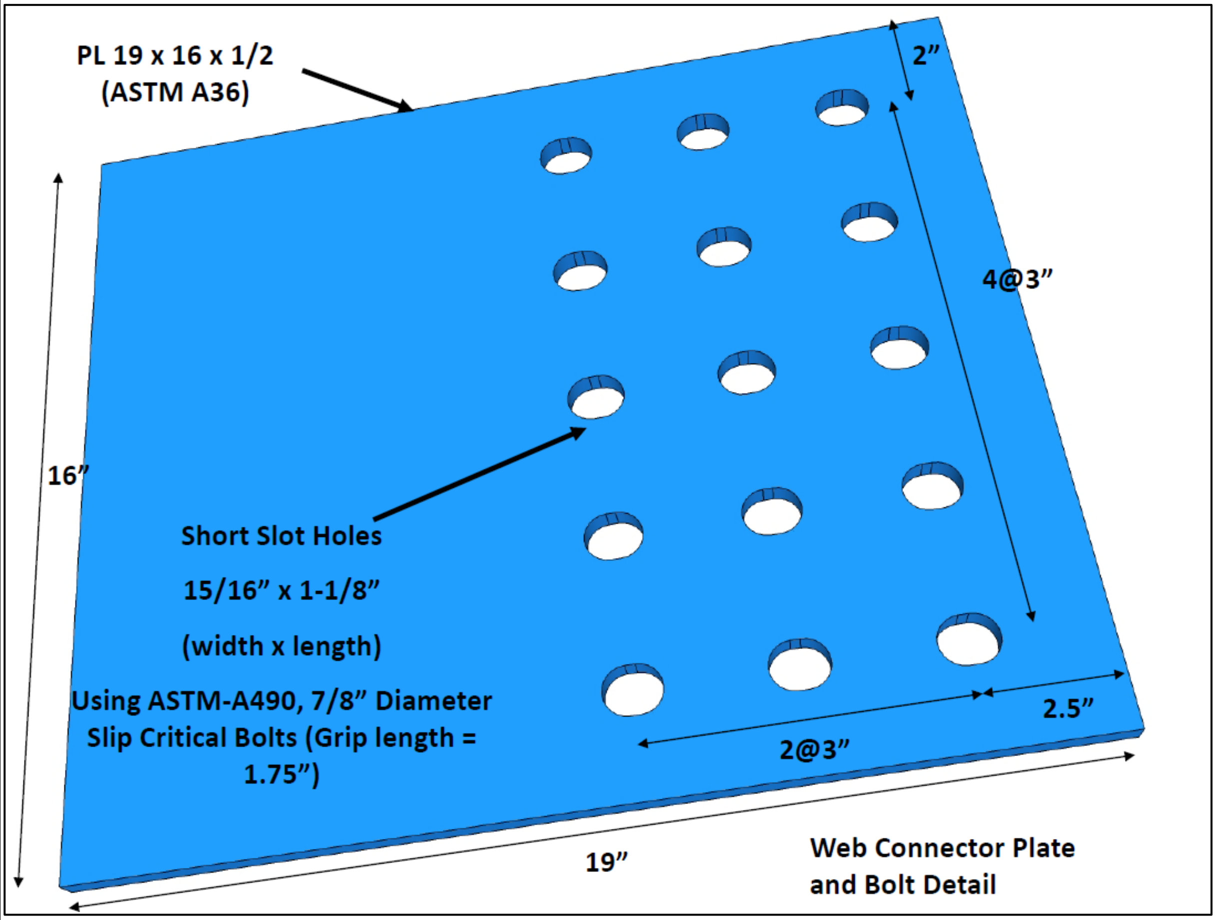
V_{mid} =	136.01	kip	(downward)	s_A =	0.00	ksi	
M_{mid} =	8.71	kip-ft	(counter-clockwise)	s_V =	7.65	ksi	
H_{mid} =	0.00	kip	(leftward)	s_M =	1.49	ksi	
				Ratio =	0.30	-	OK

Weld Size

f_v =	7.47	kip/in	
f_a =	4.35	kip/in	(averaged)
f_b =	4.77	kip/in	
f_{peak} =	11.79	kip/in	
f_{avg} =	9.64	kip/in	
f_r =	12.05	kip/in	12.04781
D >=	4.33	x 1/16	(weld size)
Use	6	x 1/16	(weld size)

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F	Lower Beam to Gusset Plate Splice	Page	13

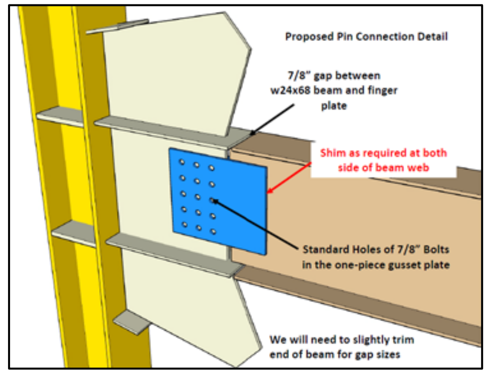
Web	Fillet welds with web plates at one end and bolted to gusset plate		
Flange	free (no weld)	(T & B)	
P_u =	300.00	kip	
R_{beam} =	21.45	kip	(Gravity)



Fillet Weld at Beam Side

L =	16	in		t_{shim} =	0.168	in	(shim as required)
w =	7	in		t_{splice} =	0.500	in	
Weld	Fillet	(Splice plates)					
F_{exx} =	70	ksi		R_n =	779.47	kip	
F_w =	42	ksi		ϕ_b =	0.75	-	
w =	7	x 1/16	inch	$\phi_b R_n$ =	584.60	kip	OK
L_{weld} =	30	in					
side =	2	sides					

Block Shear in Splice Plate



F_y =	36	ksi	
F_u =	58	ksi	
ϕ =	0.75	-	
A_{gv} =	7	in^2	
A_{nt} =	8	in^2	
side =	2	sides	
ϕR_n =	1061.40	kip	OK

Block Shear in Lower Beam Web

F_y =	50	ksi	
F_u =	65	ksi	
ϕ =	0.75	-	
A_{gv} =	5.81	in^2	
A_{nt} =	6.64	in^2	
side =	1	sides	
ϕR_n =	493.64	kip	OK

Bolt Strength Check

L_{slot} =	1.125	in	(short slot)
d_{bolt} =	0.875	in	(TC bolts)
w_{slot} =	0.9375	in	(short slot)
ϕR_n =	21.6	kip / per bolt (LRFD)	
N_{bolt} =	15	bolts	
ϕR_n =	324	kip	OK

Block Shear in Gusset Plate

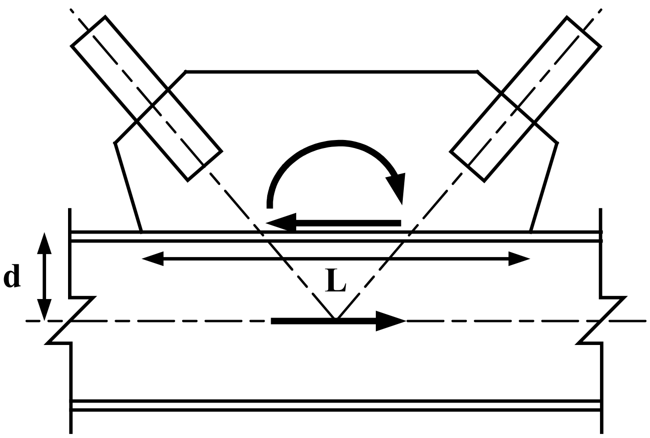
F_y =	50	ksi	
F_u =	65	ksi	
ϕ =	0.75	-	
A_{gv} =	12.38	in^2	
A_{nt} =	6.19	in^2	
A_{nv} =	8.16	in^2	
side =	1	sides	
ϕR_n =	540.21	kip	OK

Splice Plate Strength Check

F_y =	36	ksi		
F_u =	58	ksi		
ϕ =	0.75	-		
A_{gt} =	8	in^2		
A_{nt} =	5.66	in^2		
side =	2	sides		
ϕR_n =	576.00	kip	OK	(gross yielding check)
ϕR_n =	492.09	kip	OK	(net section check)

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F	Braces to Floor Beam Connection	Page	14

Braces		W8x28					
T =	453.20	kip	$\sin(\theta)$ =	0.669			
C =	360.20	kip	$\cos(\theta)$ =	0.743			
e =	0	in					
Shear =	604.60	kip					
Tension =	237.46	kip					
Moment =	0.00	kip-ft					
t_{gusset} =	0.75	in					
L =	46	in					
s_V =	17.52	ksi					
s_M =	0.00	ksi					
ϕ =	0.9	-					
$F_{y,\ gusset}$ =	50	ksi					
Ratio =	0.69	OK					
whitmo =	25.96	in	L_{min} =	38.80	in	(geometry limit)	OK
L_v =	19	in	$L_{v,\ min}$ =	19.29	in	(geometry limit)	NG!
w_{up} =	12.98	in	A_v =	14.25	in2		
w_{low} =	14.12	in	P_u =	303.17	kip		
$whit_{eff}$ =	25.96	in	ϕR_n=	384.75	kip		
ϕR_n =	876.08	kip	OK		OK		



Weld	Fillet	(Gusset to beam flange)					
F_{exx} =	70	ksi					
F_w =	42	ksi					
w =	11	x	1/16	inch	s_V =	13.52	ksi
L_{weld} =	46	in			s_M =	0.00	ksi
side =	2	sides			s_A =	5.31	ksi
t_{eff} =	0.486	in					
ϕ =	0.75	-					
Ratio =	0.76	OK					

Check Beam Web

width =	46	in	Beam	w30x391				
R_u =	0.00	kip	d =	33.2	in	t_w =	1.36	in

$N =$	23	in	$t_f =$	2.44	in	$F_{y,\ web} =$	50	ksi
$\phi =$	0.75	-	$R_n =$	4450.60	kip	$\phi R_n =$	3337.95	kip
$k_{des} =$	3.23	in				(web crippling)		OK
$\phi =$	1.00	-	$R_n =$	2662.20	kip	$\phi R_n =$	2662.20	kip
						(web local yielding)		OK

Check Gusset Plate Buckling

$L_{gb} =$	14.84	in	$kL/r =$	82.2	-	$L_c =$	15.88	in
$k =$	1.2	-	$F_e =$	42.32	ksi	$L_{c1} =$	18.06	in
$r =$	0.217	in	$0.44\ F_y =$	22	ksi	$L_{c2} =$	12.56	in
$A_g =$	19.47	in2	$R_n =$	593.65	kip	$L_{max} =$	18.56	in
$\phi =$	0.9	-	$\phi R_n =$	534.29	kip	$L_{tip} =$	9.13	in
					OK	$L_{ave} =$	14.84	in

Free Edge Buckling

$L_e =$	17.88	in	
$L_e/t_g =$	23.83	-	
Limit =	18.06	-	Edge stiffener required!

Lateral Stability of Beam

$M_r =$	6645.83	kip-ft	$Z =$	1450	in3	$L_b =$	10	ft
$C_d =$	1	-	$h_o =$	30.76	in	$L_{pd} =$	17.05	ft
$P_{br} =$	51.85	kip	$\beta_{br} =$	288.07	kip/in	$n =$	1	OK
(Nodal)			(Nodal)			$C_b =$	1	-
$M_{br} =$	79.75	kip-ft				$I_y =$	1550	in4
$P_{br} =$	31.11	kip	(torsional)					
$\beta_T =$	108666	kip-in/rad	βsec not included					
$\beta_{br} =$	114.85	kip/in	(torsional)					
$\Delta =$	0.27	in						

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F	Column Base Plate Design Check	Page	15
Column	w12x96		

Z_x =	147	in3		L =	96.15	in
F_y =	50	ksi		V_{Mp} =	152.89	kip
M_p =	7350	kip-in				
P_u =	492.31	kip		d =	12.7	in
M_u =	3114.35	kip-in		b_f =	12.2	in
N =	31.25	in		$f_{p, max}$ =	36	ksi
B =	28	in				
e =	6.33	in		q_{max} =	1008	kip/in
e_{cr} =	15.38	in	(Small Moment)			
Y =	18.60	in				
q =	26.47	kip/in	OK			
m =	9.59	in				
f_p =	0.95	ksi				
$t_{p, req}$ =	1.98	in	eq 3.3.14a (LRFD)			
use =	2.00	in				

All-thread-rods

Type	ASTM A193 B7		
d_{bolt} =	1.125	in	
F_u =	125	ksi	
F_y =	105	ksi	
F_{nt} =	93.75	ksi	
F_{nv} =	50	ksi	
A_b =	0.99	in^2	
ϕ =	0.75	-	
ϕR_n =	69.89	kip	(tension)
ϕR_n =	37.28	kip	(shear)
F_{PT} =	86.98	kip	(minimum required pretension)
V_u =	152.89	kip	(very conservative assumption)
M_u =	7350	kip-in	(very conservative assumption)
P_u =	600	kip	(very conservative assumption)
μ =	0.35	-	(class A surface)
SF =	2	-	(safety factor for not having enough bolt pretension force)

N_V =	10	bolts	(for friction shear)
N_M =	17	bolts	(for bending)
N_T =	9	bolts	(for uplifting)
$N_{req,\ total}$ =	35	bolts	
use =	34	bolts	

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
2F	Stub Beam	Page	16

$F1 =$	300	kip		$d =$	24.3	in	
$F2 =$	600	kip		$t_w =$	0.55	in	
$L_{stub}=$	19	in		$b =$	12.8	in	
Beam	w24x117			$t_f =$	0.85	in	

Column Dimension List

Column	w12x96					
$A_g =$	28.2	in^2		$b_f =$	12.2	in
$I_x =$	833	in^4		$t_f =$	0.9	in
$I_y =$	270	in^4		$d =$	12.7	in
$r_x =$	5.44	in		$t_w =$	0.55	in
$r_y =$	3.09	in		$F_y =$	50	ksi
$k_{des} =$	1.5	in		$E_s =$	29000	ksi

Column Web Local Yielding

$N =$	24.00	in	
$R_n =$	866.25	kip	
$\phi =$	1.00	-	
$\phi R_n =$	866.25	kip	OK

Column Web Crippling

$R_n =$	1382.366	kip	
$\phi =$	0.75	-	
$\phi R_n =$	1036.77	kip	OK

Column Flange Local Bending

$R_n =$	253.13	kip	$A_{web} =$	12.43	in^2	$A_s =$	34.19	in^2
$\phi =$	0.90	-	$A_{flange} =$	21.76	in^2			
$\phi R_n =$	227.81	kip	$F1_{flange}=$	190.93	kip			
			$F2_{flange}=$	381.87	kip	Continue Plate Required!		

Stub Beam Gross Yielding

$A_{s\ (beam)} =$	34.4	in^2	
$P_y =$	1720	kip	OK

Title	TCBF-B-3 Specimen Design Calculation Sheet	Date	May 28, 2009
1F	Stub Beam	Page	17

F_1 =	300	kip	d =	23.7	in
F_2 =	600	kip	t_w =	0.415	in
L_{stub}=	19	in	b =	8.97	in
Beam	w24x68		t_f =	0.585	in

Column Dimension List

Column	w12x96				
A_g =	28.2	in^2	b_f =	12.2	in
I_x =	833	in^4	t_f =	0.9	in
I_y =	270	in^4	d =	12.7	in
r_x =	5.44	in	t_w =	0.55	in
r_y =	3.09	in	F_y =	50	ksi
k_{des} =	1.5	in	E_s =	29000	ksi

Column Web Local Yielding

N =	24.00	in	
R_n =	866.25	kip	
ϕ =	1.00	-	
ϕR_n =	866.25	kip	OK

Column Web Crippling

R_n =	1382.366	kip	
ϕ =	0.75	-	
ϕR_n =	1036.77	kip	OK

Column Flange Local Bending

R_n =	253.13	kip	A_{web} =	9.35	in^2	A_s =	19.84	in^2
ϕ =	0.90	-	A_{flange} =	10.49	in^2			
ϕR_n =	227.81	kip	$F1_{flange}$=	158.65	kip			
			$F2_{flange}$=	317.31	kip		OK	

Stub Beam Gross Yielding

$A_{s\ (beam)}$ =	20.1	in^2	
P_y =	1005	kip	OK