IX. APPENDICES

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A.O ACKNOWLEDGEMENTS

- The Harmon Group - For providing my building and support through the entire process
  - Kirk Harman
  - Chris Shaffer
  - Chris Godshall
- BPG Property Group, Ltd. - For providing building statistics and architectural drawings
  - Margret Michel
- Thornton Tomasetti - For providing floor plans for my architectural breadth
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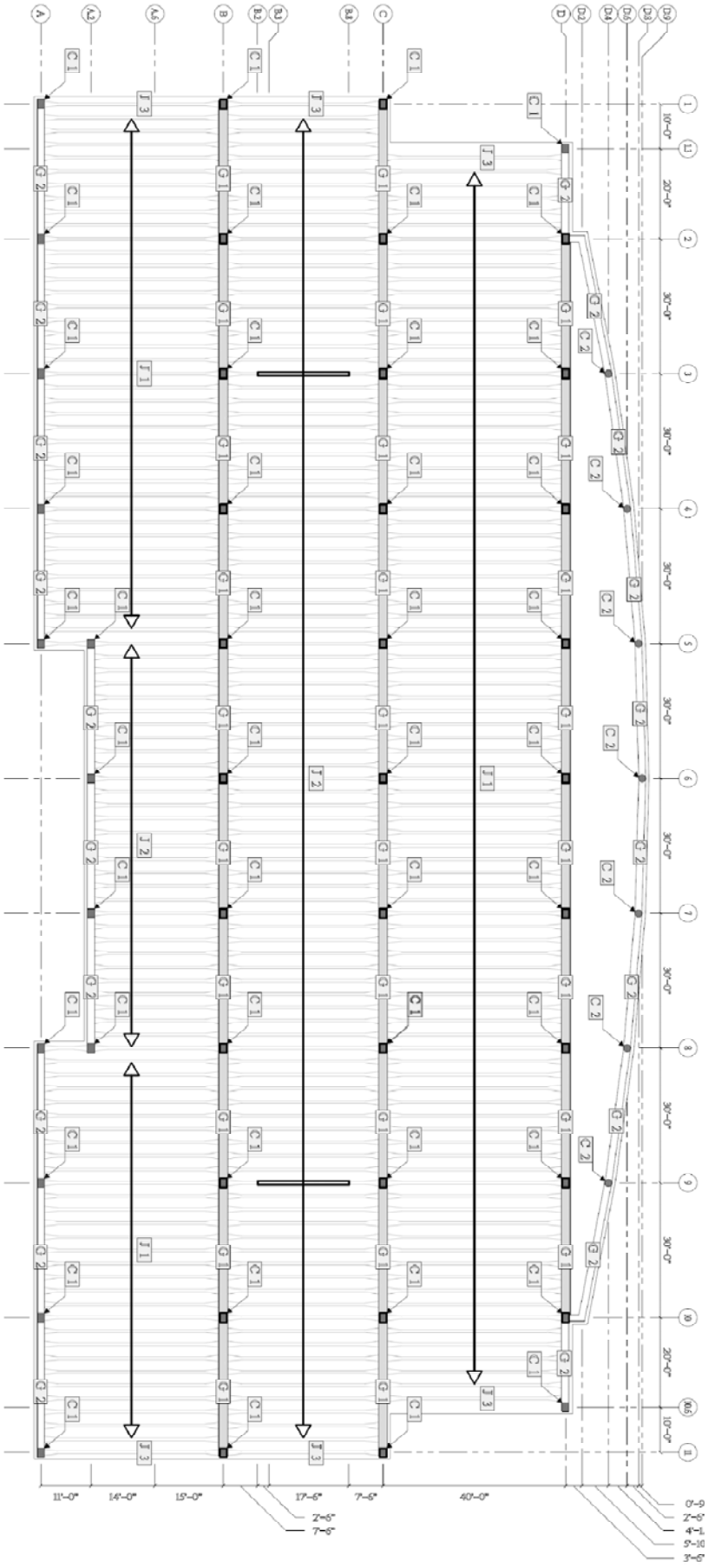
- Kristin Maruszewski - For showing me the ropes on the AGi32 software
- Eastern State Filligree – For providing technical support on filigree slab design
- Everyone else who help along the way there are too many to list.

A.1 FLOOR PLAN AND SCHEDULE

BEAM SCHEDULE						
MARK	BASE	HEIGHT	LONGITUDINAL REINFORCEMENT		STIRRUPS	
			TOP BARS	BOTTOM BARS	SIZE	SPACING
J 1	6"-7 ¾"	20"	# 6 @ 9"	(1) # 7, (1) # 8	-	-
J 2	6"-7 ¾"	20"	# 5 @ 9"	(1) # 5, (1) # 6	-	-
J 3	6"-7 ¾"	20"	# 6 @ 9	(4) #8 TWO LAYERS	-	-
G 1	24"	24 ½"	(7) # 11	(4) # 9	4 LEGS # 4	1@2", 15@8", 2@11"
G 2	18"	24 ½"	(6) # 10	(3) # 6, (2) # 8	4 LEGS # 3	1@2", 15@7", 3@11"

COLUMN SCHEDULE			
	MARK	C 1	
ROOF	DETAILS		
LEVEL 5	F'c	4000 PSI	
	SIZE	24" X 18"	
	VERT. REINF.	(12) # 6 (4 X 2)	
	TIES	#3 @ 12"	
LEVEL 4	F'c	↕	
	SIZE		
	VERT. REINF.		
	TIES		
LEVEL 3	F'c		
	SIZE		
	VERT. REINF.		
	TIES		
LEVEL 2	F'c		
	SIZE		
	VERT. REINF.		
	TIES	↕	
LEVEL 1	F'c	5500 PSI	
	SIZE	24" X 18"	
	VERT. REINF.	(12) # 6 (4 X 2)	
	TIES	#3 @ 12"	
FOUNDATION	F'c	↕	
	SIZE		
	VERT. REINF.		
	TIES		



A.2 WIND DESIGN CALCULATIONS

WIND DESIGN - ASCE 7-05 METHOD 2

1) BASIC WIND SPEED, V = 90 MPH (FIGURE 6-1)

2) IMPORTANCE FACTOR, I = 1.0 (TABLE 6-1)
BUILDING CAT. II (TABLE 1-1)

3) EXPOSURE CAT. = B (SECTION 6.5.6.3)
SURFACE ROUGHNESS = B (SECTION 6.5.6.2)

VELOCITY PRESSURE EXPOSURE COEFFICIENT, K_z
FROM (TABLE 6-3)

FLOOR	TRUE HEIGHT	EST. HEIGHT	K_z
1	13'	15'	0.57
2	26'	30'	0.70
3	39'	40'	0.76
4	52'	60'	0.85
5	65'	70'	0.89
ROOF	77' 6½"	80'	0.93

4) TOPOGRAPHIC FACTOR, $K_{zT} = 1.0$
SITE IS FLAT, THEREFORE $K_{zT} = 1.0$

5) GUST EFFECT FACTOR, G = .828 E-W OR .798 N-S

$$G = 0.925\left(\frac{(1+1.7 g_Q I_{\bar{z}} Q)}{1+1.7 g_v I_{\bar{z}}}\right) = 0.925\left(\frac{(1+1.7(3.4)(.28)(Q)}{1+1.7(3.4)(.28)}\right)$$

$h = 78'$
$c = .30$
$z_{MIN} = 30'$
$\bar{z} = 0.6h = 46.8'$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{46.8}\right)^{1/6} = .283$$

$Q = .831$ E-W or $.778$ N-S

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{B+78'}{359.5}\right)^{0.63}}} = \begin{matrix} .83 \\ \text{or} \\ .78 \end{matrix}$$

$B = 132'$ or $300'$

$h = 78'$

$l = 320'$

$\bar{\varepsilon} = 1/3.0$

$L_z = l\left(\frac{\bar{z}}{33}\right)^{\bar{\varepsilon}} = 320'\left(\frac{46.8'}{33}\right)^{1/3.0} = 359.523\ A^2$

6) ENCLOSURE CLASSIFICATION = ENCLOSED (SECTION 6-2)

7) INTERNAL PRESSURE COEFF., $GC_{pi} = \pm 0.18$ (FIGURE 6-5)

8) EXTERNAL PRESSURE COEFF. (FIGURE 6-6)

WINDWARD WALL, $C_p = 0.8$

LEEWARD WALL, $C_p = -0.3$ E-W OR -0.5 N-S

9) VELOCITY PRESSURE (EQ. 6-15)

SEE SPREAD SHEET

10) DESIGN WIND PRESSURES, P (EQ. 6-17)

SEE SPREAD SHEET

LEEWARD WALL $P = q_h GC_p - q_h(GC_{pi}) = 16.4(GC_p) - 16.4(\pm.18)$

E-W: -7.03 lb/ft²

N-S: -9.50 lb/ft²

A.3 SEISMIC DESIGN CALCULATIONS

SEISMIC

SITE LOCATION : LAT : 40.0°N LONG : 75.4°W

OCCUPANCY CATAGORY : II

IMPORTANCE FACTOR (I) = 1.0

SPECTRAL ACCELERATION

SHORT PERIODS (S_S) = 0.278

ONE SEC. PERIODS (S_1) = 0.060

SOIL SITE CLASS C

SITE COEFFICIENTS

$F_A = 1.2$

$F_V = 1.7$

MCE SPECTRAL RESPONSE PARAMETERS

$S_{MS} = F_A S_S = 1.2\ (0.278) = 0.3336$

$S_{M1} = F_V S_1 = 1.7\ (0.060) = 0.102$

DESIGN SPECTRAL ACCELERATION PARAMETERS

$S_{DS} = 2/3\ S_{MS} = 2/3\ (0.3336) = 0.2224$

$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.102) = 0.068$

SEISMIC DESIGN CATAGORY

$S_{DS} = 0.2224 \Rightarrow B$

$S_{D1} = 0.068 \Rightarrow B$

DESIGN COEFFICIENTS AND FACTORS

(E-W) ORDINARY STEEL MOMENT FRAME
$R = 3.5 \quad \Omega_0 = 3.0 \quad C_d = 3.0$

(N-S) COMP. STEEL + CONCRETE ECC. BRACED FRAME
$R = 8.0 \quad \Omega_0 = 2.0 \quad C_d = 4.0$

BUILDING FUNDAMENTAL PERIOD

$T_a = C_t h_n^x = 0.028\,(78')^{0.8} = (E\text{-}W)\,0.914_s$

$= 0.02\,(78')^{0.75} = (N\text{-}S)\,0.525_s$

SEISMIC RESPONSE COEFFICIENT

(E-W) $C_s = \frac{S_{DS}}{R/I} = \frac{.2224}{3.5/1.0} = 0.0635$

(N-S) $C_s = \frac{S_{DS}}{R/I} = \frac{.2224}{8.0/1.0} = 0.0278$

BECAUSE $T_a \leq T_L = 6$

$C_s \leq \frac{S_{D1}}{T_a\left(\frac{R}{I}\right)} = \frac{.0635}{.914(3.5/1.0)} = (E\text{-}W)\boxed{.0199}$

$= \frac{.0278}{.525(8.0/1.0)} = (N\text{-}S)\ .0066$

SEISMIC BASE SHEAR

$V = C_s W = .0199\,(17910 + 3217.5) = 419.6\ \text{kips}$

VERTICAL DISTRIBUTION : SEE SPREAD SHEET

SEISMIC WEIGHT

ROOF DL = 5 PSF
FLOOR DL = 50 PSF
SUPERIMPOSED DL = 30 PSF
PARTITION LL = 10 PSF
SNOW LOAD = NA $P_g < 30$ PSF
STORAGE LOAD = 25 PSF (.25 × 100 PSF)

APPROXIMATE FLOOR AREA
120' × 300' = 36000 SF / FLOOR

FLOOR	AREA (SF)	UNIFORM LOAD (PSF)*	WEIGHT (K)
1	36000	90 × A + 25 × .10A	3330
2	36000	90 × A + 25 × .10A	3330
3	36000	90 × A + 25 × .10A	3330
4	36000	90 × A + 25 × .10A	3330
5	36000	90 × A + 25 × .10A	3330
R	36000	35	1260
TOTAL	216000		17910

* ACTUAL STORAGE AREA UNKNOWN ASSUMED 10% FLOOR AREA

APPROXIMATE BUILDING PERIMETER
2 × 150' + 2 × 300' = 900' / FLOOR

ARCH. PANEL SELF WEIGHT = 50 PSF

FLOOR	HEIGHT	PERIMETER	PANEL SW. (PSF)	WEIGHT (K)
1	13'	900'	50	585
2	13'	900'	50	585
3	13'	900'	50	585
4	13'	900'	50	585
5	13'	900'	50	585
R	6'6"	900'	50	292.5
TOTAL				3217.5

DESIGN COEFFICIENTS AND FACTORS

(E-W) SPECIAL REINFORCED CONCRETE MOM. FR.
$R = 6.0 \quad \Omega_o = 2.5 \quad C_d = 5$

(N-S) SPECIAL REINFORCED CONCRETE SHEAR WL
$R = 8.0 \quad \Omega_o = 3.0 \quad C_d = 5.5$

BUILDING'S FUNDIMENTAL PERIOD

$T_a = C_t h_n^x =$

$C_t = 0.016 \quad (E\text{-}W) \quad x = 0.9$
$= 0.02 \quad (N\text{-}S) \quad = 0.75$

$T_{a_{E-W}} = 0.016\,(78')^{0.9} = 0.807s$

$T_{a_{N-S}} = 0.02\,(78')^{0.75} = 0.525s$

SEISMIC RESPONSE COEFFICIENT

$(E\text{-}W) \quad C_s = \dfrac{S_{DS}}{(R/I)} = \dfrac{0.2224}{(6.0/1.0)} = 0.037$

$(N\text{-}S) \quad C_s = \dfrac{0.2224}{(8.0/1.0)} = 0.028$

BECAUSE $T_{MAX} = T_a \times C_u \leq T_L = 6$

$(E\text{-}W) \quad C_s \leq \dfrac{S_{D1}}{T_a \times C_u (R/I)} = \dfrac{0.068}{.8(1.7)(6/1)} = .008 < \boxed{.01}$

$(N\text{-}S) \quad C_s \leq \dfrac{0.068}{.5(1.7)(8/1)} = .0095 < \boxed{.01}$

SEISMIC WEIGHT

- SUPERIMPOSED LOADS

ROOF FINISHES	: 5 PSF
FLOOR FINISHES	: 12 PSF
CEILING DL (MEP + FINISHES)	: 10 PSF

- SELF WEIGHTS

	(CONCRETE) (REBAR)
SLABS	: 0.74 CF/SF × 150 PCF + 2 PSF = 113 PSF
COLUMNS	: 18" × 24" × 150 PCF = 450 PLF
ROOF SLAB	: 0.74 CF/SF × 150 PCF + 2 PSF = 113 PSF
CURTIAN WALLS	: 50 PSF
BEAMS (INT)	: 24.5" × 48" × 150 PCF = 1225 PLF
(EXT)	: 24.5" × 36" × 150 PCF = 920 PLF

- SEE SPREADSHEET FOR CALCS

FLOOR	SEISMIC WEIGHT
ROOF	5986 KIPS
5	6804 KIPS
4	6804 KIPS
3	6804 KIPS
2	6875 KIPS
1	6216 KIPS
GROUND	287 KIPS
TOTAL	39775 KIPS

SEISMIC BASE SHEAR

$$V = C_s W = 0.010(39775) = 397.8 \text{ KIPS}$$

LIVE LOADS

ALL FLOORS: 100 PSF
ROOF (SNOW): 21 PSF

LIVE LOAD REDUCTION NOT APPLICABLE OTHER THAN MEMBERS SUPPORTING 2 MORE FLOORS.

DEAD LOADS (SUPERIMPOSED)

CEILING DEAD LOADS (MEP + FINISHES) : 10 PSF
ROOF FINISHES : 5 PSF
FLOOR FINISHES : 12 PSF ?? PER FLOOR SLAB DEPRESSION

APPROXIMATE FLOOR AREAS

FLOOR	AREA (SF)	PERIMETER (FT)
R	35697.5	864
5	35697.5	864
4	35697.5	864
3	35697.5	864
2	36027.5	886
1	33370.0	830
TOTAL	212187.5	5172

DISTRIBUTION OF LATERAL LOADS FROM SEISMIC ACROSS THE MOMENT FRAMES AT EACH STORY BASED ON RELATIVE STIFFNESS

VERTICAL DISTRIBUTION OF SEISMIC LOADS

ROOF	179.6 kips
5	141.8 kips
4	34.4 kips
3	28.2 kips
2	8.7 kips
1	5.2 kips

RELATIVE STIFFNESSES

	MF #1	MF #2	MF #3
ROOF	29.8%	34.2	36.0
5	29.8	35.9	34.3
4	29.8	35.9	34.3
3	29.7	35.9	34.4
2	26.6	37.8	35.6
1	33.5	52.2	14.3
	53.5	61.4	64.7
	42.2	50.8	48.7
	10.2	12.3	11.8
	8.4	10.1	9.7
	2.3	3.3	3.1
	1.7	2.7	0.7
TOTAL	118.5	140.7	138.6

A.4 PAN JOIST DESIGN

PAN JOIST DESIGN (FROM CRSI HANDBOOK)

- SUPERIMPOSED LOADING:

FLOOR LL = 100 PSF
ROOF LL = 21. PSF (SNOW LOAD)
FLOOR FINISHES = 12 PSF
ROOF FINISHES = 5 PSF
CEILING DEAD LOAD = 10 PSF (MEP + FINISHES)
BRIDGING = 2 PSF

- TOTAL FACTORED SUPERIMPOSED LOAD

$W_f = 1.2(12+2+10) + 1.6(100) = 188.8$ PSF

OR

$W_R = 1.2(5+2+10) + 1.6(21) = 54.0$ PSF

FLOORS

TRY 30" FORMS + 6" RIBS @ 36" C.C.

END SPANS = 40' - 1' = 39' = ℓ_{n1}
INT SPAN = 35 - 1' = 34' = ℓ_n

- END SPAN = 20" + 4.5"

TOP BARS: #6 @ 9" AT 1ST INTERIOR SUPPORT
BOTTOM BARS: (1)#7 + (1)#8 (PER RIB)
WEIGHT: 1.96 PSF

TOP BARS @ EXTERIOR: $A_s \approx \frac{1}{3}(0.60+0.79) \times \frac{12}{36} = 0.15$ IN²/FT

USE #4 @ 12" w/ STANDARD 90° END HOOKS

- INT SPAN

TOP BARS: #5 @ 9.5 IN
BOTTOM BARS: (1) #5 + (1) #6 (PER RIB)
WEIGHT: 1.38 PSF

- CONCRETE QUANTITY = 109 PSF

INCLUDING BRIDGING = $\left(\frac{109+2}{150}\right) = 0.74$ CF/SF

- DEFLECTION $= \left(\frac{100}{(188.8)(1.6)}\right)\left(\frac{39}{480}\right) = .027 \leq \left(\frac{39}{750}\right) = .052$ OK

ROOF

USE SAME 30" FORMS + 6" RIBS @ 36" C.C

- END SPAN

TOP BARS: #5 @ 10.5 IN
BOTTOM BARS: (2) #6 (PER RIB)
WEIGHT: 1.24 PSF

TOP @ EXTERIOR: $A_s \approx 1/3 (2 \times .44) 12/36 = .097$
USE #3 @ 12" W/ STANDARD 90° END HOOKS

- INT. SPAN

TOP BARS: #5 @ 11 IN
BOTTOM BARS: (2) #5 (PER RIB)
WEIGHT: 1.18 PSF

A.5 BEAM DESIGN

BEAM DESIGN (TYPICAL FLOOR)

SERVICE LIVE LOAD = 100 PSF

SUPERIMPOSED DEAD LOAD = 22 PSF
SLAB WEIGHT = 0.74 CF/SF × 150 PSF = 111 PSF

TOTAL SERVICE DEAD LOAD = 133 PSF

FACTORED LIVE LOAD = 1.6 (100) = 160 PSF
FACTORED DEAD LOAD = 1.2 (133) = 159.6 PSF
TOTAL LOAD = 160 + 159.6 = 320 PSF

$M_U = w_u \ell_N^2 / 12 = 320 \times \left(\frac{35+40}{2}\right) \times (30-2)^2 / 12 = 784$ kip-ft

BEAM TRIAL SIZE

$\rho = .85(.85)(4/60)(.003/.007) = 0.0206$

$\phi M_N = M_U \Rightarrow .9(0.0206)(60)bd^2\left(1 - 0.59(.0206 \times 60)/4\right)$

$\geq 784 \times 12 \Rightarrow bd^2 \geq 10343\ IN^3$

$d = 24.5" - 2.5" = 22"$

$b \geq 21.3$ TRY 24"

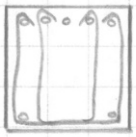
BEAM SELF WEIGHT

$1.2\left[24.5" \times 24" \times 150PCF / 144^{IN^2}/_{SF^2}\right] = .735$ KLF

$W_U = \left(\frac{35+40}{2}\right) 320$ PSF $- 111$ PSF $\times 2 = 11.8$ KLF

$\ell_N = 30' - 2' = 28'$

INTERIOR BEAM

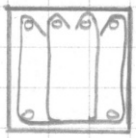


USE DESIGN 4 FROM CRSI

W = 11.9 > 11.8 KLF
STIRRUP = 155H ⇒ (15) # 5 ; 1@2", 14@10"
PROVIDE 4 LEGS
TOP BARS = (5) # 14
BOTTOM BARS = (2) # 14

EXTERIOR BEAM

$W_u = (40/2)320\ PSF = 6.4\ KLF$



USE DESIGN 3 FROM CRSI

W = 8.2 > 6.4 KLF
STIRRUP = 174H ⇒ (17) # 4 ; 1@2", 16@10"
PROVIDE 4 LEGS
TOP BARS = (4) # 14
BOTTOM BARS = (2) # 14

BEAM DESIGN (ROOF)

ROOF SNOW LOAD = 21 PSF

SUPERIMPOSED DEAD LOAD = 15 PSF
SLAB WEIGHT = 0.74 CF/SF × 150 = 111 PSF

TOTAL SERVICE DEAD LOAD = 126 PSF

FACTORED LIVE LOAD = 1.6 (21) = 33.6 PSF
FACTORED DEAD LOAD = 1.2 (126) = 151.2 PSF

TOTAL LOAD = 34 + 151 = 185 PSF

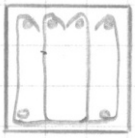
BEAM SELF WEIGHT

$$1.2\left[24.5'' \times 24' \times 150\ PCF / 144\ IN^2/SF\right] = 0.735\ KLF$$

$$W_U = \left(\frac{35+40}{2}\right) 185 - 111\ PSF \times 2 = 67.2$$

$$\ell_N = 30' - 2' = 28'$$

INTERIOR BEAM

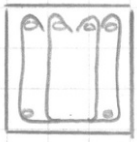


USE DESIGN 2 FROM CRSI

W = 7.2 > 6.4
STIRRUP = 133+H ⇒ (13) #3 ; 1@2", 12@10"
PROVIDE 4 LEGS
TOP BARS = (4) #11
BOTTOM BARS = (2) #10

EXTERIOR BEAM

$W_U = (40/2)\,185 = 3.7$ KLF



USE DESIGN 1 FROM CRSI

$W = 4.7 > 3.7$ KLF

STIRRUP = 133 H $\Rightarrow$ (13) #3 ; 1@2", 12@10"

PROVIDE 4 LEGS

TOP BARS = (4) #9

BOTTOM BARS = (2) #10

BEAM DESIGN 2

INTERIOR

$$W_U = \left[1.2\,(22) + 1.2\,(113) + 1.6\,(100)\right] = 322 \text{ PSF}$$

BEAM TRIAL SIZE

$$M_U = w_u l_n^2/12 = 322\,(37.5)(28')^2/12 = 789 \text{ kip}\cdot\text{ft}$$

ASSUME $\phi = 0.9$ AND $\therefore \epsilon_t = 0.005$

$$\rho = 0.85\,\beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.005} = 0.85 \times 0.85 \left(\frac{4}{60}\right)\left(\frac{0.003}{0.003+0.005}\right)$$

$$= 0.0181$$

$$M_U = \phi M_N = 789 \times 12 = 0.9 \times 0.0181 \times 60\, bd^2 \left(1 - 0.59\,\frac{0.0181 \times 60}{4}\right)$$

$$bd^2 = 11534.6 \text{ in}^2 \Rightarrow d = 22'' \Rightarrow b = 23.83 \approx 24''$$

BEAM SELF WEIGHT

$$1.2\left[24.5 \times 24 \times 145/144 \text{ in}^2/\text{ft}^2\right] = 0.711 \text{ KLF}$$

$$W_U = 322 \times 37.5' - 113 \times 3' + 711 = 12447 \text{ KLF}$$

$$M_U = \frac{12447\,(28)^2}{12} = 813 \text{ kip}\cdot\text{ft} \times 12 = 9758 \text{ in}\cdot\text{kip}$$

MINIMUM REINFORCEMENT AT ENDS

$$A_s = \rho bd = 0.0181\,(24)(24.5) = 10.64 \text{ in}^2$$

$$a = \frac{10.64 \times 60}{.85\,(4)(24)} = 7.83$$

STEEL DESIGN

$A_{S_{REQ}} = 10.64 in^2 \Rightarrow (7)\#11 = 10.92 in^2$

$b_{MIN} = 2 \times 2.5 + 13 \times 1.41 = 23.33" < 24"$ OK

MID-SPAN MOMENT

$M_U = w_u l_n^2 / 24 = \frac{12447 (28)^2}{24} = 406 \text{ kip-ft} \times 12 =$

$= 4879 \text{ in-kips}$

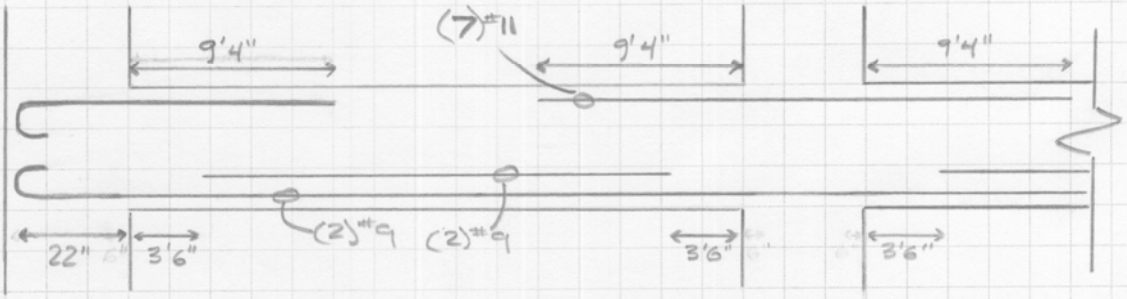
ASSUME $\phi = 0.9 \Rightarrow M_N = M_U/\phi = 4879/0.9 = 5421$

$R = \frac{M_N}{bd^2} = \frac{4879 \times 1000}{24" (22)^2} = 420.0 \text{ psi}$

FROM TBL A.5a $\Rightarrow \rho = 0.0075$

$A_s = \rho bd = 0.0075 (24)(22) = 3.96 in^2$

USE (4)#9



$l_{dh} = \left(\frac{0.02 f_y}{\sqrt{f'_c}}\right) d_b = \left(\frac{0.02(60000)}{\sqrt{4000}}\right) 1.128 = 21.4" \approx 22"$

EXTERIOR E-W

$W_u = (322\ PSF \times 20\ ft) + 1.2(50 \times 13\ ft) = 7220\ PLF$

BEAM TRIAL SIZE

$M_u = w_u l_n^2/12 = 7220 \times (30-2)^2/12 = 472\ ft\cdot kip$

ASSUME $\phi = 0.9$ AND $\therefore \varepsilon_t = 0.005$

$\rho = 0.85 \beta_1 \frac{f'_c}{f_y} \frac{\varepsilon_u}{\varepsilon_u + 0.005} = 0.0181$

$M_u = \phi M_n = 472 \times 12 = 0.9 \times 0.0181 \times 60\ bd^2 \left(1 - 0.59 \frac{0.0181 \times 60}{4}\right)$

$bd^2 = 6895\ IN^2 \Rightarrow d = 22" \Rightarrow b \geq 14.2$ USE 18"

BEAM SELF WEIGHT

$1.2\left[24.5 \times 18 \times 145/144\ IN/FT^2\right] = 533\ PLF$

$W_u = 7220 - 113 \times 1.5 + 533 = 7584\ PLF$

$M_u = 7584\ (28)^2/12 = 495\ FT\text{-}KIPS \times 12 = 5945\ IN\text{-}KIPS$

MINIMUM REINFORCEMENT AT BEAM ENDS

$A_s = \rho bd = 0.0181\ (18)(22) = 7.17\ IN^2$ TRY (6) #10

$b_{MIN} = 2 \times 1.5 + 2 \times 3.75 + 11 \times 1.27 = 17.72 < 18$ OK

$a = \frac{7.62 \times 60}{.85(4)(18)} = 7.47$

MIDSPAN MOMENT

$M_U = w_u l_n^2/24 = 7584(28)^2/24 = 248$ FT-KIPS $\times 12$

$= 2973$ IN-KIPS

ASSUME $\phi = 0.9 \Rightarrow M_N = M_U/\phi = 2973/0.9 = 3303$ IN-KIPS

$R = \frac{M_N}{bd^2} = \frac{3303}{(18)(22)^2} = 379.2$

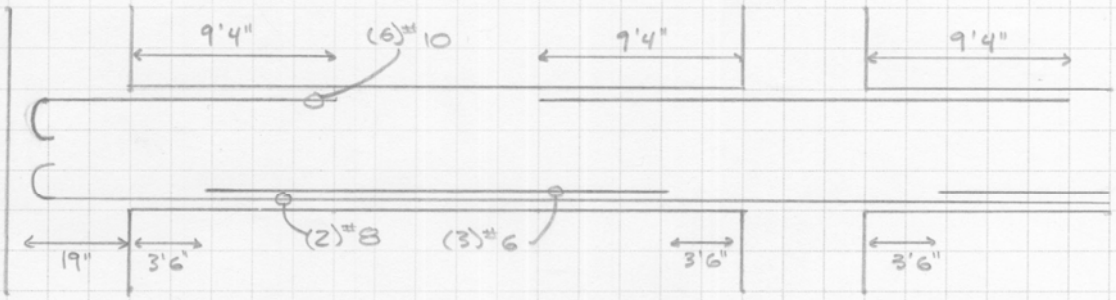
FROM TBL A.5a $\Rightarrow \rho = (0.007 - 0.0065)\left(\frac{379.2 - 368}{394 - 368}\right) + 0.0065$

$= 0.0067$

$A_S = \rho bd = 0.0067(18)(22) = 2.66$ IN^2

TRY (2)#8 + (3)#6 $= 2(.79) + 3(.44) = 2.9$ IN^2

$b_{MIN} = 2\times1.5 + 2\times.375 + 2\times1.0 + 3\times.75 + 4\times1.0 = 12$ OK



$l_{dh} = \left(\frac{0.02\,f_y}{\sqrt{f'_c}}\right) d_b = \left(\frac{0.02(60000)}{\sqrt{4000}}\right) 1.0 = 18.97$ IN ≈ 19"

EXTERIOR N-S

$W_u = [1.2(50 \times 13) + 1.2(1.5 \times 4.5/12 \times 145) + 1.2(22 \times 1.5) + 1.6(100 \times 1.5)] = 1157$ KLF

BEAM TRIAL SIZE - ASSUME $\phi = 0.9$ AND $\therefore \varepsilon_t = 0.005$

$M_u = w_u l_n^2/8 = 1157(40-1.5)^2/8 = 214$ FT-KIPS $\times 12 = 2572$ IN-KIP

$\rho = 0.85 \times 0.85 \left(\frac{4}{60}\right)\left(\frac{0.003}{0.003+0.005}\right) = 0.0181$

$M_u = \phi M_N = 2572 = 0.9 \times 0.0181 \times 60\, bd^2 \left(1 - 0.59 \frac{0.0181 \times 60}{4}\right)$

$bd^2 = 3134$ IN^2 $\Rightarrow d = 22 \Rightarrow b = 6.48 \Rightarrow 7''$ SAME AS OTHER RIBS.

BEAM SELF WEIGHT

$1.2[7'' \times 20'' \times 145/144\ \text{IN}^2/\text{FT}^2] = 169$ KLF

$M_u = w_u l_n^2/8 = (1157+169)(38.5)^2/8 = 246 \times 12 = 2948$

$M_N = M_u/\phi = 2948/0.9 = 3276$ IN-KIPS

$R = M_N/bd^2 = 3276 \times 1000/7(22)^2 = 966$ PSI

FROM TBL A.5a $\Rightarrow \rho = 0.0195$

$A_s = \rho bd = 0.0195(7 \times 22) = 3.00$ IN^2

USE 4 #8 IN TWO LAYERS $= A_s = 3.16$

$b_{MIN} = 2 \times 1.5 + 1.0 + 2 \times 1.0 = 6.0$ OK

A. 6 SHEAR CHECKS

SHEAR CHECK

PAN JOIST

$V_u \leq \phi V_N$

$V_u = [1.2(22) + 1.2(113) + 1.6(100)]\left(\frac{40' - 2}{2}\right)(3')$

$= 174283$ lbs

$V_u \leq \frac{1}{2}\phi(2\sqrt{f'_c}\, b_w d) = \frac{1}{2}(0.75)[2\sqrt{4000}(7"(22))]$

$V_u = 17$ kips $\not\leq \frac{1}{2}\phi V_c = 7.3$ kips ∴ TAPER ENDS

$b_w d = \frac{V_u}{\phi\sqrt{f'_c}} = \frac{174283}{.75\sqrt{4000}} = 367.4\ in^2$

$367.4/d = 367.4/23 = 15.9" \approx 16"$ TAPERED WIDTH OF RIBS.

BEAMS

INTERIOR

$V_u = [1.2(22) + 1.2(113) + 1.6(100)]\left[\left(\frac{30-2}{2}\right) - 22/12\right]\left(\frac{40+35}{2}\right)$

$= 147$ kips

$V_u \leq \frac{1}{2}\phi(2\sqrt{f'_c}\, b_w d) = \frac{1}{2}(0.75)[2\sqrt{4000}(24")(22")]$

$V_u = 147$ kips $\not\leq \frac{1}{2}\phi V_c = 25$ kips

SHEAR REINFORCEMENT IS NEEDED, TRY (2)#4 VERT "U"s

$12.17\left(\frac{147 - 25}{147}\right) = 12'$ FROM FACE OF SUPPORT

NO REINFORCEMENT REQ.

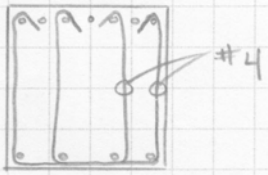
$$S_{MAX} = \begin{cases} A_v f_y / 0.75\sqrt{f'_c}\, b_w = \dfrac{4(.2)60000}{0.75(\sqrt{4000})(24)} = 42'' \le \dfrac{4(.2)(60000)}{50(24)} = 40 \\ \dfrac{d}{2} = 22/2 = 11 \leftarrow \\ \text{MIN}\ \ 24\ \text{IN} \end{cases}$$

$$S = \frac{\phi A_v f_y d}{V_u - \phi V_c} = \frac{0.75(.8)(60000)(22)}{(147-50)\times 1000} = 8.2'' \Rightarrow 8''$$

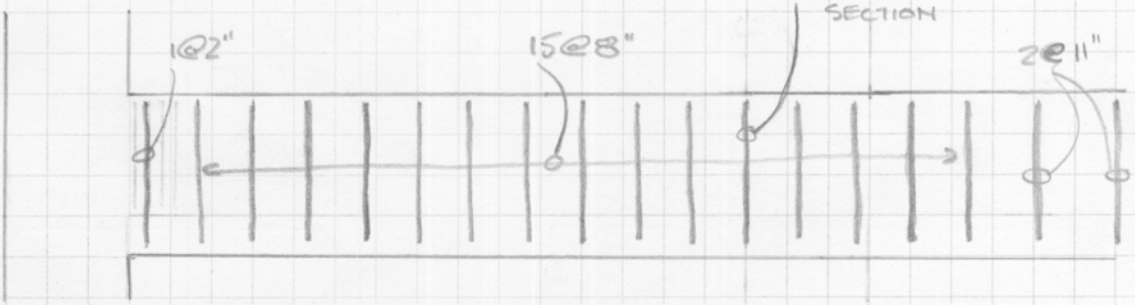
$$\frac{\phi A_v f_y d}{s} = \frac{0.75(.8)(60000)(22)}{11''} = 72000\ \text{lb}$$

$$12.17\left(\frac{147-50}{147}\right) + 22/12 = 10'$$

1 SPACE @ 2"
15 SPACES @ 8"
2 SPACES @ 11"



TYPICAL SECTION



BEAM
EXTERIOR E-W

$$V_u = W_u \times \frac{l_N}{2} - d = 7584 \times \left(\frac{28'}{2} - 22/12\right) = 92.2\ \text{kips}$$

$$V_u \le 1/2\,\phi\,(2\sqrt{f'_c}\, b_w d) = 1/2(0.75)\left[2\sqrt{4000}(18'')(22'')\right] = 18.8\ \text{kip}$$

$V_u \not< \frac{1}{2}\phi V_c = 18.8$ kips

∴ REINFORCEMENT IS NEEDED TRY (2)#3 VERT. "U"s

$12.17\left(\frac{92.2-18.8}{92.2}\right) = 9.7'$ FROM THE CRITICAL SECTION
NO REINF. REQ.

$12.17\left(\frac{92.2-37.6}{92.2}\right) = 7.2'$ FROM THE CRITICAL SECTION
MIN. REINF. REQ.

$$S_{MAX} = \begin{cases} A_v f_y / 0.75\sqrt{f'_c}\, b_w = \frac{4(.11)60000}{0.75(\sqrt{4000})(18)} = 30.9 \not< \frac{4(.11)(60000)}{50(18)} = 29.3 \\ d/2 = 22/2 = 11 \leftarrow \text{CONTROLS} \\ \text{MIN} \quad 24_{IN} \end{cases}$$

$$S = \frac{\phi A_v f_y d}{V_u - \phi V_c} = \frac{0.75(.44)(60000)(22)}{(92.2-37.6)(1000)} = 7.97 \Rightarrow 7"$$

1 SPACE @ 2"
15 SPACES @ 7"
3 SPACES @ 11"

A.7 COLUMN SPOT CHECK

EXTERIOR COLUMNS

TRIBUTARY AREA : $(20+1.5)(30) = 645$ ft²

LIVE LOAD (LIVE LOAD REDUCTION .40 L_o)

$645 \times 21 + (5)(40)(645) = 143$ kips

DEAD LOAD

$645 \times 15 + (5)(17)(645) = 65$ kips

SELF WEIGHT

$[113 \times (645 - 30 \times 3) + 940\,(30') + 11'(450)]\,5$

(SLAB) (BEAM) (COLUMN)

$= 480$ kips

TOTAL DEAD LOAD

$480 + 65 = 545$ kips

CONTROLING COMBINATION

1.2 DEAD + 1.6 LIVE

$1.2(545) + 1.6(143) = 883$ kips $= P_u$

$P_{MAX} = 1175$ kips $> P_u = 883$ kips OK

INTERIOR COLUMNS

TRIBUTARY AREA: $(20 + 17.5)(30) = 1125$ ft²

LIVE LOAD

$1125 \times 21 + (5)(100)(1125) = 586$ kips

DEAD LOAD

$1125 \times 15 + (5)(17)(1125) = 113$ kips

SELF WEIGHT

$$\left[\underbrace{113 \times (1125 - 30 \times 4)}_{\text{SLAB}} + \underbrace{(1225\ \text{PLF})(30')}_{\text{BEAM}} + \underbrace{(11')(450)}_{\text{COLUMN}}\right](5)$$

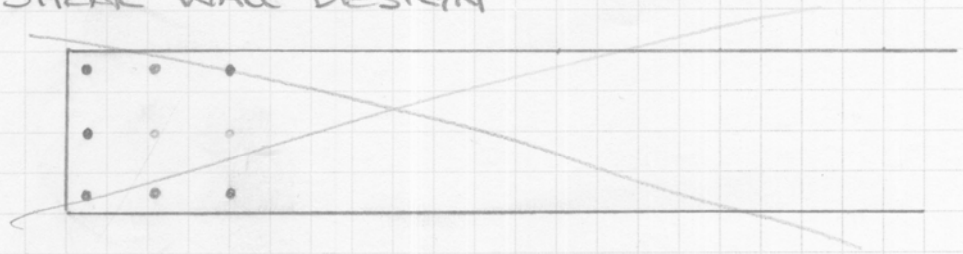
$= 776$ kips

TOTAL DEAD

$113 + 776 = 889$ kips

A.8 SHEAR WALL DESIGN

SHEAR WALL DESIGN



NOMINAL SHEAR CAPACITY

$V_N \leq 8 A_{cv} \sqrt{f'_c} = 8(12"\times 240") \sqrt{5500} \left(\frac{1\,kip}{1000\,lbs}\right)$

≤ 1708.69 kips

$\phi V_N = 0.6(1708.69 = 1025.22 > 187.16$ kips

LONG. + TRANS. REINFORCE.

$V_N \geq 2 A_{cv} \sqrt{f'_c} = 1/4(1708.69) = 427.17$ kips

$V_u = 187.16 < V_N = 592.5$ OK

REQUIRED $\rho_l = 0.0015$ $\rho_h = 0.0025$

$A_{sV} = 0.0015(12)(12) = 0.216\ in^2/ft$

#5 @ 16"

$A_{sH} = 0.0025(12)(12) = 0.36\ in^2/ft$

#5 @ 10"

CHECK NEED FOR BOUNDARY ELEMENT

$$P_{u_{BE}} = \tfrac{1}{2}P + M_u\left(\frac{1}{\ell_w - b/2}\right)$$

$$= \tfrac{1}{2}(234) + 7200\left(\frac{1}{20' - 24\tfrac{1}{2}}\right) = 495.9$$

$$A_g = 20\ ft^2$$

$$I_g = \tfrac{1}{2}(1')(20')^3 = 666.67$$

$$f_c = P_u/A_g + M_u(h_w/2)/I_g =$$

$$= 495.9/20 + \left[7200(20/2)/666.67\right]\left(\frac{1}{144}\right) =$$

$$= 25.5 > 0.2 f'_c = 1.1\ ksi$$

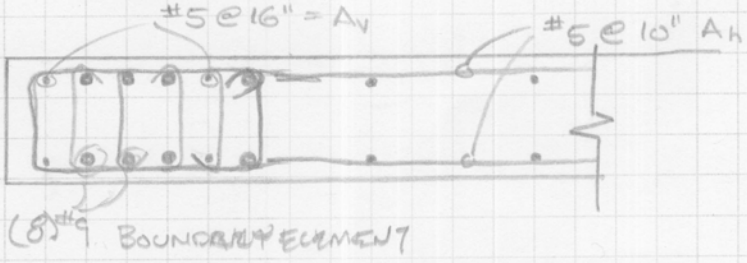
-BOUNDRY ELEMENT CAPACITY

$$\phi P_N > P_{u_{BE}} = 495.9$$

$$0.9(60)(A_{st}) > 495.5$$

$$54 A_{st} > 495.5$$

$$A_{st} \geq 9.1 \qquad (4)\#5 + (8)\#9$$



A.9 BUILDING COST ESTIMATE

RS MEANS COST ESTIMATE

ORIGINAL BUILDING

- 6 STORY, 13 FOOT STORY
- PRECAST CONCRETE FACADE
- STEEL STRUCTURAL SYSTEM
- NORRISTOWN, PA
- FLOOR AREA = 212,188 SQ.FT.
- PERIMETER = 5172 FT.

COST PER SQ. FT. = $247.22

REDESIGNED BUILDING

- 6 STORY, 13 FOOT STORY
- PRECAST CONCRETE FACADE
- REINFORCED CONCRETE STRUCTURAL SYSTEM
- NORRISTOWN, PA
- FLOOR AREA = 212,188 SQ. FT.
- PERIMETER = 5172 FT.

COST PER SQ. FT = $249.54

TOTAL COSTS

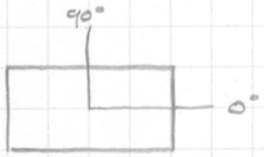
ORIGINAL	REDESIGNE
$52,457,500	$52,949,000

A.10 LIGHTING CALCULATIONS

LIGHTING DESIGN

2 GENERAL FIXTURES

RECESSED DOWNLIGHT
- COOPER LIGHTING - C7042-740061-4/2WPLT
- USE IN KITCHEN, LOBBY, WALKWAYS
- SPACING CRITERIA 0.96



RECESSED TROFFER
- LIGHTOLIER - QVL2GPFS54/32
- USE IN ALL OTHER COMMON SPACES
- SPACING CRITERIA ⇒ 0° : 1.24 90° : 1.32

APPROXIMATE SPACING

WORK SURFACE 2.5' OFF FLOOR
DISTANCE TO WORK SURFACE 10' - 2.5' = 7.5'

COOPER
0.96 × 7.5' = 7.2'
USE 6'

LIGHTOLIER
1.24 × 7.5' = 9.3'
USE 8'

1.32 × 7.5' = 9.9'
USE 10'

REQUIRED ILLUMINANCE

OFFICE - HEAVY VDT USE ⇒ ILLUMINANCE CATAGORY = D
REQUIRED ILLUMINANCE = 30 F_c

REDUCTION IN POWER USE FROM DAYLIGHTING

NUMBER OF UNNECESSARY LUMINARIES

WINTER SOLSTICE

SOUTHERN SIDE = 41
NORTHERN SIDE = 18

TOTAL = 59

SUMMER SOLSTICE

SOUTHERN SIDE = 25
NORTHERN SIDE = 30

TOTAL = 55

FULL YEAR AVERAGE = 57 FIXTURES

OVERCAST DAYS

SOUTHERN SIDE = 13
NORTHERN SIDE = 14

TOTAL = 27

ACCORDING TO THE NATIONAL CLIMATE DATA CENTER
PHILADELPHIA HAS 53% SUNNY WEATHER

TOTAL UNUSED FIXTURES = $[365 \times .53 \times 57 + 365 \times .47 \times 27]$
= 15,659 FIXTURES

TOTAL POWER SAVINGS = 15,659 x 8 hr x 108 WATTS
= 13,529 KILOWATT-HOURS

A.11 STORY DISPLACEMENTS

Story Displacements

RAM Frame v11.2
DataBase: Second Try
Building Code: IBC

04/08/08 15:58:23

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Wall Mesh Criteria :
Wall Element Type : Shell Element with No Out-of-Plane Stiffness
Max. Allowed Distance between Nodes (ft) : 8.00

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
E1	Seismic	EQ_IBC06_X_+E_F
E2	Seismic	EQ_IBC06_X_-E_F
E3	Seismic	EQ_IBC06_Y_+E_F
E4	Seismic	EQ_IBC06_Y_-E_F
W1	Wind	Wind_IBC06_1_X
W2	Wind	Wind_IBC06_1_Y
W3	Wind	Wind_IBC06_2_X+E
W4	Wind	Wind_IBC06_2_X-E
W5	Wind	Wind_IBC06_2_Y+E
W6	Wind	Wind_IBC06_2_Y-E
W7	Wind	Wind_IBC06_3_X+Y
W8	Wind	Wind_IBC06_3_X-Y
W9	Wind	Wind_IBC06_4_X+Y_CW
W10	Wind	Wind_IBC06_4_X+Y_CCW
W11	Wind	Wind_IBC06_4_X-Y_CW
W12	Wind	Wind_IBC06_4_X-Y_CCW

Level: Roof, Diaph: 1

Center of Mass (ft): (149.52, 64.84)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.00365	0.00021	-0.00000
Lp	-0.00537	0.00051	-0.00000
E1	1.17084	-0.00211	0.00006
E2	1.19742	-0.00743	0.00029
E3	0.02585	2.15831	0.00022
E4	-0.03393	2.17029	-0.00028
W1	0.37651	-0.00149	0.00004
W2	-0.00312	1.70393	-0.00002
W3	0.27435	0.00072	-0.00003
W4	0.29042	-0.00297	0.00010
W5	0.04230	1.26758	0.00036
W6	-0.04698	1.28832	-0.00039
W7	0.28005	1.27683	0.00002