Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
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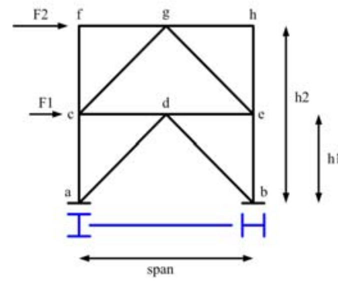
General

Building height = 2 stories

Typical floor height = 9.17 ft

$F_{1,\,max}$ = 200 kip

$F_{2,\,max}$ = 400 kip



Calculation Initialize

Items	values	units		Items	values	units
F_1 =	200	kip		V_1 =	600	kip
F_2 =	400	kip		V_2 =	400	kip
h_1 =	10.17	ft				
h_2 =	19.33	ft				
span =	20	ft	(beam span)			
h =	9.17	ft	(typical floor height, not use here)			
M_{base} =	9767	kip-ft				
P_{column} =	488	kip	(estimated max.)			
$L_{brace,1}$ =	14.26	ft	(work point to work point)			
$L_{brace,2}$ =	21.77	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 A572	50	65	1.1	1.2	
Beams	ASTM A572	50	65	1.1	1.2	
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

1.0 DL + 1.0 LL + 1.0 E per UBC-1982 Section 2303(f)

Basic Reference Codes

AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings (Nov 1, 1978

Uniform Building Code 1982

AISC Manual of Steel Construction (December,1980)

Title	Preliminary Demand-to-Capacity Ratio	Date	April 22, 2013
Author	Jiun-Wei Lai		

Brace global buckling (2F): 0.94
Brace gross section yielding (2F): 0.73

Brace global buckling (1F): 0.94
Brace gross section yielding (1F): 0.76

Brace net section failure (2F): 0.87
Brace side block shear (2F): 0.54
Brace to gusset plate weld (2F): 0.79
Gusset plate block shear (2F): 0.31
Whitmore section gross yield (2F): 0.35

Brace net section failure (1F): 0.89
Brace side block shear (1F): 0.65
Brace to gusset plate weld (1F): 0.89
Gusset plate block shear (1F): 0.45
Whitmore section gross yield (1F): 0.51

Beam (2F): 1.01 (considering max. axial force from actuator)
Beam (1F): 0.54 (considering max. axial force from actuator)

Column (2F): 0.21 (weak axis control)
Column (1F): 0.35 (weak axis control)

Center gusset plate to beam flange weld (2F): 0.64
Center gusset plate base metal shear yielding (2F): 0.48
Center gusset plate to beam, web crippling (2F): 0.37
Center gusset plate to beam, web local yielding (2F): 0.28

Center gusset plate to beam flange weld (1F): 0.63
Center gusset plate base metal shear yielding (1F): 0.71
Center gusset plate to beam, web crippling (1F): 0.60
Center gusset plate to beam, web local yielding (1F): 0.45

Corner gusset plate to beam flange weld (2F): 0.85
Corner gusset plate to column flange weld (2F): 0.95
Corner gusset plate base metal shear yielding, max (2F): 0.71
Corner gusset plate to beam, web crippling (2F): 0.44
Corner gusset plate to beam, web local yielding (2F): 0.34
Corner gusset plate to column, web crippling (2F): 0.43
Corner gusset plate to column, web local yielding (2F): 0.46

Corner gusset plate to base plate weld (1F): 0.99
Corner gusset plate to column flange weld (1F): 0.73
Corner gusset plate base metal shear yielding, max (1F): 0.99
Corner gusset plate to column, web crippling (1F): 0.49
Corner gusset plate to column, web local yielding (1F): 0.50

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
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2F-Brace			

P_u =	100.64	kip	(compression)	θ =	0.74	rad	
L_{brace} =	11.6	ft		$\sin(\theta)$ =	0.6757		
k =	1.0	-		$\cos(\theta)$ =	0.7372		
Try section	HSS6x6x3/16		(HSS-Square)				
A_s =	3.98	in^2		I_x =	22.30	in^4	
Z_x =	8.63	in^3		I_y =	22.30	in^4	
b =	6.00	in		h =	6.00	in	
t_{nom} =	0.19	in		t_{des} =	0.174	in	
r_x =	2.37	in					
r_y =	2.37	in					
F_y (brace) =	46	ksi		E_s =	29000	ksi	

Check Compression

kL/r =	58.56	-		Limit =	200	OK	(AISC limit)
C_c =	111.6	-					
F_a =	21.49	ksi		f_a =	25.29	ksi	
(1 + 1/3) F_a =	28.66	ksi	(1/3 increase in allowable stress under seismic)				
P_a =	114.05	kip	(compression)	Check		OK	

Check AISC Specification limit under compression, Sec 1.9.2.2 (1978)

Limit =	35.09	-	b/t =	31.50	OK
			h/t =	31.50	OK

Check Tension

F_t =	27.60	ksi	(gross section yield)		
(1 + 1/3) F_t =	36.80	ksi	(1/3 increase in allowable stress under seismic)		
P_a =	146.46	kip	(tension)	Check	OK

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
Author	Jiun-Wei Lai	Page	3
1F-Brace			

P_u =	151.07	kip	(compression)	θ =	0.79	rad	
L_{brace} =	12.3	ft		$\sin(\theta)$ =	0.7129		
k =	1.0	-		$\cos(\theta)$ =	0.7012		
Try section	HSS7x7x1/4		(HSS-Square)				
A_s =	6.17	in^2		I_x =	46.50	in^4	
Z_x =	15.50	in^3		I_y =	46.50	in^4	
b =	7.00	in		h =	7.00	in	
t_{nom} =	0.25	in		t_{des} =	0.233	in	
r_x =	2.75	in					
r_y =	2.75	in					
F_y (brace) =	46	ksi		E_s =	29000	ksi	

Check Compression

kL/r =	53.50	-		Limit =	200	OK	(AISC limit)
C_c =	111.6	-					
F_a =	22.21	ksi		f_a =	24.48	ksi	
(1 + 1/3) F_a =	29.62	ksi					
P_a =	182.74	kip	(compression)		Check	OK	

Check AISC Specification limit under compression, Sec 1.9.2.2 (1978)

Limit =	35.09	-	b/t =	27.00	OK
			h/t =	27.00	OK

Check Tension

F_t =	27.60	ksi	(gross section yield)		
(1 + 1/3) F_t =	36.80	ksi			
P_a =	227.06	kip	(tension)	Check	OK

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
Author	Jiun-Wei Lai	Page	4
2F	Brace to Gusset Plate Connection		
Brace	HSS6x6x3/16		

$1.25\ P_u$ =	100.6	kip	($1.25\ P_u$)			
F_uA_g =	230.8	kip	(Full Capacity, T_u)			
t_{gusset} =	0.28	in	(estimated)	F_y =	50	ksi
t_g =	0.5	in	(use)		(gusset plate)	
				F_u =	65	ksi

Check Brace Net Section

U =	0.85	-	
A_{cut} =	0.22	in^2	
A_{net} =	3.76	in^2	
A_e =	3.20	in^2	
F_t =	29.0	ksi	
$(4/3)\ F_t$ =	38.7	ksi	
P_t =	123.7	kip	OK

Brace Side Block Shear

t_{brace} =	0.174	in				
L_{use} =	12	in				
A_v =	8.35	in^2		A_t =	0	in^2
P_a =	193.8	kip	OK			

Brace to Gusset Plate Weld

L_{weld} =	12	in	
weld =	2	x 1/16 in	(fillet weld)
F_{E70XX} =	70	ksi	
F_v =	21	ksi	(allowable shear stress for weld metal)
P_a =	118.8	kip	OK

Gusset Plate Block Shear

A_v =	12	in^2	
A_t =	3.13	in^2	
P_a =	447.4	kip	OK

Whitmore Effective Width

$L_{whitmore}$ =	19.86	in	(theoretical width)	
P_a =	397.1	kip	OK	(check gross yield)

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
Author	Jiun-Wei Lai	Page	5
1F	Brace to Gusset Plate Connection		
Brace	HSS7x7x1/4		

$1.25\ P_u$ =	151.1	kip	($1.25\ P_u$)			
F_uA_g =	357.9	kip	(Full Capacity, T_u)			
t_{gusset} =	0.36	in	(estimated)	F_y =	50	ksi
t_g =	0.5	in	(use)		(gusset plate)	
				F_u =	65	ksi

Check Brace Net Section

U =	0.85	-	
A_{cut} =	0.29	in^2	
A_{net} =	5.88	in^2	
A_e =	5.00	in^2	
F_t =	29.0	ksi	
(4/3) F_t =	38.7	ksi	
P_t =	193.2	kip	OK

Brace Side Block Shear

t_{brace} =	0.233	in				
L_{use} =	12	in				
A_v =	11.18	in^2		A_t =	0	in^2
P_a =	259.5	kip	OK			

Brace to Gusset Plate Weld

L_{weld} =	12	in	
weld =	3	x 1/16 in	(fillet weld)
F_{E70XX} =	70	ksi	
F_v =	21	ksi	(allowable shear stress for weld metal)
P_a =	178.2	kip	OK

Gusset Plate Block Shear

A_v =	12	in^2	
A_t =	3.69	in^2	
P_a =	471.8	kip	OK

Whitmore Effective Width

$L_{whitmore}$ =	20.86	in	(theoretical width)	
P_a =	417.1	kip	OK	(check gross yield)

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
Author	Jiun-Wei Lai	Page	6
2F	Roof Beam Design Check		

P_u =	400.0	kip	(conservatively using the maximum actuator force)			
M_u =	13.3	kip-ft	(assuming the braces do not exist			
Try	w14x53					
A_g =	15.6	in^2		b_f =	8.06	in
I_x =	541	in^4		t_f =	0.66	in
I_y =	57.7	in^4		d =	13.9	in
r_x =	5.89	in		t_w =	0.37	in
r_y =	1.92	in		F_y =	50	ksi
λ_f =	9.19	-		b/t =	6.11	Compact
λ_w =	90.51	-		d/t_w =	37.57	Compact
L_c =	7.22	ft		Z_x =	87.1	in^3
c =	1	-		J =	1.94	in^4
C_w =	2540	in^6		h_o =	13.24	in
S_x =	77.8	in^3		r_{ts} =	2.22	in
L_r =	21.16	ft		Brace PT=	2	-
L_b =	10	ft		C_b =	1.0	(Conservatively)
L_b / r_T =	54.1	>	45.2			
	54.1	<=	101.0			
F_b =	28.55	ksi	<=	30	ksi	OK
	58.09	ksi	>	30	ksi	Note!
	38.27	ksi	>	30	ksi	Note!
F_b =	30.0	ksi				
M_p =	362.9	kip-ft				
M_b =	194.5	kip-ft				
kl/r =	31.3	-		k =	1.0	-
C_c =	107.0	-				
F_a =	27.00	ksi				
P_a =	421.2	kip				
f_a =	25.6	ksi				
f_a / F_a =	0.95					
f_b / F_b =	0.07					
C_m =	0.60		$(kl/r)_b$ =	40.7		
F_e' =	89.9	ksi				
Check	1.01	NG				

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
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1F	Lower Beam Design Check		

P_u =	200.0	kip	(conservatively using the maximum actuator force)				
M_u =	18.4	kip-ft	(assuming the braces do not exist				
Try	w14x53						
A_g =	15.6	in^2			b_f =	8.06	in
I_x =	541	in^4			t_f =	0.66	in
I_y =	57.7	in^4			d =	13.9	in
r_x =	5.89	in			t_w =	0.37	in
r_y =	1.92	in			F_y =	50	ksi
λ_f =	9.19	-			b/t =	6.11	Compact
λ_w =	90.51	-			d/t_w =	37.57	Compact
L_c =	7.22	ft			Z_x =	87.1	in^3
c =	1	-			J =	1.94	in^4
C_w =	2540	in^6			h_o =	13.24	in
S_x =	77.8	in^3			r_{ts} =	2.22	in
L_r =	21.16	ft			Brace PT=	2	-
L_b =	10	ft			C_b =	1.0	(Conservatively)
L_b / r_T =	54.1	>	45.2				
	54.1	<=	101.0				
F_b =	28.55	ksi	<=	30	ksi	OK	
	58.09	ksi	>	30	ksi	Note!	
	38.27	ksi	>	30	ksi	Note!	
F_b =	30.0	ksi					
M_p =	362.9	kip-ft					
M_b =	194.5	kip-ft					
kl/r =	31.3	-			k =	1.0	-
C_c =	107.0	-					
F_a =	27.00	ksi					
P_a =	421.2	kip					
f_a =	12.8	ksi					
f_a / F_a =	0.47						
f_b / F_b =	0.09						
C_m =	0.60		$(kl/r)_b$ =	40.7			
F_e' =	89.9	ksi					
Check	0.54	OK					

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
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2F	Column Design Check		

P_u =	85.4	kip	(revised from structural analysis)			
M_u =	9.7	kip-ft	(revised from structural analysis)			
L_{column} =	9.17	ft				
Try	w10x54					
A_g =	15.8	in^2		b_f =	10	in
I_x =	303	in^4		t_f =	0.615	in
I_y =	103	in^4		d =	10.1	in
r_x =	4.37	in		t_w =	0.37	in
r_y =	2.56	in		F_y =	50	ksi
λ_{p1} =	13.44	-		b/t =	8.13	Compact
λ_{p2} =	35.78	-		d/tw =	23.97	Compact
L_p =	9.04	ft		Z_x =	66.6	in^3
c =	1	-		J =	1.82	in^4
C_w =	2320	in^6		h_o =	9.49	in
S_x =	60	in^3		r_{ts} =	2.85	in
kl/r =	42.97	-		k =	1.0	-
C_c =	107.00					
F_a =	25.41	ksi				
P_a =	401.46	kip				
f_a / F_a =	0.21					
Check	0.21	OK				

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
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1F	Column Design Check		

P_u =	138.3	kip	(revised from structural analysis)			
M_u =	38.8	kip-ft	(revised from structural analysis)			
L_{column} =	10.17	ft				
Try	w10x54					
A_g =	15.8	in^2		b_f =	10	in
I_x =	303	in^4		t_f =	0.615	in
I_y =	103	in^4		d =	10.1	in
r_x =	4.37	in		t_w =	0.37	in
r_y =	2.56	in		F_y =	50	ksi
λ_{p1} =	13.44	-		b/t =	8.13	Compact
λ_{p2} =	35.78	-		d/tw =	23.97	Compact
L_p =	9.04	ft		Z_x =	66.6	in^3
c =	1	-		J =	1.82	in^4
C_w =	2320	in^6		h_o =	9.49	in
S_x =	60	in^3		r_{ts} =	2.85	in
kl/r =	47.66	-		k =	1.0	-
C_c =	107.00					
F_a =	24.71	ksi				
P_a =	390.45	kip				
f_a / F_a =	0.35					
Check	0.35	OK				

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
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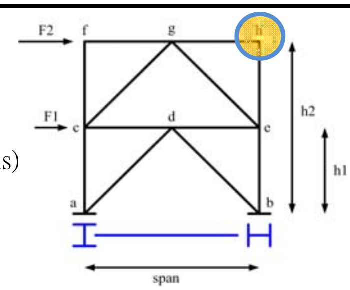
2F	Beam-Column Connection		



Type	Bolted									
H =	60.5	kip								
V =	6.6	kip								
M =	0.0	kip-ft	(revised from structural analysis)							
R_u =	60.90	kip								
Try d_b =	0.75	in		F_u =	150	ksi				
A_b =	0.44	in^2		F_{nv} =	75	ksi	(threads excluded)			
N_b =	4	bolts	(in one row)							
R_n =	132.53	kip	(bolt shear)			L_{c_ex} =	2	in		
ϕ_b =	0.75	-				L_{c_in} =	2.5	in		
$\phi_b R_n$ =	99.40	kip	OK							
L_{c1} =	1.59	in	(edge clear distance)			R_{n1} =	67.50	kip		
L_{c2} =	1.69	in	(clear distance)			R_{n2} =	135.00	kip		
t =	0.50	in	(shear tab thickness)							
R_n =	472.50	kip	(combined bolt bearing)							
ϕ_b =	0.75	-								
$\phi_b R_n$ =	354.38	kip	OK							
L_{tab} =	11.5	in		$A_{s,\ tab}$ =	5.75	in^2		R_n =	335.28	kip
w_{tab} =	4.5	in		$F_{y,\ tab}$ =	50	ksi		P_{nt} =	287.50	kip
				$F_{v,\ tab}$ =	30.0	ksi		P_{nv} =	172.50	kip
									OK	
Weld	Fillet	(shear tab)								
F_{exx} =	70	ksi			R_n =	128.06	kip			
Fw =	42	ksi			ϕ_b =	0.75	-			
w =	3	x 1/16	inch		$\phi_b R_n$ =	96.04	kip	OK		
L_{weld} =	11.5	in								
side =	2	sides								

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
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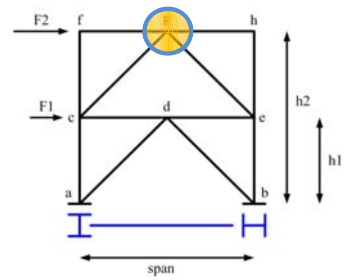
2F	Braces to Beam Connection		

Braces		HSS6x6x3/16		
T =	100.6	kip	$\sin(\theta)$ =	0.676
C =	100.6	kip	$\cos(\theta)$ =	0.737
Shear =	148.4	kip		
t_{gusset} =	0.5	in		
L =	43.5	in		
f_v =	6.8	ksi		
F_v =	20.0	ksi		
Check	OK	(base metal)		
$F_{y,\ gusset}$ =	50	ksi		



Gusset Plate to Beam Flange

Weld	Fillet			
F_{E70XX} =	70	ksi		
F_v =	21	ksi		
w =	4	x	1/16	inch
L_{weld} =	43.5	in		
side =	2	sides		
t_{eff} =	0.177	in		
P_a =	322.9	kip		
f_v / F_v =	0.5	OK		

Check Beam Web

width =	43.5	in	Beam		w14x53				
R =	68.0	kip	d =	13.9	in	t_w =	0.37	in	
N =	21.75	in	t_f =	0.66	in	$F_{y,\ web}$ =	50	ksi	
						R_a =	259.2	kip	
k_{des} =	1.25	in				(web crippling)		OK	
						R_a =	341.9	kip	
						(web local yielding)		OK	

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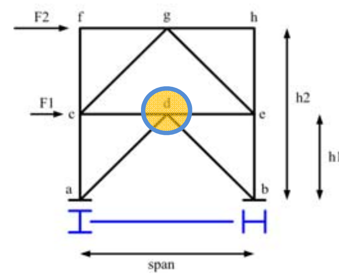
1F	Braces to Beam Connection		

Braces		HSS7x7x1/4		
T =	151.1	kip	$\sin(\theta)$ =	0.713
C =	151.1	kip	$\cos(\theta)$ =	0.701
Shear =	211.9	kip		
t_{gusset} =	0.5	in		
L =	42	in		
f_v =	10.1	ksi		
F_v =	20.0	ksi		
Check	OK	(base metal)		
$F_{y,\ gusset}$ =	50	ksi		



Gusset Plate to Beam Flange

Weld	Fillet			
F_{E70XX} =	70	ksi		
F_v =	21	ksi		
w =	6	x	1/16	inch
L_{weld} =	42	in		
side =	2	sides		
t_{eff} =	0.265	in		
P_a =	467.7	kip		
f_v / F_v =	0.5	OK		

Check Beam Web

width =	42	in	Beam		w14x53				
R =	107.7	kip	d =	13.9	in		t_w =	0.37	in
N =	21	in	t_f =	0.66	in		$F_{y,\ web}$ =	50	ksi
							R_a =	253.3	kip
k_{des} =	1.25	in					(web crippling)		OK
							R_a =	332.7	kip
							(web local yielding)		OK

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
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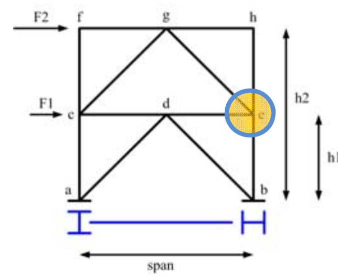
2F	Braces to Beam-Column Connection		



Braces		HSS6x6x3/16				
T =	100.6	kip	$\sin(\theta)$ =	0.676		
C =	100.6	kip	$\cos(\theta)$ =	0.737		
H =	74.2	kip				
V =	68.0	kip				
t_{gusset} =	0.5	in				
Horizontal			Vertical			
L_h =	16.5	in	L_v =	13.5	in	
f_v =	9.0	ksi	f_v =	10.1	ksi	
F_v =	20.0	ksi	F_v =	20.0	ksi	
Check	OK	(base metal)		OK		
$F_{y,\ gusset}$ =	50	ksi				

Gusset Plate to Beam Flange

Weld	Fillet					
F_{E70XX} =	70	ksi				
F_v =	21	ksi				
w =	4	x 1/16 inch				
$L_{weld,\ h}$ =	16.5	in		$L_{weld,\ v}$ =	14	in
side =	2	sides				
t_{eff} =	0.177	in				
P_a =	122.5	kip	P_a =	100.2	kip	
f_v / F_v =	0.6	OK	f_v / F_v =	0.7	OK	

Check Beam Web

width =	16.5	in	Beam		w14x53			
R =	68.0	kip	d =	13.9	in	t_w =	0.37	in
N =	16.5	in	t_f =	0.66	in	$F_{y,\ web}$ =	50	ksi
						R_a =	217.7	kip
k_{des} =	1.25	in				(web crippling)		OK
						R_a =	277.8	kip
						(web local yielding)		OK

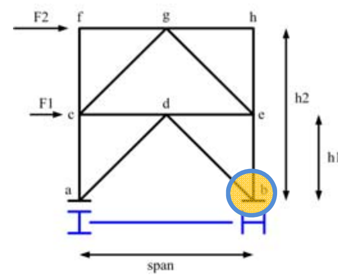
Check Column Web

width =	13.5	in	Column		w10x54			
R =	74.2	kip	d =	10.1	in	t_w =	0.37	in
N =	13.5	in	t_f =	0.615	in	$F_{y,\ web}$ =	50	ksi
						R_a =	241.9	kip
k_{des} =	1.12	in				(web crippling)		OK
						R_a =	233.2	kip
						(web local yielding)		OK

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
Author	Jiun-Wei Lai	Page	14
1F	Braces to Beam-Column Connection		

Braces HSS7x7x1/4

T =	151.1	kip	$\sin(\theta)$ =	0.713		
C =	151.1	kip	$\cos(\theta)$ =	0.701		
H =	105.9	kip				
V =	107.7	kip				
t_{gusset} =	0.5	in				
Horizontal			Vertical			
L_h =	13.5	in	L_v =	18.5	in	
f_v =	15.7	ksi	f_v =	11.6	ksi	
F_v =	20.0	ksi	F_v =	20.0	ksi	
Check	OK	(base metal)		OK		
$F_{y,\ gusset}$ =	50	ksi				



Gusset Plate to Beam Flange

Weld	Fillet					
F_{E70XX} =	70	ksi				
F_v =	21	ksi				
w =	6	x 1/16 inch				
$L_{weld,\ h}$ =	13.5	in		$L_{weld,\ v}$ =	19	in
side =	2	sides				
t_{eff} =	0.265	in				
P_a =	150.3	kip	P_a =	206.0	kip	
f_v / F_v =	0.7	OK	f_v / F_v =	0.5	OK	

Check Beam Web

width =	13.5	in	Beam		w14x53				
R =	107.7	kip	d =	13.9	in	t_w =	0.37	in	
N =	13.5	in	t_f =	0.66	in	$F_{y,\ web}$ =	50	ksi	
						R_a =	194.0	kip	
k_{des} =	1.25	in				(web crippling)		OK	
						R_a =	241.1	kip	
						(web local yielding)		OK	

Check Column Web

width =	18.5	in	Column		w10x54				
R =	105.9	kip	d =	10.1	in	t_w =	0.37	in	
N =	18.5	in	t_f =	0.615	in	$F_{y,\ web}$ =	50	ksi	
						R_a =	300.3	kip	
k_{des} =	1.12	in				(web crippling)		OK	
						R_a =	294.3	kip	
						(web local yielding)		OK	

Title	NCBF-B-1 Specimen Design Calculation Sheet	Date	April 22, 2013
Author	Jiun-Wei Lai	Page	15
1F	Column Base Plate Design Check		
Column	w10x54		

Z_x =	66.6	in3		L =	122.00	in
F_y =	50	ksi		V_{Mp} =	54.59	kip
M_p =	3330	kip-in				
P_u =	221.3	kip		d =	10.1	in
M_u =	745.0	kip-in		b_f =	10	in
N =	20	in		$f_{p,\,max}$ =	36	ksi
B =	20	in				
e =	3.37	in		q_{max} =	720	kip/in
e_{cr} =	9.85	in	(Small Moment)			
Y =	13.27	in				
q =	16.68	kip/in	OK			
m =	5.20	in				
f_p =	0.83	ksi				
$t_{p,\,req}$ =	1.01	in	eq 3.3.14a (LRFD)			
use =	1.13	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 =	54.59	kip	(very conservative assumption)
M_u =	3330	kip-in	(very conservative assumption)
P_u =	450	kip	(very conservative assumption)
μ =	0.35	-	(class A surface)
SF =	2	-	(safety factor for not having enough bolt pretension force)
N_V =	4	bolts	(for friction shear)
N_M =	5	bolts	(for bending)
N_T =	6	bolts	(for uplifting)
$N_{req,\,total}$ =	15	bolts	
use =	20	bolts	

NCBF-B-2 CFT Brace Design						Author: Barbara Simpson
Section:	HSS7X7X1/4	L =	144.4	in.	*1st story*	

PROPERTIES: *Steel:* / *Concrete:*

	As =	6.17	in^2	Ac =	42.3	in^2
	Is =	46.5	in^4	Ic = $d^4/12$ =	148.8	in^4
	b =	7	in.	B =	6.5	in.
	t =	0.25	in.			
Coupon Test:	Fy =	51.7	ksi	f'c =	2.2	ksi
	Es =	29000	ksi	Ec = 57√(f'c) =	2673.5	ksi
	b/t =	27		fr = 7.5√(f'c) =	0.352	ksi
	rx =	2.75	in^3	n = Es/Ec =	10.8	

CONCRETE-FILLED SECTION

COMPOSITE AXIAL STRENGTH

LOCAL BUCKLING / SEISMIC COMPACTNESS:

b/t = 27 < 1.4√(E/Fy) = 35.2 compact

Fy = 46 ksi

FILLED COMPOSITE MEMBERS:

As/Ac = 0.15 > 0.1 ok

t = 0.25 in. > b√(Fy/(3Es))) = 0.17 in. ok

If compact:

Pno = Pp =	398.0	kips	Pe =	798.8	kips	EI_{eff} =	1688477.4
Pp =	398.0	kips	K =	1.0		C_3 =	0.85
C_2 =	0.85					λ = √(Pe/Pp) =	0.71

Pno/Pe = 0.50 < 2.25

Pn = 323.1 kips

ϕPn = 242.3 kips ϕ = 0.75 Actual: 367 to 393 kips

STEEL TENSION STRENGTH

Tn = 319.0 kips

ϕTn = 287.1 kips ϕ = 0.9

Tc = FrAc = 14.9 kips

HOLLOW STEEL SECTION

Brace Slenderness: L = 144.4 in. = 12.0 ft

k = 1.0

kL/r = 52.5 < 200 ok λ = kl/r√(Fy/(π^2E)) = 0.71

Available Compressive Strength: Fy = 51.7 ksi

Overall Buckling: E = 29000 ksi

ϕ = 0.9

Fe = 103.8 ksi Fy/Fe < 2.25

Fcr = 42.0 ksi kl/r < 4.71√(E/Fy) inelastic buckling

Pn = Fcr*A = 259 kips

ϕPn = 233 kips

Available Tension Strength:

Gross Tensile Yielding:

ϕ = 0.9

Tn = FyAg = 319 kips

ϕTn = 287 kips

NCBF-B-2 CFT Brace Design								Author: Barbara Simpson
Section:	HSS6X6X3/16	L =	132.8	in.	2nd story			

PROPERTIES:

	Steel:				Concrete:		
	As =	3.98	in^2	Ac =	31.6	in^2
	Is =	22.3	in^4	$Ic = d^4/12 =$	83.4	in^4
	b =	6	in.	B =	5.625	in.
	t =	0.1875	in.			
Coupon Test:	Fy =	46.8	ksi	f'c =	2.2	ksi
	Es =	29000	ksi	Ec = 57√(f'c) =	2673.5	ksi
	b/t =	31.5		fr = 7.5√(f'c) =	0.352	ksi
	rx =	2.37	in^3	n = Es/Ec =	10.8	

CONCRETE-FILLED SECTION

COMPOSITE AXIAL STRENGTH

LOCAL BUCKLING / SEISMIC COMPACTNESS:

b/t = 31.5 < 1.4√(E/Fy) = 35.2 compact

Fy = 46 ksi

FILLED COMPOSITE MEMBERS:

As/Ac = 0.13 > 0.1 ok

t = 0.19 in. > b√(Fy/(3Es))) = 0.14 in. ok

If compact:

Pno = Pp =	245.4	kips	Pe =	465.1	kips	EI_{eff} =	830371.07
Pp =	245.4	kips	K =	1.0		C_3 =	0.82
C_2 =	0.85					λ = √(Pe/Pp) =	0.73

Pno/Pe = 0.53 < 2.25

Pn = 196.8 kips

ϕPn = 147.6 kips ϕ = 0.75

STEEL TENSION STRENGTH

Tn = 186.3 kips

ϕTn = 167.6 kips ϕ = 0.9

Tc = FrAc = 11.1 kips

HOLLOW STEEL SECTION

Brace Slenderness: L = 132.8 in. = 11.1 ft

k = 1.0

kL/r = 56.0 < 200 ok λ = kl/r√(Fy/(π²E)) = 0.76

Available Compressive Strength: Fy = 52.5 ksi

Overall Buckling: E = 29000 ksi

ϕ = 0.9

Fe = 91.2 ksi Fy/Fe < 2.25

Fcr = 41.3 ksi kl/r < 4.71√(E/Fy) inelastic buckling

Pn = Fcr*A = 164 kips

ϕPn = 148 kips Actual: 161 to 187 kips

Available Tension Strength:

Gross Tensile Yielding:

ϕ = 0.9

Tn = FyAg = 209 kips

ϕTn = 188 kips

NCBF-B-3SB Sample Design Calculations	Author: Barbara Simpson

Geometry:

B =	120.0	in.		
H1 =	122.0	in.		
H2 =	110.0	in.		
	Length:		**Definitions:**	
EB1 =	171.1	in.	1st story Elastic Brace	
EB2 =	162.8	in.	2nd story Elastic Brace	
BRB1 =	171.1	in.	1st story BRB	
BRB2 =	0.0	in.	2nd story BRB	

Lambda Configuration - Prototype Building (Oakland Site)

Location: 37.837°N, 122.263°W

Max Load to BRB from Collectors, Foundation, etc. (Actuators)

F_{max} =	400	kips
No. of Frames:	4	

Estimate of Base Shear, Vb, from ASCE7-10 and ASCE41-13

Vb(LS) =	5985	kips	(BSE-1E)	Design
Vb(CP) =	8978	kips	(BSE-2E)	Max
Vb(7) =	4879	kips	(ASCE7-10)	Yield
	5985.35699			

Estimated BRB Requirements:

m(LS) =	5.6	P3(LS) =	305	kips (LS)
m(CP) =	7.5	P3(CP) =	342	kips (CP)
R =	7	P3(7) =	199	kips (ASCE7)
k =	1			

BRB Capacity:

Py,LB =	199.5	kips ≈ P3(7)	ok
Tmax =	327.2	kips > P3(LS)	ok
Cmax =	357.1	kips > P3(LS)	ok

Approximate Axial Forces (Assuming simple connections):

Mode 1: 1.0F, 0.5F

F =	**400**	kips	Roof Force
0.5F =	**200**	kips	1st Floor Force
P2 =	542.6	kips	EB2
P1 =	170.7	kips	EB1
P3 =	685.0	kips	BRB1
Pc,y-y =	366.7	kips	y-y Column

Mode 2: 0.75F, 0.75F

0.75F =	**300**	kips	Roof Force
0.75F =	**300**	kips	1st Floor Force
P2 =	407.0	kips	EB2
P1 =	234.9	kips	EB1
P3 =	620.7	kips	BRB1
Pc,y-y =	275.0	kips	y-y Column

Mode 3: 1.25F, 0.25F

1.25F =	**250**	kips	Roof Force
0.25F =	**100**	kips	1st Floor Force
P2 =	339.1	kips	EB2
P1 =	88.8	kips	EB1
P3 =	410.3	kips	BRB1
Pc,y-y =	229.2	kips	y-y Column

Max Axial Force from Modes 1 thru 3: 1.1

P2 =	542.6	kips	(Design Forces)
P1 =	234.9	kips	
P3 =	685.0	kips	
Pc =	366.7	kips	
Pb =	400	kips	

Hand Calculation: Collapse Analysis Cap,BRB = 550

Pinned Beam-Col Cnxn

F = λ100 =	372	kips	λ >	3.72		
0.5F =	186	kips	F/400 =	0.93		
Vb =	558.4	kips <	660	kips		ok

Fixed Beam-Col Cnxn

F = λ100 =	423	kips	λ <	4.23		
0.5F =	212	kips	F/400 =	0.62		
Vb =	635.0	kips <	660	kips		ok

Predicted Base Shear: 558.4 kips < Vb < 635.0 kips

Max Forces from Collapse Analysis:

BRB1	1	5	(1st hinge)
Pmax	550	550	kips
Tmax	500	500	kips
EB1	1	5	
Pmax	134	175	kips
Tmax	148	188	kips
EB2	1	5	
Pmax	439	498	kips
Tmax	399	459	kips

Beam1	1	5	(2nd hinge at 1st floor beam - middle)
Pmax	167	192	kips
Tmax	152	177	kips
Mmax	726	4791	kip-in
Mmin	-726	-4791	kip-in
M	726	4791	kip-in
Vmax		40	kips

Col-xx	1	5	(3rd hinge at base of strong axis column)
Pmax	7	47	kips
Tmax	7	47	kips
Mmax	521	3663	kip-in
Vmax		36	kips

Col-yy	1	5	(4th hinge at base of weak axis column)
Pmax	267	281	kips
Tmax	294	308	kips
Mmax	300	1722	kip-in
Vmax		19	kips

Beam2	1	5	
Pmax	299	355	kips
Tmax	328	384	kips
Mmax	300	1722	kip-in
Mmin	-273	-1722	kip-in
M	300	1722	kip-in
Vmax		7	kips

NCBF-B-3SB - Inelastic Brace - 1st Story									**Author: Barbara Simpson**

Design of BRB1

Pu =	550.0	kips	H1 =	122	in.	Ly =	75.6	in.
Tu =	550.0	kips	B =	120	in.			
			L =	171.1	in.			

Material

Ry =	1.1		Fy,min =	39.9	ksi	ω =	1.64
var. =	4	ksi	Fy,max =	46	ksi	β =	1.09
						βω =	1.79

Preliminary Selection of Brace Area

Asc < Pu/Cmax =	6.07	in^2
Cmax = βωRyPysc =	90.6	*Asc
Choose: Asc =	5.00	in^2
Py,LB =	199.5	kips

Siffness Factor:

KF =	1.71

Brace Capacity:

			1st Story Drift:				
Tmax = ωPy,LB =	327.2	kips					
Cmax = βωPy,LB =	357.1	kips	Δy = δy/cosθ =	0.17	in. =	0.14	%
Py,E = FyAsc =	230.0	kips	LS = 10Δy =	1.71	in. =	1.40	%
δy = Py,E*Ly/(AE) =	0.12	in. =	CP = 13.3Δy =	2.27	in. =	1.86	%
Tdes = ωPy,E =	377.2	kips	*Roof Drift:* R =	1.9			
Cdes = βωPy,E =	411.7	kips	Δy = Rδy(1) =	0.33	in. =	0.14	%
			LS = 10Δy =	3.25	in. =	1.40	%
			CP = 13.3Δy =	4.33	in. =	1.86	%

Inelastic Interstory Drift

Inelastic Drit:

δy' = Cd*δy/I =	0.60	in.
Cd =	5.0	
I =	1.0	

Design Drift:

1% Roof Drift =	2.32	in.			
δy' =	1.63	in.	δy'(max) =	1.63	in.

Yield Length:

Ly =	75.6	in.

Inelastic Deformation

Strain @ 2δy'					
ε = 2δy'/Ly =	4.3	<	2.5	%	ng
Strain @ CP					
ε = 13.3δy/Ly =	2.1	<	2.5	%	ok

NCBF-B-3SB - Elastic Brace - 1st Story								**Author: Barbara Simpson**
Design for elastic brace (EB1): Square								
Shape:	HSS6X6X1/2		A =	9.74		I =	48.3	
W =	35.24	plf	J =	81.1		b/t =	9.9	
L =	171.1	in.	0.9L =	154.0	in.	λ:	hd	
Maximum Load from all modal combinations:								
Pu =	234.9	kips	Ω =	1.1				
ΩPu =	258.4	kips	* Assuming Mode 2: 0.75F, 0.75F					
Tu =	234.9	kips						
ΩTu =	258.4	kips						
Mu =	0.0	kip-in.						
ΩMu =	0.0	kip-in.						
Material								
Fy =	46	ksi	l =	154.0	in.	r =	2.23	in^3
Ry =	1.4	ksi	k =	1.0		kl/r =	69.1	
DCR: Assuming 0.9L								
Tn =	448.0	kips	0.58	ok	*Interaction:*			
Pn =	325.1	kips	0.80	ok		P-M:	0.80	ok
Mn =	910.8	kip-in.	0.00	ok		T-M:	0.58	ok

Expected Capacity:

TE =	627.3	kips
PE =	400.3	kips

NCBF-B-3SB - Elastic Brace - 2nd Story								**Author: Barbara Simpson**
Design for elastic brace (EB2): Square								
Shape:	HSS8X8X5/8		A =	16.4		I =	146.0	
W =	59.32	plf	J =	244.0		b/t =	10.8	
L =	162.8	in.	0.9L =	146.5	in.	λ:	hd	
Maximum Load from Plastic Analysis:								
Pu =	497.7	kips	Ω =	1.1				
ΩPu =	547.5	kips						
Tu =	458.7	kips						
ΩTu =	504.6	kips						
Mu =	0.0	kip-in.						
ΩMu =	0.0	kip-in.						
Material								
Fy =	46	ksi	l =	146.5	in.	r =	2.99	in^3
Ry =	1.4	ksi	k =	1.0		kl/r =	49.0	
DCR: Assuming 0.9L								
Tn =	754.4	kips	0.67	ok	*Interaction:*			
Pn =	641.9	kips	0.85	ok		P-M:	0.85	ok
Mn =	2056.2	kip-in.	0.00	ok		T-M:	0.67	ok

Expected Capacity:

TE =	1056.2	kips
PE =	842.4	kips

NCBF-B-3SB - Column									**Author: Barbara Simpson**
Design for Column (strong axis bending)									
Shape:	W10X54		J =	1.82			A =	15.8	in^2
W =	54	plf	bf =	10	in.		d =	10.1	in.
L =	122.0	in.	tf =	0.62	in.		tw =	0.37	in.

Maximum Modal Load:

Ω =	1.1		b/2tf =	8.15	< λhd = 7.2	ng
Pu =	47.1	kips			< λmd = 9.2	ok
ΩPu =	51.8	kips	h/tw =	21.2	< λhd = 54.4	ok
Tu =	47.1	kips			< λmd = 69.8	ok
ΩTu =	51.8	kips		Ca =	0.08	
Vu,x =	19.4	kips				
ΩVu,x =	21.4	kips				
Mu,x =	521.0	kip-in.				
ΩMu,x =	573.1	kip-in.				

Material

Fy =	50	ksi
Ry =	1.1	ksi

DCR:

					Interaction:		
Tn =	790.0	kips	0.07	ok			
Pn =	690.6	kips	0.08	ok	P-M,x:	0.21	ok
Vn,x =	112.1	kips	0.19	ok	T-M,x:	0.24	ok
Mn,x =	3330.0	kip-in.	0.17	ok			

Expected Capacity:

MpE,x =	3663.0	kip-in.
PE =	749.5	kips

NCBF-B-3SB - Elastic Column								**Author: Barbara Simpson**
Design for Elastic Column (weak axis bending)								
Shape:	W10X54		J =	1.82		A =	15.8	in^2
W =	54	plf	bf =	10	in.	d =	10.1	in.
L =	122.0	in.	tf =	0.615	in.	tw =	0.37	in.

Maximum Modal Load:

Ω =	1.1		b/2tf =	8.15	< λhd =	7.2	ng
Pu =	281.1	kips			< λmd =	9.2	ok
ΩPu =	309.2	kips	h/tw =	21.2	< λhd =	35.9	ok
Tu =	307.7	kips			< λmd =	35.9	ok
ΩTu =	338.4	kips			Ca =	0.50	
Vu,y =	19.4	kips					
ΩVu,y =	21.4	kips					
Mu,y =	300.5	kip-in.					
ΩMu,y =	330.5	kip-in.					

Material

Fy =	50	ksi
Ry =	1.1	ksi

DCR:

Tn =	790.0	kips	0.43	ok	*Interaction:*		
Pn =	690.6	kips	0.45	ok	P-M,y:	0.64	ok
Vn,y =	369.0	kips	0.06	ok	T-M,y:	0.64	ok
Mn,y =	1565.0	kip-in.	0.21	ok			

Expected Capacity:

MpE,x =	3663.0	kip-in.
MpE,y =	1721.5	kip-in.
PE =	749.5	kips

NCBF-B-3SB - Elastic Beam									**Author: Barbara Simpson**
Design for Elastic Beam (EB) - 1st floor									
Shape:	W14X53		A =	15.6	in^2	d =	13.9	in.	
W =	53	plf	tf =	0.66	in.	tw =	0.37	in.	
L =	240.0	in.	kdet =	1 1/2	in.	bf =	8.06	in^2	
			k1 =	1	in.	h0 =	13.2	in.	
			Ix =	541	in^4	Zx =	87.1	in^3	
						J =	1.94		

Maximum Modal Load:							
Ω =	1.1		b/2tf =	6.11	< λhd =	7.2	ok
Pu =	191.6	kips			< λmd =	9.2	ok
ΩPu =	210.8	kips	h/tw =	30.9	< λhd =	35.9	ok
Tu =	176.9	kips			< λmd =	35.9	ok
ΩTu =	194.6	kips		Ca =	0.34		
Mu,x =	726.1	kip-in.					
Vu,x =	39.9	kips					

Material		
Fy =	50	ksi
Ry =	1.1	ksi

EBF Link Design: Pr/Pc = 0.00 * conservative

e = 0.7L/2 =	84.0	in.	(Approximate length of link)			
Shear Yielding:						
	Alw =	4.7	in^2			
	Vp =	139.6	kips			
Flexural Yielding:	Flexural Yielding Controls					
	Vp = 1.5Mp/e =	77.8	kips	Vn =	77.8	kips
	Mp =	4355.0	kip-in.	1.25RyVn =	106.93	kips
				Ωv = 1.25RyVn/Vu =	2.7	
				ΩvMu,x =	1944.7	kip-in.

DCR:							
Tn =	780.0	kips	0.25	ok	*Interaction:*		
Pn =	618.9	kips	0.34	ok	P-M,x:	0.74	ok
Vn,x =	154.29	kips	0.69	ok	T-M,x:	0.70	ok
Mn,x =	4355	kip-in.	0.45	ok			

Expected Capacity:

Mp,E =	4790.5	kip-in.
PE =	659.6	kips

Concentrated Forces:										
N =	48									
Web Local Yielding:			d =	13.9	in.	Web Crippling:			d/2 =	6.95
			k =	1.25	in.	N/d =	3.45			
Rn =	1003.6	kips	*> d*			Rn =	942.1	kips	*Applied > d/2 from end*	
Rn =	945.8	kips	*< d*			Rn =	591.3	kips	*Applied < d/2 from end*	
ϕ =	1.0					ϕ =	0.75			

NCBF-B-3SB - Beam	**Author: Barbara Simpson**

Design for B - 2nd Floor

Shape:	W14X53		A =	15.6	in^2	d =	13.9	in.
W =	53	plf	tf =	0.66	in.	tw =	0.37	in.
L =	240.0	in.	kdet =	1 1/2	in.	bf =	8.06	in^2
			k1 =	1	in.	h0 =	13.2	in.
			Ix =	541	in^4	Zx =	87.1	in^3
						J =	1.94	

Maximum Modal Load:

Ω =	1.1		b/2tf =	6.11	< λhd =	7.2	ok
Pu =	355.0	kips			< λmd =	9.2	ok
			h/tw =	30.9	< λhd =	35.9	ok
Tu =	384.0	kips			< λmd =	35.9	ok
					Ca =	0.57	
Mu,x =	1722.0	kip-in.					
Vu,x =	7.2	kips					

Material

Fy =	50	ksi
Ry =	1.1	ksi

DCR:

					Interaction:		
Tn =	780.0	kips	0.49	ok			
Pn =	618.9	kips	0.57	ok	P-M,x:	0.93	ok
Vn,x =	154.29	kips	0.05	ok	T-M,x:	0.89	ok
Mn,x =	4355	kip-in.	0.40	ok			

Expected Capacity:

Mp,E =	4790.5	kip-in.
PE =	659.6	kips

Concentrated Forces:

N = 24

Web Local Yielding:			d =	13.9	in.	Web Crippling:		d/2 =	6.95
			k =	1.25	in.	N/d =	1.73		
Rn =	559.6	kips	*> d*			Rn =	559.1	kips	*Applied > d/2 from end*
Rn =	501.8	kips	*< d*			Rn =	336.0	kips	*Applied < d/2 from end*
ϕ =	1.0					ϕ =	0.75		