Appendix A – Structural Depth: Gravity System Calculations

King Post Truss Members

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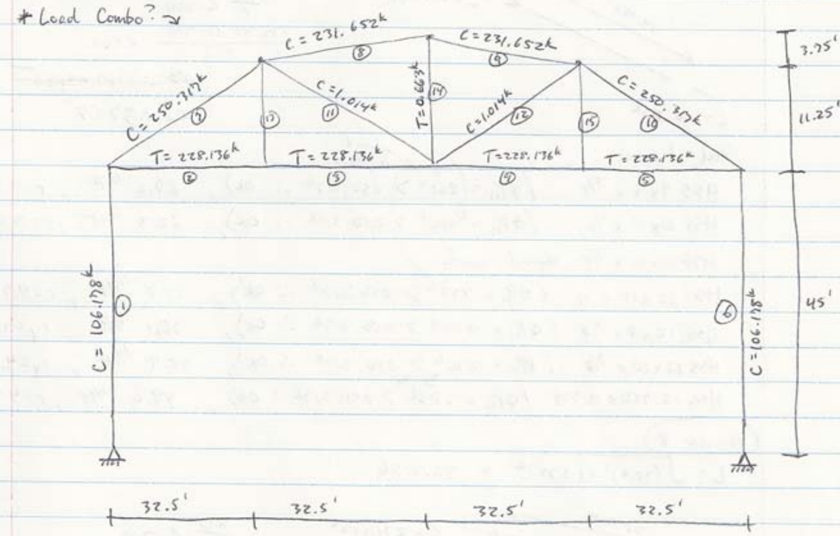
Thesis

Truss 3f) → Steel

* Load Combo? ↘



* Try to avoid slenderness ratio of $\frac{L}{r} > 300$

① $P_u = 228.136^k$

$A_{g,min} = \frac{P_u}{\phi_t F_y} = \frac{228.136^k}{(0.9)(46\ ksi)} = 5.5105\ in^2$

HSS 4x4x ½ ($A_g = 6.02\ in^2$, 21.50 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{1.41} = 276.60 < 300$ ∴ OK

HSS 5x5x ⅜ ($A_g = 6.18\ in^2$, 22.30 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{1.87} = 208.56 < 300$ ∴ OK

HSS 5½x5½x 5/16 ($A_g = 5.85\ in^2$, 21.16 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{2.11} = 184.83 < 300$ ∴ OK

$A_{e,min} = \frac{P_u}{\phi_t F_u} = \frac{228.136^k}{(0.75)(58\ ksi)} = 5.2445\ in^2$

Assume $A_e = 0.75 A_g$ → $A_{g,min} = \frac{5.2445\ in^2}{0.75} = 6.993\ in^2$

HSS 5x5x ½ ($A_g = 7.88\ in^2$, 28.30 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{1.82} = 214.29 < 300$ ∴ OK

HSS 6x6x ⅜ ($A_g = 7.58\ in^2$, 27.41 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{2.28} = 171.05 < 300$ ∴ OK

HSS 7x7x 5/16 ($A_g = 7.59\ in^2$, 27.54 lb/ft) $\frac{L}{r} = \frac{(32.5')(12)}{2.72} = 143.38 < 300$ ∴ OK

HSS 8x8x ¼ ($A_g = 7.10\ in^2$, 25.79 lb/ft) $\frac{L}{r} = \frac{(32.5')(12\ in/ft)}{3.15\ in} = 123.81 < 300$ ∴ OK

Steel Manual:

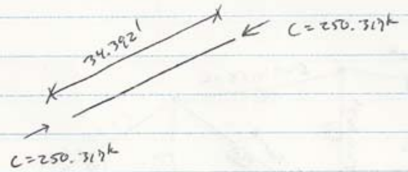
2L5x3x½ ( ) 2L6x3½x½ ( ) 2L7x4x⅜ ( )

2L5x3½x½ ( ) 2L6x4x⅜ ( ) 2L8x4x 7/16 ( )

2L5x5x⅜ ( ) 2L6x6x 5/16 ( )

Member 7

$L = \sqrt{(32.5')^2 + (11.25')^2} = 34.39204123'$



$\frac{KL}{r} < 300$

$\frac{(34.392')(12\ in/ft)}{r} < 300$

~~0.7269123642"~~

$r > 1.3757"$

Steel Manual: (36' unbraced length)

HSS 9x9 x 5/8 (ϕP_n = 261^k > 250.317^k ∴ OK), 67.6 lb/ft, r=3.40"
HSS 10x10 x 1/2 (ϕP_n = 306^k > 250.317^k ∴ OK), 62.3 lb/ft, r=3.86"
HSS 10x10 x 3/8 should work
HSS 12x12 x 1/4 (ϕP_n = 255^k > 250.317^k ∴ OK), 39.4 lb/ft, r=4.79"
HSS 12x8 x 5/8 (ϕP_n = 254^k > 250.317^k ∴ OK), 76.1 lb/ft, r_y=3.16"
HSS 12x10 x 3/8 (ϕP_n = 276^k > 250.317^k ∴ OK), 52.9 lb/ft, r_y=4.01"
HSS 12.750 x 0.375 (ϕP_n = 283^k > 250.317^k ∴ OK), 49.6 lb/ft, r=4.39"

Member 8

$L = \sqrt{(32.5)^2 + (3.75')^2} = 32.7156'$

$\frac{KL}{r} < 300$

$\frac{(32.7156')(12\ in/ft)}{r} < 300$

$r > 1.3086"$

Steel Manual: (all: 34' unbraced length)

HSS 9x9x 1/2 (ϕP_n = 248^k > 231.652^k ∴ OK), 55.5 lb/ft, r= 3.45"
HSS 10x10x 3/8 (ϕP_n = 262^k > 231.652^k ∴ OK), 47.8 lb/ft, r= 3.92"
HSS 12x12x 1/4 (ϕP_n = 267^k > 231.652^k ∴ OK), 39.4 lb/ft, r= 4.79"
HSS 10x8x 5/8 (ϕP_n = 242^k > 231.652^k ∴ OK), 67.6 lb/ft, r= 3.09"
HSS 12x8x 1/2 (ϕP_n = 241^k > 231.652^k ∴ OK), 62.3 lb/ft, r= 3.21"
HSS 12x10x 5/16 (ϕP_n = 255^k > 231.652^k ∴ OK), 44.6 lb/ft, r= 4.04"
HSS 10.000 x ~~0.500~~ 0.625 (ϕP_n = ~~238~~ 259^k > 231.652^k ∴ OK), ~~50.8~~ 62.6 lb/ft, r= ~~3.38~~ 3.34"
HSS 10.750 x 0.500 (ϕP_n = ~~237~~ 263^k > 231.652^k ∴ OK), 54.8 lb/ft, r= 3.64"
HSS 12.750 x 0.375 (ϕP_n = ~~301~~ 302^k > 231.652^k) ∴ OK, 49.6 lb/ft, r= 4.39"

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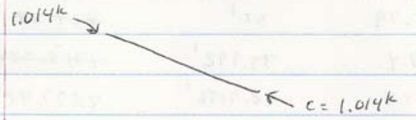
Member 11

$L = \sqrt{(32.5')^2 + (11.25')^2} = 34.39204'$



$\frac{KL}{r} < 300$

$\frac{(34.3921')(12)}{r} <$ ~~300~~ 200

$r >$ ~~1.3757"~~ $2.0635"$

Steel Manual:

HSS 5½ x 5½ x ⅛ should work, 9 lb/ft, r = 2.19"
HSS 6 x 6 x ⅛, 9.85 lb/ft, r = 2.39"
HSS 7x7x ⅛, 11.6 lb/ft, r = 2.80"
HSS 7x5 x ⅛, should work, 9.85 lb/ft, r = 2.07"
HSS 8x6 x 3/16, 17.1 lb/ft, r = 2.46"
HSS 6.000 x 0.125, 7.85 lb/ft, r = 2.08"
HSS 6.625x0.125, 8.69 lb/ft, r = 2.30"
Pipe 6 Std, 19 lb/ft, r = 2.25"

Member 13

L = 11.25'

$\frac{KL}{r} <$ ~~300~~ 200

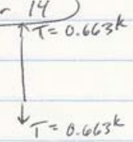
$\frac{(11.25')(12)}{r} <$ ~~300~~ 200

$r > 0.675"$

* → * Is it ok (does it seem right) that hardly any forces in the diagonal web members?

Pipe 4 Std (t = 0.221"), 10.8 lb/ft, r = 1.51"
HSS 4.000 x 0.125, 5.18 lb/ft, r = 1.37"
HSS 2x2 x ⅛, 3.04 lb/ft, r = 0.761"
HSS 4x2 x ⅛, 4.75 lb/ft, r = 0.830"

Member 14



$\frac{KL}{r} < 200$

$\frac{(15')(12)}{r} < 200$

$r > 0.90"$

HSS 2x2x ⅛ (3.04 lb/ft)
HSS 1.660 x 0.140 (2.27 lb/ft)

Using the lightest HSS members: (mostly lightest)

Member(s) #	Shape	lb/ft	Length	Weight (lb)
2, 3, 4, 5	HSS 8x8x1/4	25.79	32'	~~825.28~~ 3301.12
7, 10	HSS 12x12x1/4	39.4	34.392'	~~1355.0448~~ 2710.0896
8, 9	HSS 12x12x1/4	39.4	32.7156'	2577.98928
11, 12	HSS 5 1/2x5 1/2x1/8	9.0	34.392'	619.056
13, 15	HSS 2x2x1/8	3.04	11.25'	68.4
14	HSS 2x2x1/8	3.04	15'	45.6
				9322.25488 lb

(5 trusses)(9322.25488 lb) = **46,611.2744 lb**

* Not including bracing / diaphragm members and columns

* Do these trusses count as "king-post" trusses since they are arched?

Glulam Truss Members

Loads:

Dead Load:

Zinc Standing Seam Metal Roof Panels:	1.5 PSF
½" Moisture Resistant Gypsum Board:	2.5 PSF
4 ½" Rigid Insulation = (1.5 psf/in.)(4.5 in.):	6.75 PSF
Southern Pine 3 in. Decking:	7.6 PSF
TOTAL:	18.35 PSF
	Say = 20 PSF

D_{Total} = 20 PSF + 5 PSF (superimposed) + 5 PSF (self weight of trusses) = 30 PSF

*Applied to top chord of wood trusses (bottom of trusses is open to below; assuming superimposed loads are attached to top chord)

L_r = 20 PSF

S = 23.1 PSF

*C_s = 1.0 for roof slopes less than 30 degrees

Load Combinations (ASD):

D = 30 PSF

D + L = 20 + 0 = 20 PSF

D + (L_r or S or R) = D + S = 30 + 23.1 = **53.1 PSF**

D + 0.75L + 0.75(L_r or S or R) = D + 0.75L_r = 30 PSF + (0.75)(20 PSF) = 45 PSF

D +/- (W or 0.7E) = D = 30 PSF

D + 0.75(W or 0.7E) + 0.75L + 0.75(L_r or S or R) = D + 0.75L_r

= 30 PSF + (0.75)(20 PSF) = 45 PSF

0.6D +/- (W or 0.7E) = 0.6D = (0.6)(30 PSF) = 18 PSF

53.1 PSF controls for maximum load, but the load combination of D + S may not necessarily control. It is important to look at other load combinations as well because the duration factor (C_D) changes for other load combinations.

Load Combination: D + S

Members 13 and 22:

Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

w_{TL} = 30 PSF + (23.1 PSF)(13'/15.0833') = 49.9094 PSF

w_{TL} = (49.9094 PSF)(8') = 399.2751381 lb/ft = 0.3992751381 k/ft

Members 14 and 21:

Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

w_{TL} = 30 PSF + (23.1 PSF)(13'/14.1458') = 51.22886598 PSF

w_{TL} = (51.22886598 PSF)(8') = 409.8309278 lb/ft = 0.4098309278 k/ft

Members 15 and 20:

Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

w_{TL} = 30 PSF + (23.1 PSF)(13'/13.546875') = 52.16747405 PSF

w_{TL} = (52.16747405 PSF)(8') = 417.3397924 lb/ft = 0.4173397924 k/ft

Members 16 and 19:

Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

w_{TL} = 30 PSF + (23.1 PSF)(13'/13.1875') = 52.77156398 PSF

w_{TL} = (52.77156398 PSF)(8') = 422.1725118 lb/ft = 0.4221725118 k/ft

Members 17 and 18:

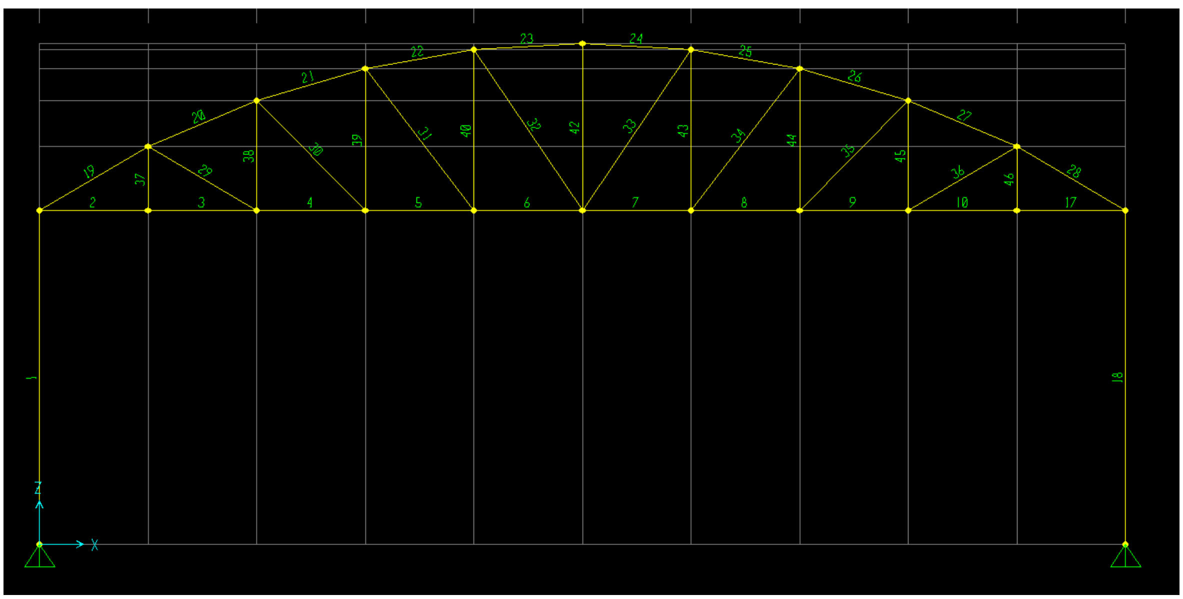
Load along roof slope:

$w_{TL} = w_D + w_S(L_2/L_1)$

$w_{TL} = 30\text{ PSF} + (23.1\text{ PSF})(13'/13.0208') = 53.06304\text{ PSF}$

$w_{TL} = (53.06304\text{ PSF})(8') = 424.50432\text{ lb/ft} = 0.42450432\text{ k/ft}$

These loads were applied to models of the glulam truss in SAP, and the results were recorded. Results for other load combinations were obtained by taking fractions of the results from the D + S load combination. For instance, since the dead load is (30 psf/53.1 psf), or 0.565 of the total load for the D + S load combination, results for just dead load were obtained by multiplying the results from the D + S load combination by 0.565. This same process was carried out to obtain results from the live roof load by itself. See Tables ____ - _____ below for a summary of the results for each load combination. In the tables, axial and shear forces are in kips and moments are in ft-kips.



Axial Load, Shear, and Moment (Unfactored) for Wood Trusses

	1 West Column	2 Bottom Chord	3 Bottom Chord	4 Bottom Chord	5 Bottom Chord	6 Bottom Chord	19 Top Chord	20 Top Chord	21 Top Chord	22 Top Chord	23 Top Chord
P_D	-16.14	24.62	24.62	25.18	25.55	25.73	-29.40	-28.03	-27.07	-26.38	-25.92
$P_{D,BOTTOM\ CHORD}$	-5.20	7.98	7.98	8.20	8.35	8.43	-9.25	-8.92	-8.70	-8.55	-8.46
P_{Lr}	-10.76	16.41	16.41	16.78	17.03	17.15	-19.60	-18.69	-18.05	-17.59	-17.28
P_S	-12.43	18.95	18.95	19.39	19.67	19.81	-22.64	-21.58	-20.84	-20.31	-19.95
$P_{W,LATERAL}$	0.00	-3.24	-3.24	-3.24	-3.24	-3.24	0.00	0.00	0.00	0.00	0.00
$P_{W,UPLIFT}$	8.90	-13.52	-13.52	-13.77	-13.94	-14.02	16.16	15.34	14.77	14.37	14.11
P_E	0.00	-4.27	-4.27	-4.27	-4.27	-4.27	0.00	0.00	0.00	0.00	0.00
V_D (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	-1.47	-1.51	-1.53	-1.55	-1.56
V_D (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	1.47	1.51	1.53	1.55	1.56
$V_{D,BOTTOM\ CHORD}$ (Top or Left)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	0.00	0.00	0.00	0.00	0.00
$V_{D,BOTTOM\ CHORD}$ (Bottom or Right)	0.00	0.52	0.52	0.52	0.52	0.52	0.00	0.00	0.00	0.00	0.00
V_{Lr} (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	-0.98	-1.00	-1.02	-1.03	-1.04
V_{Lr} (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	0.98	1.00	1.02	1.03	1.04
V_S (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	-1.13	-1.16	-1.18	-1.19	-1.20
V_S (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	1.13	1.16	1.18	1.19	1.20
$V_{W,LATERAL}$ (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$V_{W,LATERAL}$ (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$V_{W,UPLIFT}$ (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	0.84	0.84	0.84	0.84	0.84
$V_{W,UPLIFT}$ (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	-0.84	-0.84	-0.84	-0.84	-0.84
V_E (Top or Left)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
V_E (Bottom or Right)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
M_D (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	5.53	5.32	5.19	5.11	5.08
$M_{D,BOTTOM\ CHORD}$ (Max. Positive)	0.00	1.66	1.66	1.66	1.66	1.66	0.00	0.00	0.00	0.00	0.00
M_{Lr} (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	3.68	3.55	3.46	3.41	3.38
M_S (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	4.25	4.10	4.00	3.94	3.91
$M_{W,LATERAL}$ (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
$M_{W,UPLIFT}$ (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	-3.16	-2.97	-2.84	-2.77	-2.73
M_E (Max. Positive)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

D

Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-1.47	-1.51	-1.53	-1.55	-1.56
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	1.47	1.51	1.53	1.55	1.56
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	5.53	5.32	5.19	5.11	5.08
Max P_u (kips)	-21.34	32.60	32.60	33.38	33.90	34.16	-38.65	-36.95	-35.77	-34.94	-34.38

D + Lr

Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-2.44	-2.51	-2.56	-2.58	-2.60
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	2.44	2.51	2.56	2.58	2.60
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	9.21	8.87	8.65	8.52	8.46
Max P_u (kips)	-32.10	49.01	49.01	50.16	50.93	51.31	-58.25	-55.63	-53.82	-52.52	-51.66

D + S

Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-2.59	-2.66	-2.71	-2.74	-2.76
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	2.59	2.66	2.71	2.74	2.76
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	9.78	9.42	9.19	9.05	8.98
Max P_u (kips)	-33.76	51.55	51.55	52.76	53.57	53.97	-61.28	-58.53	-56.62	-55.25	-54.33

D +/- W

Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-0.63	-0.67	-0.69	-0.71	-0.72
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	0.63	0.67	0.69	0.71	0.72
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	2.37	2.36	2.35	2.35	2.34
Max P_u (kips)	-12.44	15.84	15.84	16.36	16.72	16.90	-22.49	-21.61	-21.00	-20.56	-20.27

D +/- E											
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-1.47	-1.51	-1.53	-1.55	-1.56
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	1.47	1.51	1.53	1.55	1.56
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	5.53	5.32	5.19	5.11	5.08
Max P_u (kips)	-21.34	28.32	28.32	29.11	29.63	29.89	-38.65	-36.95	-35.77	-34.94	-34.38

D + 0.75W + 0.75Lr											
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-1.57	-1.63	-1.67	-1.70	-1.71
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	1.57	1.63	1.67	1.70	1.71
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	5.92	5.76	5.66	5.60	5.56
Max P_u (kips)	-22.73	32.33	32.33	33.20	33.78	34.08	-41.23	-39.46	-38.23	-37.35	-36.75

D + 0.75W + 0.75S											
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.52	-0.52	-0.52	-0.52	-0.52	-1.68	-1.74	-1.79	-1.82	-1.83
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.52	0.52	0.52	0.52	0.52	1.68	1.74	1.79	1.82	1.83
Max $M_{MIDSPAN}$ (ft-kips)	0.00	1.66	1.66	1.66	1.66	1.66	6.35	6.17	6.06	5.99	5.96
Max P_u (kips)	-23.98	34.24	34.24	35.16	35.76	36.07	-43.50	-41.63	-40.33	-39.39	-38.76

0.6D + W											
Max $V_{TOP/LEFT}$ (kips)	0.00	-0.31	-0.31	-0.31	-0.31	-0.31	-0.04	-0.06	-0.08	-0.09	-0.10
Max $V_{BOTTOM/RIGHT}$ (kips)	0.00	0.31	0.31	0.31	0.31	0.31	0.04	0.06	0.08	0.09	0.10
Max $M_{MIDSPAN}$ (ft-kips)	0.00	0.99	0.99	0.99	0.99	0.99	0.16	0.23	0.27	0.30	0.31
Max P_u (kips)	-3.90	2.80	2.80	3.01	3.16	3.23	-7.03	-6.83	-6.69	-6.59	-6.52

Summary:

Summary of Maximum Forces, Moments, and Shears for West Column				
	Axial Force	Shear	Moment	C_D
D	-21.34	0.00	0.00	0.9
D + Lr	-32.10	0.00	0.00	1.0
D + S	-33.76	0.00	0.00	1.15
D +/- W	-12.44	0.00	0.00	1.6
D +/- E	-21.34	0.00	0.00	1.6
D + 0.75W + 0.75Lr	-22.73	0.00	0.00	1.6
D + 0.75W + 0.75S	-23.98	0.00	0.00	1.6
0.6D + W	-3.90	0.00	0.00	1.6

Summary of Maximum Forces, Moments, and Shears for Bottom Chord				
	Axial Force	Shear	Moment	C_D
D	34.16	0.52	1.66	0.9
D + Lr	51.31	0.52	1.66	1.0
D + S	53.97	0.52	1.66	1.15
D +/- W	16.90	0.52	1.66	1.6
D +/- E	29.89	0.52	1.66	1.6
D + 0.75W + 0.75Lr	34.08	0.52	1.66	1.6
D + 0.75W + 0.75S	36.07	0.52	1.66	1.6
0.6D + W	2.80	0.31	0.99	1.6

Summary of Maximum Forces, Moments, and Shears for Top Chord				
	Axial Force	Shear	Moment	C_D
D	-38.65	1.56	5.53	0.9
D + Lr	-58.25	2.60	9.21	1.0
D + S	-61.28	2.76	9.78	1.15
D +/- W	-22.49	0.72	2.37	1.6
D +/- E	-38.65	1.56	5.53	1.6
D + 0.75W + 0.75Lr	-41.23	1.71	5.92	1.6
D + 0.75W + 0.75S	-43.50	1.83	6.35	1.6
0.6D + W	-7.03	0.10	0.31	1.6

Units for Above Tables:
Axial Force: kips
Shear: kips
Moment: ft-kips

Wood Truss Member Design:

Top Chord: Combined Bending and Axial Forces (Member 6 is worst case)

Try 6 ¾" x 9 5/8"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 64.97 in^2

S = 104.2 in^3

E_{min} = 980,000 psi

LOAD COMBINATION: D + S

Axial Load: P = 61.284 kips (Compression) (from SAP2000)

Maximum Moment = 9.779 ft-kips = 117.342 in-kips (from SAP2000)

L = 15'-1" = 15.0833'

Axial Load:

$f_c = P/A$ = 61,284 lb/64.97 in^2 = 943.266 psi

$(l_e/d)_x$ = [(15.0833')(12 in/ft)]/9.625" = 18.8052 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x$ = 18.8052

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(18.8052)^2] = 1897.524$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 1897.529/1930.85 = 0.9827$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.9827]/[(2)(0.9)] = 1.1015$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.1015\} - \sqrt{\{[1.1015]^2 - [0.9827/0.9]\}}$

$= 1.1015 - 0.3485$

$= 0.7531$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.7531) = 1454.068$ psi

Axial stress ratio = $f_c/F'_c = (943.266 \text{ psi})/(1454.068 \text{ psi}) = 0.6487$

Net Section Check:

Assume connections will be made with (2) rows of ¾” diameter bolts.

Assume the hole diameter is 1/16” larger than the bolt (for stress calculations only).

$A_n = (6.75”)[9.625” - (2)(0.8125”)] = 54 \text{ in}^2$

(3/4” + 1/16” = 0.8125”)

$f_c = P/A_n = 61{,}284 \text{ lb}/54 \text{ in}^2 = 1134.889$ psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

1930.85 psi > 1134.889 psi ∴ OK

Bending:
Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case,

the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

M = 117.342 in-kips = 117,342 in-lb

S = 104.2 in^3 (for 6 ¾" x 9 5/8")

$f_b = M/S = 117{,}342$ in-lb/104.2 in^3 = 1,126.123 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/9.625'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0139 \leq 1.0$

$\therefore C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V)$ = (2100 psi)(1.15)(0.8)(1.0)(1.0) = 1932 psi

Bending stress ratio = f_b/F'_b = 1126.123 psi/1932 psi = 0.5829

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 18.80519481$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2$ = [(0.822)(816,340 psi)]/[$(18.8052)^2$] = 1897.524 psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})]$ = 1/[1 – (943.266 psi/1897.524 psi)] = 1.9885

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.6487)^2 + (1.9885)(0.5829) = 1.5799 > 1.0$ ∴ **N.G.**

Try 6 ¾" x 11"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 74.25 in^2

$S = 136.1\ in^3$

$E_{min} = 980{,}000$ psi

LOAD COMBINATION: D + S

Axial Load: P = 61.284 kips (Compression) (from SAP2000)

Maximum Moment = 9.779 ft-kips = 117.342 in-kips (from SAP2000)

L = 15'-1" = 15.083333'

Axial Load:

$f_c = P/A = 61{,}284\ lb/74.25\ in^2 = 825.374$ psi

$(l_e/d)_x = [(15.0833')(12\ in/ft)]/11" = 16.4545 < 50 \therefore$ OK

$(l_e/d)_y = 0$ because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x = 16.4545$

The larger slenderness ratio governs the adjust design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\ psi)]/[(16.4545)^2] = 2478.398$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\ psi)(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 2478.398/1930.85 = 1.2836$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 1.2836]/[(2)(0.9)] = 1.2687$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{}\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}$

$= \{1.2687\} - \sqrt{}\{[1.2687]^2 - [1.2836/0.9]\}$

$= 1.2687 - 0.4281$

$= 0.8405$

$F'_c = F_c^*(C_P) = (1930.85\ psi)(0.8405) = 1622.947\ psi > 825.374\ psi \therefore$ OK

Axial stress ratio = $f_c/F'_c = 825.374/1622.9472 = 0.5086$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[11" – (2)(0.8125")] = 63.281 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 61,284 lb/63.281 in^2 = 968.442 psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.15)(0.73)(1.0) = 1930.85 psi

1930.85 psi > 968.442 psi ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case, the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

M = 117.342 in-kips = 117,342 in-lb

S = 136.1 in^3

$f_b = M/S$ = 117,342 in-lb/136.1 in^3 = 862.175 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/11'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0072 \leq 1.0$

$\therefore C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V)$ = (2100 psi)(1.15)(0.8)(1.0)(1.0) = 1932 psi

Bending stress ratio = f_b/F'_b = 862.175/1932 = 0.4463

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x = 16.4545$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(16.4545)^2] = 2478.398 \text{ psi}$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.
Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (968.442/2478.398)] = 1.6414$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.5086)^2 + (1.6414)(0.4463) = 0.9912 < 1.0 \therefore$ OK

To be a little more conservative, use a slightly larger member.

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(2759 \text{ lb})/(74.25 \text{ in}^2) = 37.158 \text{ psi}$

$F_v = 300$ psi

$F'_v = F_v(C_D)(C_M)(C_t) = (300 \text{ psi})(1.15)(0.875)(1.0) = 301.875 \text{ psi} > 37.158 \text{ psi} \therefore$ **OK**

Try 6 ¾" x 12 3/8"

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$F_b = 2100$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 83.53 \text{ in}^2$

$S = 172.3 \text{ in}^3$

$E_{min} = 980{,}000$ psi

Load Combination: D + S

Axial Load: P = 61.284 kips (Compression) (from SAP2000)

Maximum Moment = 9.779 ft-kips = 117.342 in-kips (from SAP2000)

L = 15'-1" = 15.083333'

Axial Load:

$f_c = P/A = 61{,}284 \text{ lb}/83.53 \text{ in}^2 = 733.677 \text{ psi}$

$(l_e/d)_x = [(15.0833')(12 \text{ in/ft})]/12.375" = 14.6263 < 50 \therefore$ OK

$(l_e/d)_y = 0$ because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x = 14.6263$

The larger slenderness ratio governs the adjust design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

$E'_{min} = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(14.6263)^2] = 3136.723$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 3136.7229/1930.85 = 1.6245$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 1.6245]/[(2)(0.9)] = 1.4581$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.4581\} - \sqrt{\{[1.4581]^2 - [1.6245/0.9]\}}$

$= 1.4581 - 0.5665$

$= 0.8916$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.8916) = 1721.460 \text{ psi} > 733.677 \therefore$ OK

Axial stress ratio = $f_c/F'_c = 733.677/1721.460 = 0.4262$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[12.375" - (2)(0.8125")] = 72.5625$ in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 61{,}284 \text{ lb}/72.5625 \text{ in}^2 = 844.568$ psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$1930.85 \text{ psi} > 844.568 \text{ psi} \therefore$ OK

Bending:
Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case,

the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

M = 117.342 in-kips = 117,342 in-lb

$S = 172.3 \text{ in}^3$

$f_b = M/S = 117{,}342 \text{ in-lb}/172.3 \text{ in}^3 = 681.033 \text{ psi}$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/12.375'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0012 \leq 1.0$

$\therefore C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V) = (2100 \text{ psi})(1.15)(0.8)(1.0)(1.0) = 1932 \text{ psi}$

$> 681.033 \text{ psi} \therefore$ OK

Bending stress ratio = $f_b/F'_b = 681.033/1932 = 0.3525$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 14.62626263$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(14.6262)^2] = 3136.723 \text{ psi}$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (733.677/3136.723)] = 1.3053$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.4262)^2 + (1.3053)(0.3525) = 0.6418 < 1.0 \therefore$ OK

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(2759 \text{ lb})/(83.53 \text{ in}^2) = 49.545 \text{ psi}$

$F_v = 300 \text{ psi}$

$F'_v = F_v(C_D)(C_M)(C_t)$ = (300 psi)(1.15)(0.875)(1.0) = 301.875 psi > 49.545psi ∴ **OK**

USE 6 ¾" x 12 3/8"

LOAD COMBINATIOIN: D + L_r

Try 6 ¾" x 12 3/8"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 83.53 in^2

S = 172.3 in^3

E_{min} = 980,000 psi

Axial Load: P = 58.247 kips (Compression)

Maximum Moment = 9.208 ft-kips = 110.496 in-kips

L = 15'-1" = 15.083333'
Axial Load:

f_c = P/A = 58,247 lb/83.53 in^2 = 697.318 psi

$(l_e/d)_x$ = [(15.0833')(12 in/ft)]/12.375" = 14.6263 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x$ = 14.6263

The larger slenderness ratio governs the adjust design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.0 (for live load; load combination D + L_r)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

$c = 0.9$ (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(14.6263)^2] = 3136.723$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.0)(0.73)(1.0) = 1679$ psi

$F_{cE}/F_c^* = 3136.723/1679 = 1.8682$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 1.8682]/[(2)(0.9)] = 1.5934$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.5934\} - \sqrt{\{[1.5934]^2 - [1.8682/0.9]\}}$

$= 1.5934 - 0.6807$

$= 0.9128$

$F'_c = F_c^*(C_P) = (1679 \text{ psi})(0.9128) = 1532.579 \text{ psi} > 697.318 \text{ psi} \therefore$ OK

Axial stress ratio $= f_c/F'_c = 697.318/1532.579 = 0.4550$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[12.375" - (2)(0.8125")] = 72.5625 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 58{,}247 \text{ lb}/72.5625 \text{ in}^2 = 802.715$ psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.0)(0.73)(1.0) = 1679$ psi

$1679 \text{ psi} > 802.715 \text{ psi} \therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case, the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

$M = 110.496$ in-kips = 110,496 in-lb

$S = 172.3\ in^3$

$f_b = M/S = 110{,}496\ in\text{-}lb/172.3\ in^3 = 641.300\ psi$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/12.375'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0012 \leq 1.0$

$\therefore C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V) = (2100\ psi)(1.0)(0.8)(1.0)(1.0) = 1680\ psi$

$> 641.300\ psi \therefore$ OK

Bending stress ratio = $f_b/F'_b = 641.300/1680 = 0.3817$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x = 14.6263$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340\ psi)]/[(14.6263)^2] = 3136.723\ psi$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (697.318/3136.723)] =$

$= 1.2859$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.4550)^2 + (1.2859)(0.3817) = 0.6978 < 1.0 \therefore$ **OK**

CONTROLS OVER "D + S"

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(2{,}598\ lb)/(83.53\ in^2)] = 46.654\ psi$

$F_v = 300\ psi$

$F'_v = F_v(C_D)(C_M)(C_t) = (300 \text{ psi})(1.0)(0.875)(1.0) = 262.5 \text{ psi} > 46.654 \text{ psi} \therefore$ OK

USE 6 ¾" x 12 3/8"

LOAD COMBINATION: D

Try 6 ¾" x 12 3/8"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 83.53 \text{ in}^2$

$S = 172.3 \text{ in}^3$

E_{min} = 980,000 psi

Axial Load: P = 38.648 kips (Compression)

Maximum Moment = 5.525 ft-kips = 66.30 in-kips

L = 15'-1" = 15.083333'

Axial Load:

$f_c = P/A = 38{,}648 \text{ lb}/83.53 \text{ in}^2 = 462.684 \text{ psi}$

$(l_e/d)_x = [(15.0833')(12 \text{ in/ft})]/12.375" = 14.6263 < 50 \therefore$ OK

$(l_e/d)_y = 0$ because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x = 14.6263$

The larger slenderness ratio governs the adjust design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

$C_D = 0.9$ (for dead load; load combination D)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_M = 0.8$ for F_b (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(14.6263)^2] = 3136.723$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(0.9)(0.73)(1.0) = 1511.1$ psi

$F_{cE}/F_c^* = 3136.723/1511.1 = 2.0758$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 2.0758]/[(2)(0.9)] = 1.7088$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.7088\} - \sqrt{\{[1.7088]^2 - [2.0758/0.9]\}}$

$= 1.7088 - 0.7832$

$= 0.9255$

$F'_c = F_c^*(C_P) = (1511.1 \text{ psi})(0.9255) = 1398.581 \text{ psi} > 462.684 \text{ psi} \therefore$ OK

Axial stress ratio = $f_c/F'_c = 462.684/1398.5805 = 0.3308$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[12.375" - (2)(0.8125")] = 72.5625 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 38{,}648 \text{ lb}/72.5625 \text{ in}^2 = 532.617$ psi

At braced location there is no reduction for stability.

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(0.9)(0.73)(1.0) = 1511.1$ psi

$1511.1 \text{ psi} > 532.617 \text{ psi} \therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability. In this case,

the beam has full lateral support. Therefore, l_u and R_B are zero and the lateral stability factor is $C_L = 1.0$.

$M = 66.30$ in-kips = 66,300 in-lb

$S = 172.3\ in^3$

$f_b = M/S = 66{,}300\ \text{in-lb}/172.3\ in^3 = 384.794$ psi
$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/15.0833')^{1/20}(12''/12.375'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0012 \leq 1.0$

$\therefore C_V = 1.0$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L \text{ or } C_V) = (2100\ \text{psi})(0.9)(0.8)(1.0)(1.0) = 1512$ psi

> 384.794 psi $\therefore$ OK

Bending stress ratio = $f_b/F'_b = 384.794/1512 = 0.2545$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 14.6263$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340\ \text{psi})]/[(14.6263)^2] = 3136.723$ psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 – (f_c/F_{cEx})] = 1/[1 – (462.684/3136.723)] = 1.1730$

$(f_c/F'_c)^2 + \{1/[1 – (f_c/F_{cEx})]\}(f_b/F'_b) = (0.3308)^2 + (1.1730)(0.2545) = 0.4080 < 1.0$ $\therefore$ **OK**

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(1{,}559\ \text{lb})/(83.53\ in^2) = 27.996$ psi

$F_v = 300$ psi

$F'_v = F_v(C_D)(C_M)(C_t)$ = (300 psi)(0.9)(0.875)(1.0) = 236.25 psi > 27.996 psi ∴ OK

DOES NOT CONTROL

*Make Members 20, 21, 22, and 23 the same size cross section as Member 19 so that the entire top chord of the truss is the same size cross-section (the member size used for Member 19 will work for Members 20, 21, 22, and 23 since Members 20, 21, 22, and 23 are shorter in length and are required to carry less axial load than Member 19)

FINAL MEMBER SIZE = 6 ¾" x 12 3/8" Southern Pine Glulam I.D. #50

Bottom Chord: Combined Tension and Bending Forces (Members 3 and 4 are worst case)

LOAD COMBINATION: D + S

Axial Load: P = 53.974 kips (Tension)

Moment = 1.656 ft-kips = 19.872 in-kips = 19,872 in-lb (due to Dead Load)

Try d = 6 3/4" = 6.75" (same width as top chord members)

Axial Tension:

F_t = 1550 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.8 for F_t (p. 64, NDS Supplement)

C_t = 1.0

$F'_t = F_t(C_D)(C_M)(C_t)$ = (1550 psi)(1.15)(0.8)(1.0) = 1426 psi

$P = (F'_t)(A)$

Req'd $A_n = P/F'_t$ = 53,974 lb/1426 psi = 37.850 in^2

Assume (2) rows of ¾" diameter bolts.

Req'd $A_g = A_n + A_h$ = 37.850 in^2 + (6.75")[(2)(3/4" + 1/16")] = 48.819 in^2

Try 6 ¾" x 8 ¼" (A = 55.69 in^2 > 48.819 in^2 ∴OK)

A_n = 55.69 in^2 - (6.75")[(2)(3/4" + 1/16")] = 44.721 in^2

$f_t = T/A_n$ = (53,974 lb)/(44.721 in^2) = 1206.898 psi < 1426 psi ∴ OK

Determine tension stress at the point of maximum bending stress (midspan) for use in the interaction formula.

$f_t = T/A_g = 53{,}974 \text{ lb}/55.69 \text{ in}^2 = 969.187 \text{ psi} < 1426 \text{ psi} \therefore$ OK

Bending:

$S_x = 76.57 \text{ in}^3$

$f_b = M/S = (19{,}872 \text{ in-lb})/(76.57 \text{ in}^3) = 259.527 \text{ psi}$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.0')(12 \text{ in/ft})]/8.25'' = 18.909 > 7$

$\therefore l_e = 1.63l_u + 3d = (1.63)[(13.0')(12 \text{ in/ft})] + (3)(8.25'') = 279.03''$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(279.03'')(8.25'')/(6.75'')^2]} = 7.1080$

$F_{bE} = 1.20E'_{min}/R_B{}^2 = [(1.20)(816{,}340 \text{ psi})]/(7.1080)^2 = 19{,}388.98 \text{ psi}$

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100 \text{ psi})(1.15)(0.8)(1.0) = 1932 \text{ psi}$

$F_{bE}/F^*_b = (19{,}388.98)/(1932) = 10.0357$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 10.0357)/1.9 = 5.8083$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 5.8083 - \sqrt{(5.8083)^2 - (10.0357/0.95)]} = 0.9946$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/13.0')^{1/20}(12''/8.25'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0294 \leq 1.0 \therefore C_V = 1.0$

C_L controls over C_V

$F^*_b = F'_b = F_b(C_D)(C_M)(C_t)(C_L) = (2100 \text{ psi})(1.15)(0.8)(1.0)(0.9946) = 1921.567 \text{ psi}$

$> 259.527 \text{ psi} \therefore$ OK

Bending stress ratio $= f_b/F'_b = (259.527 \text{ psi})/(1921.567 \text{ psi}) = 0.1351$

Combined Stresses:

$(f_t/F'_t) + (f_{bx}/F^*_{bx}) = (969.187/1426 \text{ psi}) + (259.527/1921.567) = 0.8147 < 1.0 \therefore$ **OK**

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(520 \text{ lb})/(55.69 \text{ in}^2) = 14.006 \text{ psi}$

$F_v = 300 \text{ psi}$

$F'_v = F_v(C_D)(C_M)(C_t) = (300 \text{ psi})(1.15)(0.875)(1.0) = 301.875 \text{ psi} > 14.006 \text{ psi} \therefore$ OK

LOAD COMBINATION: D + L_r

<u>Try 6 ¾" x 8 ¼"</u>

Axial Load: P = 51.315 kips (Tension)

Moment = 1.656 ft-kips = 19.872 in-kips = 19,872 in-lb (due to Dead Load)

$A = 55.69 \text{ in}^2$

$S_x = 76.57 \text{ in}^3$

Axial Tension:

Assume (2) rows of ¾" diameter bolts.

$A_n = 55.69 \text{ in}^2 - (6.75")[(2)(3/4" + 1/16")] = 44.721 \text{ in}^2$

$f_t = T/A_n = (51{,}315 \text{ lb})/(44.721 \text{ in}^2) = 1147.448 \text{ psi}$

$F_t = 1550$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$C_D = 1.0$ (for live load; load combination D + L_r)

$C_M = 0.8$ for F_t (p. 64, NDS Supplement)

$C_t = 1.0$

$F'_t = F_t(C_D)(C_M)(C_t) = (1550 \text{ psi})(1.0)(0.8)(1.0) = 1240 \text{ psi} > 1147.448 \text{ psi} \therefore$ OK

Determine tension stress at the point of maximum bending stress (midspan) for use in the interaction formula.

$f_t = T/A_g = 51{,}315 \text{ lb}/55.69 \text{ in}^2 = 921.440 \text{ psi} < 1240 \text{ psi} \therefore$ OK

Bending:

$f_b = M/S = (19{,}872 \text{ in-lb})/(76.57 \text{ in}^3) = 259.527$ psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.0')(12 \text{ in/ft})]/8.25'' = 18.909 > 7$

$\therefore\ l_e = 1.63l_u + 3d = (1.63)[(13.0')(12 \text{ in/ft})] + (3)(8.25'') = 279.03''$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(279.03'')(8.25'')/(6.75'')^2]} = 7.1080$

$F_{bE} = 1.20E'_{min}/R_B^2 = [(1.20)(816{,}340 \text{ psi})]/(7.1080)^2 = 19{,}388.98$ psi

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100 \text{ psi})(1.15)(0.8)(1.0) = 1932$ psi

$F_{bE}/F^*_b = (19{,}388.98)/(1932) = 10.0357$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 10.0357)/1.9 = 5.8083$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 5.8083 - \sqrt{(5.8083)^2 - (10.0357/0.95)]} = 0.9946$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/13.0')^{1/20}(12''/8.25'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 1.0294 \leq 1.0 \therefore C_V = 1.0$

C_L controls over C_V

$F^*_b = F'_b = F_b(C_D)(C_M)(C_t)(C_L) = (2100 \text{ psi})(1.0)(0.8)(1.0)(0.9946) = 1670.928$ psi

> 259.527 psi $\therefore$ OK

Bending stress ratio = $f_{bx}/F^*_{bx} = (259.527 \text{ psi})/(1670.928 \text{ psi}) = 0.1553$

Combined Stresses:

$(f_t/F'_t) + (f_{bx}/F^*_{bx}) = (921.440/1240) + (259.527/1670.928) = 0.8984 < 1.0 \therefore$ OK

CONTROLS OVER LOAD COMBINATION "D + S"

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(520 \text{ lb})/(55.69 \text{ in}^2) = 14.006$ psi

F_v = 300 psi

$F'_v = F_v(C_D)(C_M)(C_t)$ = (300 psi)(1.0)(0.875)(1.0) = 262.5 psi > 14.006 psi ∴ **OK**

LOAD COMBINATION: D

Try 6 ¾" x 8 ¼"

Axial Load: P = 34.160 kips (Tension)

Moment = 1.656 ft-kips = 19.872 in-kips = 19,872 in-lb (due to Dead Load)

A = 55.69 in^2

S_x = 76.57 in^3

Axial Tension:
Assume (2) rows of ¾" diameter bolts.

A_n = 55.69 in^2 - (6.75")[(2)(3/4" + 1/16")] = 44.721 in^2

$f_t = T/A_n$ = (34,160 lb)/(44.721 in^2) = 763.847 psi

F_t = 1550 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

C_D = 0.9 (for dead load; load combination D)

C_M = 0.8 for F_t (p. 64, NDS Supplement)

C_t = 1.0

$F'_t = F_t(C_D)(C_M)(C_t)$ = (1550 psi)(0.9)(0.8)(1.0) = 1116 psi > 763.847 psi ∴ OK
Determine tension stress at the point of maximum bending stress (midspan) for use in the interaction formula.

$f_t = T/A_g$ = 34,160 lb/55.69 in^2 = 613.396 psi < 1116 psi ∴ OK

Bending:

f_b = M/S = (19,872 in-lb)/(76.57 in^3) = 259.527 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

C_L = 0.9946

For Southern Pine glulam: C_V = 1.0294 ≤ 1.0 ∴ C_V = 1.0

C_L controls over C_V

$F^*_b = F'_b = F_b(C_D)(C_M)(C_t)(C_L) = (2100 \text{ psi})(0.9)(0.8)(1.0)(0.9946) = 1503.835 \text{ psi}$

$> 259.527 \text{ psi} \therefore$ OK

Bending stress ratio = $f_{bx}/F^*_{bx} = (259.527 \text{ psi})/(1503.835 \text{ psi}) = 0.1726$

Combined Stresses:

$(f_t/F'_t) + (f_{bx}/F^*_{bx}) = (763.847/1116) + (259.527/1503.835) = 0.8570 < 1.0 \therefore$ **OK**

Check Shear:

$f_v = 1.5(V/A) = (1.5)[(520 \text{ lb})/(55.69 \text{ in}^2) = 14.006 \text{ psi}$

$F_v = 300$ psi

$F'_v = F_v(C_D)(C_M)(C_t) = (300 \text{ psi})(0.9)(0.875)(1.0) = 236.25 \text{ psi} > 14.006 \text{ psi} \therefore$ OK

DOES NOT CONTROL

*Use same member size for all bottom chord members (for consistency); the member size used for Member 6 will work for the rest of the bottom chord members since the axial (tensile) force in each of these other bottom chord members is less than the axial tensile force in Member 6.

FINAL MEMBER SIZE = 6 ¾” x 8 ¼” Southern Pine Glulam ID #50

Member 24 in SAP2000:

Load Combination: D + S

Axial Load: P = 0.262 kips (Compression)

L = 20’-0” = 20.0’

$(l_e/d)_{max} = 50$

$d \geq l_e/50 = [(20')(12 \text{ in/ft})]/50 = 4.8''$

Try d = 6 3/4” = 6.75”

$(l_e/d) = [(20.0')(12 \text{ in/ft})]/6.75'' = 35.556 < 50 \therefore$ OK

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66, NDS Supplement)

$E_{min} = 980{,}000$ psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(35.5556)^2] = 530.7963854$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 530.7964/1930.85 = 0.2749029626$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2749]/[(2)(0.9)] = 0.7082794237$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7082794237\} - \sqrt{\{[0.7082794237]^2 - [0.2749/0.9]\}}$

$= 0.7082794237 - 0.4429582438$

$= 0.2653211799$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.2653) = 512.2954001$ psi

$P = (F'_c)(A)$
$A_{req'd} = P/F'_c = 262 \text{ lb}/512.2954 \text{ psi} = 0.511424 \text{ in}^2$

Use 6 ¾" x 6 7/8" (A = 46.41 in^2 > 0.51 in^2 $\therefore$ OK)

*Must use width of 6 ¾" to match that of the top and bottom chord members (need to keep consistent width of members for side plates (for connections for truss members))

*Other load combinations of "D" and "D + L_r" will not require a larger size member since load is so small; width of member must be ≥ 4.8" to meet $l_e/d \leq 50$, which results in a members whose capacity is much greater than the required load it must carry

Member 32 in SAP2000:

Tension member

Very small axial force

Use 6 ¾" x 6 7/8" (minimum size with d = 6 ¾")

All web members forces are considerably small:

∴ **Use 6 ¾" x 6 7/8"** for all web members (minimum size to maintain same width as top and bottom chord members)

Member 1 (Member 1 in SAP2000 as well): Column

LOAD COMBINATION: D + S

Axial Load: P = 33.764 kips (Compression)

Analyze Column Buckling About x Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/d \leq 50$

$d \geq l_e/50 = [(40.0')(12 \text{ in/ft})]/50 = 9.6"$

Analyze Column Bucking About y Axis:

Braced at the third-points (L = 40.0'/3 = 13.3333')

$(l_e/d)_{max} = 50$

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/d \leq 50$

$d \geq l_e/50 = [(13.3333')(12 \text{ in/ft})]/50 = 3.2"$

Try d = 6 3/4" = 6.75" (to match "d" of truss members)

$(l_e/d)_y = [(13.3333')(12 \text{ in/ft})]/6.75" = 23.7037037$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(23.7037037)^2] = 1194.291867$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 1194.2919/1930.85 = 0.6185316661$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.6185]/[(2)(0.9)] = 0.8991942589$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.8991942589\} - \sqrt{\{[0.8991942589]^2 - [0.6185/0.9]\}}$

$= 0.8991942589 - 0.3482454949$

$= 0.550948764$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.5509) = 1063.799421$ psi

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/1063.7994 \text{ psi} = 31.739 \text{ in}^2$

Use 6 ¾" x 8 ¼" ($A = 55.69 \text{ in}^2 > 31.74 \text{ in}^2$ ∴OK)

However, 8 ½" < 9.6" (required dimension to prevent buckling about x axis)

Try 6 ¾" x 9 5/8" ($A = 64.97 \text{ in}^2 > 31.74 \text{ in}^2$ ∴ OK)

Check Column Dimensions:

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/9.625 = 49.8701 \leq 50$ ∴ OK [controls over $(l_e/d)_y$]

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037 \leq 50$ ∴ OK

Analyze Column Buckling About x Axis:

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/9.625 = 49.8701$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(49.87012987)^2] = 269.812$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi
$F_{cE}/F_c^* = 269.812/1930.85 = 0.1397$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.1397]/[(2)(0.9)] = 0.6332$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6332\} - \sqrt{\{[0.6332]^2 - [0.1397/0.9]\}}$

$= 0.6332 - 0.4956$

$= 0.1375$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.1375) = 265.5770$ psi

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/265.5770 \text{ psi} = 127.135 \text{ in}^2$

$A = 64.97 \text{ in}^2 < 127.135 \text{ in}^2$ ∴ **NO GOOD**

Try 6 ¾" x 16 ½" (A = 111.4 in²)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/16.5" = 29.0909$ [controls over $(l_e/d)_y$]

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(29.0909)^2] = 792.918$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 792.918/1930.85 = 0.4107$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.4107]/[(2)(0.9)] = 0.7837$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$
$= \{0.7837\} - \sqrt{\{[0.7837]^2 - [0.4107/0.9]\}}$

$= 0.7837 - 0.3974$

$= 0.3863$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.3863) = 745.956$ psi

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/745.956 \text{ psi} = 45.263 \text{ in}^2$

$A = 111.4 \text{ in}^2 > 45.263 \text{ in}^2$ ∴ **OK**

Try 6 ¾" x 15 1/8" (A = 102.1 in²)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/15.125" = 31.7355$ [controls over $(l_e/d)_y$]

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.15 (for snow load; load combination D+S)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816,340 \text{ psi})]/[(31.7355)^2] = 666.2714$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 666.2714/1930.85 = 0.3451$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.3451]/[(2)(0.9)] = 0.7473$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7473\} - \sqrt{\{[0.7473]^2 - [0.3451/0.9]\}}$

$= 0.7473 - 0.4183$

$= 0.3289$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.3289) = 635.138$ psi

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33,764 \text{ lb}/635.138 \text{ psi} = 53.160 \text{ in}^2$

$A = 111.4 \text{ in}^2 > 53.16 \text{ in}^2 \therefore$ OK

Try 6 ¾" x 13 ¾" (A = 92.81 in²)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/13.75" = 34.9091$ (controls over $(l_e/d)_y$)

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037$

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980,000$ psi

$C_D = 1.15$ (for snow load; load combination D+S)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980,000)(0.833)(1.0) = 816,340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816,340 \text{ psi})]/[(34.9091)^2] = 550.6375$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85$ psi

$F_{cE}/F_c^* = 550.6375/1930.85 = 0.2852$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2852]/[(2)(0.9)] = 0.7140$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7140\} - \sqrt{\{[0.7140]^2 - [0.2852/0.9]\}}$

$= 0.7140 - 0.4392$

$= 0.2748$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.2748) = 530.5371 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/530.5371 \text{ psi} = 63.641 \text{ in}^2$
$A = 92.81 \text{ in}^2 > 63.64 \text{ in}^2$ ∴ **OK**

Try 6 ¾" x 12 3/8" (A = 83.53 in^2)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/12.375" = 38.7879$ (controls over $(l_e/d)_y$)

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037$

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000$ psi

$C_D = 1.15$ (for snow load; load combination D+S)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340 \text{ psi}$

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(38.7879)^2] = 446.016 \text{ psi}$

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85 \text{ psi}$

$F_{cE}/F_c^* = 446.016/1930.85 = 0.2310$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2310]/[(2)(0.9)] = 0.6839$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6839\} - \sqrt{\{[0.6839]^2 – [0.2310/0.9]\}}$

$= 0.6839 – 0.4594$

$= 0.2245$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.2245) = 433.468 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/433.468 \text{ psi} = 77.893 \text{ in}^2$

$A = 83.53 \text{ in}^2 > 77.89 \text{ in}^2 \therefore$ **OK**

Use 6 ¾” x 12 3/8”

Try 6 ¾” x 11” (A = 74.25 in^2)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/11" = 43.6364$ [controls over $(l_e/d)_y$]

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/6.75 = 23.7037$

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000$ psi

$C_D = 1.15$ (for snow load; load combination D+S)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340 \text{ psi}$

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(43.6364)^2] = 352.408 \text{ psi}$

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85 \text{ psi}$

$F_{cE}/F_c^* = 352.408/1930.85 = 0.1825$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.1825]/[(2)(0.9)] = 0.6570$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 – [F_{cE}/F_c^*]/c\}}$

$= \{0.6570\} - \sqrt{\{[0.6570]^2 – [0.1825/0.9]\}}$

$= 0.6570 – 0.4783$

$= 0.1786$

$F'_c = F_c^*(C_P) = (1930.85 \text{ psi})(0.1786) = 344.907 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764 \text{ lb}/344.907 \text{ psi} = 97.893 \text{ in}^2$

$A = 74.25 \text{ in}^2 < 97.89 \text{ in}^2$ ∴ **NO GOOD**

Try 5 ½" x 13 ¾" (A = 75.63 in^2)

$(l_e/d)_x = [(1.0)(40.0')(12 \text{ in/ft})]/13.75" = 34.9091$ (controls over $(l_e/d)_y$)

$(l_e/d)_y = [(1.0)(13.333')(12 \text{ in/ft})]/5.5 = 29.0909$

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000$ psi

$C_D = 1.15$ (for snow load; load combination D+S)

$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340 \text{ psi}$

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(34.9091)^2] = 550.6375 \text{ psi}$

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.15)(0.73)(1.0) = 1930.85 \text{ psi}$

$F_{cE}/F_c^* = 550.6375/1930.85 = 0.2852$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2852]/[(2)(0.9)] = 0.7140$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 – [F_{cE}/F_c^*]/c\}}$

$= \{0.7140\} - \sqrt{\{[0.7140]^2 – [0.2852/0.9]\}}$

$= 0.7140 – 0.4392$

$= 0.2748$

$F'_c = F_c^*(C_P) = (1930.85\ psi)(0.2748) = 530.537\ psi$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764\ lb/530.537\ psi = 63.641\ in^2$

$A = 75.63\ in^2 > 63.64\ in^2 \therefore$ **OK**

Try 5 ½" x 12 3/8" ($A = 68.06\ in^2$)

$(l_e/d)_x = [(1.0)(40.0')(12\ in/ft)]/12.375" = 38.7879$ (controls over $(l_e/d)_y$)

$(l_e/d)_y = [(1.0)(13.333')(12\ in/ft)]/5.5 = 29.0909$

$F_c = 2300\ psi$ (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$E_{min} = 980{,}000\ psi$

$C_D = 1.15$ (for snow load; load combination D+S)
$C_M = 0.73$ for F_c (p. 64, NDS Supplement)

$C_M = 0.833$ for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340\ psi$

$c = 0.9$ (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\ psi)]/[(38.7879)^2] = 446.016\ psi$

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\ psi)(1.15)(0.73)(1.0) = 1930.85\ psi$

$F_{cE}/F_c^* = 446.016/1930.85 = 0.2310$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2310]/[(2)(0.9)] = 0.6839$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6839\} - \sqrt{\{[0.6839]^2 - [0.2310/0.9]\}}$

$= 0.6839 - 0.4594$

$= 0.2245$

$F'_c = F_c^*(C_P) = (1930.85\ psi)(0.2245) = 433.468\ psi$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 33{,}764\ lb/433.468\ psi = 77.893\ in^2$

A = 68.06 in^2 > 77.89 in^2 $\therefore$ **N.G.**

LOAD COMBINATION: D+W (Combined Bending and Axial Forces)

Try 6 ¾" x 16 ½"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 111.4 in^2

S = 306.3 in^3

E_{min} = 980,000 psi

Axial Load: P = 12,438 lb (Compression)

Maximum Moment:

W = 26.85 k + 51.49 k + 44.89 k = 123.23 k

(123.23 k)/[(156')(40')] = 0.019748 ksf = 19.7484 psf

w = (19.7484 psf)(8') = 157.987 lb/ft = 0.157987 k/ft

$M_{max} = wL^2/8 = (0.157987 \text{ k/ft})(40')^2/8$ = 31.599 k-ft = 31,599 ft-lb = 379,188 in-lb

L = 40.0'

Axial Load:

f_c = P/A = 12,438 lb/111.4 in^2 = 111.652 psi

$(l_e/d)_x$ = [(40')(12 in/ft)]/16.5" = 29.0909 < 50 $\therefore$ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 $\therefore$ OK

$(l_e/d)_{max} = (l_e/d)_x$ = 29.0909

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(29.0909)^2]$ = 792.919 psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.6)(0.73)(1.0) = 2686.4 psi

F_{cE}/F_c^* = 792.919/2686.4 = 0.2952

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.2952]/[(2)(0.9)] = 0.7195

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7195\} - \sqrt{\{[0.7195]^2 - [0.2952/0.9]\}}$

= 0.2839

$F'_c = F_c^*(C_P)$ = (2686.4 psi)(0.2839) = 762.727 psi

Axial stress ratio = f_c/F'_c = (111.652 psi)/(762.727 psi) = 0.1464

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[16.5" – (2)(0.8125")] = 97.03 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 12,438 lb/97.03 in^2 = 128.187 psi

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P)$ = (2300 psi)(1.6)(0.73)(1.0)(0.2839) = 762.669 psi

762.669 psi > 128.187 psi $\therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

$M = 379{,}188$ in-lb

$S = 306.3 \text{ in}^3$

$f_b = M/S = 379{,}188 \text{ in-lb}/306.3 \text{ in}^3 = 1237.963$ psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.333')(12 \text{ in/ft})]/16.5'' = 9.697 > 7$

$\therefore\ l_e = 1.63l_u + 3d = (1.63)[(13.333')(12 \text{ in/ft})] + (3)(16.5'') = 310.30''$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(310.30'')(16.5'')/(6.75'')^2]} = 10.601$

$F_{bE} = 1.20E'_{min}/R_B{}^2 = [(1.20)(816{,}340 \text{ psi})]/(10.601)^2 = 8717.544$ psi

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100 \text{ psi})(1.6)(0.8)(1.0) = 2688$ psi

$F_{bE}/F^*_b = (8717.544)/(2688) = 3.2431$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 3.2431)/1.9 = 2.233$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.233 - \sqrt{(2.233)^2 - (3.2431/0.95)]} = 0.9786$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12''/16.5'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 0.9400 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688 \text{ psi})(0.9400) = 2526.72$ psi

Bending stress ratio = $f_b/F'_b = (1237.98 \text{ psi})/(2526.72 \text{ psi}) = 0.4830$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 29.0909$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(29.0909)^2] = 792.919$ psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 – (f_c/F_{cEx})] = 1/[1 – (111.652 \text{ psi}/792.919 \text{ psi})] = 1.1639$

$(f_c/F'_c)^2 + \{1/[1 – (f_c/F_{cEx})]\}(f_b/F'_b) = (0.1464)^2 + (1.1639)(0.4830) = 0.5836 < 1.0 \therefore$ **OK**

Try 6 ¾" x 15 1/8"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 102.1 \text{ in}^2$

$S = 257.4 \text{ in}^3$

E_{min} = 980,000 psi

Axial Load: P = 12,438 lb (Compression)

Maximum Moment: M_{max} = 379,188 in-lb

L = 40.0'

Axial Load:

$f_c = P/A = 12{,}438 \text{ lb}/102.1 \text{ in}^2 = 121.822 \text{ psi}$

$(l_e/d)_x = [(40')(12 \text{ in/ft})]/15.125" = 31.7355 < 50 \therefore$ OK

$(l_e/d)_y = [(13.333')(12 \text{ in/ft})]/6.75" = 23.7037 < 50 \therefore$ OK

$(l_e/d)_{max} = (l_e/d)_x = 31.7355$

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\text{ psi})]/[(31.7355)^2] = 666.271\text{ psi}$

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\text{ psi})(1.6)(0.73)(1.0) = 2686.4\text{ psi}$

$F_{cE}/F_c^* = 666.271/2686.4 = 0.2480$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2480]/[(2)(0.9)] = 0.6933$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6933\} - \sqrt{\{[0.6933]^2 - [0.2480/0.9]\}}$

$= 0.2403$

$F'_c = F_c^*(C_P) = (2686.4\text{ psi})(0.2403) = 645.663\text{ psi}$

Axial stress ratio = $f_c/F'_c = (121.822\text{ psi})/(645.663\text{ psi}) = 0.1887$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[15.125" - (2)(0.8125")] = 91.125\text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 12{,}438\text{ lb}/91.125\text{ in}^2 = 136.494\text{ psi}$

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300\text{ psi})(1.6)(0.73)(1.0)(0.2403) = 645.542\text{ psi}$

$645.542\text{ psi} > 136.494\text{ psi} \therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 379,188 in-lb

$S = 257.4\text{ in}^3$

$f_b = M/S = 379{,}188\text{ in-lb}/257.4\text{ in}^3 = 1473.147\text{ psi}$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.333')(12\ in/ft)]/15.125'' = 10.579 > 7$

$\therefore\ l_e = 1.63l_u + 3d = (1.63)[(13.333')(12\ in/ft)] + (3)(15.125'') = 306.17''$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(306.17'')(15.125'')/(6.75'')^2]} = 10.082$

$F_{bE} = 1.20E'_{min}/R_B{}^2 = [(1.20)(816{,}340\ psi)]/(10.082)^2 = 9638.174\ psi$

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100\ psi)(1.6)(0.8)(1.0) = 2688\ psi$

$F_{bE}/F^*_b = (9638.174)/(2688) = 3.5856$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 3.5856)/1.9 = 2.4135$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.4135 - \sqrt{(2.4135)^2 - (3.5856/0.95)]} = 0.9815$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12''/15.125'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 0.9441 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688\ psi)(0.9441) = 2537.741\ psi$

Bending stress ratio $= f_b/F'_b = (1473.147\ psi)/(2537.741\ psi) = 0.5805$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x = 31.7355$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340\ psi)]/[(31.7355)^2] = 666.271\ psi$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor $= 1/[1 - (f_c/F_{cEx})] = 1/[1 - (121.822\ psi/666.271\ psi)] = 1.2238$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.1887)^2 + (1.2238)(0.5805) = 0.746 < 1.0$ $\therefore$ **OK**

Try 6 ¾" x 13 ¾"

$F_c = 2300\ psi$ (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 92.81\ in^2$

$S_x = 212.7\ in^3$

E_{min} = 980,000 psi

Axial Load: P = 12,438 lb (Compression)

Maximum Moment: M_{max} = 379,188 in-lb

L = 40.0'

Axial Load:

$f_c = P/A = 12{,}438\ lb/92.81\ in^2 = 134.016$ psi

$(l_e/d)_x = [(40')(12\ in/ft)]/13.75'' = 34.9091 < 50 \therefore$ OK

$(l_e/d)_y = [(13.333')(12\ in/ft)]/6.75'' = 23.7037 < 50 \therefore$ OK

$(l_e/d)_{max} = (l_e/d)_x = 34.9091$

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\ psi)]/[(34.9091)^2] = 550.638$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\ psi)(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 550.638/2686.4 = 0.2050$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2050]/[(2)(0.9)] = 0.6694$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6694\} - \sqrt{\{[0.6694]^2 - [0.2050/0.9]\}}$

$= 0.2000$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.2000) = 537.220 \text{ psi}$

Axial stress ratio = $f_c/F'_c = (134.016 \text{ psi})/(537.220 \text{ psi}) = 0.2495$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[13.75" - (2)(0.8125")] = 81.84 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 12{,}438 \text{ lb}/81.84 \text{ in}^2 = 151.979 \text{ psi}$

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300 \text{ psi})(1.6)(0.73)(1.0)(0.200) = 537.28 \text{ psi}$

537.28 psi > 151.979 psi $\therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 379,188 in-lb

$S = 257.4 \text{ in}^3$

$f_b = M/S = 379{,}188 \text{ in-lb}/212.7 \text{ in}^3 = 1782.736 \text{ psi}$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.333')(12 \text{ in/ft})]/13.75" = 11.636 > 7$

$\therefore l_e = 1.63l_u + 3d = (1.63)[(13.333')(12 \text{ in/ft})] + (3)(13.75") = 302.05"$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(302.05")(13.75")/(6.75")^2]} = 9.547$

$F_{bE} = 1.20E'_{min}/R_B^2 = [(1.20)(816{,}340 \text{ psi})]/(9.547)^2 = 10{,}746.782 \text{ psi}$

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100\text{ psi})(1.6)(0.8)(1.0) = 2688\text{ psi}$

$F_{bE}/F^*_b = (10{,}176.782)/(2688) = 3.9981$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 3.9981)/1.9 = 2.6306$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.6306 - \sqrt{(2.6306)^2 - (3.9981/0.95)]} = 0.9840$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12''/13.75'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 0.9486 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688\text{ psi})(0.9486) = 2549.837\text{ psi}$

Bending stress ratio = $f_b/F'_b = (1782.736\text{ psi})/(2549.837\text{ psi}) = 0.6992$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 34.9091$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340\text{ psi})]/[(34.9091)^2] = 550.637\text{ psi}$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (134.016\text{ psi}/550.637\text{ psi})] = 1.3217$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.2495)^2 + (1.3217)(0.6992) = 0.9864 < 1.0 \therefore$ **OK**

LOAD COMBINATION: D + 0.75W + 0.75 S

<u>Try 6 ¾” x 13 ¾”</u>

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 92.81\text{ in}^2$

S_x = 212.7 in^3

E_{min} = 980,000 psi

Axial Load: P = 23,983 lb (Compression)

Maximum Moment: M_{max} = 23.700 k-ft = 23,700 ft-lb = 284,400 in-lb

L = 40.0'

Axial Load:

f_c = P/A = 23,983 lb/92.81 in^2 = 258.410 psi

$(l_e/d)_x$ = [(40')(12 in/ft)]/13.75" = 34.9091 < 50 ∴ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 ∴ OK

$(l_e/d)_{max} = (l_e/d)_x$ = 34.9091
The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(34.9091)^2]$ = 550.638 psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.6)(0.73)(1.0) = 2686.4 psi

F_{cE}/F_c^* = 550.638/2686.4 = 0.2050

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.2050]/[(2)(0.9)] = 0.6694

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6694\} - \sqrt{\{[0.6694]^2 - [0.2050/0.9]\}}$

= 0.2000

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.2000) = 537.220 \text{ psi}$

Axial stress ratio = $f_c/F'_c = (258.410 \text{ psi})/(537.220 \text{ psi}) = 0.4810$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[13.75" – (2)(0.8125")] = 81.84 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 23{,}983 \text{ lb}/81.84 \text{ in}^2 = 293.047 \text{ psi}$

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300 \text{ psi})(1.6)(0.73)(1.0)(0.200) = 537.28 \text{ psi}$

537.28 psi > 293.047 psi ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 284,400 in-lb

$S = 257.4 \text{ in}^3$

$f_b = M/S = 284{,}400 \text{ in-lb}/212.7 \text{ in}^3 = 1337.094 \text{ psi}$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.333')(12 \text{ in/ft})]/13.75" = 11.636 > 7$

∴ $l_e = 1.63 l_u + 3d = (1.63)[(13.333')(12 \text{ in/ft})] + (3)(13.75") = 302.05"$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(302.05")(13.75")/(6.75")^2]} = 9.547$

$F_{bE} = 1.20E'_{min}/R_B^2 = [(1.20)(816{,}340 \text{ psi})]/(9.547)^2 = 10{,}746.782 \text{ psi}$

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100 \text{ psi})(1.6)(0.8)(1.0) = 2688 \text{ psi}$

$F_{bE}/F^*_b = (10{,}176.782)/(2688) = 3.9981$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 3.9981)/1.9 = 2.6306$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.6306 - \sqrt{(2.6306)^2 - (3.9981/0.95)]} = 0.9840$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12''/13.75'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 0.9486 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688 \text{ psi})(0.9486) = 2549.837 \text{ psi}$

Bending stress ratio = $f_b/F'_b = (1337.094 \text{ psi})/(2549.837 \text{ psi}) = 0.5244$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 34.9091$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(34.9091)^2] = 550.637 \text{ psi}$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (258.410 \text{ psi}/550.637 \text{ psi})] = 1.8843$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.4810)^2 + (1.8843)(0.5244) = 1.219 > 1.0 \therefore$ **N.G.**

Try 6 ¾" x 15 1/8"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 102.1 \text{ in}^2$

$S = 257.4 \text{ in}^3$

E_{min} = 980,000 psi

Axial Load: P = 23,983 lb (Compression)

Maximum Moment: M_{max} = 284,400 in-lb

L = 40.0'

Axial Load:

f_c = P/A = 23,983 lb/102.1 in^2 = 234.897 psi

$(l_e/d)_x$ = [(40')(12 in/ft)]/15.125" = 31.7355 < 50 ∴ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 ∴ OK

$(l_e/d)_{max} = (l_e/d)_x$ = 31.7355

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2]$ = [(0.822)(816,340 psi)]/[(31.7355)^2] = 666.271 psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.6)(0.73)(1.0) = 2686.4 psi

F_{cE}/F_c^* = 666.271/2686.4 = 0.2480

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.2480]/[(2)(0.9)] = 0.6933

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6933\} - \sqrt{\{[0.6933]^2 - [0.2480/0.9]\}}$

= 0.2403

$F'_c = F_c^*(C_P)$ = (2686.4 psi)(0.2403) = 645.663 psi

Axial stress ratio = f_c/F'_c = (234.897 psi)/(645.663 psi) = 0.3638

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[15.125" – (2)(0.8125")] = 91.125 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 23,983 lb/91.125 in^2 = 263.188 psi

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P)$ = (2300 psi)(1.6)(0.73)(1.0)(0.2403) = 645.542 psi

645.542 psi > 263.188 ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 284,400 in-lb

S = 257.4 in^3

f_b = M/S = 284,400 in-lb/257.4 in^3 = 1104.895 psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: l_u/d = [(13.333')(12 in/ft)]/15.125" = 10.579 > 7

∴ $l_e = 1.63l_u + 3d$ = (1.63)[(13.333')(12 in/ft)] + (3)(15.125") = 306.17"

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(306.17")(15.125")/(6.75")^2]}$ = 10.082

$F_{bE} = 1.20E'_{min}/R_B^2$ = [(1.20)(816,340 psi)]/(10.082)^2 = 9638.174 psi

$F^*_b = F_b(C_D)(C_M)(C_t)$ = (2100 psi)(1.6)(0.8)(1.0) = 2688 psi

F_{bE}/F^*_b = (9638.174)/(2688) = 3.5856

$(1 + F_{bE}/F^*_b)/1.9$ = (1 + 3.5856)/1.9 = 2.4135

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 2.4135 - \sqrt{(2.4135)^2 - (3.5856/0.95)]} = 0.9815$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/40')^{1/20}(12"/15.125")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

$C_V = 0.9441 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V)$ = (2688 psi)(0.9441) = 2537.741 psi

Bending stress ratio = f_b/F'_b = (1104.895 psi)/(2537.741 psi) = 0.4354

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 31.7355$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(31.7355)^2] = 666.271$ psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (234.897 \text{ psi}/666.271 \text{ psi})] = 1.5445$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.3638)^2 + (1.5445)(0.4354) = 0.805 < 1.0$ ∴ **OK**

FINAL SECTION SIZE: 6 ¾" x 15 1/8" Southern Pine Glulam ID #50

SUMMARY	
Top Chord	6 3/4" x 12 3/8"
Bottom Chord	6 3/4" x 8 1/4"
Web Members	6 3/4" x 6 7/8"
West Column	6 3/4" x 15 1/8"
All members are Southern Pine, Glulam I.D. #50	

Deflection Check in SAP2000:

Member 1 (Column): 6 ¾" x 15 1/8"(Southern Pine, Glulam ID # 50)

$A = 102.1 \text{ in}^2$

$I_x = bh^3/12 = (6.75")(15.125")^3/12 = 1946 \text{ in}^4$

$I_y = bh^3/12 = (15.125")(6.75")^3/12 = 387.6 \text{ in}^4$

E = 1,900,000 psi

Member 13 (Top Chord): 6 ¾" x 9 5/8" (Southern Pine, Glulam ID #50)

$A = 64.97 \text{ in}^2$

$I_x = bh^3/12 = (6.75")(9.625")^3/12 = 501.6 \text{ in}^4$

$I_y = bh^3/12 = (9.625")(6.75")^3/12 = 246.7 \text{ in}^4$

E = 1,900,000 psi

Member 6 (Bottom Chord): 6 ¾" x 6 7/8" (Southern Pine, Glulam ID #50)

$A = 46.41 \text{ in}^2$

$I_x = bh^3/12 = (6.75")(6.875")^3/12 = 182.8 \text{ in}^4$

$I_y = bh^3/12 = (6.875")(6.75")^3/12 = 176.2 \text{ in}^4$

E = 1,900,000 psi

Total Load: D + S

Deflection at mid-span of truss (top chord) = 1.582" (from SAP2000 model)

1.582" < L/240 = [(130')(12 in/ft)]/240 = 6.5" ∴ **OK**

Deflection at mid span of truss (bottom chord) = 1.584" (from SAP2000 model)

1.584" < L/240 = [(130')(12 in/ft)]/240 = 6.5" ∴ **OK**
Deflections include distributed dead load of (10 PSF)(8') = 80 lb/ft = 0.080 k/ft to the bottom chord.

Live Load: L_r

Deflection at mid-span of truss (top chord) = 0.513"

0.513" < L/360 = [(130')(12 in/ft)]/360 = 4.333" ∴ **OK**

Deflection at mid-span of truss (bottom chord) = 0.512"

0.512" < < L/360 = [(130')(12 in/ft)]/360 = 4.333" ∴ **OK**

All Top Chord Members:

Load along roof slope:

w_{Lr} = (20 PSF)(8') = 160 lb/ft = 0.160 k/ft (due to roof live load)

Cost Comparison Using RS Means

From RS Means Building Construction Cost Data (2009)
(costs include material, labor, and equipment)

Wood Roof System:

Connector Plates, steel, with bolts, straight = ($34/plate)(22)(19 trusses) = $14,212

Laminated Roof Deck:
Cedar, 3" thick = ($5.61/SF)(20,280 SF) = $113,770.80
(values for Southern Pine were not given, so Cedar was conservatively assumed)

Sheathing, Plywood on Roofs:
3/8" thick = ($0.87/SF)(20,280 SF) = $17,643.60

Glued-Laminated Beams:
Bowstring trusses, 20' o.c., 120' clear span
= ($8.09/SF)(20280 SF) = $164,065.20
Although 8' o.c. is not listed in the tables, it is listed for other similar framing systems. On average, the total cost of various trusses @ 8' o.c. is only about $1/SF more than the same trusses @ 16' o.c. For this analysis, look at radial arches:
120' clear span, frames 8' o.c. = $13.86/SF
120' clear span, frames 16' o.c. = $12.34/SF
Increased by $13.86/$12.34 = 1.1232
So, for the bowstring trusses at 8' o.c., 120' clear span, assume:
(1.1232)($8.09/SF) = $9.09/SF
($9.09/SF)(20280 SF) = $184,274.20
For pressure treating, add 35" to material cost:
Material cost: (1.1232)($7.24/SF) = $8.14/SF
(1.35)($8.14/SF) = $10.99/SF
Total cost = $10.99/SF + (1.1232)($0.53/SF) + (1.1232)($0.31/SF) =
= $11.93/SF
($11.93/SF)(20280 SF) = $242,011.14

High-Strength Bolts:
¾" diameter x 8" long = ($9.26/bolt)(846 bolts/truss)(19 trusses) = $148,845.24

Original Steel Roof System:

Paints and Protective Coatings:
Galvanizing steel in shop:
Steel trusses: 1 ton to 20 tons = ($795/ton)(19.1865 tons) = $15,253.27
Long-span metal roof deck (galvanized and painted):
Galvanized steel, 18 ga, corrugated (2 ½" and 3") = 2.4 psf
For 7 ½", assume = (2)(2.4 psf) = 4.8 psf
(4.8 psf)(20280 SF) = 97.344 k = 48.672 tons
Over 20 tons: ($735/ton)(48.672 tons) = $35,773.92

Welded Rigid Frame:

Minimum: (\$3,475/ton)[(38.373 k + 45.595 k)/2] = \$145,894.40
Maximum: (\$5,055/ton)[(38.373 k + 45.595 k)/2] = \$212,229.12

Or use "roof trusses":

Minimum: (\$4,615/ton)[(38.373 k + 45.595 k)/2] = \$193,756.16
Maximum: (\$5,751/ton)[(38.373 k + 45.595 k)/2] = \$241,449.98

For projects 25 to 49 tons, add 30% to material costs:

Welded Rigid Frame:

Minimum: (1.30)(\$3,125/ton) = \$4,062.5/ton
Total = \$4062.5/ton + \$223/ton + \$127/ton = \$4,412.5/ton
(\$4,412.5/ton)(41.984 tons) = \$185,254.40
Maximum: (1.30)(\$4050/ton) = \$5,265/ton
Total = \$5,265/ton + \$640/ton + \$365/ton = \$6,270/ton
(\$6,270/ton)(41.984 tons) = \$263,239.68

Or use "roof trusses":

Minimum: (1.30)(\$4,200/ton) = \$5,460/ton
Total = \$5460/ton + \$271/ton + \$144/ton = \$5,875/ton
(\$5875/ton)(41.984 tons) = \$246,656.00
Maximum: (1.30)(\$5100/ton) = \$6,630/ton
Total = \$6,630/ton + \$425/ton + \$226/ton = \$7,281/ton
(\$7281/ton)(41.984 tons) = \$305,685.50

Average of all four = \$1,000,835.58/4 = \$250,208.90

Plus, the actual cost would probably be toward the maximum end anyway due to the complex truss configuration.

Steel Deck:
7 ½" deep, long span, 18 gauge: \$16.30/SF
For acoustical perforated, with fiberglass, add: \$1.91/SF
Total = \$16.30/SF + \$1.91/SF = \$18.21/SF
(\$18.21/SF)(20,280 SF) = \$369,298.80

Concrete Moment Frames:

Forms in place, beams and girders:

24" wide, 4 use = \$6.64/SFCA
Column line 2: SFCA = (8 beams)[(2*24")+(2*30")/12](32') = 2304 SFCA
Column line 1.8: SFCA = (4 beams)[(2*24")+(2*26")/12](32') = 1066.67 SFCA
East/West frame: SFCA = (5 beams)[(2*24")+(2*26")/12](32') = 1333.33 SFCA
Total = 4,704.00 SFCA

($6.64/SFCA)(4704.00 SFCA) = $22,381.23

Forms in place, columns:

24"x24" columns, 4 use = $5.91/SFCA
Column line 2: SFCA = (5 columns)[(4*24")/12](40') = 1,600 SFCA
Column line 1.8: SFCA = (5 columns)[(4*24")/12](10.5') = 420 SFCA
Total = 2020 SFCA
($5.91/SFCA)(2,020 SFCA) = $11,938.20

Concrete in place:

Columns, 24"x24", average reinforcing = $1,068/CY
Column line 2: (5 columns)[(2')(2')(40')/27] = 29.630 CY
Column line 1.8: (5 columns)[(2')(2')(10.5')/27] = 7.778 CY
Total = 29.630 CY + 7.778 CY = 37.407 CY
($1,068/CY)(37.407 CY) = $39,951.08

Beams, 25' span = $901/CY
Column line 2: (8 beams)[(2')(2.5')(32')/27] = 47.407 CY
Column line 1.8: (4 beams)[(2')(2.1667')(32')/27] = 20.543 CY
East/West frame: (5 beams)[(2')(2.1667')(23')/27] = 18.457 CY
Total = 47.407 CY + 20.543 CY + 18.457= 86.407 CY
($901/CY)(86.407 CY) = $77,852.52

Reinforcing steel:

Beams and Girders: #3 to #7 = $2440/ton
Columns: #8 to #18 = $2170/ton

Beams: Use ρ_g = 0.015
Column line 2: (8 beams)[((24"*30")/144)(32')] = 1,280 ft^3
(0.015)(1280 ft^3) = 19.2 ft^3
(490 lb/ft^3)(19.2 ft^3) = 9,408 lb = 4.704 tons
($2,440/ton)(4.704 tons) = $11,477.76
Column line 1.8: (4 beams)[((24"*26")/144)(32')] = 554.667 ft^3
(0.015)(554.667 ft^3) = 8.32 ft^3
(490 lb/ft^3)(8.32 ft^3) = 4,076.80 lb = 2.038 tons
($2,440/ton)(2.038 tons) = $4,973.70
East/West frame: (5 beams)[((24"*26")/144)(23')] = 498.333 ft^3
(0.015)(498.333 ft^3) = 7.475 ft^3
(490 lb/ft^3)(7.475 ft^3) = 3,662.75 lb = 1.831 tons
($2,440/ton)(1.831 tons) = $4,468.56

Columns: Use ρ_g = 0.015
Column line 2: (5 columns)[((24"*24")/144)(40')] = 800 ft^3
(0.015)(800 ft^3) = 12.0 ft^3
(490 lb/ft^3)(12.0 ft^3) = 5,880 lb = 2.94 tons
($2440/ton)(2.94 tons) = $7173.60
Column line 1.8: (5 columns)[((24"*24")/144)(10.5')] = 210 ft^3

(0.015)(210 ft^3) = 3.15 ft^3
(490 lb/ft^3)(3.15 ft^3) = 1,543.50 lb = 0.772 tons
($2440/ton)(0.772 tons) = $1,883.07

Steel Moment Frame (Original Design):

Structural tubing, heavy sections = $1.63/lb
Column line 2:
Columns: (5) HSS18x18x5/8
(5)[(127 lb/ft)(37')] = 23,495 lb
($1.63/lb)(23,495 lb) = $38,296.85
Beams: (8) HSS12x12x3/8
(8)[(58.03 lb/ft)(30')] = 13,927.20 lb
($1.63/lb)(13,927.20 lb) = $22,701.34
Column line 1.8:
Columns: (5) HSS14x14x1/2
(5)[(89.55 lb/ft)(10.5')] = 4,701.375 lb
($1.63/lb)(4,701.375 lb) = $7,663.24
Beams: (4) W27x84
(4)(30') = 120'
($143.54/ft)(120') = $17,224.80
East/West frame:
Beams: (5) W27x84
(5)(23') = 115'
($143.54/ft)(115') = $16,507.10

Decking

From "AITC 112*-81: Standard for Tongue-and-Groove Heavy Timber Roof Decking"

1) Sizes (tongue-and-groove decking)
   Two-inch decking
   Three-inch decking
   Four-inch decking
   (nominal dimensions are given)

2) Patterns
   Controlled Random Layup
   Cantilever Spans with Controlled Random Layup
   Cantilevered Pieces Intermixed
   Combination Simple and Two-Span Continuous
   Two-Span Continuous

3) V-groove for architectural aspect since decking will be exposed from below.

4) Southern Pine
   Select Quality
      Bending Stress = 1650 psi
      Modulus of Elasticity = 1,600,000 psi
   Commercial Quality
      Bending Stress = 1650 psi
      Modulus of Elasticity = 1,600,000 psi

*"When decking is used where the moisture content will exceed 19% for an extended period of time, bending stress values should be multiplied by a factor of 0.86 and modules of elasticity by a factor of 0.97."

*These values include repetitive member factor

Adjusted Values for Southern Pine (moisture content exceeding 19% since natatorium):
   Select Quality
      Bending Stress = (0.86)(1650 psi) = **1419 psi**
      Modulus of Elasticity = (0.97)(1,600,000 psi) = **1,552,000 psi**

5) Table 4: "Two Inch Nominal Thickness, Allowable Roof Load Limited by Bending"
   Simple Span, 8 ft, Bending Stress = 1400 psi
      =66 psf
   Controlled Random Layup Span, 8 ft, Bending Stress = 1400 psi
      =55 psf

6) Table 5: "Two Inch Nominal Thickness, Allowable Roof Load Limited by Deflection"

Simple Span, 8 ft, Modulus of Elasticity = 1,500,000 psi
L/180……….29 psf
L/240……….22 psf
L/360……….(29 psf)(0.5) = 14.5 psf
Controlled Random Layup Span, 8 ft, Modulus of Elasticity = 1,500,000 psi
L/180……….38 psf
L/240……….29 psf
L/360……….(38 psf)(0.5) = 19 psf
Cantilevered Pieces Intermixed, 8 ft, Modulus of Elasticity = 1,500,000 psi
L/180……….(38 psf)(1.05) = 39.9 psf
L/240……….(29 psf)(1.05) = 30.45 psf
L/360……….(39.9 psf)(0.5) = 19.95 psf
Combination Simple Span and Two-Span Continuous, 8 ft, E = 1,500,000 psi
L/180……….(38 psf)(1.31) = 49.78 psf
L/240……….(29 psf)(1.31) = 37.99 psf
L/360……….(49.78 psf)(0.5) = 24.89 psf
Two-Span Continuous, 8 ft, E = 1,500,000 psi
L/180……….(38 psf)(1.85) = 70.3 psf
L/240……….(29 psf)(1.85) = 53.65 psf
L/360……….(70.3 psf)(0.5) = 35.15 psf

7) Table 6: "Three and Four Inch Nominal Thickness, Allowable Roof Load Limited by Bending, Simple Span and Controlled Random Layups (3 or more spans)"
3 in. Nominal Thickness, 8 ft, Bending Stress = 1400 psi
= 182 psf
4 in. Nominal Thickness, 8 ft, Bending Stress = 1400 psi
= 357 psi

8) Table 7: "Three and Four Inch Nominal Thickness, Allowable Roof Load Limited by Deflection, Simple Span Layup"
3 in. Nominal Thickness, 8 ft, E = 1,500,000 psi
L/180……….136 psf
L/240………..102 psf
L/360……….(136 psf)(0.5) = 68 psf
4 in. Nominal Thickness, 8 ft, E = 1,500,000 psi
L/180……….347 psf
L/240……….261 psf
L/360……….(347 psf)(0.5) = 173.5 psf

9) Table 8: "Three and Four Inch Nominal Thickness, Allowable Roof Load Limited by Deflection, Controlled Random Layup (3 or more spans)"
3 in. Nominal Thickness, 8 ft, E = 1,500,000 psi
L/180……….205 psf
L/240……….154 psf
L/360……….(205 psf)(0.5) = 102.5 psf
Cantilevered Pieces Intermixed, 3 in., 8 ft, E = 1,500,000 psi

L/180……….(205 psf)(0.90) = 184.5 psf
L/240……….(154 psf)(0.90) = 138.6 psf
L/360……….(184.5 psf)(0.5) = 92.25 psf

Combination Simple Spans and Two-Span Continuous, 3 in., 8 ft

L/180……….(205 psf)(1.13) = 231.65 psf
L/240……….(154 psf)(1.13) = 174.02 psf
L/360……….(231.65 psf)(0.5) = 115.825 psf

Two-Span Continuous, 3 in., 8 ft, E = 1,500,000 psi

L/180……….(205 psf)(1.59) = 325.95 psf
L/240……….(154 psf)(1.59) = 244.86 psf
L/360……….(325.95 psf)(0.5) = 162.975 psf

4 in. Nominal Thickness, 8 ft, E = 1,500,00 psi

L/180……….562 psf
L/240……….421 psf
L/360……….(562 psf)(0.5) = 281 psf

Cantilevered Pieces Intermixed, 4 in. 8 ft, E = 1,500,000 psi

L/180……….(562 psf)(0.90) = 505.8 psf
L/240……….(421 psf)(0.90) = 378.9 psf
L/360……….(505.8 psf)(0.5) = 252.9 psf

Combination Simple Spans and Two-Span Continuous, 4 in., 8 ft

L/180……….(562 psf)(1.13) = 635.06 psf
L/240……….(421 psf)(1.13) = 475.73 psf
L/360……….(635.06 psf)(1.13) = 717.6178 psf

Two-Span Continuous, 4 in., 8 ft, E = 1,500,000 psi

L/180……….(562 psf)(1.59) = 893.58 psf
L/240……….(421 psf)(1.59) = 669.39 psf
L/360……….(893.58 psf)(0.5) = 446.79 psf

Wood Diaphragm:

Support for gravity loads applied to the roof is provided by the 3-inch tongue-and-groove decking. Plywood will be nailed directly into the tongue-and-groove decking to ensure diaphragm action of the roof system.

From ANSI / AF&PA SDPWS-2005 "Special Design Provisions for Wind and Seismic":

Section 4.2.4: Diaphragm Aspect Ratios (p. 14)

Wood structural panel, blocked: Maximum L/W ratio = 3:1

Aspect ratio = (156'/130'):1 = 1.2:1 < 3:1 $\therefore$ OK

Section 4.2.3: Unit Shear Capacities

For ASD allowable unit shear capacity, divide table values (nominal unit shear capacity) by 2.0 (the ASD reduction factor).

Lateral Loads to Sheathing:

SEISMIC LOADS:

Will only see "Building 1" seismic loads

Total load = 8.96 k (level 1) + 31.43 k (level 2) + 40.79 k (level 3) = 81.16 k

(assuming that all lateral load is transferred to roof diaphragm: worst-case scenario)

Longitudinal Direction (North/South):

Assume load is evenly distributed: w_u = (81.16 k)/130' = 0.6243 k/ft

V_u = (0.6243 k/ft)(130')/2 = 40.58 k

$\nu_u = V_u/b$ = (40.58 k)/(156') = 0.26013 k/ft = 260.13 lb/ft

Transverse Direction (East/West):

Assume load is evenly distributed: w_u = (81.16 k)/156' = 0.5203 k/ft

V_u = (0.5203 k/ft)(156')/2 = 40.58 k

$\nu_u = V_u/b$ = (40.58 k)/(130') = 0.31215 k/ft = 312.15 lb/ft

Roof Unit Shears (ASD):

From load combinations: Use 0.7E

Longitudinal Direction: ν = 0.7E = (0.7)(260.13 lb/ft) = 182.09 lb/ft

Transverse Direction: $\nu = 0.7E = (0.7)(312.15 \text{ lb/ft}) = 218.51 \text{ lb/ft}$

Wood Structural Panel Sheathing and Nailing:

Assume load cases 2 and 4.

Transverse Direction (Case 4):

Need table value (from Table A.4.2A) of (218.51 lb/ft)(2) = 437.01 lb/ft

Use:

3/8" Structural I plywood
All edges supported and nailed into 3 in. minimum nominal framing
(blocking is provided by tongue-and-groove decking)
8d common nails at:
- 6-in. o.c. boundary and continuous panel edges
- 6-in. o.c. other panel edges (blocking is provided)
- 12-in. o.c. in field

Allowable $\nu = 600 \text{ lb/ft}/2 = 300 \text{ lb/ft} > 218.51 \text{ lb/ft}$ ∴ OK
$> 182.09 \text{ lb/ft}$ ∴ OK

WIND LOADS:

North/South Direction:

Total load = 66.68 k (level 1) + 46.46 k (level 2) + 37.63 k (level 3) = 150.77 k

Assume that half of total lateral load is transferred to roof diaphragm:

150.77 k/2 = 75.39 k

Longitudinal Direction (North/South):

Assume load is evenly distributed: $w_u = (75.385 \text{ k})/130' = 0.5799 \text{ k/ft}$

$V_u = (0.5799 \text{ k/ft})(130')/2 = 37.69 \text{ k}$

$\nu_u = V_u/b = (37.69 \text{ k})/(156') = 0.24162 \text{ k/ft} = 241.62 \text{ lb/ft}$

East/West Direction:

Total load = 44.89 k (level 1) + 51.49 k (level 2) + 26.85 k (level 3) = 123.23 k

Assume that half of total lateral load is transferred to roof diaphragm:

123.23 k/2 = 61.62 k

Transverse Direction (East/West):

Assume load is evenly distributed: $w_u = (61.62 \text{ k})/156' = 0.3950 \text{ k/ft}$

$V_u = (0.3950 \text{ k/ft})(156')/2 = 30.81 \text{ k}$

$\nu_u = V_u/b = (30.81 \text{ k})/(130') = 0.2370 \text{ k/ft} = 236.98 \text{ lb/ft}$

Roof Unit Shears (ASD):

From load combinations: Use 1.0W

Longitudinal Direction: $\nu = 1.0W = (1.0)(241.62 \text{ lb/ft}) = 241.62 \text{ lb/ft}$

Transverse Direction: $\nu = 1.0W = (1.0)(236.98 \text{ lb/ft}) = 236.98 \text{ lb/ft}$

Wood Structural Panel Sheathing and Nailing:

Assume load cases 2 and 4.

Transverse Direction (Case 4):

Need table value (from Table A.4.2A) of (241.62 lb/ft)(2) = 483.24 lb/ft

Use:

5/16" Structural I plywood
All edges supported and nailed into 3 in. minimum nominal framing
(blocking is provided by tongue-and-groove decking)
6d common nails at:
- 6-in. o.c. boundary and continuous panel edges
- 6-in. o.c. other panel edges (blocking is provided)
- 12-in. o.c. in field

Allowable ν = 590 lb/ft/2 = 300 lb/ft > 241.62 lb/ft ∴ OK
> 236.98 lb/ft ∴ OK

Seismic load requirements control

∴ Use: 3/8" Structural I plywood
All edges supported and nailed into 3 in. minimum nominal framing
(blocking is provided by tongue-and-groove decking)
8d common nails at:
- 6-in. o.c. boundary and continuous panel edges
- 6-in. o.c. other panel edges (blocking is provided)
- 12-in. o.c. in field

Allowable ν = 600 lb/ft/2 = 300 lb/ft > 218.51 lb/ft ∴ OK
> 182.09 lb/ft ∴ OK

Design of Chords:

Longitudinal Direction:

SEISMIC LOADS:

$M_{u,max} = wL^2/8 = (0.6243\ k/ft)(130')^2/8 = 1318.83$ k-ft

$T_u = C_u = M_u/b = 1318.83$ k-ft/156' = 8.454 k

WIND LOADS:

$M_{u,max} = wL^2/8 = (0.5799\ k/ft)(130')^2/8 = 1225.039$ k-ft

$T_u = C_u = M_u/b = 1225.039$ k-ft/156' = 7.853 k

$\therefore$ Seismic controls

Check the 3 ½" x 5 ½" Southern Pine glulam ID #50 member already designed for the braced frames at column line 1.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 19.25 in^2

E_{min} = 980,000 psi

LOAD COMBINATION: E

Axial Compression:

P = 8.454 kips (Compression)

L = 8.0'

$f_c = P/A$ = 8,454 lb/19.25 in^2 = 439.169 psi

$(l_e/d)_x$ = [(8.0')(12 in/ft)]/5.5" = 17.4545 < 50 $\therefore$ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x = 17.4545$

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for seismic load; load combination E)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(17.4545)^2] = 2202.562$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 2202.562/2686.4 = 0.8199$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.8199]/[(2)(0.9)] = 1.0111$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{1.0111\} - \sqrt{\{[1.0111]^2 - [0.8199/0.9]\}}$

$= 0.6776$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.6776) = 1820.239 \text{ psi} > f_c = 439.169$ psi $\therefore$ **OK**

Axial Load: P = 8.454 kips (Tension)

Axial Tension:

P = 8.454 kips (Tension)

$F_t = 1550$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$C_D = 1.6$ (for seismic load; load combination E)

$C_M = 0.8$ for F_t (p. 64, NDS Supplement)

$C_t = 1.0$

$F'_t = F_t(C_D)(C_M)(C_t) = (1550 \text{ psi})(1.6)(0.8)(1.0) = 1984$ psi

$P = (F'_t)(A)$

Req'd $A_n = P/F'_t = 8{,}454 \text{ lb}/1984 \text{ psi} = 4.261 \text{ in}^2$

Assume (2) rows of ¾" diameter bolts.

Req'd $A_g = A_n + A_h = 4.261 \text{ in}^2 + (3.5")[(2)(3/4" + 1/16")] = 9.949 \text{ in}^2$

Try 3 ½" x 5 ½" ($A = 19.25 \text{ in}^2 > 9.95 \text{ in}^2$ $\therefore$ OK)

A_n = 19.25 in^2 - (3.5")[(2)(3/4" + 1/16")] = 13.56 in^2

f_t = T/A_n = (8,454 lb)/(13.56 in^2) = 623.34 psi < F'_t = 1984 psi ∴ **OK**

Use 3 ½" x 5 ½" Southern Pine glulam ID #50

Transverse Direction:

SEISMIC LOADS:

$M_{u,max}$ = wL^2/8 = (0.5203 k/ft)(156')^2/8 = 1582.75 k-ft

T_u = C_u = M_u/b = 1582.75 k-ft/130' = 12.175 k

WIND LOADS:

$M_{u,max}$ = wL^2/8 = (0.3950 k/ft)(156')^2/8 = 1201.59 k-ft

T_u = C_u = M_u/b = 1201.59 k-ft/130' = 9.243 k

∴ Seismic controls

Check the 5" x 6 7/8" Southern Pine glulam ID #50 member already designed for the braced frames in the East/West direction.

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 34.38 in^2

E_{min} = 980,000 psi

LOAD COMBINATION: W

Axial Compression:

P = 12.175 kips (Compression)

L = 26.0'

f_c = P/A = 12,175 lb/34.38 in^2 = 354.130 psi

$(l_e/d)_x$ = [(26.0')(12 in/ft)]/5.0" = 62.4 > 50 ∴ **N.G.**

Try 6 ¾" x 6 7/8"

A = 46.41 in^2

f_c = P/A = 12,175 lb/46.41 in^2 = 262.336 psi

$(l_e/d)_x$ = [(26.0')(12 in/ft)]/6.75" = 46.222 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x$ = 46.222

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

C_D = 1.6 (for seismic load; load combination E)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t)$ = (980,000)(0.833)(1.0) = 816,340 psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816,340 \text{ psi})]/[(46.222)^2]$ = 314.081 psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t)$ = (2300 psi)(1.6)(0.73)(1.0) = 2686.4 psi

F_{cE}/F_c^* = 314.081/2686.4 = 0.1169

$[1 + F_{cE}/F_c^*]/(2c)$ = [1 + 0.1169]/[(2)(0.9)] = 0.6205

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6205\} - \sqrt{\{[0.6205]^2 - [0.1169/0.9]\}}$

= 0.1154

$F'_c = F_c^*(C_P)$ = (2686.4 psi)(0.1154) = 309.969 psi < f_c = 354.130 psi ∴ **N.G.**

Try 6 ¾" x 8 ¼"

A = 55.69 in^2

f_c = P/A = 12,175 lb/55.69 in^2 = 218.621 psi

$(l_e/d)_x$ = [(26.0')(12 in/ft)]/8.25" = 37.818 < 50 ∴ OK

$(l_e/d)_y$ = 0 because of lateral support provided by roof diaphragm

$(l_e/d)_{max} = (l_e/d)_x$ = 37.818

The larger slenderness ratio governs the adjusted design value. Therefore, the strong axis of the member is critical, and $(l_e/d)_x$ is used to determine F'_c.

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(37.818)^2] = 469.182$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 469.182/2686.4 = 0.1747$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.1747]/[(2)(0.9)] = 0.6526$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6526\} - \sqrt{\{[0.6526]^2 - [0.1747/0.9]\}}$

$= 0.1712$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.1712) = 459.888 \text{ psi} > f_c = 218.621 \text{ psi} \therefore$ **O.K.**

Axial Tension:

P = 12.175 kips (Tension)

$F_t = 1550$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$C_D = 1.6$ (for seismic load; load combination E)

$C_M = 0.8$ for F_t (p. 64, NDS Supplement)

$C_t = 1.0$

$F'_t = F_t(C_D)(C_M)(C_t) = (1550 \text{ psi})(1.6)(0.8)(1.0) = 1984$ psi

$P = (F'_t)(A)$

Req'd $A_n = P/F'_t = 12{,}175 \text{ lb}/1984 \text{ psi} = 6.137 \text{ in}^2$

Assume (2) rows of ¾" diameter bolts.

Req'd $A_g = A_n + A_h = 6.137 \text{ in}^2 + (6.75")[(2)(3/4" + 1/16")] = 17.106 \text{ in}^2$

Try 6 ¾" x 8 ¼" ($A = 55.69 \text{ in}^2 > 17.106 \text{ in}^2 \therefore$ OK)

$A_n = 55.69 \text{ in}^2 - (6.75")[(2)(3/4" + 1/16")] = 44.721 \text{ in}^2$

$f_t = T/A_n = (12{,}175 \text{ lb})/(44.72 \text{ in}^2) = 272.242 \text{ psi} < F'_t = 1984 \text{ psi} \therefore$ **OK**

Use 6 ¾" x 8 ¼" Southern Pine glulam ID #50

Wood Truss Member Connections

Bolted Metal Side Plates

Bottom Chord Heel Connections

Maximum tension force at heel (from bottom chord):

$D + S = (24.616\text{ k} + 7.979\text{ k}) + 18.954\text{ k} = 51.549\text{ k}$

$D + L_r = (24.616\text{ k} + 7.979\text{ k}) + 16.411\text{ k} = 49.006\text{ k}$

Other load combinations will not control by inspection.

LOAD COMBINATION: D + S

For 6 ¾" thick southern pine glulam member, wit h ¼" steel side plates, load applied parallel to grain, the nominal design value "Z" of a ¾" bolt in double shear is:

Z = 3460 lb (Table 11I, p. 90, NDS)

The allowable bolt design value is:

$Z' = (Z)(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_{eg})(C_{di})(C_{tn})$

$C_D = 1.15$

$C_M = 0.7$ (for dowel-type fasteners with in-service moisture content > 19%)

$C_t = 1.0$

$C_{eg} = C_{di} = C_{tn} = 1.0$

$Z' = (3480\text{ lb})(1.15)(0.7)(1.0)(C_g)(C_\Delta)(1.0)(1.0)(1.0) = (2801.4\text{ lb})(C_g)(C_\Delta)$

Check bolt spacing and edge distances:

Bottom Chord: 6 ¾" x 8 ¼"

Table 11.5.1A: Edge Distance Requirements

Parallel to Grain:

l/D = minimum of $[l_m/D$ or $l_s/D]$

$l_m/D = 6.75"/0.75" = 9$

$l_s/D = (2)(1/4")/0.75" = 0.667$ (Governs) l/D = 0.667 < 6 ∴ Min. Edge Distance = 1.5D = (1.5)(0.75") = 1.125"

Table 11.5.1B: End Distance Requirements

Direction of Loading is Parallel to Grain, Tension: (fastener bearing toward member end)

For softwoods: Minimum End Distance for C_Δ = 0.5 is 3D = (3)(0.75") = 2.625"

Minimum End Distance for C_Δ = 1.0 is 7D = (7)(0.75") = 5.25"

Table 11.5.1C: Spacing Requirements for Fasteners in a Row

Direction of Loading is Parallel to Grain:

Minimum Spacing = 3D = (3)(0.75") = 2.25"

Minimum Spacing for C_Δ = 1.0 is 4D = (4)(0.75") = 3.0"

Table 11.5.1D: Spacing Requirements Between Rows

Direction of Loading is Parallel to Grain:

Minimum Spacing = 1.5D = (1.5)(0.75") = 1.125"

Spacing between outer rows of bolts ≤ 5"

Assuming that all bolt spacing, edge distances, and end distances meet the requirements for C_Δ = 1.0

Z' = (2801.4 lb)(C_g)(C_Δ) = (2801.4 lb)(C_g)(1.0) = 2801.4 lb(C_g)

# of bolts required = (51,549 lb)/(2801.4 lb/bolt) = 18.4 bolts ∴ try 20 bolts

Try (20) ¾" bolts arranged in (2) rows of ten each.

Check bolt capacity with group action:

Area of main member: A_m = (6.75")(8.25") = 55.69 in^2

Area of side plates, assuming ¼" x 6", is

A_s = (2)[(0.25")(6")] = 3.0 in^2

A_m/A_s = (55.69 in^2)/(3.0 in^2) = 18.5633

Table 10.3.6C (NDS): Group Action Factors, C_g, for Bolt or Lag Screw Connections with Steel Side Plates

(Tabulated group action factors (C_g) are conservative for D < 1" or s < 4")

For $A_m/A_s = 18$:

$A_m = 40$ in^2……..(10) fasteners per row……..$C_g = 0.80$

$A_m = 64$ in^2……..(10) fasteners per row……..$C_g = 0.86$

Interpolate for $A_m = 55.69$ in^2: $C_g = 0.8392$

For $A_m/A_s = 24$:

$A_m = 40$ in^2……..(10) fasteners per row……..$C_g = 0.79$

$A_m = 64$ in^2……..(10) fasteners per row……..$C_g = 0.85$

Interpolate for $A_m = 55.69$ in^2: $C_g = 0.8292$

Interpolate for $A_m/A_s = 18.5633$: $C_g = 0.8383$

Connection Capacity = (20 bolts)(2801.4 lb)(0.8383) = 46,968 lb < 51,549 lb ∴ N.G.

Try (22) ¾" bolts arranged in (2) rows of eleven each.

Table 10.3.6C (NDS): Group Action Factors, C_g, for Bolt or Lag Screw Connections with Steel Side Plates

(Tabulated group action factors (C_g) are conservative for D < 1" or s < 4")

For $A_m/A_s = 18$:

$A_m = 40$ in^2……..(11) fasteners per row……..$C_g = 0.77$

$A_m = 64$ in^2……..(11) fasteners per row……..$C_g = 0.83$

Interpolate for $A_m = 55.69$ in^2: $C_g = 0.8092$

For $A_m/A_s = 24$:

$A_m = 40$ in^2……..(11) fasteners per row……..$C_g = 0.76$

$A_m = 64$ in^2……..(11) fasteners per row……..$C_g = 0.83$

Interpolate for $A_m = 55.69$ in^2: $C_g = 0.8058$

Interpolate for $A_m/A_s = 18.5633$: $C_g = 0.8089$

Connection Capacity = (22 bolts)(2801.4 lb)(0.8089) = 49,853 lb < 51,549 lb ∴ N.G.

Try (24) ¾" bolts arranged in (2) rows of twelve each.

Table 10.3.6C (NDS): Group Action Factors, C_g, for Bolt or Lag Screw Connections with Steel Side Plates

(Tabulated group action factors (C_g) are conservative for D < 1" or s < 4")

For $A_m/A_s = 18$:

$A_m = 40\ in^2$……..(11) fasteners per row……..$C_g = 0.73$

$A_m = 64\ in^2$……..(11) fasteners per row……..$C_g = 0.81$

Interpolate for $A_m = 55.69\ in^2$: $C_g = 0.7823$

For $A_m/A_s = 24$:

$A_m = 40\ in^2$……..(11) fasteners per row……..$C_g = 0.72$

$A_m = 64\ in^2$……..(11) fasteners per row……..$C_g = 0.80$

Interpolate for $A_m = 55.69\ in^2$: $C_g = 0.7723$

Interpolate for $A_m/A_s = 18.5633$: $C_g = 0.7814$

Connection Capacity = (24 bolts)(2801.4 lb)(0.7814) = 52,536 lb > 51,549 lb ∴ O.K.

LOAD COMBINATION: D + L_r

P = 49,006 lb

$C_D = 1.0$

The allowable bolt design value is:

$Z' = (Z)(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_{eg})(C_{di})(C_{tn})$

$Z' = (3480\ lb)(1.0)(0.7)(1.0)(C_g)(C_\Delta)(1.0)(1.0)(1.0) = (2436\ lb)(C_g)(C_\Delta)$

Assuming that all bolt spacing, edge distances, and end distances meet the requirements for $C_\Delta = 1.0$

$Z' = (2436\ lb)(C_g)(C_\Delta) = (2436\ lb)(C_g)(1.0) = 2436\ lb(C_g)$

\# of bolts required = (49,006 lb)/(2436 lb/bolt) = 20.12 bolts ∴ try 22 bolts

Try (22) ¾" bolts arranged in (2) rows of eleven each.

$C_g = 0.8089$

Connection Capacity = (22 bolts)(2436 lb)(0.8089) = 43,351 lb < 49,006 lb ∴ N.G.

Try (24) ¾" bolts arranged in (2) rows of twelve each.

$C_g = 0.7814$

Connection Capacity = (24 bolts)(2436 lb)(0.7814) = 45,684 lb < 49,006 lb $\therefore$ N.G.

Try (26) ¾" bolts arranged in (2) rows of thirteen each.

Group Action Factor, C_g

$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$

n = number of fasteners in a row = 13

R_{EA} = lesser of $(E_sA_s)/(E_mA_m)$ or $(E_mA_m)/(E_sA_s)$

$E_s = 29{,}000{,}000$ psi

$A_s = 3.0\ in^2$

$E_m = 1{,}900{,}000$ psi

$A_m = 55.69\ in^2$

$(E_sA_s)/(E_mA_m) = [(29{,}000{,}000\ psi)(3.0\ in^2)]/[(1{,}900{,}000\ psi)(55.69\ in^2)]$

$= 0.8222$

$(E_mA_m)/(E_sA_s) = [(1{,}900{,}000\ psi)(55.69\ in^2)]/[(29{,}000{,}000\ psi)(3.0\ in^2)]$

$= 1.2162$

$\therefore R_{EA} = 0.8222$

s = 3"

$\gamma = (270{,}000)(D^{1.5}) = (270{,}000)(0.75)^{1.5} = 175{,}370.14$

$u = 1 + (\gamma)(s/2)[(1/(E_mA_m)) + (1/(E_sA_s))]$

$= 1 + (175{,}370.14)(3/2)[(1/(1{,}900{,}000)(55.69))+(1/(29{,}000{,}000)(3.0))]$

$= 1.005510$

$m = u - \sqrt{(u^2 - 1)} = 1.005510 - \sqrt{(1.005510^2 - 1)} = 0.90039$

$C_g = \{[(0.90039)(1 - (0.90039)^{2(13)})]/[(13)(1+(0.8222)(0.90039)^{13})(1+0.90039) - 1 +$

$+ (0.90039)^{2(13)})\}[(1+0.8222)/(1-0.90039)]$

= 0.8675

Connection Capacity = (26 bolts)(2436 lb)(0.8675) = 54,944 lb > 49,006 lb ∴ **O.K.**

Try (24) ¾" bolts arranged in (2) rows of twelve each using calculated C_g from equation.

Group Action Factor, C_g

$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$

n = number of fasteners in a row = 12

R_{EA} = 0.8222 (from previous)

s = 3"

γ = 175,370.14 (from previous)

u = 1.005510 (from previous)

m = 0.90039 (from previous)

$C_g = \{[(0.90039)(1 - (0.90039)^{2(12)})]/[(12)((1+(0.8222)(0.90039)^{12})(1+0.90039) - 1 +$

$+ (0.90039)^{2(12)})\}[(1+0.8222)/(1-0.90039)]$

= 0.8858

Connection Capacity = (24 bolts)(2436 lb)(0.8858) = 51,787 lb > 49,006 lb ∴ **O.K.**

Try 4-in-diameter shear plates with ¾" bolts.

For Southern Pine, the specific gravity G = 0.55

Table 12A: Species Group B (for $0.49 \leq G < 0.60$)

The capacity of a 4-in shear plate with steel side plates, ¾" bolt, using species group B, loaded parallel to grain per NDS Table 12.2B:

P = 4320 lb

Table 12.3: Geometry Factors, C_Δ, for Split Ring and Shear Plate Connectors

Edge Distance: Parallel to Grain Loading

Minimum for C_Δ = 1.0 is 2 ¾"

End Distance: Parallel to Grain Loading, Tension Member

Minimum for C_Δ = 1.0 is 7"

Spacing: Parallel to Grain Loading

Spacing Parallel to Grain:

Minimum for $C_\Delta = 1.0$ is 9”

Spacing Perpendicular to Grain:

Minimum for $C_\Delta = 1.0$ is 5”

Assuming that all bolt spacing, edge distances, and end distances meet the requirements for $C_\Delta = 1.0$

$C_{st} = 1.11$ (Table 12.2.4, Species Group B)

$P' = (P)(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_d)(C_{st})$

$= (4230\text{ lb})(1.0)(0.7)(1.0)(C_g)(1.0)(1.0)(1.11)$

$= (3286.71\text{ lb})(C_g)$

Number of shear plates required is:

(49,006 lb)/(3286.71 lb) = 14.91 = 15 shear plates

Due to excessive number of shear plates and required room for spacing of shear plates, use the (24) ¾” bolts for the connection.

Check Minimum End Distance for Steel Plates:

¾” bolts, ¼” steel plates (A36)

Assume end distance for steel plates = 1.5”

End bolts: L_c = 1.5” – (1/2)(3/4” + 1/16”) = 1.094” < 2d = (2)(0.75”) = 1.5”

∴ Tear-out Controls

$\phi r_n = \phi 1.2F_uL_ct$ = (0.75)(1.2)(58 ksi)(1.094”)(0.25”) = 14.273 k

Bolt Shear Strength: ϕr_n = 15.9 k (for single ¾” A325N bolts)

Interior Bolts: L_c = 3 – (3/4” + 1/16”) = 2.188” > 2d = 1.5”

∴ Bearing Controls

$\phi r_n = \phi 2.4dtF_u$ = (0.75)(2.4)(0.75”)(0.25”)(58 ksi) = 19.575 k

∴ Bolt shear strength controls for interior bolts.

ϕR_n = (2)(14.273 k) + (22)(15.9 k) = 378.346 k

P_u = 1.2D + 1.6S = (1.2)(24.616 k + 7.979 k) + (1.6)(18.954 k) = 69.440 k

P_u for each steel plate = (69.440 k)/2 = 34.720 k

ϕR_n = 378.346 k > P_u = 34.720 k ∴ **OK**

Block shear strength of steel plates is OK by inspection.

FINAL CONNECTION:

Use (24) ¾" bolts arranged in two rows of (12) each with ¼" steel side plates.

<u>Bottom Chord Splice Connections</u>

LOAD COMBINATION: D + L_r (controls)

Assume bottom chord is spliced at quarter points.

Maximum tension force at splice = 51,315 lb

Assume same steel side plates, spacing, and edge distances as used for the bottom chord heel connection.

(24) ¾" bolts arranged in (2) rows of twelve each will work (from previous calculations):

Connection Capacity = (24 bolts)(2436 lb)(0.8858) = 51,787 lb > 51,315 lb ∴ **O.K.**

Check Minimum End Distance for Steel Plates:

¾" bolts, ¼" steel plates (A36)

Assume end distance for steel plates = 1.5"

End bolts: L_c = 1.5" – (1/2)(3/4" + 1/16") = 1.094" < 2d = (2)(0.75") = 1.5"

∴ Tear-out Controls

$\phi r_n = \phi 1.2 F_u L_c t$ = (0.75)(1.2)(58 ksi)(1.094")(0.25") = 14.273 k

Bolt Shear Strength: ϕr_n = 15.9 k (for single ¾" A325N bolts)

Interior Bolts: L_c = 3 – (3/4" + 1/16") = 2.188" > 2d = 1.5"

∴ Bearing Controls

$\phi r_n = \phi 2.4 d t F_u$ = (0.75)(2.4)(0.75")(0.25")(58 ksi) = 19.575 k

∴ Bolt shear strength controls for interior bolts.

ϕR_n = (2)(14.273 k) + (22)(15.9 k) = 378.346 k

P_u = 1.2D + 1.6S = (1.2)(25.732 k + 8.428 k) + (1.6)(19.814 k) = 72.694 k

P_u for each steel plate = (72.694 k)/2 = 36.347 k

ϕR_n = 378.346 k > P_u = 36.347 k ∴ **OK**

Block shear strength of steel plates is OK by inspection.

FINAL CONNECTION:

Use (24) ¾" bolts arranged in two rows of (12) each with ¼" steel side plates.

Top Chord Member Connections

LOAD COMBINATON: D + L_r (controls)

P = 58,247 lb (compression)

C_D = 1.0

For 6 ¾" thick southern pine glulam member, wit h ¼" steel side plates, load applied parallel to grain, the nominal design value "Z" of a ¾" bolt in double shear is:

Z = 3460 lb (Table 11I, p. 90, NDS)

The allowable bolt design value is:

$Z' = (Z)(C_D)(C_M)(C_t)(C_g)(C_\Delta)(C_{eg})(C_{di})(C_{tn})$

Z' = (3480 lb)(1.0)(0.7)(1.0)$(C_g)(C_\Delta)$(1.0)(1.0)(1.0) = (2436 lb)$(C_g)(C_\Delta)$

Assuming that all bolt spacing, edge distances, and end distances meet the requirements for C_Δ = 1.0

Z' = (2436 lb)$(C_g)(C_\Delta)$ = (2436 lb)(C_g)(1.0) = 2436 lb(C_g)

# of bolts required = (58,247 lb)/(2436 lb/bolt) = 23.91 bolts ∴ try 24 bolts

Try (24) ¾" bolts arranged in (2) rows of twelve each.

Group Action Factor, C_g

$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$

n = number of fasteners in a row = 12

R_{EA} = lesser of $(E_sA_s)/(E_mA_m)$ or $(E_mA_m)/(E_sA_s)$

E_s = 29,000,000 psi

$A_s = (2)[(1/4”)(8”)] = 4.0\ in^2$

E_m = 1,900,000 psi

$A_m = 83.53\ in^2$

$(E_sA_s)/(E_mA_m) = [(29{,}000{,}000\ psi)(4.0\ in^2)]/[(1{,}900{,}000\ psi)(83.53\ in^2)]$

$= 0.7309$

$(E_mA_m)/(E_sA_s) = [(1{,}900{,}000\ psi)(83.53\ in^2)]/[(29{,}000{,}000\ psi)(4.0\ in^2)]$

$= 1.3682$

$\therefore R_{EA} = 0.7309$

s = 3”

$\gamma = (270{,}000)(D^{1.5}) = (270{,}000)(0.75)^{1.5} = 175{,}370.14$

$u = 1 + (\gamma)(s/2)[(1/(E_mA_m)) + (1/(E_sA_s))]$

$= 1 + (175{,}370.14)(3/2)[(1/(1{,}900{,}000)(83.53))+(1/(29{,}000{,}000)(4.0))]$

$= 1.003925$

$m = u - \sqrt{(u^2 - 1)} = 1.003925 - \sqrt{(1.003925^2 -1)} = 0.91524$

$C_g = \{[(0.91524)(1 - (0.91524)^{2(12)})]/[(12)((1+(0.7309)(0.91524)^{12})(1+0.91524) - 1 +$

$+ (0.91524)^{2(12)})\}[(1+0.7309)/(1-0.91524)]$

$= 0.9034$

Connection Capacity = (24 bolts)(2436 lb)(0.9034) = 52,816 lb < 58,247 lb $\therefore$ **N.G.**

Try (26) ¾” bolts arranged in (2) rows of thirteen each.

Group Action Factor, C_g

$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$

n = number of fasteners in a row = 13

R_{EA} = 0.7309 (from previous)

s = 3”

γ = 175,370.14 (from previous)

u = 1.003925 (from previous)

m = 0.91524 (from previous)

$$C_g = \{[(0.91524)(1 - (0.91524)^{2(13)})]/[(13)((1+(0.7309)(0.91524)^{13})(1+0.91524) - 1 + (0.91524)^{2(13)})\}[(1+0.7309)/(1-0.91524)]$$

$= 0.8876$

Connection Capacity = (26 bolts)(2436 lb)(0.8876) = 56,217 lb < 58,247 lb ∴ **N.G.**

Try (28) ¾” bolts arranged in (2) rows of fourteen each.

Group Action Factor, C_g

$$C_g = \{[(m)(1-m^{2n})]/[(n)((1+R_{EA}m^n)(1+m) - 1 + m^{2n})]\}[(1+R_{EA})/(1-m)]$$

n = number of fasteners in a row = 14

R_{EA} = 0.7309 (from previous)

s = 3”

γ = 175,370.14 (from previous)

u = 1.003925 (from previous)

m = 0.91524 (from previous)

$$C_g = \{[(0.91524)(1 - (0.91524)^{2(14)})]/[(14)((1+(0.7309)(0.91524)^{14})(1+0.91524) - 1 + (0.91524)^{2(14)})\}[(1+0.7309)/(1-0.91524)] = 0.8712$$

Connection Capacity = (28 bolts)(2436 lb)(0.8712) = 59,423 lb > 58,247 lb ∴ **O.K.**

FINAL CONNECTION:

Use (28) ¾” bolts arranged in two rows of (14) each with ¼” steel side plates.

Appendix B – Structural Depth: Lateral System Calculations

Wind Calculations

Method 2 – Analytical Procedure

Building Natural Frequency = n_1

For concrete moment-resisting frames: $n_1 = 43.5/H^{0.9}$

H = building height = 60'

$n_1 = (43.5)/((60)^{0.9}) = 43.5/39.842 = 1.092 > 1$ Hz therefore $\therefore$ Structure is rigid

*Building and Other Structure, Flexible: Slender buildings and other structures that have a fundamental natural frequency less than 1 Hz (p. 21).

$g_Q = g_v = 3.4$

$z = 0.6h = (0.6)(60') = 36' > z_{min} = 15'$ (Table 6-2, Exposure C)

Use maximum roof height for "h" (most conservative) instead of trying to estimate mean roof height of curved roof.

$I_z = c[(33/z)^{1/6}] = (0.20)[(33/36)^{1/6}] = 0.1971$

c = 0.20 (Table 6-2, Exposure C)

$L_z = l(z/33)^{\epsilon} = (500')(36/33)^{0.20} = 508.7773$

l = 500' (Table 6-2, Exposure C)

$\in = 1/5.0 = 0.20$ (Table 6-2, Exposure C)

$Q = \sqrt{[1/(1 + 0.63((B+h)/L_z)^{0.63})]}$

North/South:

B = 183'

L = 156'

$Q_{N/S} = \sqrt{[1/(1 + 0.63((183'+36')/508.777')^{0.63})]} = 0.9272$

East/West:

B = 156'

L = 183'

$Q_{E/W} = \sqrt{[1/(1 + 0.63((156'+36')/508.777')^{0.63})]} = 0.8636$

G = 0.85 or

$G = 0.925[(1 + 1.7g_QI_zQ)/(1 + 1.7g_vI_z)]$

North/South:

$G_{N/S} = 0.925[(1 + 1.7g_QI_zQ_{N/S})/(1 + 1.7g_vI_z)]$

$= 0.925[(1 + [(1.7)(3.4)(36)(0.9272)]/(1 + 1.7(3.4)(36))] = 0.8579848361$

$\therefore$ use $G_{N/S} = 0.8580$

East/West:

$G_{E/W} = 0.925[(1 + 1.7g_QI_zQ_{E/W})/(1 + 1.7g_vI_z)]$

$= 0.925[(1 + [(1.7)(3.4)(36)(0.8636)]/(1 + 1.7(3.4)(36))] = 0.7994$

$\therefore$ use $G_{E/W} = 0.85$

Velocity Pressure:

V = 90 m.p.h. (Figure 6-1)

$K_d = 0.85$ (Table 6-4)

I = 1.15 (Table 6-1, Occupancy Category III)

Exposure Category = C

$K_{zt} = 1.0$ (ASCE 7-05, 6.5.7.2)

Level	Height	K_z
1	10.50'	0.85
2	24.67'	0.937
3	40.00'	1.04
4	60.00'	1.13

(Values of K_z from Table 6-2, Exposure C)

$K_h = 1.13$ (using maximum roof height to be conservative)

$q_z = 0.00256K_zK_{zt}K_dV^2I$

Level 1: $q_z = (0.00256)(0.85)(1.0)(0.85)(90^2)(1.15) = 17.2290$ psf

Level 2: $q_z = (0.00256)(0.937)(1.0)(0.85)(90^2)(1.15) = 18.9992$ psf

Level 3: $q_z = (0.00256)(1.04)(1.0)(0.85)(90^2)(1.15) =$ 21.0802 psf

Level 4: $q_z = (0.00256)(1.13)(1.0)(0.85)(90^2)(1.15) =$ 22.9045 psf
$= q_h =$ 22.9045 psf

Pressure Coefficients, C_p, for the Walls and Roof (Figure 6-6):

Wall Pressure Coefficients, C_p

North/South:

Windward Wall: $C_p = 0.8$

Leeward Wall: C_p = L/B = 156'/183' = 0.852 ∴ $C_p = -0.5$

Side Wall: $C_p = -0.7$

East/West:

Windward Wall: $C_p = 0.8$

Leeward Wall: C_p = L/B = 183'/156' = 1.173 ∴ $C_p = -0.4654$

Side Wall: $C_p = -0.7$

Roof Pressure Coefficients, C_p, for use with q_h

Since roof slope, θ, for curved roof is less than 10° for most of the roof, use "Normal to ridge for <10 and Parallel to ridge for all θ."

North/South:

h/L = 60'/156' = 0.3846

Horizontal Distance from Windward Edge	C_p
0 to h/2	-0.9, -0.18
h/2 to	-0.9, -0.18
h to 2h	-0.5, -0.18
>2h	-0.3, -0.18

Use worst case scenario: $C_p = -0.9$ for entire roof

East/West:

h/L = 60'/183' = 0.3279

Same chart (above, for North/South) applies

Use worst case scenario: $C_p = -0.9$ for entire roof

Or use "Arched Roofs", Figure 6-8, ASCE 7-05

Rise-to-Span Ratio: r = 20'/130' = 0.1538 < 0.2

∴ C_p for Windward Quarter = -0.9

C_p for Center Half = -0.7 – r = -0.7 – 0.1538 = -0.8538

C_p for Leeward Quarter = -0.5

Conservatively use C_p = -0.9 for entire roof

Internal Pressure Coefficients (GC_{pi}) (Figure 6-5):

Enclosed Buildings: GC_{pi} = +0.18
= -0.18

Design Wind Pressures:

Windward Walls: $p_z = q_zGC_p - q_i(GC_{pi})$

However, internal pressures cancel on MLFRS

∴ $p_z = q_zGC_p$

Leeward Walls, Side Walls, and Roofs: $p_h = q_hGC_p - q_i(GC_{pi})$

However, internal pressures cancel on MLFRS

∴ $p_h = q_hGC_p$

North/South:

Windward Walls:

$p_z = q_zGC_p = (q_z)(0.858)(0.8) = 0.6864(q_z)$

(Varies by level, see Table)

Leeward Walls:

$p_h = q_hGC_p = (21.080)(0.858)(-0.5) = -9.0433$ psf

Side Walls:

$p_h = q_hGC_p = (21.080)(0.858)(-0.7) = -12.6606$ psf

Roof:

$p_h = q_hGC_p = (21.080)(0.858)(-0.9) = -16.2779$ psf

East/West:

Windward Walls:

$p_z = q_zGC_p = (q_z)(0.85)(0.8) = 0.68(q_z)$

(Varies by level, see Table)

Leeward Walls:

$p_h = q_hGC_p = (21.080)(0.85)(-0.4654) = -8.3391$ psf

Side Walls:

$p_h = q_hGC_p = (21.080)(0.85)(-0.7) = -12.5427$ psf

Roof:

$p_h = q_hGC_p = (21.080)(0.85)(-0.9) = -16.1264$ psf

*Forces, base shear, and moments are shown in spreadsheets

<u>Wind Forces for Lateral Force Resisting System:</u>

W = Wind Load

North/South: "Building 1"

Level 1:

W = (11.83 PSF + 9.04 PSF)(742.7109 SF) + (13.04 PSF + 9.04 PSF)(1002.0703 SF) =

= 37,626.09 lb = 37.626 kips

Level 2:

W = (13.04 PSF + 9.04 PSF)(1002.0703 SF) + (14.47 PSF + 9.04 PSF)(1034.8958 SF) =

= 46,456.11 lb = 46.456 kips

Level 3:

W = (14.47 PSF + 9.04 PSF)(996.6667 SF) + (15.72 PSF + 9.04 PSF)(1746.6029 SF) =

= 66,677.52 lb = 66.678 kips

OR if only looking at Level 2 and Level 3 for wind loads for "Building 1":

Level 2:

W = (13.04 PSF + 9.04 PSF)(1744.7813 SF) + (14.47 PSF + 9.04 PSF)(1034.8958 SF) =

= 62,855.17 lb = 62.855 kips

Level 3:

W = (14.47 PSF + 9.04 PSF)(996.6667 SF) + (15.72 PSF + 9.04 PSF)(1746.6029 SF) =

= 66,667.52 lb = 66.678 kips

North/South: "Building 4"

Level 2:

W = (13.04 PSF + 9.04 PSF)(499.8854 SF) + (14.47 PSF + 9.04 PSF)(135.1042 SF) =

= 14,213.77 lb = 14.214 kips

East/West:

Level 1:

W = (11.72 PSF + 8.34 PSF)(920.9375 SF) + (12.92 PSF + 8.34 PSF)(1242.5347 SF) =

= 44,890.29 lb = 44.890 kips

Level 2:

W = (12.92 PSF + 8.34 PSF)(1153.4239 SF) + (14.33 PSF + 8.34 PSF)(1189.5000 SF) =

= 51,487.76 lb = 51.488 kips

Level 3:

W = (14.33 PSF + 8.34 PSF)(1184.5000 SF) = 26.852 kips

Seismic Calculations

Equivalent Lateral Force Procedure

S_S = 0.20 (Figure 22-1, ASCE 7-05) (Also from www.seismicfactor.com)

S_1 = 0.054 (Figure 22-1, ASCE 7-05) (Also from www.seismicfactor.com)

Occupancy Category III, Site Class C

F_a = 1.2 (Table 11.4-1) ($S_S \leq 0.25$, Site Class C)

F_v = 1.7 (Table 11.4-2) ($S_1 \leq 0.1$, Site Class C)

$S_{MS} = F_aS_S = (1.2)(0.20) = 0.24$ (Eq. 11.4-1)

$S_{M1} = F_vS_1 = (1.7)(0.054) = 0.0918$ (Eq. 11.4-2)

$S_{DS} = (2/3)(S_{MS}) = (2/3)(0.24) = 0.16$ (Eq. 11.4-3)

$S_{D1} = (2/3)(S_{M1}) = (2/3)(0.0918) = 0.0612$ (Eq. 11.4-4)

Seismic Design Category based on S_{DS} (Table 11.6-1):

$S_{DS} = 0.16 < 0.167$, Occupancy Category III: SDC A

Seismic Design Category based on S_{D1}:

$S_{D1} = 0.0612 < 0.067$, Occupancy Category III: SDC A

Use most severe of the two Seismic Design Categories: (same in this case)

Seismic Design Category A

Could use methods of 11.7 “Design Requirements for Seismic Design Category A” (Lateral Forces: $F_x = 0.01w_x$) but continue to solve for C_s instead.

For Wood Braced Frames:

R = 4 (Table 12.2-1) (Light-framed wall systems using flat strap bracing)

I = 1.25 (Table 11.5-1) (Occupancy Category III)

$T_a = C_th_n^x$

C_t = 0.02 (Table 12.8-2)

h_n = 60’

x = 0.75 (Table 12.8-2)

$T_a = (0.02)(60')^{0.75} = 0.4312$

T_L = 6 seconds (Figure 22-15)

$T = T_a = 0.4312$ (this is allowed per Section 12.8.2, ASCE 7-05)

$< C_u T_a = (1.7)(0.4312) = 0.7330$

C_s = minimum of

$S_{DS}/(R/I) = 0.16/(4/1.25) = 0.05$

$S_{D1}/[(T)(R/I)] = 0.0612/[(0.4312)(4/1.25)] = 0.044353$

$\mathbf{C_s = 0.044353}$

For Concrete Moment Frames:

R = 3 (Table 12.2-1) (Ordinary reinforced concrete moment frames)

I = 1.25 (Table 11.5-1) (Occupancy Category III)

$T_a = C_t h_n^x$

$C_t = 0.016$ (Table 12.8-2)

$h_n = 60'$

x = 0.9 (Table 12.8-2)

$T_a = (0.016)(60')^{0.9} = 0.6375$

T_L = 6 seconds (Figure 22-15)

$T = T_a = 0.4312$ (this is allowed per Section 12.8.2, ASCE 7-05)

$< C_u T_a = (1.7)(0.6375) = 1.0837$

C_s = minimum of

$S_{DS}/(R/I) = 0.16/(3/1.25) = 0.066667$

$S_{D1}/[(T)(R/I)] = 0.0612/[(0.6375)(3/1.25)] = 0.040002$

$\mathbf{C_s = 0.040002}$

Use $\mathbf{C_s = 0.044353}$ for entire building (worst case)

$V = C_s W$ (see spreadsheets for weights of building components, seismic forces, and story shears)

Stiffness Values

The stiffness of each frame at each applicable level was determined by applying a 1 kip load to the frame at that particular level and determining the displacement of the frame at that level. SAP was used to determine the displacements. The stiffness is equal to the 1 kip load divided by the displacement.

$$k = P/\Delta$$

Stiffness Values (k-values) - North/South Direction				
	Level	P (kips)	Deflection (in.)	k = P/Defl. (kip/in)
Braced Frame - Column Line 1	1	1	0.010448	95.712
Braced Frame - Column Line 1	2	1	0.032685	30.595
Braced Frame - Column Line 1	3	1	0.077295	12.937
Moment Frame - Column Line 1.8	1	1	0.002836	352.609
Moment Frame - Column Line 2	2	1	0.006298	158.781
Moment Frame - Column Line 2	3	1	0.014274	70.057
Moment Frame - Column Line 4	2	1	0.046756	21.388

Table ____ - Stiffness Values for Wood Braced Frames, Concrete Moment Frames, and Steel Moment Frame – North/South Direction

Stiffness Values (k-values) - East/West Direction				
	Level	P (kips)	Deflection (in.)	k = P/Defl. (kip/in)
Concrete Moment Frame	1	1	0.014789	67.618
Concrete Moment Frame	2	1	0.017769	56.278
Concrete Moment Frame	3	1	0.108563	9.211
Wood Braced Frame	1	1	0.002595	385.356
Wood Braced Frame	2	1	0.007476	133.761
Wood Braced Frame	3	1	0.015516	64.450

Table ____ - Stiffness Values for Concrete Moment Frames – East/West Direction

Center of Mass

The center of mass at each level was determined by hand. Tributary areas were used for building elements that did not exactly line up with a level or that crossed over several levels. The reference point used for the center of mass was the Southwest corner of the façade of the building. Center of mass values for each level are found in Tables ____ - ____ below. Calculations for the center of mass at each level are found in Appendix ____.

Center of Mass x = {∑[(weight)(x)]}/∑weight

Center of Mass y = {∑[(weight)(y)]}/∑weight

Center of Mass - Entire Building - Level 1			
	Weight (kips)	Center of Mass	
		x (ft)	y (ft)
Building 1 - Level 1	496.085	31.6634	80.7836
Building 2 - Level 1	404.340	112.6943	78.0000
Building 3 - Level 1	1089.540	125.7531	78.2569
TOTAL=	1989.965	99.6438	78.8346

Table ____ - Center of Mass of Entire Building at Level 1

Center of Mass - Entire Building - Level 2			
	Weight (kips)	Center of Mass	
		x (ft)	y (ft)
Building 1 - Level 2	740.563	55.8277	80.1876
Building 2 - Level 2	329.779	124.6779	75.2708
Building 4 - Level 2	760.650	151.5494	75.1941
TOTAL=	1830.992	107.9940	77.2276

Table ____ - Center of Mass of Entire Building at Level 2

Center of Mass - Entire Building - Level 3			
	Weight (kips)	Center of Mass	
		x (ft)	y (ft)
Building 1 - Level 3	593.006	52.7936	78.0000
TOTAL=	593.006	52.7936	78.0000

Table ____ - Center of Mass of Entire Building at Level 3

Center of Rigidity

The center of rigidity was calculated for each level using the stiffness values of the frames that contribute to that level. The reference point used for the center of rigidity was the Southwest corner of the façade of the building (the same as that used for the center of mass). The center of rigidity at each level for the North/South direction is found in Tables ____-____, and the center of rigidity for the East/West direction is found in Tables ____-____ below. Table ____ shows the overall center of rigidity at each level.

Center of Rigidity (x) = [sum($k_{iy}x_i$)]/[sum(k_{iy})]

Center of Rigidity - North/South Direction - Entire Building - Level 1					
	k_{iy}	x_i (ft)	Quantity	($k_{iy}x_i$)	Center of Rigidity x (ft)
Braced Frames - Column Line 1	95.712	1.1510	10	1101.6850	
Moment Frame - Column Line 1.8	352.609	111.9010	1	39457.3144	
TOTAL=	1309.729		TOTAL=	40558.9994	30.9675

Table ____ - Center of Rigidity for North/South Direction – Level 1

Center of Rigidity - North/South Direction - Entire Building - Level 2					
	k_{iy}	x_i (ft)	Quantity	($k_{iy}x_i$)	Center of Rigidity x (ft)
Braced Frames - Column Line 1	30.595	1.1510	10	352.1612	
Moment Frame - Column Line 2	158.781	130.3177	1	20691.9760	
Moment Frame - Column Line 4	21.388	171.6510	1	3671.2089	
TOTAL=	486.119		TOTAL=	24715.3461	50.8422

Table ____ - Center of Rigidity for North/South Direction – Level 2

Center of Rigidity - North/South Direction - Entire Building - Level 3					
	k_{iy}	x_i (ft)	Quantity	($k_{iy}x_i$)	Center of Rigidity x (ft)
Braced Frames - Column Line 1	12.937	1.1510	10	148.9103	
Moment Frame - Column Line 2	70.057	130.3177	1	9129.6677	
TOTAL=	199.427		TOTAL=	9278.5780	46.5262

Table ____ - Center of Rigidity for North/South Direction – Level 3

Center of Rigidity (y) = [sum($k_{ix}y_i$)]/[sum(k_{ix})]

Center of Rigidity - East/West Direction - Entire Building - Level 1					
	k_{ix}	$_{yi}$ (ft)	Quantity	(k_{ixyi})	Center of Rigidity y (ft)
Concrete Moment Frame	67.618	18.0000	1	1217.1208	
Concrete Moment Frame	67.618	48.0000	1	3245.6556	
Concrete Moment Frame	67.618	78.0000	1	5274.1903	
Concrete Moment Frame	67.618	108.0000	1	7302.7250	
Concrete Moment Frame	67.618	138.0000	1	9331.2597	
Wood Braced Frame	385.357	4.2500	2	3275.5303	
Wood Braced Frame	385.357	151.7500	2	116955.6978	
TOTAL=	1879.515		TOTAL=	146602.1794	78.0000

Table ____ - Center of Rigidity for East/Direction Direction – Level 1

Center of Rigidity - East/West Direction - Entire Building - Level 2					
	k_{ix}	y_i (ft)	Quantity	($k_{ix}y_i$)	Center of Rigidity y (ft)
Concrete Moment Frame	56.278	18.0000	1	1013.0002	
Concrete Moment Frame	56.278	48.0000	1	2701.3338	
Concrete Moment Frame	56.278	78.0000	1	4389.6674	
Concrete Moment Frame	56.278	108.0000	1	6078.0010	
Concrete Moment Frame	56.278	138.0000	1	7766.3346	
Wood Braced Frame	133.761	4.2500	2	1136.9719	
Wood Braced Frame	133.761	151.7500	2	40596.5849	
TOTAL=	816.435		TOTAL=	63681.8938	78.0000

Table ____ - Center of Rigidity for East/West Direction – Level 2

Center of Rigidity - East/West Direction - Entire Building - Level 3					
	k_{ix}	y_i (ft)	Quantity	($k_{ix}y_i$)	Center of Rigidity y (ft)
Concrete Moment Frame	9.211	18.0000	1	165.8023	
Concrete Moment Frame	9.211	48.0000	1	442.1396	
Concrete Moment Frame	9.211	78.0000	1	718.4768	
Concrete Moment Frame	9.211	108.0000	1	994.8141	
Concrete Moment Frame	9.211	138.0000	1	1271.1513	
Wood Braced Frame	64.450	4.2500	2	547.8216	
Wood Braced Frame	64.450	151.7500	2	19560.4536	
TOTAL=	303.855		TOTAL=	23700.6593	78.0000

Table ____ - Center of Rigidity for East/West Direction – Level 3

Center of Rigidity - Entire Building		
Level	Center of Rigidity x (ft)	Center of Rigidity y (ft)
1	30.9675	78.0000
2	50.8422	78.0000
3	46.5262	78.0000

Table ____ - Center of Rigidity for Entire Building at Each Level

Direct Shear

The direct shear values for each lateral force resisting frame and each level were calculated by hand and are found in Tables _____ - _____ below. Calculations for direct shear are found in Appendix _____. Direct shear values in the North/South direction for "Building 1" were based on tributary area since the wood roof diaphragm is considered to be a flexible diaphragm.

$$\text{Direct Load: } F_{iy} = [(k_{iy}/\sum k_{iy})](P_y)$$

Due to Seismic Loads:

1.2D + 1.0E + L + 0.2S

North/South Direction:

Direct Shear - North/South Direction - "Building 1"							
			Distributed Force (kips)				
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Braced Frame - Column Line 1 - Level 1	Braced Frame - Column Line 1 - Level 2	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 2 - Level 3
Level 1	8.96	8.96	0.90				
Level 2	31.41	31.41		1.57		15.71	
Level 3	40.79	40.79			2.04		20.40

Table _____ - Direct Shear Values due to Seismic Loads for "Building 1" (North/South)
*Assuming flexible diaphragm for "Building 1"
*Based on 10 braced frames at Column Line 1

Direct Shear - North/South Direction - "Building 2"				
			Distributed Force (kips)	
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Moment Frame - Column Line 1.8 - Level 1	Moment Frame - Column Line 2 - Level 2
Level 1	11.17	11.17	11.17	
Level 2	21.39	21.39		21.39

Table _____ - Direct Shear Values due to Seismic Loads for "Building 2" (North/South)

Direct Shear - North/South Direction - "Building 3"			
			Distributed Force (kips)
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Moment Frame - Column Line 1.8 - Level 1
Level 1	48.32	48.32	48.32

Table _____ - Direct Shear Values due to Seismic Loads for "Building 3" (North/South)

Direct Shear - North/South Direction - "Building 4"				
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)	
			Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2
Level 2	33.74	33.74	29.73	4.01

Table _____ - Direct Shear Values due to Seismic Loads for "Building 4" (North/South)

Total Direct Shear - North/South Direction							
Load Combination = 1.2D+1.0E+L+0.2S	Distributed Force (kips)						
	Braced Frame - Column Line 1 - Level 1	Braced Frame - Column Line 1 - Level 2	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 1.8 - Level 1	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 2 - Level 3	Moment Frame - Column Line 4 - Level 2
Level 1	0.90			59.49			
Level 2		1.57			66.83		4.01
Level 3			2.04			20.40	

Table _____ - Total Direct Shear Values due to Seismic Loads (North/South)

East/West Direction:

Total Direct Shear - East/West Direction					
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)		
			Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	68.45	68.45	14.04	12.64	0.26
Level 2	86.54	86.54	17.75	14.81	0.92
Level 3	40.79	40.79	8.37	5.46	1.19

Table ____ - Total Direct Shear Values due to Seismic Loads (East/West)

Due to Wind Loads:

1.2D + 1.6W + L + 0.5(Lr or S or R)

North/South Direction:

Direct Shear - North/South Direction - "Building 1"							
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 1	Braced Frame - Column Line 1 - Level 2	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 2 - Level 3
Level 1	37.63	60.21	6.02				
Level 2	46.46	74.34		3.72		37.17	
Level 3	66.68	106.69			5.33		53.34

Table ____ - Direct Shear Values due to Wind Loads for "Building 1" (North/South)

Direct Shear - North/South Direction - "Building 4"				
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)	
			Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2
Level 2	14.10	22.56	19.88	2.68

Table ____ - Direct Shear Values due to Wind Loads for "Building 4" (North/South)

Total Direct Shear - North/South Direction						
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Distributed Force (kips)					
	Braced Frame - Column Line 1 - Level 1	Braced Frame - Column Line 1 - Level 2	Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 2 - Level 3	Moment Frame - Column Line 4 - Level 2
Level 1	6.02					
Level 2		3.72		57.05		2.68
Level 3			5.33		53.34	

Table ____ - Total Direct Shear Values due to Wind Loads (North/South)

East/West Direction:

Total Direct Shear - East/West Direction					
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)		
			Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	44.89	71.82	14.73	9.61	2.10
Level 2	51.49	82.38	16.90	11.02	2.41
Level 3	26.85	42.96	8.81	5.75	1.26

Table ____ - Total Direct Shear Values due to Wind Loads (East/West)

Direct Shear Calculations:

Based on Seismic Load:

"Building 1" seismic loads are distributed to the lateral force resisting frames based on tributary area. "Building 4" seismic loads are distributed to the lateral force resisting frames based on the relative stiffness of each frame.

Direct Shear – North/South Direction – "Building 4"

Moment Frame – Column Line 2 – Level 2

F = [158.781/(158.781+21.388)][33.74 k] = **29.7347 k**

Moment Frame – Column Line 4 – Level 2

F = [21.388/(158.781+21.388)][33.74 k] = **4.0053 k**

Direct Shear – East/West Direction

Tributary Width of Moment Frames:

Inside Frames: 32.0'

Outer Frames: 16.0' + 4.875' = 20.875'

Tributary Width of Wood Braced Frames (2 of 4) = 4.875 + 4.25’ = 9.125’

Total Width = 156’

For Level 1: Assume that the 8.96 k load from “Building 1” is distributed to all lateral force resisting frames in the East/West direction. Assume that the 11.17 k load from “Building 2” and the 48.32 k from “Building 3” are taken only by the concrete moment frames.

Inside Moment Frame – Level 1

F_{BLDG1} = [32.0/156][8.96 k] = 1.8379 k

$F_{BLDG2,3}$ = [32.0/156][11.17 k + 48.32 k] = 12.2031 k

F_{TOTAL} = 1.8379 k + 12.2031 k = **14.0410 k**

Outer Moment Frame – Level 1

F_{BLDG1} = [20.875/156][8.96 k] = 1.1990 k

$F_{BLDG2,3}$ = [(11.17 k + 48.32 k) - (3)(12.2031 k)]/2 = 11.4404 k

F_{TOTAL} = 1.1990 k + 11.4404 k = **12.6394 k**

Wood Braced Frame (2 of 4) – Level 1

F = [9.125/156][8.96 k] = 0.5241 k

Each Wood Braced Frame: F = (0.5241 k)/2 = **0.2621 k**

For Level 2: Assume that the 31.41 k load from “Building 1” is distributed to all lateral force resisting frames in the East/West direction. Assume that the 21.39 k load from “Building 2” and the 33.74 k load from “Building 4” are taken only by the concrete moment frames.

Inside Moment Frame – Level 2

F_{BLDG1} = [32.0/156][31.41 k] = 6.4431 k

$F_{BLDG2,4}$ = [32.0/156][21.39 k + 33.74 k] = 11.3087 k

F_{TOTAL} = 6.4431 k + 11.3087 k = **17.7518 k**

Outer Moment Frame – Level 2

F_{BLDG1} = [20.875/156][31.41 k] = 4.2031 k

$F_{BLDG2,4}$ = [(21.39 k + 33.74 k) - (3)(11.3087 k)]/2 = 10.6020 k

F_{TOTAL} = 4.2031 k + 10.6020 k = **14.8051 k**

Wood Braced Frame (2 of 4) – Level 1

F = [9.125/156][31.41 k] = 1.8373 k

Each Wood Braced Frame: F = (1.8373 k)/2 = **0.9186 k**

For Level 3: Assume that the 40.79 k load from "Building 1" is distributed to all lateral force resisting frames in the East/West direction.

Inside Moment Frame – Level 3

F_{BLDG1} = [32.0/156][40.79 k] = **8.3672 k**

Outer Moment Frame – Level 3

F_{BLDG1} = [20.875/156][40.79 k] = **5.4583 k**

Wood Braced Frame (2 of 4) – Level 1

F = [9.125/156][40.79 k] = 2.3860 k

Each Wood Braced Frame: F = (2.3860 k)/2 = **1.1930 k**

Based on Wind Load:

Direct Shear – North/South Direction – "Building 4"(Factored Load)

Moment Frame – Column Line 2 – Level 2

F = [158.781/(158.781+21.388)][22.56 k] = **19.8819 k**

Moment Frame – Column Line 4 – Level 2

F = [21.388/(158.781+21.388)][22.56 k] = **2.6781 k**

Direct Shear – North/South Direction – "Building 4"(Unfactored Load)

Moment Frame – Column Line 2 – Level 2

F = [158.781/(158.781+21.388)][14.10 k] = **12.4262 k**

Moment Frame – Column Line 4 – Level 2

F = [21.388/(158.781+21.388)][14.10 k] = **1.6738 k**

Direct Shear – East/West Direction (Factored Load)

Tributary Width of Moment Frames:

Inside Frames: 32.0’

Outer Frames: 16.0’ + 4.875’ = 20.875’

Tributary Width of Wood Braced Frames (2 of 4) = 4.875 + 4.25’ = 9.125’

Total Width = 156’

Inside Moment Frame – Level 1

F = [32.0/156][71.82 k] = **14.7323 k**

Outer Moment Frame – Level 1

F = [20.875/156][71.82 k] = **9.6105 k**

Wood Braced Frame (2 of 4) – Level 1

F = [9.125/156][71.82 k] = 4.2010 k

Each Wood Braced Frame: F = (4.2010 k)/2 = **2.1005 k**

Inside Moment Frame – Level 2

F = [32.0/156][82.38 k] = **16.8985 k**

Outer Moment Frame – Level 2

F = [20.875/156][82.38 k] = **11.0236 k**

Wood Braced Frame (2 of 4) – Level 2

F = [9.125/156][82.38 k] = 4.8187 k

Each Wood Braced Frame: F = (4.8187 k)/2 = **2.4094 k**

Inside Moment Frame – Level 3

F = [32.0/156][42.96 k] = **8.8123 k**

Outer Moment Frame – Level 3

F = [20.875/156][42.96 k] = **5.7487 k**

Wood Braced Frame (2 of 4) – Level 3

F = [9.125/156][42.96 k] = 2.5129 k

Each Wood Braced Frame: F = (2.5129 k)/2 = **1.2564 k**

Torsional Shear

The torsional shear values for each lateral force resisting frame and each level were calculated by hand and are found in Tables ____-____ below. Rather than breaking up the building into the four different "buildings" as was done when determining the direct shear values, torsional shear values due to loads in the North/South direction were calculated looking at the entire building at each level. Torsional shear values due to wind loads were determined for both Wind Load Cases 1 and 2. Wind Load Case 1 just looks at the total wind load in one direction. Wind Load Case 2 used (0.75)(wind load) but adds in an eccentricity of (0.15)(building width). Wind Load Case 1 was found to control over Wind Load Case 2. Torsional shear due to loads in the East/West direction were neglected since the center of mass and center of rigidity are located at the same point or within one foot of each other in that direction. Plus, the five concrete frames in the East/West direction are evenly spaced at 32'-0" apart and are centered on the center of the building in the East/West direction. Therefore, it was assumed that torsional shear values in this direction would be negligible. Torsional shear due to eccentricities from Wind Load Case 2 was also neglected and assumed not to control for the East/West direction. Calculations for torsional shear are found in Appendix ____.

$$\text{Torsional Shear: } F_{it} = [(k_i)(d_i)(P_y)(e_x)]/[\sum((k_j)(d_j)^2)]$$

Due to Seismic Loads:

1.2D + 1.0E + L + 0.2S

Torsional Shear - North/South Direction - Level 1							
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 1	Moment Frame - Column Line 1.8 - Level 1	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	68.45	68.45	1.10	10.96	0.83	1.66	10.92

Table ____ - Torsional Shear Values due to Seismic Loads for Level 1 (North/South)

Torsional Shear - North/South Direction - Level 2								
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)					
			Braced Frame - Column Line 1 - Level 2	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 2	86.54	86.54	1.35	11.23	2.30	1.60	3.21	8.78

Table ____ - Torsional Shear Values due to Seismic Loads for Level 2 (North/South)

Torsional Shear - North/South Direction - Level 3							
Load Combination = 1.2D+1.0E+L+0.2S	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 3	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 3	40.79	40.79	0.07	0.67	0.03	0.07	0.54

Table ____ - Torsional Shear Values due to Seismic Loads for Level 3 (North/South)

Due to Wind Loads:

1.2D + 1.6W + L + 0.5(L_r or S or R)

Load Case 1:

Torsional Shear - North/South Direction - Level 1							
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 1	Moment Frame - Column Line 1.8 - Level 1	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	37.63	60.21	0.49	4.94	0.37	0.75	4.92

Table ____ - Torsional Shear Values due to Wind Load Case 1 for Level 1 (North/South)

Torsional Shear - North/South Direction - Level 2								
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)					
			Braced Frame - Column Line 1 - Level 2	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 2	60.67	97.07	0.95	7.85	1.61	1.12	2.24	6.14

Table ____ - Torsional Shear Values due to Wind Load Case 1 for Level 2 (North/South)

Torsional Shear - North/South Direction - Level 3							
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 3	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 3	66.68	106.69	0.55	5.45	0.27	0.55	4.41

Table ____ - Torsional Shear Values due to Wind Load Case 1 for Level 3 (North/South)

Load Case 2:

Torsional Shear - North/South Direction - Level 1							
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 1	Moment Frame - Column Line 1.8 - Level 1	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 1	28.22	45.15	0.64	6.44	0.49	0.98	6.41

Table ____ - Torsional Shear Values due to Wind Load Case 2 for Level 1 (North/South)

Torsional Shear - North/South Direction - Level 2								
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)					
			Braced Frame - Column Line 1 - Level 2	Moment Frame - Column Line 2 - Level 2	Moment Frame - Column Line 4 - Level 2	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 2	45.50	72.80	1.23	10.17	2.08	1.45	2.90	7.95

Table ____ - Torsional Shear Values due to Wind Load Case 2 for Level 2 (North/South)

Torsional Shear - North/South Direction - Level 3							
Load Combination = 1.2D+1.6W+L+0.5(Lr or S or R)	Force (k)	Factored Force (k)	Distributed Force (kips)				
			Braced Frame - Column Line 1 - Level 3	Moment Frame - Column Line 2 - Level 3	Inside Concrete Moment Frame (1 of 3)	Outer Concrete Moment Frame (1 of 2)	Wood Braced Frame (1 of 4)
Level 3	50.01	80.02	0.95	9.49	0.48	0.95	7.68

Table ____ - Torsional Shear Values due to Wind Load Case 2 for Level 3 (North/South)

Torsional Load Calculations

Torsional Load: $F_{it} = [(k_i)(d_i)(P_y)(e_x)]/[\sum((k_j)(d_j)^2)]$

For torsional loads, the entire building was analyzed per level instead of using "Buildings 1, 2, 3, and 4". The results can be seen below.

<u>North/South Direction:</u>

Level 1: Seismic Load (unfactored)

e_x = 99.6438' – 30.9675' = 68.6763'

P_y = 8.96 k + 11.17 k + 48.32 k = 68.45 k

$\sum k_j d_j^2 = (10)(95.712)(29.8165')^2 + (352.609)(80.9335')^2 + (2)(67.618)(32')^2 + (2)(67.618)(64')^2 + (4)(385.357)(73.75')^2 = 12{,}236{,}893.56$

Braced Frame (column line 1):
F_{it} = (95.712 k/in)(29.8165')(68.45 k)(68.6763')/12,236,893.56 = **1.0963 k**

Moment Frame (column line 1.8):
F_{it} = (352.609 k/in)(80.9335')(68.45 k)(68.6763')/12,236,893.56 = **10.9630 k**

Inside Moment Frames (column lines D and F):
F_{it} = (67.618 k/in)(32')(68.45 k)(68.6763')/12,236,893.56 = **0.8312 k**

Outer Moment Frames (column lines C and G):
F_{it} = (67.618 k/in)(64')(68.45 k)(68.6763')/12,236,893.56 = **1.6625 k**

Braced Frames (East/West Direction):
F_{it} = (385.357 k/in)(73.75')(68.45 k)(68.6763')/12,236,893.56 = **10.9178 k**

Level 2: Seismic Load (unfactored)

e_x = 107.9940' – 50.8422' = 57.1518'

P_y = 31.41 k + 21.39 k + 33.74 k = 86.54 k

$\sum k_j d_j^2 = (10)(30.595)(49.6912')^2 + (158.781)(79.4755')^2 + (21.388)(120.8088')^2 + (2)(56.278)(32')^2 + (2)(56.278)(64')^2 + (4)(133.761)(73.75)^2 = 5{,}556{,}958.898$

Braced Frame (column line 1):

F_{it} = (30.595 k/in)(49.6912')(86.54 k)(57.1518')/5,556,958.898 = **1.3531 k**

Moment Frame (column line 2):
F_{it} = (158.781 k/in)(79.4755')(86.54 k)(57.1518')/5,556,958.898 = **11.2316 k**

Moment Frame (column line 4):
F_{it} = (21.388 k/in)(120.8088')(86.54 k)(57.1518')/5,556,958.898 = **2.2997 k**

Inside Moment Frames (column lines D and F):
F_{it} = (56.278 k/in)(32')(86.54 k)(57.1518')/5,556,958.898 = **1.6029 k**

Outer Moment Frames (column lines C and G):
F_{it} = (56.278 k/in)(64')(86.54 k)(57.1518')/5,556,958.898 = **3.2057 k**

Braced Frames (East/West Direction):
F_{it} = (133.761 k/in)(73.75')(86.54 k)(57.1518')/5,556,958.898 = **8.7802 k**

Level 3: Seismic Load (unfactored)

e_x = 52.7936' – 46.5262' = 6.2674'

P_y = 40.79 k

$\sum k_j d_j^2$ = (10)(12.937)(45.3752')^2 + (70.057)(83.7915')^2 + (2)(9.211)(32')^2 + (2)(9.211)(64')^2 + (4)(64.450)(73.75')^2 = 2,254,734.207

Braced Frame (column line 1):
F_{it} = (12.937 k/in)(45.3752')(40.79 k)(6.2674')/2,254,734.207 = **0.06656 k**

Moment Frame (column line 2):
F_{it} = (70.057 k/in)(83.7915')(40.79 k)(6.2674')/2,254,734.207 = **0.6656 k**

Inside Moment Frames (column lines D and F):
F_{it} = (9.211 k/in)(32')(40.79 k)(6.2674')/2,254,734.207 = **0.03342 k**

Outer Moment Frames (column lines C and G):
F_{it} = (9.211 k/in)(64')(40.79 k)(6.2674')/2,254,734.207 = **0.06684 k**

Braced Frames (East/West Direction):
F_{it} = (64.450 k/in)(73.75')(40.79 k)(6.2674')/2,254,734.207 = **0.5389 k**

Level 1: Wind Load (Unfactored) – Load Case 1

e_x = 66.1510' – 30.9675' = 35.1835'

P_y = 37.63 k

$\sum k_j d_j^2$ = (10)(95.712)(29.8165')^2 + (352.609)(80.9335')^2 + (2)(67.618)(32')^2 + (2)(67.618)(64')^2 + (4)(385.357)(73.75')^2 = 12,236,893.56

Braced Frame (column line 1):
F_{it} = (95.712 k/in)(29.8165')(37.63 k)(35.1835')/12,236,893.56 = **0.3088 k**

Moment Frame (column line 1.8):
F_{it} = (352.609 k/in)(80.9335')(37.63 k)(35.1835')/12,236,893.56 = **3.0876 k**

Inside Moment Frames (column lines D and F):
F_{it} = (67.618 k/in)(32')(37.63 k)(35.1835')/12,236,893.56 = **0.2341 k**

Outer Moment Frames (column lines C and G):
F_{it} = (67.618 k/in)(64')(37.63 k)(35.1835')/12,236,893.56 = **0.4682 k**

Braced Frames (East/West Direction):
F_{it} = (385.357 k/in)(73.75')(37.63 k)(35.1835')/12,236,893.56 = **3.0749 k**

Level 2: Wind Load (Unfactored) – Load Case 1

e_x = 86.4479' – 50.8422' = 35.6057'

P_y = 46.46 k + 14.21 k = 60.67 k

$\sum k_j d_j^2$ = (10)(30.595)(49.6912')^2 + (158.781)(79.4755')^2 + (21.388)(120.8088')^2 + (2)(56.278)(32')^2 + (2)(56.278)(64')^2 + (4)(133.761)(73.75)^2 = 5,556,958.898

Braced Frame (column line 1):
F_{it} = (30.595 k/in)(49.6912')(60.67 k)(35.6057')/5,556,958.898 = **0.5910 k**

Moment Frame (column line 2):
F_{it} = (158.781 k/in)(79.4755')(60.67 k)(35.6057')/5,556,958.898 = **4.9056 k**

Moment Frame (column line 4):
F_{it} = (21.388 k/in)(120.8088')(60.67 k)(35.6057')/5,556,958.898 = **1.0044 k**

Inside Moment Frames (column lines D and F):
F_{it} = (56.278 k/in)(32')(60.67 k)(35.6057')/5,556,958.898 = **0.7001 k**

Outer Moment Frames (column lines C and G):
F_{it} = (56.278 k/in)(64')(60.67 k)(35.6057')/5,556,958.898 = **1.4002 k**

Braced Frames (East/West Direction):
F_{it} = (133.761 k/in)(73.75')(60.67 k)(35.6057')/5,556,958.898 = **3.8349 k**

Level 3: Wind Load (Unfactored) – Load Case 1

e_x = 66.1510' – 46.5262' = 19.6248'

P_y = 66.68 k

$\sum k_j d_j^2$ = (10)(12.937)(45.3752')^2 + (70.057)(83.7915')^2 + (2)(9.211)(32')^2 + (2)(9.211)(64')^2 + (4)(64.450)(73.75')^2 = 2,254,734.207

Braced Frame (column line 1):
F_{it} = (12.937 k/in)(45.3752')(66.68 k)(19.6248')/2,254,734.207 = **0.3407 k**

Moment Frame (column line 2):
F_{it} = (70.057 k/in)(83.7915')(66.68 k)(19.6248')/2,254,734.207 = **3.4069 k**

Inside Moment Frames (column lines D and F):
F_{it} = (9.211 k/in)(32')(68.68 k)(19.6248')/2,254,734.207 = **0.1711 k**

Outer Moment Frames (column lines C and G):
F_{it} = (9.211 k/in)(64')(66.68 k)(19.6248')/2,254,734.207 = **0.3421 k**

Braced Frames (East/West Direction):
F_{it} = (64.450 k/in)(73.75')(66.68 k)(19.6248')/2,254,734.207 = **2.7586 k**

Load Case 2: Multiply loads by 0.75 and use an eccentricity of $0.15b_x$

Level 1: Wind Load (Unfactored) – Load Case 2

e_x = 35.1835' + (0.15)(172.8958') = 61.1179'

P_y = (0.75)(37.63 k) = 28.22 k

$\sum k_j d_j^2 = (10)(95.712)(29.8165')^2 + (352.609)(80.9335')^2 + (2)(67.618)(32')^2 + (2)(67.618)(64')^2 + (4)(385.357)(73.75')^2 = 12{,}236{,}893.56$

Braced Frame (column line 1):
F_{it} = (95.712 k/in)(29.8165')(28.22 k)(61.1179')/12,236,893.56 = **0.4022 k**

Moment Frame (column line 1.8):
F_{it} = (352.609 k/in)(80.9335')(28.22 k)(61.1179')/12,236,893.56 = **4.0223 k**

Inside Moment Frames (column lines D and F):
F_{it} = (67.618 k/in)(32')(28.22 k)(61.1179')/12,236,893.56 = **0.3050 k**

Outer Moment Frames (column lines C and G):
F_{it} = (67.618 k/in)(64')(28.22 k)(61.1179')/12,236,893.56 = **0.6100 k**

Braced Frames (East/West Direction):
F_{it} = (385.357 k/in)(73.75')(28.22 k)(61.1179')/12,236,893.56 = **4.0057 k**

Level 2: Wind Load (Unfactored) – Load Case 2

e_x = 35.6057' + (0.15)(172.8958') = 61.5401'

P_y = (0.75)(60.67 k) = 45.50 k

$\sum k_j d_j^2 = (10)(30.595)(49.6912')^2 + (158.781)(79.4755')^2 + (21.388)(120.8088')^2 + (2)(56.278)(32')^2 + (2)(56.278)(64')^2 + (4)(133.761)(73.75)^2 = 5{,}556{,}958.898$

Braced Frame (column line 1):
F_{it} = (30.595 k/in)(49.6912')(45.50 k)(61.5401')/5,556,958.898 = **0.7661 k**

Moment Frame (column line 2):
F_{it} = (158.781 k/in)(79.4755')(45.50 k)(61.5401')/5,556,958.898 = **6.3586 k**

Moment Frame (column line 4):
F_{it} = (21.388 k/in)(120.8088')(45.50 k)(61.5401')/5,556,958.898 = **1.3020 k**

Inside Moment Frames (column lines D and F):
F_{it} = (56.278 k/in)(32')(45.50 k)(61.5401')/5,556,958.898 = **0.9074 k**

Outer Moment Frames (column lines C and G):
F_{it} = (56.278 k/in)(64')(45.50 k)(61.5401')/5,556,958.898 = **1.8149 k**

Braced Frames (East/West Direction):
F_{it} = (133.761 k/in)(73.75')(45.50 k)(61.5401')/5,556,958.898 = **4.9708 k**

Level 3: Wind Load (Unfactored) – Load Case 2

e_x = 19.6248' + (0.15)(172.8958') = 45.5592'

P_y = (0.75)(66.68 k) = 50.01 k

$\sum k_j d_j^2$ = (10)(12.937)(45.3752')^2 + (70.057)(83.7915')^2 + (2)(9.211)(32')^2 + (2)(9.211)(64')^2 + (4)(64.450)(73.75')^2 = 2,254,734.207

Braced Frame (column line 1):
F_{it} = (12.937 k/in)(45.3752')(50.01 k)(45.5592')/2,254,734.207 = **0.5932 k**

Moment Frame (column line 2):
F_{it} = (70.057 k/in)(83.7915')(50.01 k)(45.5592')/2,254,734.207 = **5.9318 k**

Inside Moment Frames (column lines D and F):
F_{it} = (9.211 k/in)(32')(50.01 k)(45.5592')/2,254,734.207 = **0.2978 k**

Outer Moment Frames (column lines C and G):
F_{it} = (9.211 k/in)(64')(50.01 k)(45.5592')/2,254,734.207 = **0.5957 k**

Braced Frames (East/West Direction):
F_{it} = (64.450 k/in)(73.75')(50.01 k)(45.5592')/2,254,734.207 = **4.8031 k**

East/West Direction:

Torsional effects were not accounted for in the East/West direction since the center of mass and center of rigidity either match up perfectly in the y-direction for each floor level or were only off by less than one foot. Hence, for seismic loads the eccentricity would be zero or very close to zero. Similarly, Wind Load Case 1 was not considered since the wind load would basically be applied at the center of the building in the East/West direction, which lines up with the center of rigidity in the East/West direction. Therefore, this case would also produce little or no eccentricity. Wind Load Case 2 was not considered for the East/West direction either because it was assumed that any small torsional effects would not control in this direction. The five moment frames and four braced frames in the East/West direction are centered on the building and spaced symmetrically on both sides of the building, so torsional effects should be minimal in this direction.

Total Shear

Total shear values were determined by combining the direct shear at each frame and level with the torsional shear at each frame and level. Torsional shear was either added or subtracted to the direct shear depending on which side of the center of rigidity the frames were located and which side of the center of rigidity the load was applied.

$$F_i = F_{i,direct} +/- F_{i,torsion}$$

Due to Seismic Loads:

North/South Direction:

Total Shear - North/South Direction - Braced Frame at Column Line 1			
Load Combination = 1.2D+1.0E+L+0.2S	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	0.90	-1.10	-0.20
Level 2	1.57	-1.35	0.22
Level 3	2.04	-0.07	1.97

Table ____ - Total Shear Values due to Seismic Loads for Braced Frame at Column Line 1 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 2			
Load Combination = 1.2D+1.0E+L+0.2S	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 2	66.83	11.23	78.06
Level 3	20.40	0.67	21.07

Table ____ - Total Shear Values due to Seismic Loads for Moment Frame at Column Line 2 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 1.8			
Load Combination = 1.2D+1.0E+L+0.2S	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	59.49	10.96	70.45

Table ____ - Total Shear Values due to Seismic Loads for Moment Frame at Column Line 1.8 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 4			
Load Combination = 1.2D+1.0E+L+0.2S	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 2	4.01	2.30	6.31

Table ____ - Total Shear Values due to Seismic Loads for Moment Frame at Column Line 4 (North/South)

East/West Direction:

Total Shear - East/West Direction - Inside Concrete Moment Frame			
Load Combination = 1.2D+1.0E+L+0.2S	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	14.04	0.83	14.87
Level 2	17.75	1.60	19.35
Level 3	8.37	0.03	8.40

Table ____ - Total Shear Values due to Seismic Loads for Inside Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Outer Concrete Moment Frame			
Load Combination = 1.2D+1.0E+L+0.2S	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	12.64	1.66	14.30
Level 2	14.81	3.21	18.02
Level 3	5.46	0.07	5.53

Table ____ - Total Shear Values due to Seismic Loads for Outer Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Wood Braced Frame			
Load Combination = 1.2D+1.0E+L+0.2S	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	0.26	10.92	11.18
Level 2	0.92	8.78	9.70
Level 3	1.19	0.54	1.73

Table ____ - Total Shear Values due to Seismic Loads for Wood Braced Frame (East/West)

Due to Wind Loads:

Load Case 1:

North/South Direction

Total Shear - North/South Direction - Braced Frame at Column Line 1			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	6.02	-0.49	5.53
Level 2	3.72	-0.95	2.77
Level 3	5.33	-0.55	4.78

Table ____ - Total Shear Values due to Wind Load Case 1 for Braced Frame at Column Line 1 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 2			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 2	57.05	7.85	64.90
Level 3	53.34	5.45	58.79

Table ____ - Total Shear Values due to Wind Load Case 1 for Moment Frame at Column Line 2 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 4			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 2	2.68	1.61	4.29

Table ____ - Total Shear Values due to Wind Load Case 1 for Moment Frame at Column Line 4 (North/South)

East/West Direction:

Total Shear - East/West Direction - Inside Concrete Moment Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	14.73	0.37	15.10
Level 2	16.90	1.12	18.02
Level 3	8.81	0.27	9.08

Table ____ - Total Shear Values due to Wind Load Case 1 for Inside Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Outer Concrete Moment Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	9.61	0.75	10.36
Level 2	11.02	2.24	13.26
Level 3	5.75	0.55	6.30

Table ____ - Total Shear Values due to Wind Load Case 1 for Outer Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Wood Braced Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	2.10	4.92	7.02
Level 2	2.41	6.14	8.55
Level 3	1.26	4.41	5.67

Table ____ - Total Shear Values due to Wind Load Case 1 for Wood Braced Frame (East/West)

Load Case 2:

North/South Direction:

Total Shear - North/South Direction - Braced Frame at Column Line 1			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 1	4.52	-0.64	3.88
Level 2	2.79	-1.23	1.56
Level 3	4.00	-0.95	3.05

Table ____ - Total Shear Values due to Wind Load Case 2 for Braced Frame at Column Line 1 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 2			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 2	42.79	10.17	52.96
Level 3	40.01	9.49	49.50

Table ____ - Total Shear Values due to Wind Load Case 2 for Moment Frame at Column Line 2 (North/South)

Total Shear - North/South Direction - Moment Frame at Column Line 4			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	Factored Direct Shear Force (k)	Factored Torsional Shear Force (k)	Total Factored Shear (k)
Level 2	2.01	2.08	4.09

Table ____ - Total Shear Values due to Wind Load Case 2 for Moment Frame at Column Line 4 (North/South)

East/West Direction:

Total Shear - East/West Direction - Inside Concrete Moment Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	11.05	0.49	11.54
Level 2	12.68	1.45	14.13
Level 3	6.61	0.48	7.09

Table ____ - Total Shear Values due to Wind Load Case 2 for Inside Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Outer Concrete Moment Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	7.21	0.98	8.19
Level 2	8.27	2.90	11.17
Level 3	4.31	0.95	5.26

Table ____ - Total Shear Values due to Wind Load Case 2 for Outer Concrete Moment Frame (East/West)

Total Shear - East/West Direction - Wood Braced Frame			
Load Combination = 1.2D+1.6W+L+0.5 (Lr or S or R)	**Factored Direct Shear Force (k)**	**Factored Torsional Shear Force (k)**	**Total Factored Shear (k)**
Level 1	1.58	6.41	7.99
Level 2	1.81	7.95	9.76
Level 3	0.95	4.80	5.75

Table ____ - Total Shear Values due to Wind Load Case 2 for Wood Braced Frame (East/West)

Drift and Displacement

Drift and displacement values were determined for each frame at each applicable level by applying the total forces due to direct loads and torsional loads to the SAP models of each frame. Drifts due to seismic loads were multiplied by a C_d factor of 3 ½ and divided by an importance factor of 1.25. Since two different seismic force-resisting systems were considered for the natatorium, the worst case C_d factor was used. For the wood braced frames, a C_d factor of 3 ½ applies to light-framed wall systems using flat strap bracing. For the concrete moment frames, a C_d factor of 2 ½ applies to ordinary reinforced concrete moment frames. Therefore, a C_d factor of 3 ½ was conservatively assumed to apply to all frames. This value was then compared to $0.015h_{sx}$ for each story, where h_{sx} is the story height below Level x. All frames met the seismic load drift limits.

For drift due to seismic loads:

$\Delta_x = (C_d)(\Delta_{xe})/I$

C_d = 3 ½ (Light-framed wall systems using flat strap bracing)

I = 1.25

Table 12.12.1 (ASCE 7-05):

Allowable Story Drift = $0.015h_{sx}$ (all other structures, Occupancy Category III)

Drifts due to unfactored wind loads were compared to an allowable limit of H/400, with H being the elevation height of the level, or with H being the story height.

North/South Direction:

Story Drifts - North/South Direction - Braced Frame at Column Line 1					
Unfactored Seismic	Deflection (in)	$Defl._x = (C_d * Defl._{xe})/I$	Story Height (ft)	Limit = $0.015h_{sx}$ (in)	
Level 1	0.0203	0.0569	13.33	2.4000	OK
Level 2	0.0053	0.0148	13.33	2.4000	OK
Level 3	0.0015	0.0042	13.33	2.4000	OK

Table ____ - Story Drifts due to Seismic Loads for Braced Frame at Column Line 1 (North/South)

Deflections - North/South Direction - Braced Frame at Column Line 1				
Unfactored Wind	Deflection from SAP (in)	Elevation (ft)	Limit =H/400 (in)	
Level 1	0.1270	13.33	0.4000	OK
Level 2	0.2764	26.67	0.8000	OK
Level 3	0.4236	40.00	1.2000	OK

Table ____ - Deflections due to Wind Loads for Braced Frame at Column Line 1 (North/South)

Story Drifts - North/South Direction - Braced Frame at Column Line 1				
Unfactored Wind	Deflection (in)	Story Height (ft)	Limit =H/400 (in)	
Level 1	0.1270	13.33	0.4000	OK
Level 2	0.1495	13.33	0.4000	OK
Level 3	0.1471	13.33	0.4000	OK

Table ____ - Story Drifts due to Wind Loads for Braced Frame at Column Line 1 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 2					
Unfactored Seismic	Deflection from SAP (in)	$Defl._x = (C_d*Defl._{xe})/I$	Story Height (ft)	Limit = $0.015h_{sx}$ (in)	
Level 2	0.6591	1.8455	22.50	4.0500	OK
Level 3	0.2621	0.7339	17.50	3.1500	OK

Table ____ - Story Drifts due to Seismic Loads for Moment Frame at Column Line 2 (North/South)

Deflections - North/South Direction - Moment Frame at Column Line 2				
Unfactored Wind	Deflection from SAP (in)	Elevation (ft)	Limit =H/400 (in)	
Level 2	0.5475	22.50	0.6750	OK
Level 3	0.8469	40.00	1.2000	OK

Table ____ - Deflections due to Wind Loads for Moment Frame at Column Line 2 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 2				
Unfactored Wind	Deflection from SAP (in)	Elevation (ft)	Limit =H/400 (in)	
Level 2	0.5475	22.50	0.6750	OK
Level 3	0.2994	17.50	0.5250	OK

Table ____ - Story Drifts due to Wind Loads for Moment Frame at Column Line 2 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 1.8					
Unfactored Seismic	Deflection from SAP (in)	$Defl._x = (C_d*Defl._{xe})/I$	Elevation (ft)	Limit = $0.015h_{sx}$ (in)	
Level 1	0.0624	0.1748	10.50	1.8900	OK

Table ____ - Story Drifts due to Seismic Loads for Moment Frame at Column Line 1.8 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 4					
Unfactored Seismic	Deflection from SAP (in)	$Defl._x = (C_d*Defl._{xe})/I$	Elevation (ft)	Limit = $0.015h_{sx}$ (in)	
Level 2	0.2950	0.8261	24.67	4.4400	OK

Table ____ - Story Drifts due to Seismic Loads for Moment Frame at Column Line 4 (North/South)

Deflections - North/South Direction - Moment Frame at Column Line 4				
Unfactored Wind	Deflection from SAP (in)	Elevation (ft)	Limit =H/400 (in)	
Level 2	0.1253	24.67	0.7400	OK

Table ____ - Deflections due to Wind Loads for Moment Frame at Column Line 4 (North/South)

Story Drifts - North/South Direction - Moment Frame at Column Line 4				
Unfactored Wind	Deflection from SAP (in)	Elevation (ft)	Limit =H/400 (in)	
Level 2	0.1253	24.67	0.7400	OK

Table ____ - Story Drifts due to Wind Loads for Moment Frame at Column Line 4 (North/South)

East/West Direction:

Story Drifts - East/West Direction - Concrete Moment Frame					
Unfactored Seismic	Deflection (in)	$Defl._x = (C_d*Defl._{xe})/I$	Story Height (ft)	Limit = $0.015h_{sx}$ (in)	
Level 1	0.2298	0.6434	10.50	1.8900	OK
Level 2	-0.0011	-0.0030	12.00	2.1600	OK
Level 3	0.6772	1.8963	17.50	3.1500	OK

Table ____ - Story Drifts due to Seismic Loads for Moment Frame (East/West)

Deflections - East/West Direction - Concrete Moment Frame				
Unfactored Wind	**Deflection from SAP (in)**	**Elevation (ft)**	**Limit =L/400 (in)**	
Level 1	0.1434	10.50	0.3150	OK
Level 2	0.1420	22.50	0.6750	OK
Level 3	0.5964	40.00	1.2000	OK

Table ____ - Deflections due to Wind Loads for Moment Frame (East/West)

Story Drifts - East/West Direction - Concrete Moment Frame				
Unfactored Wind	**Deflection (in)**	**Story Height (ft)**	**Limit =L/400 (in)**	
Level 1	0.1434	10.50	0.3150	OK
Level 2	-0.0014	12.00	0.3600	OK
Level 3	0.4543	17.50	0.5250	OK

Table ____ - Story Drifts due to Wind Loads for Moment Frame (East/West)

Story Drifts - East/West Direction - Braced Frame					
Unfactored Seismic	**Deflection (in)**	**Defl.$_x$ = (C_d*Defl.$_{xe}$)/I**	**Story Height (ft)**	**Limit = 0.015h_{sx} (in)**	
Level 1	0.0733	0.2052	13.33	2.4000	OK
Level 2	0.0595	0.1666	13.33	2.4000	OK
Level 3	0.0367	0.1028	13.33	2.4000	OK

Table ____ - Story Drifts due to Seismic Loads for Braced Frame (East/West)

Deflections - East/West Direction - Braced Frame				
Unfactored Wind	**Deflection from SAP (in)**	**Elevation (ft)**	**Limit =H/400 (in)**	
Level 1	0.0875	13.33	0.4000	OK
Level 2	0.1719	26.67	0.8000	OK
Level 3	0.2325	40.00	1.2000	OK

Table ____ - Deflections due to Wind Loads for Braced Frame (East/West)

Story Drifts - East/West Direction - Braced Frame				
Unfactored Wind	**Deflection (in)**	**Story Height (ft)**	**Limit =H/400 (in)**	
Level 1	0.0875	13.33	0.4000	OK
Level 2	0.0844	13.33	0.4000	OK
Level 3	0.0606	13.33	0.4000	OK

Table ____ - Story Drifts due to Wind Loads for Braced Frame (East/West)

Wood Braced Frame – Column Line 1

Design of Diagonal Members:

Controlling Load Combination: D + 0.75W + 0.75S

D + 0.75W + 0.75S = 6.391 k + (0.75)(9.291 k) + (0.75)(5.015 k) = 17.121 k (compression)

Analyze Member Buckling About x Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_x = [(1.0)(15.5492')(12\ in/ft)]/d \leq 50$

$d \geq l_e/50 = [(15.5492')(12\ in/ft)]/50 = 3.73''$

Analyze Member Bucking About y Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_y = [(1.0)(7.7746')(12\ in/ft)]/d \leq 50$

$d \geq l_e/50 = [(7.7746')(12\ in/ft)]/50 = 1.87''$

Try 3 ½” x 5 ½”

$(l_e/d)_x = [(15.5492)(12\ in/ft)]/5.5'' = 33.9255$

$(l_e/d)_y = [(7.7746')(12\ in/ft)]/3.5'' = 26.6558$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.6 (for wind load))

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\ psi)]/[(33.9255)^2] = 583.029$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\ psi)(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 583.029/2686.4 = 0.2170$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.2170]/[(2)(0.9)] = 0.6761$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6761\} - \sqrt{\{[0.6761]^2 - [0.2170/0.9]\}}$

$= 0.2113$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.2113) = 567.641 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 17{,}121 \text{ lb}/567.641 \text{ psi} = 30.16 \text{ in}^2 > A_{provided} = 19.25 \text{ in}^2 \therefore$ **N.G.**

Try 3 ½” x 6 7/8”

$(l_e/d)_x = [(15.5492)(12 \text{ in/ft})]/6.875” = 27.1404$

$(l_e/d)_y = [(7.7746’)(12 \text{ in/ft})]/3.5” = 26.6558$

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(27.1404)^2] = 910.982 \text{ psi}$

$F_{cE}/F_c^* = 910.982/2686.4 = 0.3391$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.3391]/[(2)(0.9)] = 0.7439$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7439\} - \sqrt{\{[0.7439]^2 - [0.3391/0.9]\}}$

$= 0.3236$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.3236) = 869.221 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 17{,}121 \text{ lb}/869.221 \text{ psi} = 19.70 \text{ in}^2 < A_{provided} = 24.06 \text{ in}^2 \therefore$ **OK**

Use 3 ½” x 6 7/8” for all diagonal members

Concrete Moment Frame – Column Line 1.8

Beams

*Use rebar cover of 1.5(1.5") = 2.25" due to corrosive environment (natatorium) (see ACI 7.7.6.1)

Design beams as a continuous beam.
Design beams for worst case and make all four beams the same size.

Shear and Moment (Unfactored) for Column Line 1.8 (24x24 Columns and 24x26 Beams)

	Beam 2	Beam 4	Beam 6	Beam 8	Column 1 (Exterior Column)	Column 9 (Exterior Column)	Column 7 (Interior Column)
V_D (Top or Left)	-30.38	-31.95	-31.76	-33.31	-18.93	-19.28	1.71
V_D (Bottom or Right)	33.37	31.81	32.00	30.44	-18.93	-19.28	1.71
V_L (Top or Left)	-28.96	-30.45	-30.27	-31.75	-18.04	-18.38	1.62
V_L (Bottom or Right)	31.81	30.32	30.50	29.02	-18.04	-18.38	1.62
V_E (Top or Left)	2.25	1.83	1.75	1.94	13.25	-11.13	-14.78
V_E (Bottom or Right)	2.25	1.83	1.75	1.94	13.25	-11.13	-14.78
$V_{E,REVERSED}$ (Top or Left)	-1.94	-1.75	-1.83	-2.25	-11.13	13.25	16.26
$V_{E,REVERSED}$ (Bottom or Right)	-1.94	-1.75	-1.83	-2.25	-11.13	13.25	16.26
M_D (Top or Left)	-137.17	-171.67	-168.68	-184.05	137.17	-138.17	11.57
M_D (Bottom or Right)	-184.95	-169.40	-172.48	-138.17	-61.62	64.25	-6.37
M_L (Top or Left)	-130.71	-163.60	-160.66	-175.40	130.71	-131.72	10.99
M_L (Bottom or Right)	-176.31	-161.48	-164.41	-131.72	-58.61	61.30	-6.01
M_E (Top or Left)	38.11	29.42	28.31	29.75	-38.11	84.46	97.75
M_E (Bottom or Right)	-33.88	-29.16	-27.71	-32.40	101.00	-32.40	-57.47
$M_{E,REVERSED}$ (Top or Left)	-32.40	-27.71	-29.16	-33.88	32.40	-101.00	-107.38
$M_{E,REVERSED}$ (Bottom or Right)	29.75	28.31	29.42	38.11	-84.46	38.11	63.30
P_D					-30.38	-30.44	-65.32
P_L					-28.96	-29.02	-62.25
P_E					2.25	-1.94	0.19
$P_{E,REVERSED}$					-1.94	2.25	-0.42
M_D (Midspan)	93.96	84.49	84.49	93.91			
M_L (Midspan)	89.56	80.53	80.53	89.51			
M_E (Midspan)	2.12	0.13	0.30	-1.33			
$M_{E,REVERSED}$ (Midspan)	-1.33	0.30	0.13	2.12			
1.2D +/- 1.0E + 1.0L							
Max $V_{TOP/LEFT}$ (kips)	-67.36	-70.54	-70.21	-73.97	-51.89	-52.65	19.93
Max $V_{BOTTOM/RIGHT}$ (kips)	74.10	70.32	70.65	67.49	-51.89	-52.65	19.93
Max $M_{TOP/LEFT}$ (ft-kips)	-327.72	-397.32	-392.24	-430.14	327.72	**-398.52**	122.62
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-432.13	-393.92	-399.10	-329.93	-217.02	176.51	-71.12
Max $M_{MIDSPAN}$ (ft-kips)	204.43	182.21	182.21	204.32			
Max P_u (kips)					-67.36	-67.49	-141.05
1.2D + 1.6L							
Max $V_{TOP/LEFT}$ (kips)	-82.79	-87.06	-86.54	-90.77	-51.58	-52.54	4.64
Max $V_{BOTTOM/RIGHT}$ (kips)	**90.94**	86.68	87.20	82.96	-51.58	-52.54	4.64
Max $M_{TOP/LEFT}$ (ft-kips)	-373.74	-467.76	-459.47	-501.50	373.74	-376.56	31.47
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	**-504.04**	-461.65	-470.03	-376.56	-167.72	175.18	-17.26
Max $M_{MIDSPAN}$ (ft-kips)	**256.05**	230.24	230.24	255.91			
Max P_u (kips)					-82.80	-82.96	**-177.98**

Tables Account for Torsional Effects

BEAM DESIGN:

$V_{u,max}$ = 90.94 kips (1.2D + 1.6L)
$M_{u,max}$ at Supports = 504.04 k-ft (1.2D + 1.6L)
$M_{u,max}$ at Midspan = 256.05 k-ft (1.2D + 1.6L)

Use normal-weight concrete with f'_c = 4000 psi
f_y = 60,000 psi for flexural reinforcement
f_{yt} = 60,000 psi for stirrups

1) Choose the actual size of the beam stem.

a) Calculate the minimum depth based on deflections.

Use worst case scenario (one-end continuous instead of both ends continuous).

ACI Table 9.5(a):

Minimum thickness, h = L/18.5 = [(32')(12 in/ft)]/18.5 = 20.76"

b) Determine the minimum depth based on the maximum negative moment.

$M_{u,max}$ at Supports = 504.04 k-ft

$\rho(\text{initial}) = [(\beta_1 f'_c)/(4f_y)] = [(0.85)(4 \text{ ksi})/(4)(60 \text{ ksi})] = 0.0142$

$\omega = \rho(f_y/f'_c) = (0.0142)(60 \text{ ksi}/4 \text{ ksi}) = 0.213$

$R = \omega f'c(1 - 0.59\omega) = (0.213)(4 \text{ ksi})[1 - (0.59)(0.213)] = 0.745 \text{ ksi}$

$bd^2 \geq M_u/\phi R = [(504.04 \text{ ft-kips})(12 \text{ in/ft})]/[(0.9)(0.745 \text{ ksi})] = 9020.85 \text{ in}^3$

Assuming b = 24 in.

$d \geq 19.39$ in.

h ≅ 19.39" + 3.25" = 22.64" (accounting for 2.25" clear cover due to corrosive environment; see ACI 7.7.6.1; (1.5)(1.5") = 2.25")

Try h = 26" > 20.76" ∴ Meets deflection criteria

d ≅ 26" – 3.25" = 22.75"

c) Check the shear capacity of the beam.

$V_u = \phi(V_c + V_s)$

$V_{u,max}$ = 90.94 kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24")(22.75")/1000 = 69.06 kips

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$V_s = 8\sqrt{f'_c}b_wd$ = (8) √4000 psi (24")(22.75")/1000 = 276.26 kips

Thus, the absolute maximum ϕV_n = 0.75(69.06 k + 276.26 k) = 258.99 kips

≥ $V_{u,max}$ = 90.94 kips ∴ OK

d) Summary. Use:

b = 24"
h = 26"
d = 22.75"

2) Compute the dead load of the stem, and recompute the total moment.

Weight of 24"x26" concrete beam = [(24")(26")/144 in^2/ft^2][(150 lb/ft^3)/1000]

= 0.650 k/ft

Original dead load = 1.9923 k/ft

New dead load = 1.9923 k/ft + (0.650 k/ft – 0.375 k/ft) = 2.2673 k/ft

(2.2673 k/ft)/(1.9923 k/ft) = 1.1380

New $M_{u,max}$ at Supports ≅ (1.2)(-184.95 k-ft*1.1380) + (1.6)(-176.31 k-ft) = 534.66 k-ft

New $M_{u,max}$ at Midspan ≅ (1.2)(93.96 k-ft*1.1380) + (1.6)(89.56 k-ft) = 271.61 k-ft

New $V_{u,max}$ ≅ (1.2)(33.37 k*1.1380) + (1.6)(31.81 k) = 96.47 k < ϕV_n = 258.99 kips

∴ Shear capacity is still OK.

3) Design the flexural reinforcement.

a) Compute the area of steel required at the point of maximum negative moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and ϕ = 0.90

$A_s \cong (534.66 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(0.9)(22.75")] = 5.80 \text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (5.80 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 4.267"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (534.66 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(22.75" - 4.267"/2)]$

$= 5.76 \text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (5.76 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 4.238"$

$c = a/\beta_1 = 4.238"/0.85 = 4.985" < (3/8)(d) = (3/8)(22.75") = 8.531"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$A_s \cong (271.61 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(0.95)(22.75")] = 2.79 \text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (2.79 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 2.053"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (271.61 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(22.75" - 2.053"/2)]$

$= 2.78 \text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (2.78 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 2.043"$

$c = a/\beta_1 = 2.043"/0.85 = 2.404" < (3/8)(d) = (3/8)(22.75") = 8.531"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

c) Calculate the minimum reinforcement (using ACI Code Section 10.5.1).

$A_{s,\,min}$ = max. of:

$[3\sqrt{f'_c}/f_y]b_w d = [3\sqrt{4000 \text{ psi}}/60000 \text{ psi}](24")(22.75") = 1.73 \text{ in}^2$

$200b_w d/f_y = (200)(24")(22.75")/60000 \text{ psi} = 1.82 \text{ in}^2$

$\therefore A_{s,min} = 1.82 \text{ in}^2$

4) Calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 5.76 \text{ in}^2 > A_{s,min} = 1.82 \text{ in}^2 \therefore$ OK

Use (10) #7 bars [$A_s = (10)(0.60 \text{ in}^2) = 6.00 \text{ in}^2 > 5.76 \text{ in}^2 \therefore$ OK]

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 2.78 \text{ in}^2 > A_{s,min} = 1.82 \text{ in}^2 \therefore$ OK

Use (5) #7 bars [$A_s = (5)(0.60 \text{ in}^2) = 3.00 \text{ in}^2 > 2.78 \text{ in}^2 \therefore$ OK]

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

c_c = 2.25 in. cover + 0.5 in. stirrups = 2.75"

The maximum bar spacing is

$s = 15(40{,}000/f_s) - 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000 \text{ ksi}) = 40{,}000 \text{ ksi}$

$s = 15(40{,}000/40{,}000) - (2.5)(2.75") = 8.125"$

Spacing of bars is less than 8.125" by inspection.

Minimum bar spacing:

s_c = max of [1", d_b, $(4/3)s_a$]; Assume s_a = 1" aggregate

s_c = max of [1”, 0.875”, (4/3)(1”) = 1.333”]; Assume s_a = 1” aggregate

s_c = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24” > (10)(0.875”) + (10-1)(1.333”) + (2)(0.5”) + (2)(2.25”)

24” < 26.25” ∴ Need two rows of reinforcing in negative-moment regions

Minimum vertical spacing between layers of reinforcement

= max. of: (4/3)(s_a) or 1”

= max. of (4/3)(1”) = 1.333”, or 1”

=1.333”

New d_{eff} = 26” – 2.25” – 0.5” – 0.875” – (1/2)(1.333”) = 21.708”

1) Re-check the shear capacity of the beam with d = 21.708”.

$V_u = \phi(V_c + V_s)$

$V_{u,max}$ = 96.47 kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$V_c = 2\lambda\sqrt{f'_c}b_w d$ = (2)(1.0) √4000 psi (24”)(21.708”)/1000 = 65.90 kips

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$V_s = 8\sqrt{f'_c}b_w d$ = (8) √4000 psi (24”)(21.708”)/1000 = 263.60 kips

Thus, the absolute maximum ϕV_n = 0.75(65.90 k + 263.60 k) = 247.13 kips

≥ $V_{u,max}$ = 96.47 kips ∴ OK

Shear capacity is OK when accounting for weight of 24”x26” beam.

2) Re-design the flexural reinforcement with d = 21.708”.

a) Compute the area of steel required at the point of maximum negative moment.

$A_s \geq M_u/[\phi f_y(d – a/2)] \cong M_u/[\phi f_y(jd)]$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and $\phi = 0.90$

$A_s \cong (534.66 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(0.9)(21.708")] = 6.08 \text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (6.08 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 4.472"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (534.66 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(21.708" - 4.472"/2)]$

$= 6.10 \text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (6.10 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 4.487"$

$c = a/\beta_1 = 4.487"/0.85 = 5.278" < (3/8)(d) = (3/8)(21.708") = 8.141"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$A_s \cong (271.61 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(0.95)(21.708")] = 2.93 \text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (2.93 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 2.154"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (271.61 \text{ k-ft})(12 \text{ in/ft})/[(0.9)(60 \text{ ksi})(21.708" - 2.154"/2)]$

$= 2.93 \text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (2.93 \text{ in}^2)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(24")] = 2.151"$

$c = a/\beta_1 = 2.151\text{"}/0.85 = 2.531\text{"} < (3/8)(d) = (3/8)(21.708\text{"}) = 8.141\text{"}$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

c) Calculate the minimum reinforcement (using ACI Code Section 10.5.1).

$A_{s,\,min}$ = max. of:

$[3\sqrt{f'_c}/f_y]b_w d = [3\sqrt{4000\text{ psi}}/60000\text{ psi}](24\text{"})(21.708\text{"}) = 1.65\text{ in}^2$

$200b_w d/f_y = (200)(24\text{"})(21.708\text{"})/60000\text{ psi} = 1.74\text{ in}^2$

$\therefore A_{s,min} = 1.74\text{ in}^2$

3) Re-calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 6.10\text{ in}^2 > A_{s,min} = 1.74\text{ in}^2 \therefore$ OK

Use (5) #8 bars and (5) #7 bars in two rows.

$[A_s = (5)(0.79\text{ in}^2) + (5)(0.60\text{ in}^2) = 6.95\text{ in}^2 > 6.10\text{ in}^2 \therefore \text{OK}]$

$a = A_s f_y/0.85f'_c b = (6.95\text{ in2})(60\text{ ksi})/[(0.85)(4\text{ ksi})(24\text{"})] = 5.110\text{"}$

$a = \beta_1 c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 5.110\text{"}/0.85 = 6.012\text{"}$

$d_{actual} = 26\text{"} - 2.25\text{"} - 0.5\text{"} - 1.0\text{"} - (1/2)(1.333\text{"}) = 21.583\text{"}$

$\varepsilon_s = (d-c)(\varepsilon_u)/c = (21.583\text{"} - 6.012\text{"})(0.003)/6.012\text{"} = 0.00777 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.00777 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_s f_y(d - a/2) = (0.9)(6.95\text{ in}^2)(60\text{ ksi})(21.583\text{"} - 5.110\text{"}/2)/(12\text{ in/ft}) =$

$= 595.10\text{ k-ft} > 534.66\text{ k-ft} \therefore$ OK

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 2.93\text{ in}^2 > A_{s,min} = 1.74\text{ in}^2 \therefore$ OK

Use (5) #7 bars in one row $[A_s = (5)(0.60\text{ in}^2) = 3.00\text{ in}^2 > 2.93\text{ in}^2 \therefore \text{OK}]$

*Using d = 21.708” for positive-moment region was conservative since using only one row of rebar in this region (actual “d” for this region will be greater than 21.708”)

$a = A_s f_y / 0.85 f'_c b = (3.00\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.206"$

$a = \beta_1 c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 2.206"/0.85 = 2.595"$

$\varepsilon_s \cong (d-c)(\varepsilon_u)/c = (21.708" – 2.595")(0.003)/2.595" = 0.02210 > \varepsilon_y = 0.00207$

(actual “d” for positive-moment region is larger since only have one row of reinforcement)

$\varepsilon_t \cong \varepsilon_s = 0.02210 > 0.005$ ∴ Tension-controlled Section ∴ $\phi = 0.9$

$\phi M_n = \phi A_s f_y (d – a/2) = (0.9)(3.00\ in^2)(60\ ksi)(21.708" – 2.206"/2)/(12\ in/ft) =$

= 278.17 k-ft > 271.61 k-ft ∴ OK

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

c_c = 2.25 in. cover + 0.5 in. stirrups = 2.75”

The maximum bar spacing is:

$s = 15(40{,}000/f_s) – 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000\ ksi) = 40{,}000\ ksi$

$s = 15(40{,}000/40{,}000) - (2.5)(2.75") = 8.125"$

Spacing of bars is less than 8.125” by inspection.

Minimum bar spacing:

s_c = max of [1”, d_b, (4/3)s_a]; Assume s_a = 1” aggregate

s_c = max of [1”, 0.875”, (4/3)(1”) = 1.333”]; Assume s_a = 1” aggregate

s_c = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

18" > (5)(1.00") + (5-1)(1.333") + (2)(0.5") + (2)(2.25")

24" > 15.83" ∴ OK

b) Positive-moment Region

The maximum bar spacing is 8.125". Spacing of bars is less than 8.125" by inspection.

Minimum bar spacing = 1.333"

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24" > (5)(0.875") + (5-1)(1.333") + (2)(0.5") + (2)(2.25")

24" > 15.21" ∴ OK

6) Design the shear reinforcement.

a) The critical section for shear is located at the support. ACI Code Section 11.4.6.1 requires stirrups if $V_u \geq \phi V_c/2$

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24")(21.708")/1000 = 65.90 kips

$V_c/2$ = 65.90 kips/2 = 32.95 kips

V_u/ϕ = (96.47 kips)/(0.75) = 128.63 kips > $V_c/2$ = 32.95 kips

∴ Stirrups are required.

b) Determine shear strength required by shear reinforcing.

$V_s = V_u/\phi - V_c$ = [(96.47 kips)/(0.75)] – 65.90 kips = 62.73 kips

$V_s \leq 8\sqrt{f'_c}b_wd$ = 8√4000 psi (24")(21.708")/1000 = 263.60 kips ∴ OK

c) Determine maximum spacing of shear reinforcing (ACI 318-08 Sections 11.4.5.1 and 11.4.5.3).

For $V_s \leq 8\sqrt{f'_c}b_wd$: s_{max} = min of {d/2, 24"}

d/2 = 21.708"/2 = 10.854"

s_{max} = 10"

d) Determine minimum shear reinforcement (ACI 318-08 Section 11.4.6.3).

$A_{v,min}$ = max of {$0.75\sqrt{f'_c}b_ws/f_{yt}$, $50b_ws/f_{yt}$}

$0.75\sqrt{f'_c}b_ws/f_{yt} = 0.75\sqrt{4000 \text{ psi}}(24")(10")/60{,}000 \text{ psi} = 0.190 \text{ in}^2$

$50b_ws/f_{yt} = 50(24")(10")/60{,}000 \text{ psi} = 0.200 \text{ in}^2$

$\therefore A_{v,min} = 0.200 \text{ in}^2$

Use #3 stirrups @ 10" as minimum shear reinforcement.

(A_v = 2 legs x 0.11 in^2/leg = 0.22 in^2 > 0.200 in^2 $\therefore$ OK)

e) Design the shear reinforcement.

$V_s = A_vf_{yt}d/s$

Rearranging: $s = A_vf_{yt}d/V_s$ = (0.22 in^2)(60 ksi)(21.708")/62.73 kips = 4.57"

Usually absolute minimum "s" is 4".

Use (2) #3 stirrups @ 4", starting 2" from face of support.

Or use #4 stirrups instead of #3 stirrups.

For #4 stirrups: (A_v = 2 legs x 0.20 in^2/leg = 0.40 in^2 > 0.200 in^2 $\therefore$ OK)

$s = A_vf_{yt}d/V_s$ = (0.40 in^2)(60 ksi)(21.708")/62.73 kips = 8.305"

Use (2) #4 stirrups @ 8", starting 2" from face of support.

Use this stirrup layout throughout the entire length of the beam since lateral loads can change the shear forces (shear diagram) throughout the beam length (since the beam is part of a concrete moment frame).

FINAL DESIGN: Use 24" x 26" beam with (5) #8 and (5) #7 bars for negative moment reinforcement (at the supports) and (5) #7 bars for positive moment reinforcement. Use (2) #4 stirrups @ 8" throughout length of beam.

COLUMN DESIGN:

Load Case 1: 1.2D + 1.6L (Gravity Load Case)

Exterior Column:

P_u = 177.98 kips

M_2 = 31.47 k-ft

M_1 = -17.26 k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y\rho_g)$

$A_{g(trial)} \geq$ 177.98 kips/[0.40(4 ksi + (60 ksi)(0.015))] = 90.81 in^2

$\cong$ (9.53 in.)^2

Try 18"x18" column

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \text{ x } \Delta_o]/[V_{us} \text{ x } l_c]$

$\sum P_u \cong$ (5)(177.98 k) = 889.90 k

V_{us} = 1 kip

Δ_o = 0.017769"

l_c = 10.5' = 126"

Q = [(889.90 kips)(0.017769")]/[(1 kip)(126")] = 0.02002 < 0.05

$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

r = 0.3h = (0.3)(18") = 5.4"

kl_u/r = (1.2)(126")/5.4" = 28 > 22 $\therefore$ Column is slender

4) Find δ_{ns} for the column.

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] \geq 1.0$

$C_m = 0.6 + 0.4(M_1/M_2) = 0.6 + 0.4(-17.26 \text{ k-ft}/31.47 \text{ k-ft}) = 0.3806$
$P_c = \pi^2 EI/(kl_u)^2$

a) Calculation of EI values

$EI = [0.2E_cI_g + E_sI_{se}]/[1 + \beta_{dns}]$

$I_g = bh^3/12 = (18")(18")^3/12 = 8748 \text{ in}^4$

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000} \text{ psi} = 3{,}605{,}000 \text{ psi} = 3605 \text{ ksi}$

$E_s = 29{,}000 \text{ ksi}$

$I_{se} \cong 2.2\rho_g\gamma^2 \times I_g$ (Table 12-1 in textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor)

Assume total steel ratio $\rho_g = 0.015$

For an 18"x18" column: $\gamma = [18" – (2)(2.5")]/18" = 0.7222$

$I_{se} \cong 2.2(0.015)(0.7222)^2 \times 8748 \text{ in}^4 = 150.58 \text{ in}^4$

Assuming that only the dead load is considered to cause a sustained axial load on the columns:

β_{dns} = (maximum factored sustained axial load)/(total factored axial load)

$\beta_{dns} = (1.2)(65.32 \text{ kips})/177.98 \text{ kips} = 0.6644$

$EI = [(0.2)(3605 \text{ ksi})(8748 \text{ in}^4) + (29{,}000 \text{ ksi})(150.58 \text{ in}^4)]/[1 + 0.6644]$

$= 6{,}413{,}198.75 \text{ kip-in}^2 = 6.4132 \times 10^6 \text{ kip-in}^2$

b) Calculation of P_c

$P_c = \pi^2 EI/(kl_u)^2 = \pi^2(6{,}413{,}198.75 \text{ kip-in}^2)/[(1 \times 126")^2] = 3986.88 \text{ kips}$

c) Calculation of δ_{ns}

$\delta_{ns} = C_m/[1 – (P_u/(0.75P_c))] = 0.3806/[1 – (177.98 \text{ kips}/(0.75)(3986.88 \text{ kips}))]$

$= 0.4047 \therefore$ Use $\delta_{ns} = 1.0$

Thus, the moments do not need to be magnified for this loading case.

5) Check initial column sections for gravity-load case.

$e = M_c/P_u = (31.47 \text{ k-ft})(12 \text{ in/ft})/(177.98 \text{ kips}) = 2.12"$

$e/h = 2.12''/18'' = 0.1179$

Fig. A-9b (from textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor):

Using $\gamma = 0.722 \cong 0.75$, $e/h = 0.1179$, and $\rho_g = 0.015$

$\phi P_n/A_g = 2.20$ ksi

$A_g \geq P_u/2.20 \text{ ksi} = 177.98 \text{ kips}/2.20 \text{ ksi} = 80.90 \text{ in}^2$

$A_g = (18'')(18'') = 324 \text{ in}^2 > 80.90 \text{ in}^2 \therefore$ OK

6) Select the longitudinal bars for this column.

$A_{st} = \rho_g A_g = (0.015)(324 \text{ in}^2) = 4.86 \text{ in}^2$

Select (12) #6 bars [$A_s = (12)(0.44 \text{ in}^2) = 5.28 \text{ in}^2 > 4.86 \text{ in}^2 \therefore$ OK]

It is OK to be a little conservative due to the corrosive natatorium environment.

$\phi P_n(\max) = \phi \times 0.80[0.85f'_c(A_g - A_{st}) + f_y A_{st}]$

$= (0.65)(0.80)[(0.85)(4 \text{ ksi})(324 \text{ in}^2 - 5.28 \text{ in}^2) + (60 \text{ ksi})(5.28 \text{ in}^2)]$

$= 728.23 \text{ kips} > 177.98 \text{ kips} \therefore$ OK

*Could reduce reinforcement ratio and go back to graph, obtain new value, and use less reinforcement as long as the column still works

Load Case 2: Gravity Plus Lateral (Earthquake) Loads

Exterior Column:

$P_u = 67.49$ kips

$M_2 = -398.52$ k-ft

$M_1 = 176.51$ k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y\rho_g)$

$A_{g(trial)} \geq 67.49 \text{ kips}/[0.40(4 \text{ ksi} + (60 \text{ ksi})(0.015))] = 34.43 \text{ in}^2$

$\cong (5.87 \text{ in.})^2$

Try 18"x18" column (due to the large moments)

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \times \Delta_o]/[V_{us} \times l_c]$

$\sum P_u \cong (5)(177.98\text{ k}) = 889.90\text{ k}$

$V_{us} = 1$ kip

$\Delta_o = 0.002836"$

$l_c = 10.5' = 126"$

$Q = [(889.90\text{ kips})(0.002836")]/[(1\text{ kips})(126")] = 0.02002 < 0.05$

$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

$r = 0.3h = (0.3)(18") = 5.4"$

$kl_u/r = (1.2)(126")/5.4" = 28 > 22$ $\therefore$ Column is slender

4) Find δ_{ns} for the column.

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] \geq 1.0$

$C_m = 0.6 + 0.4(M_1/M_2) = 0.6 + 0.4(176.51\text{ k-ft}/-398.52\text{ k-ft}) = 0.4228$

$P_c = \pi^2 EI/(kl_u)^2$

a) Calculation of EI values

$EI = [0.2E_cI_g + E_sI_{se}]/[1 + \beta_{dns}]$

$I_g = bh^3/12 = (18")(18")^3/12 = 8748\text{ in}^4$

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000}\text{ psi} = 3{,}605{,}000\text{ psi} = 3605\text{ ksi}$

$E_s = 29{,}000$ ksi

$I_{se} \cong 2.2\rho_g\gamma^2 \times I_g$ (Table 12-1 in textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor)

Assume total steel ratio $\rho_g = 0.015$

For an 18"x18" column: $\gamma = [18" - (2)(2.5")]/18" = 0.7222$

$I_{se} \cong 2.2(0.015)(0.7222)^2 \times 8748\text{ in}^4 = 150.58\text{ in}^4$

Assuming that only the dead load is considered to cause a sustained axial load on the columns:

β_{dns} = (maximum factored sustained axial load)/(total factored axial load)

β_{dns} = (1.2)(30.44 kips)/67.49 kips = 0.5412

EI = [(0.2)(3605 ksi)(8748 in^4) + (29,000 ksi)(150.58 in^4)]/[1 + 0.5412]

= 6,925,855.18 kip-in^2 = 6.9259 x 10^6 kip-in^2

b) Calculation of P_c

$P_c = \pi^2 EI/(kl_u)^2 = \pi^2$(6,925,855.18 kip-in^2)/[(1 x 126")^2] = 4305.58 kips

c) Calculation of δ_{ns}

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))]$ = 0.4228/[1 – (67.49 kips/(0.75)(4305.58 kips))]

= 0.4318 ∴ Use δ_{ns} = 1.0

Thus, the moments do not need to be magnified for this loading case.

5) Check initial column sections for gravity-load case.

e = M_c/P_u = (398.52 k-ft)(12 in/ft)/(67.49 kips) = 70.86"

e/h = 70.86"/18" = 3.94

Exceeds moment capacity of column.

Use interaction diagrams (Fig. A-9b) to determine required ρ_g:

The interaction diagrams are entered with:

$\phi P_n/A_g = P_u/A_g$ = (67.49 k)/(18"x18") = 0.208

$\phi M_n/A_g h = M_u/A_g h$ = (398.52 k-ft)(12 in/ft)/[(18"x18")(18")] = 0.820

Required ρ_g = 0.04 (which is too high)

∴ Must increase column size.

Try a 24"x24" column.

1) Use interaction diagrams (Fig. A-9b) to determine required ρ_g:

The interaction diagrams are entered with:

$\phi P_n/A_g = P_u/A_g$ = (67.49 k)/(24"x24") = 0.117

$\phi M_n/A_g h = M_u/A_g h$ = (398.52 k-ft)(12 in/ft)/[(24"x24")(24")] = 0.346

Required $\rho_g \cong 0.014$ ∴ OK to use 24"x24" column

2) Select the reinforcement

$A_{st} = \rho_g A_g$ = (0.014)(24"x24") = 8.064 in^2

Use (12) #8 bars [A_{st} = (12)(0.79 in^2) = 9.48 in^2 > 8.064 in^2 ∴ OK]

It is ok to be a little conservative due to the corrosive natatorium environment.

FINAL DESIGN: Use 24"x24" columns with (12) #8 bars.

<u>Concrete Moment Frame – Column Line 2</u>

<u>Beams</u>

*Use rebar cover of 1.5(1.5") = 2.25" due to corrosive environment (natatorium) (see ACI 7.7.6.1)

Design beams as a continuous beam.
Design beams for worst case and make all four beams the same size.

Axial Load and Moment (Unfactored) for Column Line 2 (24x24 Columns and 24x30 Beams)

	Beam 20	Beam 21	Beam 24	Beam 25	Column 10 Bottom, Exterior	Column 12 Bottom, Interior	Column 11 Top, Exterior	Column 13 Top, Interior
P_D					-130.28	-190.87	-67.61	-104.26
P_L					-29.47	-29.47	0.00	0.00
P_{Lr}					-59.92	-113.03	-24.93	-42.76
P_S					-35.71	-64.91	-28.28	-50.04
P_W					11.43	-1.55	3.55	-0.44
$P_{W,REVERSED}$					-11.39	1.52	-3.58	0.47
P_E					10.91	-1.31	2.51	-0.10
$P_{E,REVERSED}$					-11.13	1.48	-2.76	0.30
V_D (Top or Left)	-22.13	-22.72	-28.30	-31.31	-1.59	-0.11	-12.42	1.39
V_D (Bottom or Right)	23.37	22.78	33.63	30.62	-1.59	-0.11	-12.42	1.39
V_{Lr} (Top or Left)	-30.82	-32.38	-14.53	-15.69	-3.51	0.21	-9.46	0.89
V_{Lr} (Bottom or Right)	33.72	32.16	16.67	15.51	-3.51	0.21	-9.46	0.89
V_S (Top or Left)	-7.43	-7.39	-16.27	-18.26	-0.12	-0.15	-6.69	0.80
V_S (Bottom or Right)	7.48	7.51	19.77	17.77	-0.12	-0.15	-6.69	0.80
V_W (Top or Left)	7.88	6.77	3.55	3.11	14.23	16.60	3.91	9.72
V_W (Bottom or Right)	7.88	6.77	3.55	3.11	14.23	16.60	3.91	9.72
$V_{W,REVERSED}$ (Top or Left)	-7.81	-6.76	-3.58	-3.11	-13.85	-16.39	-4.08	-9.80
$V_{W,REVERSED}$ (Bottom or Right)	-7.81	-6.76	-3.58	-3.11	-13.85	-16.39	-4.08	-9.80
V_E (Top or Left)	8.40	7.19	2.51	2.41	18.74	21.20	0.63	6.21
V_E (Bottom or Right)	8.40	7.19	2.51	2.41	18.74	21.20	0.63	6.21
$V_{E,REVERSED}$ (Top or Left)	-8.37	-7.19	-2.76	-2.46	-17.84	-20.72	-1.43	-6.73
$V_{E,REVERSED}$ (Bottom or Right)	-8.37	-7.19	-2.76	-2.46	-17.84	-20.72	-1.43	-6.73
M_D (Top or Left)	-107.72	-120.68	-134.03	-203.29	23.39	1.41	134.03	-16.04
M_D (Bottom or Right)	-127.58	-121.71	-219.33	-192.13	-12.27	-1.03	-84.33	8.31
M_{Lr} (Top or Left)	-152.38	-188.86	-80.19	-118.16	**59.37**	**-62.90**	**80.19**	**-24.01**
M_{Lr} (Bottom or Right)	-190.66	-189.38	-124.66	-113.20	**-29.47**	**33.23**	**-93.02**	**70.78**
M_S (Top or Left)	-39.61	-38.48	-79.55	-125.40	1.46	2.14	79.55	-10.15
M_S (Bottom or Right)	-40.29	-40.42	-135.55	-117.56	-1.15	-1.26	-38.15	3.96
M_W (Top or Left)	132.76	107.83	59.58	49.47	-123.63	-159.89	-59.58	-103.48
M_W (Bottom or Right)	-119.47	-108.93	-54.01	-49.92	195.36	212.20	9.13	67.40
$M_{W,REVERSED}$ (Top or Left)	-131.34	-107.46	-60.23	-49.56	119.83	157.64	60.23	103.96
$M_{W,REVERSED}$ (Bottom or Right)	118.49	108.74	54.40	49.96	-190.65	-209.68	-11.51	-68.31
M_E (Top or Left)	141.68	114.25	41.03	38.46	-171.63	-209.94	-41.03	-77.86
M_E (Bottom or Right)	-126.96	-115.73	-39.40	-38.68	248.42	265.22	-29.94	31.27
$M_{E,REVERSED}$ (Top or Left)	-141.13	-114.26	-45.80	-39.47	161.78	204.63	45.80	81.99
$M_{E,REVERSED}$ (Bottom or Right)	126.69	115.73	42.51	39.19	-238.16	-259.92	20.65	-36.32
$M_{D,MIDSPAN}$	64.33	60.79	132.28	111.25				
$M_{Lr,MIDSPAN}$	94.47	85.42	69.86	61.87				
$M_{S,MIDSPAN}$	19.68	20.18	84.64	70.71				
$M_{W,MIDSPAN}$	6.64	-0.55	2.79	-0.23				
$M_{W,REVERSED,MIDSPAN}$	-6.43	0.64	-2.92	0.20				
$M_{E,MIDSPAN}$	7.36	-0.74	0.82	-0.11				
$M_{E,REVERSED,MIDSPAN}$	-7.22	0.74	-1.64	-0.14				

Torsional Effects are Included in Table

1.2D +/- 1.0E + 0.2S								
Max $V_{TOP/LEFT}$ (kips)	-36.411	-35.929	-39.974	-43.682	-19.773	-20.885	-17.672	8.034
Max $V_{BOTTOM/RIGHT}$ (kips)	37.935	36.025	46.823	42.709	-19.773	-20.885	-17.672	8.034
Max $M_{TOP/LEFT}$ (ft-kips)	-278.3137	-266.7756	-222.5419	-308.5008	190.1374	206.753	222.5419	-99.1366
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-288.1182	-269.8611	-329.7016	-292.7521	-253.1161	-261.4039	-138.7701	42.032
Max $M_{MIDSPAN}$ (ft-kips)	88.4919	77.7193	176.4816	147.5001				
Max P_u (kips)					-174.6034	-243.3334	-89.548	-135.222

1.2D + 1.6(Lr or S) + 0.8W								
Max $V_{TOP/LEFT}$ (kips)	-82.11	-84.48	-62.86	-69.28	-18.60	-13.48	-33.30	10.87
Max $V_{BOTTOM/RIGHT}$ (kips)	**88.30**	84.21	74.83	67.66	-18.60	-13.48	-33.30	10.87
Max $M_{TOP/LEFT}$ (ft-kips)	-478.15	-532.95	-336.30	-484.23	218.92	131.23	**336.30**	-90.13
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	**-553.73**	-536.20	-523.28	-458.59	-214.40	-170.99	-259.23	177.14
Max $M_{MIDSPAN}$ (ft-kips)	233.66	210.13	**272.74**	232.65				
Max P_u (kips)					-261.32	**-411.13**	-129.25	-205.53

1.2D + 1.6W + 0.5(Lr or S)								
Max $V_{TOP/LEFT}$ (kips)	-54.457	-54.264	-47.827	-51.678	-25.824	-26.424	-26.162	17.662
Max $V_{BOTTOM/RIGHT}$ (kips)	57.515	54.254	55.921	50.599	-25.824	-26.424	-26.162	17.662
Max $M_{TOP/LEFT}$ (ft-kips)	-415.59795	-411.17805	-296.9865	-385.9405	249.48145	254.9868	296.9865	-189.89
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-439.5792	-415.0268	-417.3857	-369.2095	-334.50205	**-337.3473**	-166.1165	153.20445
Max $M_{MIDSPAN}$ (ft-kips)	135.061	116.6864	205.5139	169.1805				
Max P_u (kips)					-204.5154	-288.0394	-101.004	-150.842

1.2D + 1.6L + 0.5(L_r or S)								
Max P_u (kips)					-233.44	-332.70	-93.60	-146.49

1.4D								
Max P_u (kips)					-182.39	-267.21	-94.65	-145.96

Torsional Effects are Included in Tables

BEAM DESIGN

$V_{u,max}$ = 88.30 kips (1.2D + 1.6L_r + 0.8W)
$M_{u,max}$ at Supports = - 553.73 k-ft (1.2D + 1.6L_r + 0.8W)
$M_{u,max}$ at Midspan = 272.74 k-ft (1.2D + 1.6L_r + 0.8W)

Use normal-weight concrete with f'_c = 4000 psi
f_y = 60,000 psi for flexural reinforcement
f_{yt} = 60,000 psi for stirrups

1) Choose the actual size of the beam stem.

a) Calculate the minimum depth based on deflections.

Use worst case scenario (one-end continuous instead of both ends continuous).

ACI Table 9.5(a):

Minimum thickness, h = L/18.5 = [(32')(12 in/ft)]/18.5 = 20.76"

b) Determine the minimum depth based on the maximum negative moment.

$M_{u,max}$ at Supports = 553.73 k-ft

$\rho(\text{initial}) = [(\beta_1 f'_c)/(4f_y)] = [(0.85)(4\text{ ksi})/(4)(60\text{ ksi})] = 0.0142$

$\omega = \rho(f_y/f'_c) = (0.0142)(60\text{ ksi}/4\text{ ksi}) = 0.213$

$R = \omega f'c(1 - 0.59\omega) = (0.213)(4\text{ ksi})[1 - (0.59)(0.213)] = 0.745\text{ ksi}$

$bd^2 \geq M_u/\phi R = [(553.73\text{ ft-kips})(12\text{ in/ft})]/[(0.9)(0.745\text{ ksi})] = 9910.16\text{ in}^3$

Assuming b = 24 in. (for 24" x 24" column)

$d \geq 20.32$ in.

h ≅ 20.32" + 3.25" = 23.57" (accounting for 2.25" clear cover due to corrosive environment and assuming #4 stirrups and #8 bars; see ACI 7.7.6.1)

[(1.5)(1.5") = 2.25"; 2.25" + 0.5" + (1/2)(1.00") = 3.25"]

Try h = 30"

h = 30" > 20.76" ∴ Meets deflection criteria

d ≅ 30" – 3.25" = 26.75"

c) Check the shear capacity of the beam.

$V_u = \phi(V_c + V_s)$

$V_{u,max} = 88.30$ kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$$V_c = 2\lambda\sqrt{f'_c}b_wd = (2)(1.0)\sqrt{4000 \text{ psi}}\,(24")(26.75")/1000 = 81.21 \text{ kips}$$

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$$V_s = 8\sqrt{f'_c}b_wd = (8)\sqrt{4000 \text{ psi}}\,(24")(26.75")/1000 = 324.83 \text{ kips}$$

Thus, the absolute maximum $\phi V_n = 0.75(81.21 \text{ k} + 324.83 \text{ k}) = 304.53$ kips

$\geq V_{u,max} = 88.30$ kips $\therefore$ OK

d) Summary. Use:

b = 24"
h = 30"
d = 26.75"

2) Compute the dead load of the stem, and recompute the total moment.

Weight of 24"x30" concrete beam = $[(24")(30")/144 \text{ in}^2/\text{ft}^2][(150 \text{ lb/ft}^3)/1000]$

= 0.720 k/ft

Original dead load = 1.42 k/ft

New dead load = 1.42 k/ft + 0.720 k/ft = 2.14 k/ft

(2.14 k/ft)/(1.42 k/ft) = 1.507

New $M_{u,max}$ at Supports ≅

Beam 20: $1.2D + 1.6L_r + 0.8W$

= (1.2)(-127.58 k-ft*1.507) + (1.6)(-190.66 k-ft) + (0.8)(-119.47 k-ft) =

= -631.35 k-ft

New $M_{u,max}$ at Midspan ≅

Beam 24: $1.2D + 1.6S + 0.8W$

= (1.2)(132.28 k-ft*1.507) + (1.6)(84.64 k-ft) + (0.8)(2.79 k-ft)

= 376.87 k-ft

New $V_{u,max} \cong$

Beam 20: $1.2D + 1.6L_r + 0.8W$

$= (1.2)(23.37\text{ k}*1.507) + (1.6)(33.72\text{ k}) + (0.8)(7.88\text{ k}) =$

$= 102.52\text{ k} < \phi V_n = 304.53\text{ kips}$

∴ Shear capacity is still OK.

3) Design the flexural reinforcement.

a) Compute the area of steel required at the point of maximum negative moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that $j = 0.9$ and $\phi = 0.90$

$A_s \cong (631.35\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.9)(26.75\text{”})] = 5.83\text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (5.83\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24\text{”})] = 4.285\text{”}$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (631.35\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(26.75\text{”} - 4.285\text{”}/2)]$

$= 5.70\text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (5.70\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24\text{”})] = 4.192\text{”}$

$c = a/\beta_1 = 4.192\text{”}/0.85 = 4.932\text{”} < (3/8)(d) = (3/8)(26.75\text{”}) = 10.031\text{”}$

∴ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Assume that the compression zone is rectangular, and take $j = 0.95$ for the first calculation of A_s.

$A_s \cong (376.87\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.95)(26.75\text{”})] = 3.30\text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (3.30\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.423"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (376.87\ k\text{-}ft)(12\ in/ft)/[(0.9)(60\ ksi)(26.75" - 2.423"/2)]$

$= 3.28\ in^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (3.28\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.411"$

$c = a/\beta_1 = 2.411"/0.85 = 2.837" < (3/8)(d) = (3/8)(26.75") = 10.031"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

c) Calculate the minimum reinforcement (using ACI Code Section 10.5.1).

$A_{s,\ min}$ = max. of:

$[3\sqrt{f'_c}/f_y]b_w d = [3\sqrt{4000\ psi}/60000\ psi](24")(26.75") = 2.03\ in^2$

$200b_w d/f_y = (200)(24")(26.75")/60000\ psi = 2.14\ in^2$

$\therefore A_{s,min} = 2.14\ in^2$

4) Calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 5.70\ in^2 > A_{s,min} = 2.14\ in^2 \therefore$ OK

Use (10) #7 bars [$A_s = (10)(0.60\ in^2) = 6.00\ in^2 > 5.70\ in^2 \therefore$ OK]

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 3.28\ in^2 > A_{s,min} = 2.14\ in^2 \therefore$ OK

Use (6) #7 bars [$A_s = (6)(0.60\ in^2) = 3.60\ in^2 > 3.28\ in^2 \therefore$ OK]

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

c_c = 2.25 in. cover + 0.5 in. stirrups = 2.75”

The maximum bar spacing is

$s = 15(40{,}000/f_s) – 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000 \text{ ksi}) = 40{,}000 \text{ ksi}$

s = 15(40,000/40,000) - (2.5)(2.75”) = 8.125”

Spacing of bars is less than 8.125” by inspection.

Minimum bar spacing:

s_c = max of [1”, d_b, (4/3)s_a]; Assume s_a = 1” aggregate

s_c = max of [1”, 0.875”, (4/3)(1”) = 1.333”]; Assume s_a = 1” aggregate

s_c = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24” > (10)(0.875”) + (10-1)(1.333”) + (2)(0.5”) + (2)(2.25”)

24” < 26.25” ∴ Need two rows of reinforcing in negative-moment region

Minimum vertical spacing between layers of reinforcement

= max. of: (4/3)(s_a) or 1”

= max. of (4/3)(1”) = 1.333”, or 1”

=1.333”

New d_{eff} = 30” – 2.25” – 0.5” – 0.875” – (1/2)(1.333”) = 25.708”

1) Re-check the shear capacity of the beam with d = 25.708”.

$V_u = \phi(V_c + V_s)$

$V_{u,max}$ = 102.52 kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24”)(25.708”)/1000 = 78.04 kips

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$$V_s = 8\sqrt{f'_c}b_w d = (8)\sqrt{4000\text{ psi}}\,(24'')(25.708'')/1000 = 312.18\text{ kips}$$

Thus, the absolute maximum $\phi V_n = 0.75(78.04\text{ k} + 312.18\text{ k}) = 292.67$ kips

$\geq V_{u,max} = 102.52$ kips $\therefore$ OK

Shear capacity is OK when accounting for weight of 24x30 beam.

2) Re-design the flexural reinforcement with d = 25.708".

a) Compute the area of steel required at the point of maximum negative moment.

$$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and ϕ = 0.90

$$A_s \cong (631.35\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.9)(25.708'')] = 6.06\text{ in.}^2$$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$$a = A_s f_y/0.85f'_c b = (6.06\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24'')] = 4.459''$$

and then recalculating the required A_s with this calculated value of a:

$$A_s \geq M_u/[\phi f_y(d - a/2)] = (631.35\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(25.708'' - 4.459''/2)]$$

$$= 5.98\text{ in}^2$$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$$a = A_s f_y/0.85f'_c b = (5.98\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24'')] = 4.394''$$

$$c = a/\beta_1 = 4.394''/0.85 = 5.169'' < (3/8)(d) = (3/8)(25.708'') = 9.641''$$

$\therefore$ Section is tension-controlled and can be designed using ϕ = 0.90

b) Compute the area of steel required at the point of maximum positive moment.

$$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$$A_s \cong (376.87\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.95)(25.708'')] = 3.43\text{ in.}^2$$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y / 0.85 f'_c b = (3.43\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.521"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (376.87\ k\text{-}ft)(12\ in/ft)/[(0.9)(60\ ksi)(25.708" - 2.521"/2)]$

$= 3.43\ in^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y / 0.85 f'_c b = (3.43\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.522"$

$c = a/\beta_1 = 2.522"/0.85 = 2.967" < (3/8)(d) = (3/8)(25.708") = 9.641"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

3) Re-calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 5.98\ in^2 > A_{s,min} = 2.14\ in^2 \therefore$ OK

Use (10) #7 bars in two rows.

$[A_s = (10)(0.60\ in^2) = 6.00\ in^2 > 5.98\ in^2 \therefore$ OK]

$a = A_s f_y / 0.85 f'_c b = (6.00\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 4.4118"$

$a = \beta_1 c =$ where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 4.4118"/0.85 = 5.1903"$

$\varepsilon_s = (d-c)(\varepsilon_u)/c = (25.708" - 5.1903")(0.003)/5.1903" = 0.01186 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.01186 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_s f_y(d - a/2) = [(0.9)(6.00\ in^2)(60\ ksi)(25.708" - 4.4118"/2)]/(12\ in/ft) =$

$= 634.56\ k\text{-}ft > 631.35\ k\text{-}ft \therefore$ OK

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 3.43\ in^2 > A_{s,min} = 2.14\ in^2 \therefore$ OK

Use (8) #6 bars in two rows [$A_s = (8)(0.44\ in^2) = 3.52\ in^2 > 3.43\ in^2 \therefore$ OK]

$a = A_sf_y/0.85f'_cb = (3.52\ in^2)(60\ ksi)/[(0.85)(4\ ksi)(24")] = 2.5882"$

$a = \beta_1 c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 2.5882"/0.85 = 3.0450"$

$\varepsilon_s \cong (d-c)(\varepsilon_u)/c = (25.708" - 3.0450")(0.003)/3.0450" = 0.02233 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.02233 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_sf_y(d - a/2) = (0.9)(3.52\ in^2)(60\ ksi)(25.708" - 2.5882"/2)/(12\ in/ft) =$

$= 386.72$ k-ft > 376.87 k-ft $\therefore$ OK

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

c_c = 2.25 in. cover + 0.5 in. stirrups = 2.75"

The maximum bar spacing is:

$s = 15(40{,}000/f_s) - 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000\ ksi) = 40{,}000\ ksi$

$s = 15(40{,}000/40{,}000) - (2.5)(2.75") = 8.125"$

Spacing of bars is less than 8.125" by inspection.

Minimum bar spacing:

s_c = max of [1", d_b, $(4/3)s_a$]; Assume s_a = 1" aggregate

s_c = max of [1", 0.875", (4/3)(1") = 1.333"]; Assume s_a = 1" aggregate

$s_c = 1.333"$

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

$24" > (5)(0.875") + (5-1)(1.333") + (2)(0.5") + (2)(2.25")$

$24" > 15.21" \therefore$ OK

b) Positive-moment Region

The maximum bar spacing is 8.125”. Spacing of bars is less than 8.125” by inspection.

Minimum bar spacing = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24” > (4)(0.75”) + (4-1)(1.333”) + (2)(0.5”) + (2)(2.75”)

24” > 12.50” ∴ OK

6) Design the shear reinforcement.

a) The critical section for shear is located at the support. ACI Code Section 11.4.6.1 requires stirrups if $V_u \geq \phi V_c/2$

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24”)(25.708”)/1000 = 78.04 kips

$V_c/2$ = 78.04 kips/2 = 39.02 kips

V_u/ϕ = (102.52 kips)/(0.75) = 136.69 kips > $V_c/2$ = 39.02 kips

∴ Stirrups are required.

b) Determine shear strength required by shear reinforcing.

$V_s = V_u/\phi - V_c$ = [(102.52 kips)/(0.75)] – 78.04 kips = 58.65 kips

$V_s \leq 8\sqrt{f'_c}b_wd$ = 8√4000 psi (24”)(25.708”)/1000 = 312.18 kips ∴ OK

c) Determine maximum spacing of shear reinforcing (ACI 318-08 Sections 11.4.5.1 and 11.4.5.3).

For $V_s \leq 8\sqrt{f'_c}b_wd$: s_{max} = min of {d/2, 24”}

d/2 = 25.708”/2 = 12.854”

s_{max} = 12”

d) Determine minimum shear reinforcement (ACI 318-08 Section 11.4.6.3).

$A_{v,min}$ = max of $\{0.75\sqrt{f'_c}b_ws/f_{yt}, 50b_ws/f_{yt}\}$

$0.75\sqrt{f'_c}b_ws/f_{yt}$ = 0.75√4000 psi (24”)(12”)/60,000 psi = 0.23 in^2

$50b_w s/f_{yt}$ = 50(24")(12")/60,000 psi = 0.24 in^2

$\therefore A_{v,min}$ = 0.24 in^2

$s = A_v f_{yt} d/V_s$ = (0.24 in^2)(60 ksi)(25.708")/58.65 kips = 6.312"

Use #4 stirrups @ 6" as minimum shear reinforcement.

e) Design the shear reinforcement.

$V_s = A_v f_{yt} d/s$

Rearranging: $s = A_v f_{yt} d/V_s$ = (0.24 in^2)(60 ksi)(25.708")/58.65 kips = 6.312"

Use #4 stirrups.

For #4 stirrups: (A_v = 2 legs x 0.20 in^2/leg = 0.40 in^2 > 0.24 in^2 $\therefore$ OK)

$s = A_v f_{yt} d/V_s$ = (0.40 in^2)(60 ksi)(25.708")/58.65 kips = 10.52"

Use (2) #4 stirrups @ 10", starting 2" from face of support.

Use this stirrup layout throughout the entire length of the beam since lateral loads can change the shear forces (shear diagram) throughout the beam length (since the beam is part of a concrete moment frame).

FINAL DESIGN: Use 24" x 30" beam with (10) #7 bars for negative moment reinforcement (at the supports) and (8) #6 bars for positive moment reinforcement.

COLUMN DESIGN

Load Case 1: 1.2D + 1.6W + 0.5L_r

Interior Column (worse case): Column 12 (bottom, interior)

P_u = 288.04 kips (compression)

M_2 = -337.35 k-ft

M_1 = 254.99 k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y \rho_g)$

$A_{g(trial)} \geq$ 288.04 kips/[0.40(4 ksi + (60 ksi)(0.015))] = 146.96 in^2

$\cong$ (12.12 in.)^2

Try 24”x24” column (due to large moments on column)

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \times \Delta_o]/[V_{us} \times l_c]$

$\sum P_u \cong (2)(204.52)$ kips + $(3)(288.04)$ kips = 1273.16

$V_{us} = 1$ kip

$\Delta_o = 0.006298$”

$l_c = 22.5' = 270$”

Q = [(1273.16 kips)(0.006298”)]/[(1 kips)(270”)] = 0.02970 < 0.05

$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

r = 0.3h = (0.3)(24”) = 7.2”

kl_u/r = (1.2)(270”)/7.2” = 45 > 22 $\therefore$ Column is slender

4) Find δ_{ns} for the column.

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] \geq 1.0$

$C_m = 0.6 + 0.4(M_1/M_2) = 0.6 + 0.4(254.99 \text{ k-ft}/-337.35 \text{ k-ft}) = 0.2977$

$P_c = \pi^2 EI/(kl_u)^2$

a) Calculation of EI values

$EI = [0.2E_cI_g + E_sI_{se}]/[1 + \beta_{dns}]$

$I_g = bh^3/12$ = (24”)(24”)^3/12 = 27,648 in^4

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000}$ psi = 3,605,000 psi = 3605 ksi

E_s = 29,000 ksi

$I_{se} \cong 2.2\rho_g\gamma^2 \times I_g$ (Table 12-1 in textbook “Reinforced Concrete Mechanics and Design by Wight and MacGregor)

Assume total steel ratio $\rho_g = 0.015$

For a 24”x24” column: γ = [24” – (2)(2.5”)]/24” = 0.7917

$I_{se} \cong 2.2(0.015)(0.7917)^2 \times 27{,}648 \text{ in}^4 = 571.82 \text{ in}^4$

Assuming that only the dead load is considered to cause a sustained axial load on the columns:

β_{dns} = (maximum factored sustained axial load)/(total factored axial load)

$\beta_{dns} = (1.2)(190.87 \text{ kips})/288.04 \text{ kips} = 0.7952$

$EI = [(0.2)(3605 \text{ ksi})(27{,}648 \text{ in}^4) + (29{,}000 \text{ ksi})(571.82 \text{ in}^4)]/[1 + 0.7952]$

$= 20{,}341{,}459 \text{ kip-in}^2 = 20.3415 \times 10^6 \text{ kip-in}^2$

b) Calculation of P_c

$P_c = \pi^2 EI/(kl_u)^2 = \pi^2(20{,}341{,}459 \text{ kip-in}^2)/[(1 \times 270")^2] = 2753.94 \text{ kips}$

c) Calculation of δ_{ns}

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] = 0.2977/[1 - (288.04 \text{ kips}/(0.75)(2753.94 \text{ kips}))]$

$= 0.3459 \therefore$ Use $\delta_{ns} = 1.0$

Thus, the moments do not need to be magnified for this loading case.

5) Check initial column sections.

$e = M_{c/}P_u = [(337.35 \text{ k-ft})(12 \text{ in/ft})]/(288.04 \text{ kips}) = 14.054"$

$e/h = 14.054"/24" = 0.5856$

Fig. A-9b (from textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor):

Using $\gamma = 0.7917 \cong 0.75$, $e/h = 0.5856$, and $\rho_g = 0.015$

$\phi P_n/A_g = 0.85 \text{ ksi}$

$A_g \geq P_u/0.45 \text{ ksi} = 288.04 \text{ kips}/0.85 \text{ ksi} = 338.87 \text{ in}^2$

$A_g = (24")(24") = 576 \text{ in}^2 > 338.87 \text{ in}^2 \therefore$ OK

$\phi M_n/bh^2 = 0.47 \text{ ksi}$

$bh^2 \geq [(337.35 \text{ k-ft})(12 \text{ in/ft})]/0.47 \text{ ksi} = 8{,}613.19 \text{ in}^3$

$h \geq \sqrt{[(8{,}613.19 \text{ in}^3)/(b)]} = \sqrt{[(13{,}042 \text{ in}^3)/(24")]} = 18.94"$
$h = 24" > 18.94" \therefore$OK

6) Select the longitudinal bars for this column.

$A_{st} = \rho_g A_g = (0.015)(576 \text{ in}^2) = 8.64 \text{ in}^2$

Select (12) #8 bars [$A_s = (12)(0.79 \text{ in}^2) = 9.48 \text{ in}^2 > 8.64 \text{ in}^2 \therefore$ OK]

It is OK to be a little conservative due to the corrosive natatorium environment.

$\phi P_n(\max) = \phi \times 0.80[0.85f'_c(A_g - A_{st}) + f_y A_{st}]$

$= (0.65)(0.80)[(0.85)(4 \text{ ksi})(576 \text{ in}^2 - 9.48 \text{ in}^2) + (60 \text{ ksi})(9.48 \text{ in}^2)]$

$= 1297.38 \text{ kips} > 288.04 \text{ kips} \therefore$ OK

Load Case 2: 1.2D + 1.6L_r + 0.8W

Interior Column (worst case): Column 12 (bottom, interior)

$P_u = 411.13$ kips (compression)

$M_2 = -170.99$ k-ft

$M_1 = 131.23$ k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y\rho_g)$

$A_{g(trial)} \geq 411.13 \text{ kips}/[0.40(4 \text{ ksi} + (60 \text{ ksi})(0.015))] = 209.76 \text{ in}^2$

$\cong (14.48 \text{ in.})^2$

Try 24"x24" column (due to large moments on column)

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \times \Delta_o]/[V_{us} \times l_c]$

$\sum P_u \cong (2)(261.32) \text{ kips} + (3)(411.12) \text{ kips} = 1756 \text{ kips}$

$V_{us} = 1$ kip

$\Delta_o = 0.006298"$

$l_c = 22.5' = 270"$

$Q = [(1756 \text{ kips})(0.006298")]/[(1 \text{ kips})(270")] = 0.04096 < 0.05$

$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

$r = 0.3h = (0.3)(24") = 7.2"$

$kl_u/r = (1.2)(270")/7.2" = 45 > 22$ ∴ Column is slender

4) Find δ_{ns} for the column.

$\delta_{ns} = C_m/[1 - (P_u/(0.75P_c))] \geq 1.0$

$C_m = 0.6 + 0.4(M_1/M_2) = 0.6 + 0.4(131.23 \text{ k-ft}/-170.99 \text{ k-ft}) = 0.2930$

$P_c = \pi^2 EI/(kl_u)^2$

a) Calculation of EI values

$EI = [0.2E_cI_g + E_sI_{se}]/[1 + \beta_{dns}]$

$I_g = bh^3/12 = (24")(24")^3/12 = 27{,}648 \text{ in}^4$

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{4000 \text{ psi}} = 3{,}605{,}000 \text{ psi} = 3605 \text{ ksi}$

$E_s = 29{,}000 \text{ ksi}$

$I_{se} \cong 2.2\rho_g\gamma^2 \times I_g$ (Table 12-1 in textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor)

Assume total steel ratio $\rho_g = 0.015$

For a 24"x24" column: $\gamma = [24" - (2)(2.5")]/24" = 0.7917$

$I_{se} \cong 2.2(0.015)(0.7917)^2 \times 27{,}648 \text{ in}^4 = 571.82 \text{ in}^4$

Assuming that only the dead load is considered to cause a sustained axial load on the columns:

β_{dns} = (maximum factored sustained axial load)/(total factored axial load)

$\beta_{dns} = (1.2)(190.87 \text{ kips})/411.13 \text{ kips} = 0.5571$

$EI = [(0.2)(3605 \text{ ksi})(27{,}648 \text{ in}^4) + (29{,}000 \text{ ksi})(571.82 \text{ in}^4)]/[1 + 0.5571]$

$= 23{,}451{,}922.16 \text{ kip-in}^2 = 23.4519 \times 10^6 \text{ kip-in}^2$

b) Calculation of P_c

$P_c = \pi^2 EI/(kl_u)^2 = \pi^2(23{,}451{,}922.16 \text{ kip-in}^2)/[(1 \times 270")^2] = 3175.05 \text{ kips}$

c) Calculation of δ_{ns}

$\delta_{ns} = C_m/[1 – (P_u/(0.75P_c))] = 0.2930/[1 – (411.13 \text{ kips}/(0.75)(3175.05 \text{ kips}))]$

$= 0.3541 \therefore \text{ Use } \delta_{ns} = 1.0$

Thus, the moments do not need to be magnified for this loading case.

5) Check initial column sections.

$e = M_c/P_u = [(170.99 \text{ k-ft})(12 \text{ in/ft})]/(411.13 \text{ kips}) = 4.9908''$

$e/h = 4.9908''/24'' = 0.2080$

Fig. A-9b (from textbook "Reinforced Concrete Mechanics and Design by Wight and MacGregor):

Using $\gamma = 0.7917 \cong 0.75$, $e/h = 0.2080$, and $\rho_g = 0.015$

$\phi P_n/A_g = 1.70 \text{ ksi}$

$A_g \geq P_u/0.45 \text{ ksi} = 411.13 \text{ kips}/1.70 \text{ ksi} = 241.84 \text{ in}^2$

$A_g = (24'')(24'') = 576 \text{ in}^2 > 241.84 \text{ in}^2 \therefore \text{ OK}$

$\phi M_n/bh^2 = 0.34 \text{ ksi}$

$bh^2 \geq [(170.99 \text{ k-ft})(12 \text{ in/ft})]/0.34 \text{ ksi} = 6{,}034.94 \text{ in}^3$

$h \geq \sqrt{[(6{,}034.94 \text{ in}^3)/(b)]} = \sqrt{[(13{,}042 \text{ in}^3)/(24'')]} = 15.86''$

$h = 24'' > 15.86'' \therefore \text{OK}$

6) Select the longitudinal bars for this column.

$A_{st} = \rho_g A_g = (0.015)(576 \text{ in}^2) = 8.64 \text{ in}^2$

Select (12) #8 bars [$A_s = (12)(0.79 \text{ in}^2) = 9.48 \text{ in}^2 > 8.64 \text{ in}^2 \therefore$ OK]

It is OK to be a little conservative due to the corrosive natatorium environment.

$\phi P_n(\text{max}) = \phi \times 0.80[0.85f'_c(A_g – A_{st}) + f_y A_{st}]$

$= (0.65)(0.80)[(0.85)(4 \text{ ksi})(576 \text{ in}^2 – 9.48 \text{ in}^2) + (60 \text{ ksi})(9.48 \text{ in}^2)]$

$= 1297.38 \text{ kips} > 288.04 \text{ kips} \therefore \text{ OK}$

FINAL DESIGN: Use 24" x 24" column with (12) #8 bars.

Concrete Moment Frame – East/West Direction

Beams

*Use rebar cover of 1.5(1.5”) = 2.25” due to corrosive environment (natatorium) (see ACI 7.7.6.1)

Shear and Moment (Unfactored) for Columns and Sloped Concrete Beams (E/W Direction)

	Beam 13/14	West Column (C.L. 1.8)	East Column (C.L. 2) - Bottom	East Column (C.L. 2) - Top
V_D (Top or Left)	-22.29	-4.08	-4.08	0.00
V_D (Bottom or Right)	28.65	-4.08	-4.08	0.00
V_L (Top or Left)	-6.89	-4.92	-4.92	0.00
V_L (Bottom or Right)	34.57	-4.92	-4.92	0.00
V_E (Top or Left)	11.43	36.62	6.00	8.40
V_E (Bottom or Right)	30.06	36.62	6.00	8.40
$V_{E,REVERSED}$ (Top or Left)	-11.43	-36.62	-6.00	-8.40
$V_{E,REVERSED}$ (Bottom or Right)	-30.06	-36.62	-6.00	-8.40
V_W (Top or Left)	7.26	23.01	3.37	5.68
V_W (Bottom or Right)	18.91	23.01	3.37	5.68
$V_{W,REVERSED}$ (Top or Left)	-7.26	-23.01	-3.37	-5.68
$V_{W,REVERSED}$ (Bottom or Right)	-18.91	-23.01	-3.37	-5.68
M_D (Top or Left)	-50.27	50.27	0.00	0.00
M_D (Bottom or Right)	-91.83	7.41	-91.83	0.00
M_L (Top or Left)	-60.64	60.64	0.00	0.00
M_L (Bottom or Right)	-110.79	8.94	-110.79	0.00
M_E (Top or Left)	136.74	-136.74	-8.88	0.00
M_E (Bottom or Right)	-155.88	247.73	126.21	147.00
$M_{E,REVERSED}$ (Top or Left)	-136.74	136.74	8.88	0.00
$M_{E,REVERSED}$ (Bottom or Right)	155.88	-247.73	-126.21	-147.00
M_W (Top or Left)	86.20	-86.20	0.16	0.00
M_W (Bottom or Right)	-99.16	155.61	75.97	99.31
$M_{W,REVERSED}$ (Top or Left)	-86.20	86.20	-0.16	0.00
$M_{W,REVERSED}$ (Bottom or Right)	99.16	-155.36	-75.97	-99.31
P_D	-21.35	-30.59	-28.65	0.00
P_L	-25.75	-36.90	-34.57	0.00
P_E	35.29	30.06	-30.06	0.00
$P_{E,REVERSED}$	-35.29	-30.06	30.06	0.00
P_W	22.11	18.91	-18.91	0.00
$P_{W,REVERSED}$	-22.11	-18.91	18.91	0.00
M_D (Midspan)	65.63	28.84	-45.92	0.00
M_L (Midspan)	79.19	34.79	-55.40	0.00
M_E (Midspan)	**20.49**	**55.49**	58.66	73.50
$M_{E,REVERSED}$ (Midspan)	**-20.49**	**-55.49**	-58.66	-73.50
M_W (Midspan)	**12.42**	**34.58**	38.06	49.66
$M_{W,REVERSED}$ (Midspan)	**-12.42**	**-34.58**	-38.06	-49.66

Table Accounts for Torsional Effects

1.2D +/- 1.0E + 1.0L				
Max $V_{TOP/LEFT}$ (kips)	-45.07	-46.44	-15.83	-8.40
Max $V_{BOTTOM/RIGHT}$ (kips)	99.01	26.79	-15.83	8.40
Max $M_{TOP/LEFT}$ (ft-kips)	-257.71	**257.71**	-8.88	0.00
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-376.87	**265.56**	**-347.20**	147.00
Max $M_{MIDSPAN}$ (ft-kips)	178.44	124.89	-169.16	73.50
Max P_u (kips)	-86.66	-103.67	-99.01	0.00

1.2D + 1.6L				
Max $V_{TOP/LEFT}$ (kips)	-37.77	-12.78	-12.78	0.00
Max $V_{BOTTOM/RIGHT}$ (kips)	89.70	-12.78	-12.78	0.00
Max $M_{TOP/LEFT}$ (ft-kips)	-157.35	**157.35**	0.00	0.00
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	-287.46	**23.20**	-287.46	0.00
Max $M_{MIDSPAN}$ (ft-kips)	205.46	90.27	-143.73	0.00
Max P_u (kips)	-66.82	-95.75	-89.70	0.00

1.2D + 1.6W + 1.0L				
Max $V_{TOP/LEFT}$ (kips)	-45.24	-46.63	-15.21	-9.08
Max $V_{BOTTOM/RIGHT}$ (kips)	**99.20**	-46.63	-15.21	-9.08
Max $M_{TOP/LEFT}$ (ft-kips)	**-258.88**	**258.88**	-0.25	-158.90
Max $M_{BOTTOM/RIGHT}$ (ft-kips)	**-379.64**	**266.81**	-342.54	158.90
Max $M_{MIDSPAN}$ (ft-kips)	177.83	124.73	-171.39	79.45
Max P_u (kips)	-86.74	-103.86	-99.20	0.00

Table Accounts for Torsional Effects

BEAM DESIGN:

$V_{u,max}$ = 99.20 kips (1.2D + 1.6W + 1.0L)
$M_{u,max}$ at Supports = -379.64 k-ft (1.2D + 1.6W + 1.0L)
$M_{u,max}$ at Midspan = 205.46 k-ft (1.2D + 1.6L)

Use normal-weight concrete with f'_c = 4000 psi
f_y = 60,000 psi for flexural reinforcement
f_{yt} = 60,000 psi for stirrups

1) Choose the actual size of the beam stem.

a) Calculate the minimum depth based on deflections.

Use worst case scenario (use "simply supported" criteria).

ACI Table 9.5(a):

Minimum thickness, h = L/16 = [(23')(12 in/ft)]/16 = 17.25"

b) Determine the minimum depth based on the maximum negative moment.

$M_{u,max}$ at Supports = 379.64 k-ft

$\rho(\text{initial}) = [(\beta_1 f'_c)/(4f_y)] = [(0.85)(4\text{ ksi})/(4)(60\text{ ksi})] = 0.0142$

$\omega = \rho(f_y/f'_c) = (0.0142)(60\text{ ksi}/4\text{ ksi}) = 0.213$

$R = \omega f'c(1 - 0.59\omega) = (0.213)(4\text{ ksi})[1 - (0.59)(0.213)] = 0.745\text{ ksi}$

$bd^2 \geq M_u/\phi R = [(379.64\text{ ft-kips})(12\text{ in/ft})]/[(0.9)(0.745\text{ ksi})] = 6794.45\text{ in}^3$

Assuming b = 24 in.

$d \geq 16.83$ in.

h ≅ 16.83" + 3.25" = 20.08" (accounting for 2.25" clear cover due to corrosive environment; see ACI 7.7.6.1; (1.5)(1.5") = 2.25")

Try h = 26" > 20.76" ∴ Meets deflection criteria

d ≅ 26" – 3.25" = 22.75"

c) Check the shear capacity of the beam.

$V_u = \phi(V_c + V_s)$

$V_{u,max}$ = 99.20 kips

From ACI Code Section 11.2.1.1, the nominal V_c is

$$V_c = 2\lambda\sqrt{f'_c}b_wd = (2)(1.0)\sqrt{4000 \text{ psi}}(24")(22.75")/1000 = 69.06 \text{ kips}$$

ACI Code Section 11.4.7.9 sets the maximum nominal V_s as

$$V_s = 8\sqrt{f'_c}b_wd = (8)\sqrt{4000 \text{ psi}}(24")(22.75")/1000 = 276.26 \text{ kips}$$

Thus, the absolute maximum $\phi V_n = 0.75(69.06 \text{ k} + 276.26 \text{ k}) = 258.99$ kips

$\geq V_{u,max} = 99.20$ kips $\therefore$ OK

d) Summary. Use:

b = 24”
h = 26”
d = 22.75”

2) Compute the dead load of the stem, and recompute the total moment.

Weight of 24”x26” concrete beam = $[(24")(26")/144 \text{ in}^2/\text{ft}^2][(150 \text{ lb/ft}^3)/1000]$

= 0.650 k/ft

Original dead load = 2.6524 k/ft

New dead load = 2.6524 k/ft + (0.650 k/ft – 0.375 k/ft) = 2.9274 k/ft

(2.9274 k/ft)/(2.6524 k/ft) = 1.1037

New $M_{u,max}$ at Supports $\cong$ (1.2)(-91.83 k-ft*1.1037) + (1.6)(-99.16 k-ft) – 100.79 =

= 381.07 k-ft

New $M_{u,max}$ at Midspan $\cong$ (1.2)(65.63 k-ft*1.1037) + (1.6)(79.19 k-ft) = 213.63 k-ft

New $V_{u,max} \cong$ (1.2)(28.65 k*1.1037) + (1.6)(18.91 k) + 34.57 k = 102.77 k

$< \phi V_n = 258.99$ kips $\therefore$ Shear capacity is still OK.

3) Design the flexural reinforcement.

a) Compute the area of steel required at the point of maximum negative moment.

$$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$$

Because there is negative moment at the support, the beams acts as a rectangular beam with compression in the web. Assume that j = 0.9 and $\phi = 0.90$

$A_s \cong (381.07\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.9)(22.75")] = 4.14\text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (4.14\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 3.041"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (381.07\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(22.75" - 3.041"/2)]$

$= 3.99\text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (3.99\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 2.933"$

$c = a/\beta_1 = 2.933"/0.85 = 3.451" < (3/8)(d) = (3/8)(22.75") = 8.531"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

b) Compute the area of steel required at the point of maximum positive moment.

$A_s \geq M_u/[\phi f_y(d - a/2)] \cong M_u/[\phi f_y(jd)]$

Assume that the compression zone is rectangular, and take j = 0.95 for the first calculation of A_s.

$A_s \cong (213.63\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(0.95)(22.75")] = 2.20\text{ in.}^2$

This value can be improved with one iteration to find the depth of the compression stress block, a:

$a = A_s f_y/0.85f'_c b = (2.20\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 1.618"$

and then recalculating the required A_s with this calculated value of a:

$A_s \geq M_u/[\phi f_y(d - a/2)] = (213.63\text{ k-ft})(12\text{ in/ft})/[(0.9)(60\text{ ksi})(22.75" - 1.618"/2)]$

$= 2.16\text{ in}^2$

Before proceeding, it must be confirmed that this is a tension-controlled section. This can be done by showing that the neutral axis, c, is less than 3/8 of d.

$a = A_s f_y/0.85f'_c b = (2.16\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 1.591"$

$c = a/\beta_1 = 1.591"/0.85 = 1.872" < (3/8)(d) = (3/8)(22.75") = 8.531"$

$\therefore$ Section is tension-controlled and can be designed using $\phi = 0.90$

c) Calculate the minimum reinforcement (using ACI Code Section 10.5.1).

$A_{s,\,min}$ = max. of:

$[3\sqrt{f'_c}/f_y]b_wd = [3\sqrt{4000\text{ psi}}/60000\text{ psi}](24")(22.75") = 1.73\text{ in}^2$

$200b_wd/f_y = (200)(24")(22.75")/60000\text{ psi} = 1.82\text{ in}^2$

$\therefore A_{s,min} = 1.82\text{ in}^2$

4) Calculate the area of steel and select the bars.

a) Negative-moment Region

$A_{s,req} = 3.99\text{ in}^2 > A_{s,min} = 1.82\text{ in}^2 \therefore$ OK

Use (7) #7 bars [$A_s = (7)(0.60\text{ in}^2) = 4.20\text{ in}^2 > 3.99\text{ in}^2 \therefore$ OK]

$a = A_sf_y/0.85f'_cb = (4.20\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 3.088"$

$a = \beta_1c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 3.088"/0.85 = 3.633"$

$d_{actual} = 26" - 2.25" - 0.5" - (1/2)(0.875") = 22.8125$

$\varepsilon_s = (d-c)(\varepsilon_u)/c = (22.8125" - 3.633")(0.003)/3.633" = 0.01584 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.01584 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_sf_y(d - a/2) = (0.9)(4.20\text{ in}^2)(60\text{ ksi})(22.8125" - 3.088"/2)/(12\text{ in/ft}) =$

$= 401.97\text{ k-ft} > 381.07\text{ k-ft} \therefore$ OK

Small bars were selected at the supports because the bars have to be hooked into the exterior supports and there may not be enough room for a standard hook on larger bars.

b) Positive-moment Region

$A_{s,req} = 2.16\text{ in}^2 > A_{s,min} = 1.82\text{ in}^2 \therefore$ OK

Use (4) #7 bars [$A_s = (4)(0.60\text{ in}^2) = 2.40\text{ in}^2 > 2.16\text{ in}^2 \therefore$ OK]

$a = A_sf_y/0.85f'_cb = (2.40\text{ in}^2)(60\text{ ksi})/[(0.85)(4\text{ ksi})(24")] = 1.765"$

$a = \beta_1 c$ = where $\beta = 0.85$ for $f'_c = 4{,}000$ psi

$c = a/\beta 1 = 1.765''/0.85 = 2.076''$

$\varepsilon_s \cong (d-c)(\varepsilon_u)/c = (22.8125'' - 2.076'')(0.003)/2.076'' = 0.02997 > \varepsilon_y = 0.00207$

$\varepsilon_t \cong \varepsilon_s = 0.02997 > 0.005 \therefore$ Tension-controlled Section $\therefore \phi = 0.9$

$\phi M_n = \phi A_s f_y(d - a/2) = (0.9)(2.40\text{ in}^2)(60\text{ ksi})(22.8125'' - 1.765''/2)/(12\text{ in/ft}) =$

$= 236.84$ k-ft > 213.63 k-ft $\therefore$ OK

5) Check the distribution of the reinforcement (spacing requirements).

a) Negative-moment Region

$c_c = 2.25$ in. cover + 0.5 in. stirrups = 2.75”

The maximum bar spacing is

$s = 15(40{,}000/f_s) - 2.5c_c$

$f_s = (2/3)(f_y) = (2/3)(60{,}000\text{ ksi}) = 40{,}000$ ksi

$s = 15(40{,}000/40{,}000) - (2.5)(2.75'') = 8.125''$

Spacing of bars is less than 8.125” by inspection.

Minimum bar spacing:

s_c = max of [1”, d_b, $(4/3)s_a$]; Assume s_a = 1” aggregate

s_c = max of [1”, 0.875”, (4/3)(1”) = 1.333”]; Assume s_a = 1” aggregate

$s_c = 1.333''$

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

$24'' > (7)(0.875'') + (7-1)(1.333'') + (2)(0.5'') + (2)(2.25'')$

$24'' > 19.62'' \therefore$ OK

b) Positive-moment Region

The maximum bar spacing is 8.125”. Spacing of bars is less than 8.125” by inspection.

Minimum bar spacing = 1.333”

Side spacing and cover:

$b > (n)(d_b) + (n-1)(s_c) + 2d_{tr} + 2c_c$

24” > (4)(0.875”) + (4-1)(1.333”) + (2)(0.5”) + (2)(2.25”)

24” > 14.00” ∴ OK

6) Design the shear reinforcement.

a) The critical section for shear is located at the support. ACI Code Section 11.4.6.1 requires stirrups if $V_u \geq \phi V_c/2$

$V_c = 2\lambda\sqrt{f'_c}b_wd$ = (2)(1.0) √4000 psi (24”)(22.8175”)/1000 = 69.27 kips

$V_c/2$ = 69.27 kips/2 = 34.63 kips

V_u/ϕ = (102.77 kips)/(0.75) = 137.03 kips > $V_c/2$ = 34.63 kips

∴ Stirrups are required.

b) Determine shear strength required by shear reinforcing.

$V_s = V_u/\phi - V_c$ = [(102.77 kips)/(0.75)] – 69.27 kips = 67.76 kips

$V_s \leq 8\sqrt{f'_c}b_wd$ = 8√4000 psi (24”)(22.8125”)/1000 = 277.02 kips ∴ OK

c) Determine maximum spacing of shear reinforcing (ACI 318-08 Sections 11.4.5.1 and 11.4.5.3).

For $V_s \leq 8\sqrt{f'_c}b_wd$: s_{max} = min of {d/2, 24”}

d/2 = 22.8125”/2 = 11.41”

s_{max} = 11”

d) Determine minimum shear reinforcement (ACI 318-08 Section 11.4.6.3).

$A_{v,min}$ = max of $\{0.75\sqrt{f'_c}b_ws/f_{yt}, 50b_ws/f_{yt}\}$

$0.75\sqrt{f'_c}b_ws/f_{yt}$ = 0.75√4000 psi (24”)(11”)/60,000 psi = 0.209 in^2

$50b_ws/f_{yt}$ = 50(24”)(11”)/60,000 psi = 0.220 in^2

∴ $A_{v,min}$ = 0.220 in^2

Use #3 stirrups @ 11” as minimum shear reinforcement.

(A_v = 2 legs x 0.11 in^2/leg = 0.22 $in^2 \geq$ 0.220 in^2 $\therefore$ OK)

e) Design the shear reinforcement.

$V_s = A_v f_{yt} d/s$

Rearranging: $s = A_v f_{yt} d/V_s$ = (0.22 in^2)(60 ksi)(22.8125")/67.76 kips = 4.44"

Usually absolute minimum "s" is 4".

Use (2) #3 stirrups @ 4", starting 2" from face of support.

Or use #4 stirrups instead of #3 stirrups.

For #4 stirrups: (A_v = 2 legs x 0.20 in^2/leg = 0.40 in^2 > 0.200 in^2 $\therefore$ OK)

$s = A_v f_{yt} d/V_s$ = (0.40 in^2)(60 ksi)(22.8125")/67.76 kips = 8.08"

Use (2) #4 stirrups @ 8", starting 2" from face of support.

Use this stirrup layout throughout the entire length of the beam since lateral loads can change the shear forces (shear diagram) throughout the beam length (since the beam is part of a concrete moment frame).

FINAL DESIGN: Use 24" x 26" beam with (7) #7 bars in a single layer for negative moment reinforcement (at the supports) and (4) #7 bars for positive moment reinforcement. Use (2) #4 stirrups @ 8" throughout length of beam.

COLUMN DESIGN:

<u>Columns at Column Line 1.8:</u>

These columns were already designed for gravity forces and lateral forces in the North/South direction. The design resulted in 24"x24" concrete columns with (12) #8 bars.

Check this column size and reinforcement for gravity loads and lateral loads in the East/West direction. The total P_u will be the same (may vary depending on load cases), but the moments (M_1 and M_2) at the top and bottom of the column will change. The P_u used for the North/South design already been calculated and that value for P_u will thus be used for this column check.

Controlling Load Case: 1.2D + 1.6L

P_u = 177.98 kips (same as the design for the North/South direction)

M_2 = 266.81 k-ft

M_1 = 258.88 k-ft

1) Preliminary column size

$A_{g(trial)} \geq P_u/[0.40(f'_c + f_y\rho_g)$

$A_{g(trial)} \geq$ 177.98 kips/[0.40(4 ksi + (60 ksi)(0.015))] = 90.81 in^2

$\cong$ (9.53 in.)^2

Try 24"x24" column (already designed for North/South direction)

2) Is the story being designed sway or nonsway?

$Q = [\sum P_u \times \Delta_o]/[V_{us} \times l_c]$

$\sum P_u \cong$ (5)(177.98 k) = 889.90 k

V_{us} = 1 kip

Δ_o = 0.014789"

l_c = 10.5' = 126"

Q = [(889.90 kips)(0.014789")]/[(1 kip)(126")] = 0.01045 < 0.05

$\therefore$ Nonsway (but assume sway story because $\sum P_u$ will actually be higher due to loads at other columns around the building at that level)

3) Are the columns slender?

$r = 0.3h = (0.3)(24") = 7.2"$
$kl_u/r = (1.2)(126")/7.2" = 21 < 22$ (for a sway frame) $\therefore$ Column is not slender

2) Compute γ

For a 24"x24" column: $\gamma = [24" – (2)(2.5")]/24" = 0.7917$

3) Use interaction diagrams to determine ρ_g

$\phi P_n/A_g = P_u/A_g = 177.98 \text{ k}/[(24")(24")] = 0.3099$

$\phi M_n/A_g h = M_u/A_g h = (379.64 \text{ k-ft})(12 \text{ in/ft})/[(24"x24")(24")] = 0.3295$

From Fig. A-9b (from "Reinforced Concrete Mechanics and Design" by White and MacGregor):

$\rho_g = 0.010 < 0.016$ (provided) $\therefore$ OK

$\rho_{g,provided} = (12)(0.79 \text{ in}^2)/[(24")(24")] = 0.016$

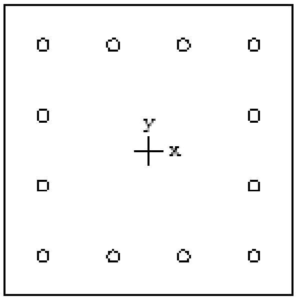
The 24"x24"column with (12) #8 bars is OK

PCA Column was also used to check the 24"x24" column with (12) #8 bars

(Pu,Mu) = (177.98 k, 266.81 k-ft)

This point lies within the boundaries on the interaction diagram from PCA column (see diagram below).

$\therefore$ Column is OK



24 x 24 in

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

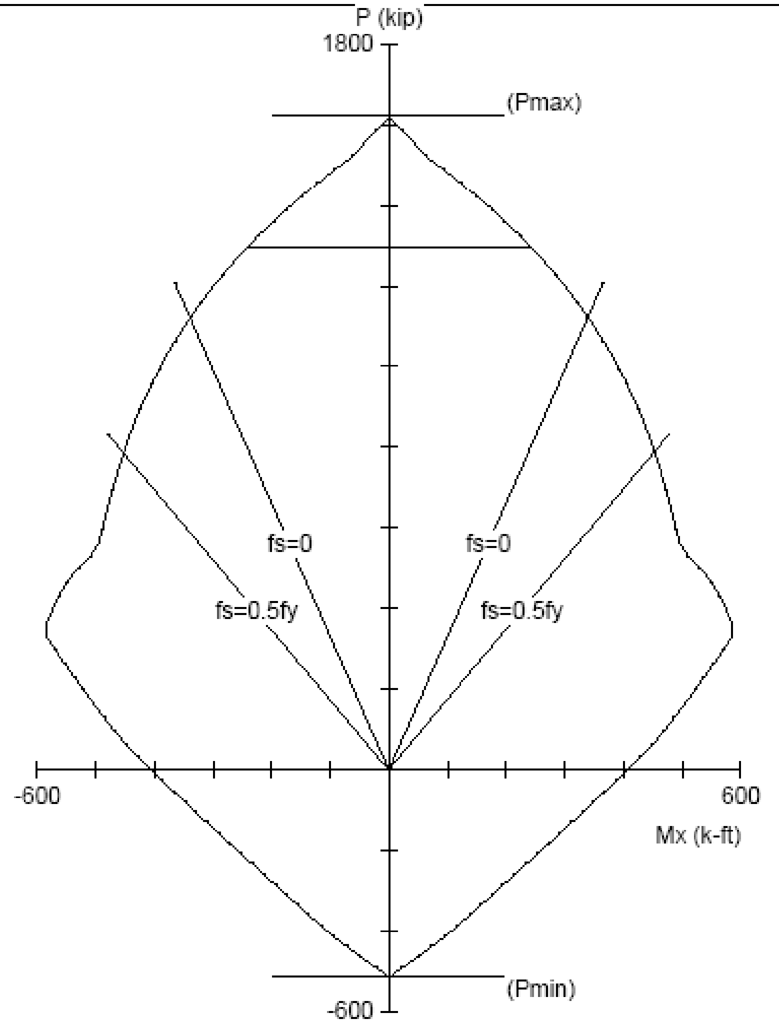
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/22/10

Time: 21:24:58



pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D

File: F:\AE 897G\3-22-10\18x18 with (8) #8 bars.col

Project: Kukorlo - Thesis - 18x18

Column:		Engineer:	
f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	12 #8 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 9.48 in^2	Rho = 1.65%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Clear spacing = 4.83 in	Clear cover = 2.75 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

Wood Braced Frame – East/West Direction

Design of Diagonal Members:

P_u = 13.72 k (compression)

Analyze Member Buckling About x Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_x = [(1.0)(26.2552')(12\ in/ft)]/d \leq 50$

$d \geq l_e/50 = [(26.2552')(12\ in/ft)]/50 = 6.30''$

Analyze Member Bucking About y Axis:

$(l_e/d)_{max} = 50$

$(l_e/d)_y = [(1.0)(13.1276')(12\ in/ft)]/d \leq 50$

$d \geq l_e/50 = [(13.1276')(12\ in/ft)]/50 = 3.15''$

Try 5”” x 6 7/8”

$(l_e/d)_x = [(26.2662')(12\ in/ft)]/6.875'' = 45.846$

$(l_e/d)_y = [(13.1276')(12\ in/ft)]/5'' = 31.5062$

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

E_{min} = 980,000 psi

C_D = 1.6 (for wind load))

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

$C_t = 1.0$

$E'_{min} = (E_{min})(C_M)(C_t) = (980{,}000)(0.833)(1.0) = 816{,}340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340\ psi)]/[(45.846)^2] = 319.257$ psi

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300\ psi)(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 319.257/2686.4 = 0.1188$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.1188]/[(2)(0.9)] = 0.6216$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.6216\} - \sqrt{\{[0.6216]^2 - [0.1188/0.9]\}}$

$= 0.1173$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.1173) = 315.004 \text{ psi}$

$P = (F'_c)(A)$

$A_{req'd} = P/F'_c = 13{,}720 \text{ lb}/315.004 \text{ psi} = 43.56 \text{ in}^2 > A_{provided} = 34.38 \text{ in}^2$ $\therefore$ **N.G.**

Try 6 ¾" x 6 7/8"

$(l_e/d)_x = [(26.2662')(12 \text{ in/ft})]/6.875" = 45.846$

$(l_e/d)_y = [(13.1276')(12 \text{ in/ft})]/6.75" = 23.338$

Same C_P and Areq'd

$A_{req'd} = P/F'_c = 13{,}720 \text{ lb}/315.004 \text{ psi} = 43.56 \text{ in}^2 < A_{provided} = 46.41 \text{ in}^2$ $\therefore$ **OK**

Use 6 ¾" x 6 7/8" Southern Pine glulam ID #50

Wind Columns

Try truss design with 3'-0" depth:

LOAD COMBINATION: D+W (Combined Bending and Axial Forces) (Controls)

"Top Chord"

P_{max} = 22.238 k + (30 psf/53.1 psf)(5.5522 k) = 25.375 k (Compression)

M_{max} = 4.1695 ft-k = 4169.5 ft-lb = 50,034 in-lb

Try 6 ¾" x 11"

F_c = 2300 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

F_b = 2100 psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

A = 74.25 in^2

S = 136.1 in^3

E_{min} = 980,000 psi

Axial Load: P_{max} = 25,375 lb (Compression)

Maximum Moment: M_{max} = 50,034 in-lb

L = 6.667'

Axial Load:

f_c = P/A = 25,375 lb/74.25 in^2 = 341.751 psi

$(l_e/d)_x$ = [(6.667')(12 in/ft)]/11" = 7.2727 < 50 ∴ OK

$(l_e/d)_y$ = [(13.333')(12 in/ft)]/6.75" = 23.7037 < 50 ∴ OK

$(l_e/d)_{max}$ = $(l_e/d)_x$ = 23.7037

The larger slenderness ratio governs the adjusted design value. Therefore, the weak axis of the member is critical, and $(l_e/d)_y$ is used to determine F'_c.

C_D = 1.6 (for wind load; load combination D+W)

C_M = 0.73 for F_c (p. 64, NDS Supplement)

C_M = 0.833 for E and E_{min} (p. 64, NDS Supplement)

C_M = 0.8 for F_b (p. 64, NDS Supplement)
C_t = 1.0

$E'_{min} = (E_{min})(C_M)(C_t) = (980,000)(0.833)(1.0) = 816,340$ psi

c = 0.9 (glulam)

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816,340 \text{ psi})]/[(27.7037)^2] = 874.314$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 874.314/2686.4 = 0.3255$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.3255]/[(2)(0.9)] = 0.7364$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 - [F_{cE}/F_c^*]/c\}}$

$= \{0.7364\} - \sqrt{\{[0.7364]^2 - [0.3255/0.9]\}}$

$= 0.3115$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.3115) = 836.723$ psi

Axial stress ratio = f_c/F'_c = (341.751 psi)/(836.723 psi) = 0.4084

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

A_n = (6.75")[11" – (2)(0.8125")] = 63.28 in^2

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n$ = 25,375 lb/63.28 in^2 = 400.988 psi

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300 \text{ psi})(1.6)(0.73)(1.0)(0.3115) = 836.814$ psi

836.814 psi > 400.988 psi ∴ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

M = 50,034 in-lb

$S = 136.1\ in^3$

$f_b = M/S = 50{,}034$ in-lb/$136.1\ in^3 = 367.627$ psi

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.333')(12\ in/ft)]/11'' = 14.545 > 7$

$\therefore\ l_e = 1.63l_u + 3d = (1.63)[(13.333')(12\ in/ft)] + (3)(11'') = 293.799''$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(293.799'')(11'')/(6.75'')^2]} = 8.422$

$F_{bE} = 1.20E'_{min}/R_B^2 = [(1.20)(816{,}340\ psi)]/(8.422)^2 = 13{,}810.721$ psi

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100\ psi)(1.6)(0.8)(1.0) = 2688$ psi

$F_{bE}/F^*_b = (13810.721)/(2688) = 5.1379$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 5.1379)/1.9 = 3.2305$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 3.2305 - \sqrt{(3.2305)^2 - (5.1379/0.95)]} = 0.9882$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12''/d)^{1/20}(5.125''/b)^{1/20} \leq 1.0$

$C_V = (21'/60')^{1/20}(12''/11'')^{1/20}(5.125''/6.75'')^{1/20} \leq 1.0$

$C_V = 0.9400 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688\ psi)(0.9400) = 2526.72$ psi

Bending stress ratio $= f_b/F'_b = (367.627\ psi)/(2526.72\ psi) = 0.1455$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{bending\ moment} = (l_e/d)_x = 7.2727$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340\ psi)]/[(7.2727)^2] = 12686.784$ psi

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 – (f_c/F_{cEx})] = 1/[1 – (341.751 \text{ psi}/12686.784 \text{ psi})] = 1.0277$

$(f_c/F'_c)^2 + \{1/[1 – (f_c/F_{cEx})]\}(f_b/F'_b) = (0.4084)^2 + (1.0277)(0.1455) = 0.3163 < 1.0 \therefore$ **OK**

Try 6 ¾” x 6 7/8”

$F_c = 2300$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$F_b = 2100$ psi (Glulam ID #50, S.P.) (p. 66 NDS Supplement)

$A = 46.41 \text{ in}^2$

$S = 53.17 \text{ in}^3$

$E_{min} = 980{,}000$ psi

Axial Load: $P_{max} = 25{,}375$ lb (Compression)

Maximum Moment: $M_{max} = 50{,}034$ in-lb

L = 6.667’

Axial Load:

$f_c = P/A = 25{,}375 \text{ lb}/46.41 \text{ in}^2 = 546.757$ psi

$(l_e/d)_x = [(6.667’)(12 \text{ in/ft})]/6.875” = 11.6364 < 50 \therefore$ OK

$(l_e/d)_y = [(13.333’)(12 \text{ in/ft})]/6.75” = 23.7037 < 50 \therefore$ OK

$(l_e/d)_{max} = (l_e/d)_x = 23.7037$

The larger slenderness ratio governs the adjusted design value. Therefore, the weak axis of the member is critical, and $(l_e/d)_y$ is used to determine F'_c.

$F_{cE} = [0.822E'_{min}]/[(l_e/d)^2] = [(0.822)(816{,}340 \text{ psi})]/[(27.7037)^2] = 874.314$ psi

Here, l_e/d is based on $(l_e/d)_{max}$.

$F_c^* = F_c(C_D)(C_M)(C_t) = (2300 \text{ psi})(1.6)(0.73)(1.0) = 2686.4$ psi

$F_{cE}/F_c^* = 874.314/2686.4 = 0.3255$

$[1 + F_{cE}/F_c^*]/(2c) = [1 + 0.3255]/[(2)(0.9)] = 0.7364$

$C_P = \{[1 + F_{cE}/F_c^*]/(2c)\} - \sqrt{\{[(1 + F_{cE}/F_c^*)/(2c)]^2 – [F_{cE}/F_c^*]/c\}}$

$= \{0.7364\} - \sqrt{\{[0.7364]^2 – [0.3255/0.9]\}}$

$= 0.3115$

$F'_c = F_c^*(C_P) = (2686.4 \text{ psi})(0.3115) = 836.723 \text{ psi}$

Axial stress ratio $= f_c/F'_c = (546.757 \text{ psi})/(836.723 \text{ psi}) = 0.6535$

Net Section Check:

Assume connections will be made with (2) rows of ¾" diameter bolts.

Assume the hole diameter is 1/16" larger than the bolt (for stress calculations only).

$A_n = (6.75")[6.875" - (2)(0.8125")] = 35.44 \text{ in}^2$

(3/4" + 1/16" = 0.8125")

$f_c = P/A_n = 25{,}375 \text{ lb}/35.44 \text{ in}^2 = 715.999 \text{ psi}$

$F'_c = F_c^* = F_c(C_D)(C_M)(C_t)(C_P) = (2300 \text{ psi})(1.6)(0.73)(1.0)(0.3115) = 836.814 \text{ psi}$

$836.814 \text{ psi} > 715.999 \text{ psi} \therefore$ OK

Bending:

Bending is about the strong axis of the cross section. The adjusted bending design value for a glulam is governed by the smaller of two criteria: volume effect or lateral stability.

$M = 50{,}034 \text{ in-lb}$

$S = 53.17 \text{ in}^3$

$f_b = M/S = 50{,}034 \text{ in-lb}/53.17 \text{ in}^3 = 941.019 \text{ psi}$

$F'_b = F_b(C_D)(C_M)(C_t)(C_L)$ or

$F'_b = F_b(C_D)(C_M)(C_t)(C_V)$

For C_L: $l_u/d = [(13.333')(12 \text{ in/ft})]/6.875" = 23.272 > 7$

$\therefore l_e = 1.63l_u + 3d = (1.63)[(13.333')(12 \text{ in/ft})] + (3)(6.875") = 281.425"$

$R_B = \sqrt{l_e d/b^2} = \sqrt{[(281.425")(6.875")/(6.75")^2]} = 6.516$

$F_{bE} = 1.20E'_{min}/R_B^2 = [(1.20)(816{,}340 \text{ psi})]/(6.516)^2 = 23{,}068.884 \text{ psi}$

$F^*_b = F_b(C_D)(C_M)(C_t) = (2100 \text{ psi})(1.6)(0.8)(1.0) = 2688 \text{ psi}$

$F_{bE}/F^*_b = (23068.884)/(2688) = 8.5821$

$(1 + F_{bE}/F^*_b)/1.9 = (1 + 8.5821)/1.9 = 5.0432$

$C_L = [(1 + F_{bE}/F^*_b)/1.9] - \sqrt{\{[(1 + F_{bE}/F^*_b)/1.9]^2 - [F_{bE}/F^*_b/0.95]\}}$

$= 5.0432 - \sqrt{(5.0432)^2 - (8.5821/0.95)]} = 0.9935$

For Southern Pine glulam:

$C_V = (21'/L)^{1/20}(12"/d)^{1/20}(5.125"/b)^{1/20} \leq 1.0$

$C_V = (21'/60')^{1/20}(12"/6.875")^{1/20}(5.125"/6.75")^{1/20} \leq 1.0$

$C_V = 0.9623 \leq 1.0$

C_V governs of C_L

$F'_b = F^*_b(C_V) = (2688 \text{ psi})(0.9623) = 2586.662 \text{ psi}$

Bending stress ratio = $f_b/F'_b = (941.019 \text{ psi})/(2586.662 \text{ psi}) = 0.3638$

Combined Stresses:

The bending moment is about the strong axis of the cross section, and the amplification for P-Δ is measured by the column slenderness ratio about the x axis.

$(l_e/d)_{\text{bending moment}} = (l_e/d)_x = 11.6364$

$F_{cEx} = [0.822E'_{min}]/[(l_e/d)_x]^2 = [(0.822)(816{,}340 \text{ psi})]/[(11.6364)^2] = 4955.707 \text{ psi}$

*Here, (l_e/d) is based on the axis about which the bending moment occurs.

Amplification factor = $1/[1 - (f_c/F_{cEx})] = 1/[1 - (546.757 \text{ psi}/4955.707 \text{ psi})] = 1.1240$

$(f_c/F'_c)^2 + \{1/[1 - (f_c/F_{cEx})]\}(f_b/F'_b) = (0.6535)^2 + (1.1240)(0.3638) = 0.8360 < 1.0 \therefore$ **OK**

FINAL MEMBER SIZE = 6 ¾” x 6 7/8” Southern Pine Glulam ID #50

Overturning Check

Wood Braced Frame at Column Line 1:

Look at load combination: 0.9D + 1.6W (controlling load combination)

Tributary area for each frame = (8')(130'/2) = 520 SF

Wind uplift = 16.28 PSF

Upward/overturning force due to 1.6W (applied lateral force)

= 36.71 k (from SAP model)

Upward/overturning force due to wind uplift = (1.6)(16.28 PSF)(520 SF)/1000 =

= 13.54 k

Total upward force at base = 36.71 k + 13.54 k = 50.25 k

Resistance is provided by applied dead load plus dead load of concrete footing and concrete pier.

Dead load applied to column = 21.34 k (from SAP model)

Footing: [(19')(19')(2')](150 PCF)/1000 = 108.3 k

Pier: [(9.667')(8.333')(10')](150 PCF)/1000 = 106.3 k

These footing and pier sizes are from the original building, which had columns spaced at 30'-0" o.c. at column line 1. Since the design with the wood trusses has columns spaced at 8' o.c., it will be assumed that the dead load of the footing and pier will be about one-quarter of that from the original design.

Footing ≅ (1/4)(108.3 k) = 27.035 k

Pier ≅ (1/4)(106.3 k) = 26.575 k

Total resistance due to dead load = (0.9)(21.34 k + 27.035 k + 26.575 k) = 67.46 k

67.46 k > 50.25 k ∴ OK

The dead weight of the roof load plus the estimated self weight of the concrete footings and piers at this location was able to resist the upward forces caused by the overturning moments due to the wind loads. However, since the weight of the footings and piers is only an estimate, overturning will need to be investigated more closely using the final concrete footing and piers sizes. The applied live roof load was conservatively omitted from this check and would help resist overturning as well.

Concrete Moment Frame at Column Line 2 (North/South Direction):

Look at load combination: 0.9D + 1.6W

Tributary area for each frame = (32')(130'/2) = 2080 SF

Wind uplift = 16.28 PSF

Upward/overturning force due to 1.6W (applied lateral force)

= (1.6)(11.43 k) = 18.29 k (from SAP model)

Upward/overturning force due to wind uplift = (1.6)(16.28 PSF)(2080 SF)/1000 =

= 54.18 k

Total upward force at base = 18.29 k + 54.18 k = 72.47 k

Resistance is provided by applied dead load plus dead load of concrete column, concrete footing, and concrete pier.

Dead load applied to column = 130.28 k (from SAP model)

Resistance due to dead load = (0.9)(130.28 k) = 117.25 k

117.25 k > 72.47 k ∴ OK

The dead weight applied to the exterior column of the concrete moment frame at column line 2 was able to resist the overturning forces by itself. Therefore, there was no need to consider the self weight of the concrete column, concrete footing, and pier, which also help to resist the overturning moment. Hence, overturning is not a concern at the moment frame at column line 2.

<u>Concrete Moment Frame in East/West Direction:</u>

Look at load combination: 0.9D + 1.6W (controlling load combination)

Tributary area for each frame = (32')(130'/2) = 2080 SF

Wind uplift = 16.28 PSF

Upward/overturning force due to 1.6W (applied lateral force)

= (1.6)(18.91 k) = 30.26 (from SAP model)

Upward/overturning force due to wind uplift = (1.6)(16.28 PSF)(2080 SF)/1000 =

= 54.18 k

Total upward force at base = 30.26 k + 54.18 k = 84.44 k

Resistance is provided by applied dead load plus the self weight of the concrete footing and the concrete column.

Dead load applied to column = 30.59 k (from SAP model)

Footing: [(13.5')(13.5')(2.75')](150 PCF)/1000 = 75.18 k

Total resistance due to dead load = (0.9)(30.59 k + 75.18 k) = 95.19 k

95.19 k > 84.44 k ∴ OK

The applied dead load and self weight of the concrete footing can resist the overturning moment due to wind. The self weight of the column was conservatively not considered, but would assist in resisting overturning as well.

Wood Braced Frame in East/West Direction:

Look at load combination: 0.9D + 1.6W (controlling load combination)

Tributary area for each frame = (26')(9.125') = 237.25 SF

Wind uplift = 16.28 PSF

Upward/overturning force due to 1.6W (applied lateral force)

= (1.6)(17.55 k) = 28.08 k (from SAP model)

Upward/overturning force due to wind uplift = (1.6)(16.28 PSF)(237.25 SF)/1000 =

= 6.18 k

Total upward force at base = 28.08 k + 6.18 k = 34.26 k

Resistance is provided by applied dead load plus the self weight of the concrete footing.

Dead load applied to column = 5.10 k

Footing: [(5')(5')(1')](150 PCF)/1000 = 3.75 k

Total resistance due to dead load = (0.9)(5.10 k + 3.75 k) = 8.00 k

8.00 k < 34.26 k ∴ N.G.

The applied dead load and self weight of the concrete footing cannot resist the overturning moment due to wind. Therefore, connections at the base of the column need to be investigated further (connections must be able to resist the uplift forces and hence prevent overturning).

Foundation Check

Concrete Moment Frame – Column Line 2

P_D = 190.87 k

P_{Lr} = 113.03 k

P_W = 1.55 k

P_u = 411.13 k (1.2D + 1.6L_r + 0.8W) + Weight of Concrete Column

[(24")(24")]/(144 in^2/ft^2) = 4 SF

(4 SF)(40') = 160 ft^3

Weight of Concrete Column = (160 ft^3)(150 lb/ft^3)/1000 = 24 k

P_u = 411.13 k + (1.2)(24 k) = 439.93 k

M_D = 1.03 k-ft

M_{Lr} = 1.26 k-ft

M_W = 209.68 k-ft

M_u = 170.99 k-ft (1.2D + 1.6L_r + 0.8W)

Foundation Size: 15'-0" x 15'-0" x 2'-9" with (17) #7 bars each way, top and bottom

q_a = 2500 psf

f'_c = 4,000 psi

P = P_D + P_L + P_W = 190.87 k + 113.03 k + 1.55 k = 305.45 k

M = M_D + M_{Lr} + M_W = 1.03 k-ft + 1.26 k-ft + 209.68 k-ft = 211.97 k-ft

M = (P)(e)

211.97 k-ft = (305.45 k)(e)

e = 0.694' = 8.328"

$q_a \geq$ P/A + M/S

S = $bh^2/6$

2.5 ≥ = (305.45 k)/[(15')(15')] + (211.97 k-ft)/[(15')$(15')^2$/6] = 1.358 ksf + 0.377 ksf = 1.734 ksf

$\therefore$ OK

B/6 = 15'/6 = 2.5' > e = 0.694' $\therefore$ In the kern (do not need to worry about overturning)

L' = L – 2e = 15' – (2)(0.694') = 13.612'

A' = (B)(L') = (15')(13.612') = 204.18 ft^2

P/A' = (305.45 k)/(204.18 ft^2) = 1.496 ksf < 2.5 ksf = q_a $\therefore$ OK

∑M = [(305.45 k)(15'/2) – 211.97 k-ft] = +2078.91 k-ft ($\therefore$ Stable since positive)

$M_{resisting}$ = (305.45)(15'/2) = 2290.88 k-ft

$M_{overturning}$ = 211.97 k-ft

P_u = 439.93 k

M_u = 170.99 k-ft

e = M_u/P_u = (170.99 k-ft)/(439.93 k) = 0.389' = 4.664"

L' = L – 2e = 15' – (2)(0.346') = 14.308'

A' = (B)(L') = (15')(14.31') = 214.65 ft^2

q = P_u/A' = (439.93 k)/(214.65 ft^2) = 2.050 ksf

Wide Beam Shear:

V_u = (2.050 ksf)[[(15'-2')/2] – d/12](1') = (0.75)(2)√4000(12")(d)/1000

13.325 – 0.1708d = 1.138d

d ≥ 10.178"

$d_{provided}$ > 10.178" $\therefore$ OK

Punching Shear:

$v_c = P_u/\{[2d(b+d) + 2d(c+d)]\}$

$4d^2 + 2d(b+c) = P_u/v_c$

$v_c = \phi v_c = \phi(2 + 4/\beta)\sqrt{f'_c} = \phi(2 + 4/1)\sqrt{f'_c} = \phi 6\sqrt{f'_c}$

$= \phi 4\sqrt{f'c} = (0.75)(4)\sqrt{4000} = 189.737$ psi

$4d^2 + 2d(24" + 24") = (439{,}930 \text{ lb})/(189.737 \text{ psi})$

$4d^2 + 96d - 2318.63 = 0$

$d \geq 14.90"$

With #7 bars: $h = 14.90" + 3" + 0.875" = 18.78" > h = 33" \therefore$ OK

Assume $d = 33" - 3" - (1/2)(0.875") = 20.563"$

Flexure:

$l = (15'\text{-}2')/2 = 6.5'$

$M = ql^2/2 = (2.050 \text{ ksf})(6.5')^2/2 = 43.31 \text{ k-ft}$

$a = A_sf_y/0.85f'_cb = (A_s)(60 \text{ ksi})/[(0.85)(4 \text{ ksi})(12")] = 1.471A_s$

$\phi M_n = \phi A_sf_y(d - a/2)$

$(43.31 \text{ k-ft})(12 \text{ in/ft}) = (0.9)(A_s)(60 \text{ ksi})(29.563" - 1.471A_s/2)$

$519.72 = 1596.40A_s - 39.717A_s^2$

$39.717A_s^2 - 1596.40A_s + 519.72 = 0$

$A_s \geq 0.328 \text{ in}^2/\text{ft}$

$A_{s,provided} = (17)(0.60 \text{ in}^2)/15' = 0.680 \text{ in}^2/\text{ft} > 0.328 \text{ in}^2/\text{ft} \therefore$ **OK**

Appendix C – Glass Strength Calculations

1) Determination of the Load Resistance of a Solar-Control Low-E Insulating-Glass Unit

Location: South Façade, Enclosing Lobby Area

Outer Lite: ¼” Fully Tempered (FT) Clear Float Glass, Monolithic

Inner Lite: ¼” Annealed Clear Float Glass, Monolithic

Air Space: ½”

Dimensions: 5’-0” x 9’-2” = 60” x 110”

Maximum Wind Pressure = 13.04 psf

NFL = Non-Factored Load, GTF = Glass Type Factor, LS = Load Share Factor

LR = Load Resistance

Assume an 8 in 1,000 breakage probability

Outer Lite (for Short Duration Load):

NFL = 1.18 kPa (Fig. A1.6, p. 12, E 1300) = (1.8 kPa)(20.9 psf/kPa) = 24.662 psf

Plate Length = 110”, Plate Width = 60”, Four Sides Simply Supported

GTF = 3.8 (Table 2, p. 2, E 1300, Fully Tempered, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (24.662 psf)(3.8)(2.00) = 187.43 psf

Inner Lite (for Short Duration Load):

NFL = 1.18 kPa (Fig. A1.6, p. 12, E 1300) = (1.8 kPa)(20.9 psf/kPa) = 24.662 psf

Plate Length = 110”, Plate Width = 60”, Four Sides Simply Supported

GTF = 1.0 (Table 2, p. 2, E 1300, Annealed, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (24.662 psf)(1.0)(2.00) = 49.32 psf

Outer Lite (for Long Duration Load):

NFL = 1.18 kPa (Fig. A1.6, p. 12, E 1300) = (1.8 kPa)(20.9 psf/kPa) = 24.662 psf

Plate Length = 110", Plate Width = 60", Four Sides Simply Supported

GTF = 2.85 (Table 3, p. 2, E 1300, Fully Tempered, Long Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (24.662 psf)(2.85)(2.00) = 140.57 psf

Inner Lite (for Long Duration Load):

NFL = 1.18 kPa (Fig. A1.6, p. 12, E 1300) = (1.8 kPa)(20.9 psf/kPa) = 24.662 psf

Plate Length = 110", Plate Width = 60", Four Sides Simply Supported

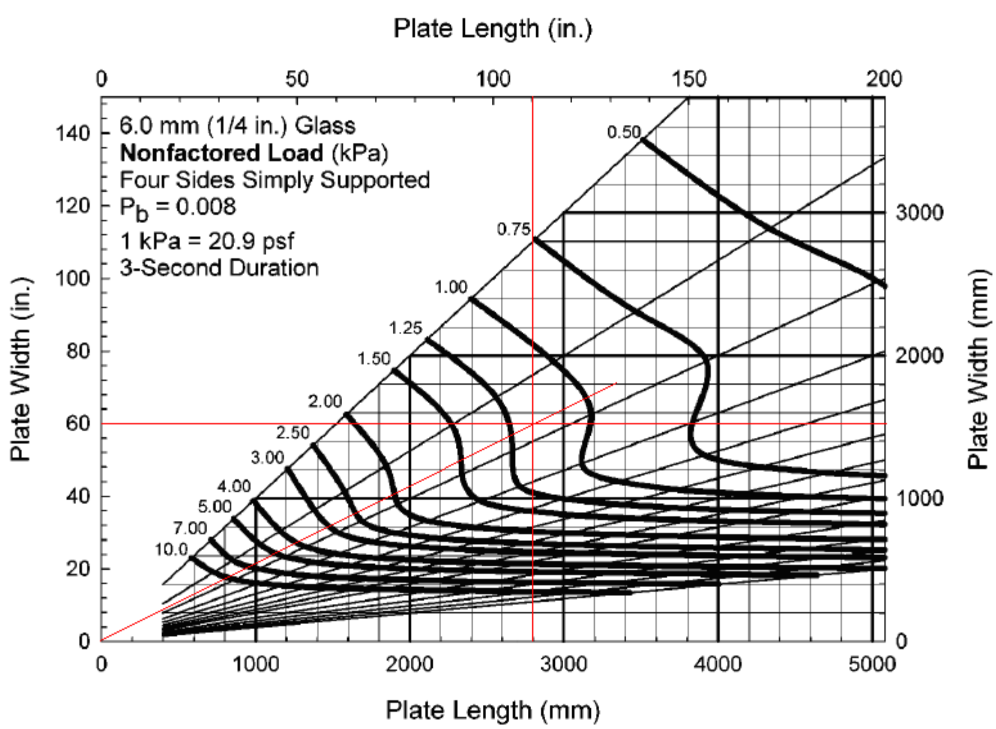
GTF = 0.5 (Table 3, p. 2, E 1300, Annealed, Long Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (24.662 psf)(0.5)(2.00) = 24.66 psf (Controls)

The load resistance of the IGU is 24.66 psf, being the least of the four values: 187.43, 49.32, 140.57, or 24.66 psf

LR = 24.66 psf > 13.04 psf ∴ **OK**



ASTM E-1300 Fig. A1.6

TABLE 2 Glass Type Factors (GTF) for Insulating Glass (IG), Short Duration Load

Lite No. 1 Monolithic Glass or Laminated Glass Type	Lite No. 2 Monolithic Glass or Laminated Glass Type					
	AN		HS		FT	
	GTF1	GTF2	GTF1	GTF2	GTF1	GTF2
AN	0.9	0.9	1.0	1.9	1.0	3.8
HS	1.9	1.0	1.8	1.8	1.9	3.8
FT	3.8	1.0	3.8	1.9	3.6	3.6

ASTM E 1300 – Table 2 – Glass Type Factors for Insulating Glass, Short Duration Load

E 1300 – 04$^{\epsilon 1}$

TABLE 5 Load Share (LS) Factors for Insulating Glass (IG) Units

NOTE 1—Lite No. 1 Monolithic glass, Lite No. 2 Monolithic glass, short or long duration load, or Lite No. 1 Monolithic glass, Lite No. 2 Laminated glass, short duration load only, or Lite No. 1 Laminated Glass, Lite No. 2 Laminated Glass, short or long duration load.

Lite No. 1		Lite No. 2																					
Monolithic Glass		Monolithic Glass, Short or Long Duration Load or Laminated Glass, Short Duration Load Only																					
Nominal Thickness		2.5 (3⁄32)		2.7 (lami)		3 (1⁄8)		4 (5⁄32)		5 (3⁄16)		6 (1⁄4)		8 (5⁄16)		10 (3⁄8)		12 (1⁄2)		16 (5⁄8)		19 (3⁄4)	
mm	(in.)	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2	LS1	LS2
2.5	(3⁄32)	2.00	2.00	2.73	1.58	3.48	1.40	6.39	1.19	10.5	1.11	18.1	1.06	41.5	1.02	73.8	1.01	169.	1.01	344.	1.00	606.	1.00
2.7	(lami)	1.58	2.73	2.00	2.00	2.43	1.70	4.12	1.32	6.50	1.18	10.9	1.10	24.5	1.04	43.2	1.02	98.2	1.01	199.	1.01	351.	1.00
3	(1⁄8)	1.40	3.48	1.70	2.43	2.00	2.00	3.18	1.46	4.83	1.26	7.91	1.14	17.4	1.06	30.4	1.03	68.8	1.01	140.	1.01	245.	1.00
4	(5⁄32)	1.19	6.39	1.32	4.12	1.46	3.18	2.00	2.00	2.76	1.57	4.18	1.31	8.53	1.13	14.5	1.07	32.2	1.03	64.7	1.02	113.	1.01
5	(3⁄16)	1.11	10.5	1.18	6.50	1.26	4.83	1.57	2.76	2.00	2.00	2.80	1.56	5.27	1.23	8.67	1.13	18.7	1.06	37.1	1.03	64.7	1.02
6	(1⁄4)	1.06	18.1	1.10	10.9	1.14	7.91	1.31	4.18	1.56	2.80	2.00	2.00	3.37	1.42	5.26	1.23	10.8	1.10	21.1	1.05	36.4	1.03
8	(5⁄16)	1.02	41.5	1.04	24.5	1.06	17.4	1.13	8.53	1.23	5.27	1.42	3.37	2.00	2.00	2.80	1.56	5.14	1.24	9.46	1.12	15.9	1.07
10	(3⁄8)	1.01	73.8	1.02	43.2	1.03	30.4	1.07	14.5	1.13	8.67	1.23	5.26	1.56	2.80	2.00	2.00	3.31	1.43	5.71	1.21	9.31	1.12
12	(1⁄2)	1.01	169.	1.01	98.2	1.01	68.8	1.03	32.2	1.06	18.7	1.10	10.8	1.24	5.14	1.43	3.31	2.00	2.00	3.04	1.49	4.60	1.28
16	(5⁄8)	1.00	344.	1.01	199.	1.01	140.	1.02	64.7	1.03	37.1	1.05	21.1	1.12	9.46	1.21	5.71	1.49	3.04	2.00	2.00	2.76	1.57
19	(3⁄4)	1.00	606.	1.00	351.	1.00	245.	1.01	113.	1.02	64.7	1.03	36.4	1.07	15.9	1.12	9.31	1.28	4.60	1.57	2.76	2.00	2.00

ASTM E 1300 – Table 5 – Load Share Factors for Insulating Glass Units

TABLE 3 Glass Type Factors (GTF) for Insulating Glass (IG), Long Duration Load

Lite No. 1 Monolithic Glass or Laminated Glass Type	Lite No. 2 Monolithic Glass or Laminated Glass Type					
	AN		HS		FT	
	GTF1	GTF2	GTF1	GTF2	GTF1	GTF2
AN	0.45	0.45	0.5	1.25	0.5	2.85
HS	1.25	0.5	1.25	1.25	1.25	2.85
FT	2.85	0.5	2.85	1.25	2.85	2.85

ASTM E 1300 – Table 3 – Glass Type Factors for Insulating Glass, Long Duration Load

2) <u>Determination of the Load Resistance of a Solar-Control Low-E Insulating-Glass Unit</u>

Location: East Façade, Enclosing Concessions Area

Outer Lite: ¼" Fully Tempered (FT) Clear Float Glass, Monolithic

Inner Lite: ¼" Annealed Clear Float Glass, Monolithic

Air Space: ½"

Dimensions: 5'-0" x 12'-6" = 60" x 150"

Maximum Wind Pressure = 12.92 psf

NFL = Non-Factored Load, GTF = Glass Type Factor, LS = Load Share Factor

LR = Load Resistance

Assume an 8 in 1,000 breakage probability

Outer Lite (for Short Duration Load):

NFL = 0.75 kPa (Fig. A1.6, p. 12, E 1300) = (0.75 kPa)(20.9 psf/kPa) = 15.675 psf

Plate Length = 150", Plate Width = 60", Four Sides Simply Supported

GTF = 3.8 (Table 2, p. 2, E 1300, Fully Tempered, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (15.675 psf)(3.8)(2.00) = 119.13 psf

Inner Lite (for Short Duration Load):

NFL = 0.75 kPa (Fig. A1.6, p. 12, E 1300) = (0.75 kPa)(20.9 psf/kPa) = 15.675 psf

Plate Length = 150", Plate Width = 60", Four Sides Simply Supported

GTF = 1.0 (Table 2, p. 2, E 1300, Annealed, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (15.675 psf)(1.0)(2.00) = 31.35 psf

Outer Lite (for Long Duration Load):

NFL = 0.75 kPa (Fig. A1.6, p. 12, E 1300) = (0.75 kPa)(20.9 psf/kPa) = 15.675 psf

Plate Length = 150", Plate Width = 60", Four Sides Simply Supported

GTF = 2.85 (Table 3, p. 2, E 1300, Fully Tempered, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (15.675 psf)(2.85)(2.00) = 89.35 psf

Inner Lite (for Long Duration Load):

NFL = 0.75 kPa (Fig. A1.6, p. 12, E 1300) = (0.75 kPa)(20.9 psf/kPa) = 15.675 psf

Plate Length = 150”, Plate Width = 60”, Four Sides Simply Supported

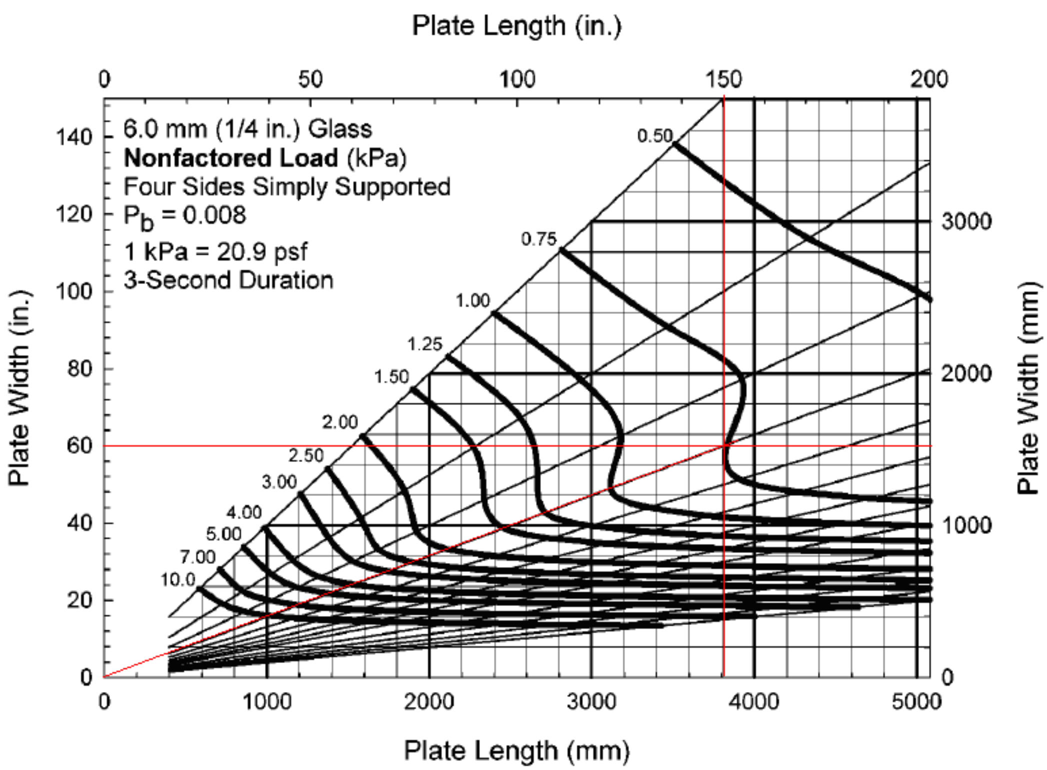
GTF = 0.5 (Table 3, p. 2, E 1300, Annealed, Short Duration Load)

LS = 2.00 (Table 5, p. 5, E 1300)

LR = (NFL)(GTF)(LS) = (15.675 psf)(0.5)(2.00) = 15.675 psf

The load resistance of the IGU is 15.675 psf, being the least of the four values: 119.13, 31.35, 89.35, or 15.675 psf

LR = 15.675 psf > 12.92 psf ∴ **OK**



See ASTM E-1300 Tables 2, 3, and 5 from #1 (above)