Structural Technical Report 1

Advisor: Boothby

10-05-2005

Executive Summary

This focus of this report is to introduce and analyze the existing conditions for the structural system of the Christina Landing Apartment Tower. The building is a 22 story high rise which is part of a residential building project in Wilmington, Delaware. The project site is located on the fringe of center city just south of the Christina River. Included in the housing development are 63 townhouses, a river-walk, and a 2 acre park. The tower is highly visible and able to be view from both interstate 95 and interstate 495, which bypass the heart of Wilmington.

This report covers the design criteria used and all relevant codes. It also includes detailed descriptions of the structural system incorporated to illustrate how the building resists the loads applied to it. Building schematics have been included to allow for a better understanding of the building layout. All required loads are given as well as calculations detailing wind, seismic, and snow loading. Finally various structural elements are checked for size and capacity compatibility to the existing conditions. All calculations done for this report are given in the appendix.

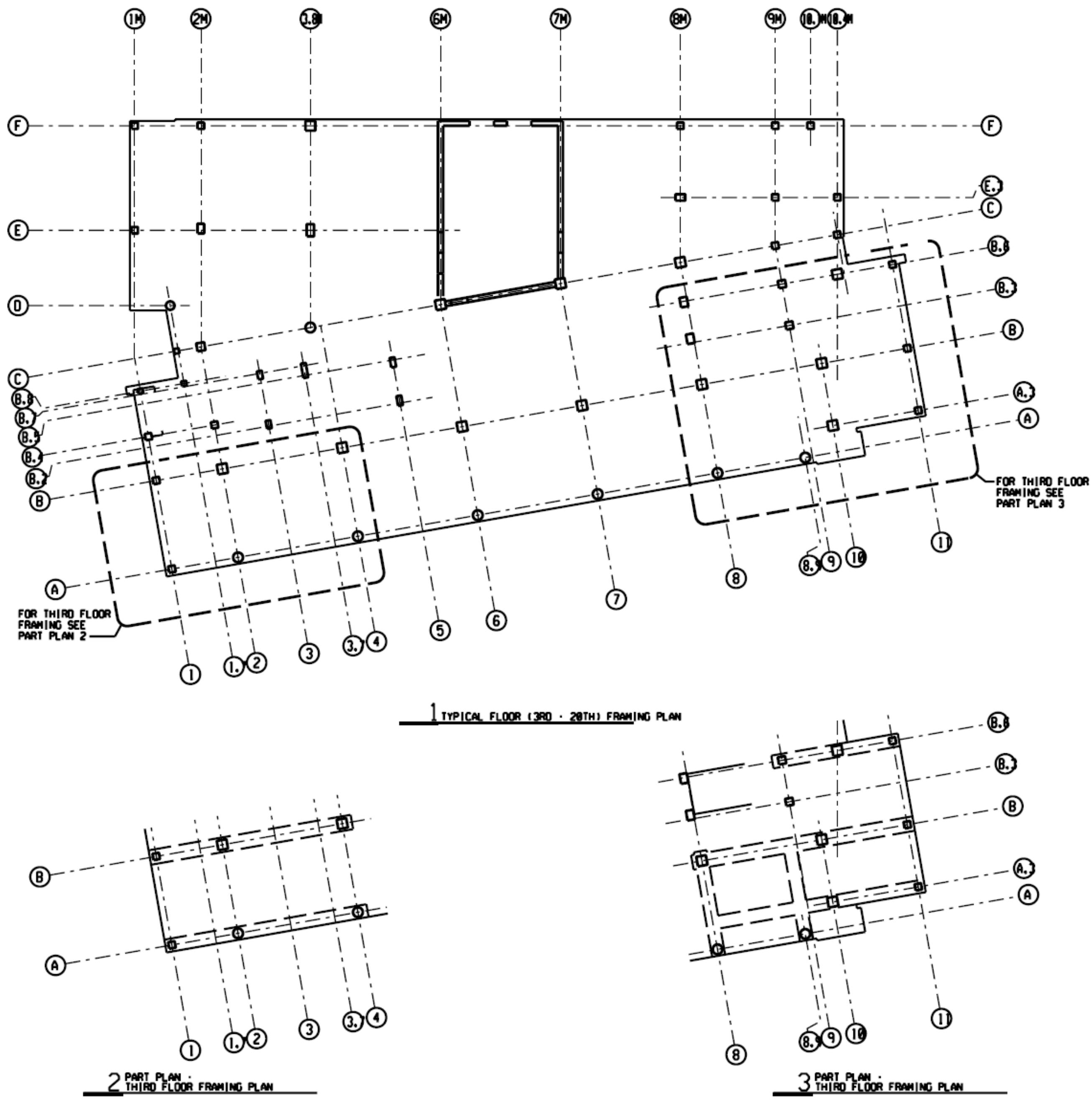
Introduction/Summary of Structural System

The Christina Landing Apartment Tower is a predominantly cast-in-place concrete building. Its floors are supported by a two way flat slab system. It also incorporates some small areas of reinforced concrete beams or post-tensioned beams. Spans between columns are on average approximately 20 to 25 feet. The floors are supported by square and round concrete columns of various sizes. The entire building is supported by a foundation system of H-piles and pile caps. Concrete strengths differ throughout the structure, ranging from 4000psi to 8000psi.

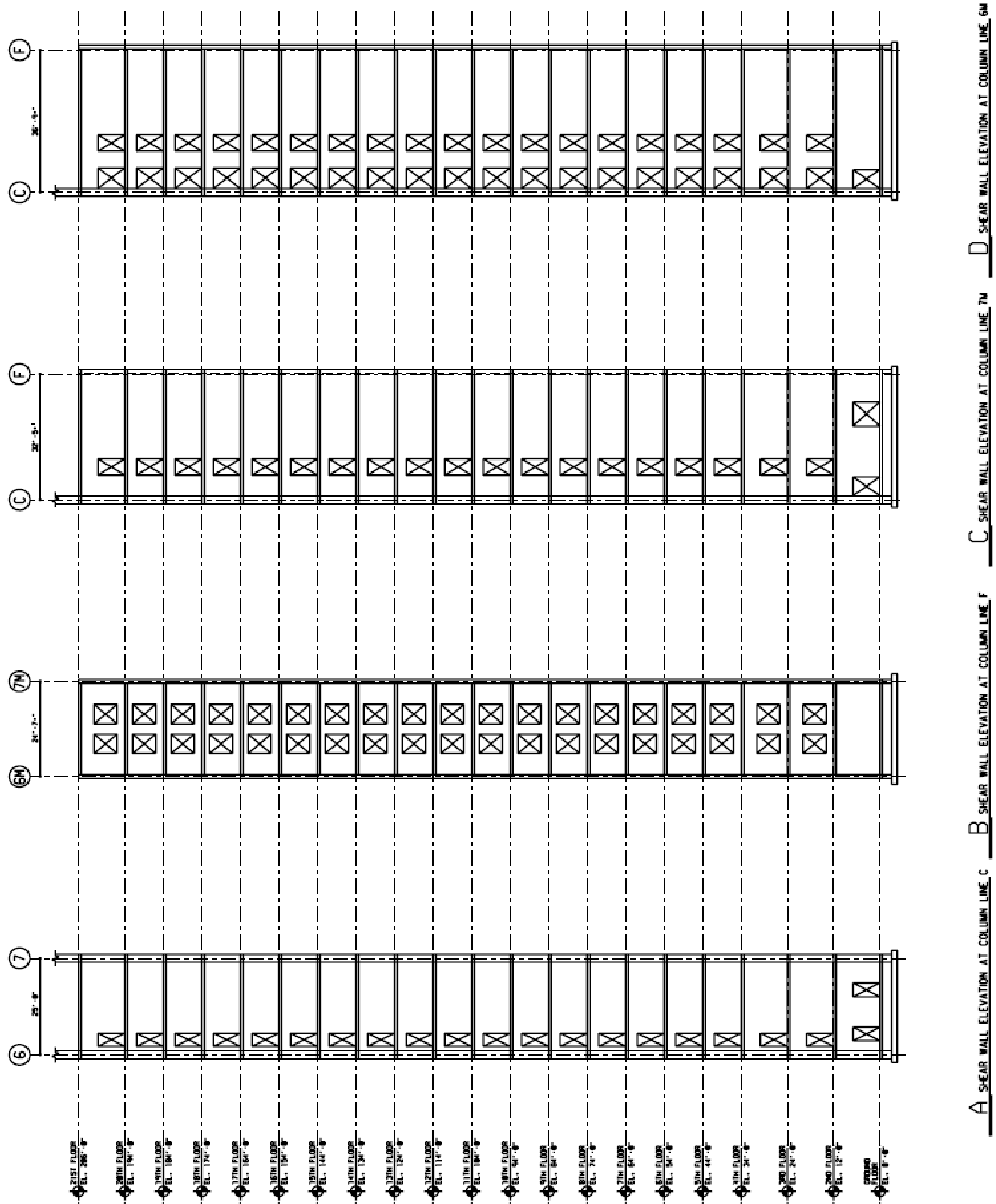
Codes Followed for Design

-Building Officials and Code Administrators (BOCA) National Building Code / 1996 with City of Wilmington Amendments.
-American Society of Civil Engineers 7-1995 (ASCE7-95)
-Council of American Building Officials/American National Standards Institute (CABO/ANSI)
-ANSI/ASME Standard A17.1 Safety Code for Elevators and Escalators

Building Sketches



<u>Basic Building Layout</u>
Typical Floors (3-20)


A SHEAR WALL ELEVATION AT COLUMN LINE C
B SHEAR WALL ELEVATION AT COLUMN LINE F
C SHEAR WALL ELEVATION AT COLUMN LINE 7M
D SHEAR WALL ELEVATION AT COLUMN LINE 6M

Shear Walls

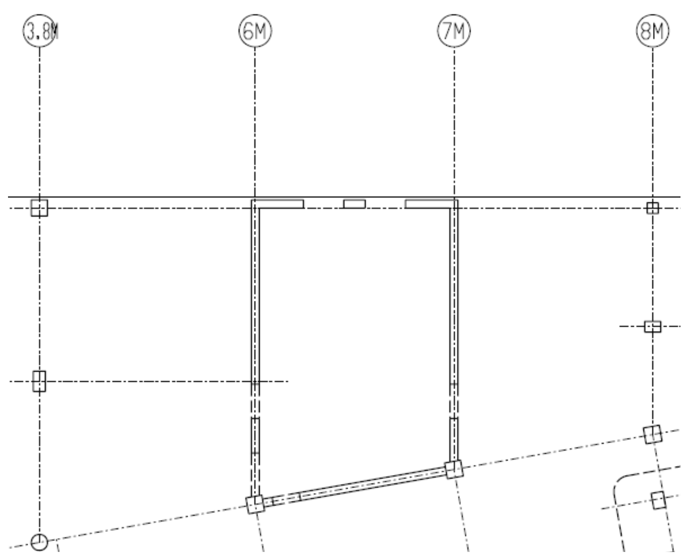
Description of Structure

Slab and Framing System

All the floors in the building have the same two way flat slab system, including the roof and the ground floor. It is an 8" slab with #6 bars at 10" on center, each way in the top and #4 bars at 10" on center, each way in the bottom. The strength of the concrete in the floor system is 5,600psi from the ground floor to the fifth floor and 4,500psi above the fifth floor. Each floor also has small sections of concrete framing. Some of these beams are post-tensioned concrete framing. The member sizes range from 12"x 16" to 36"x 60".

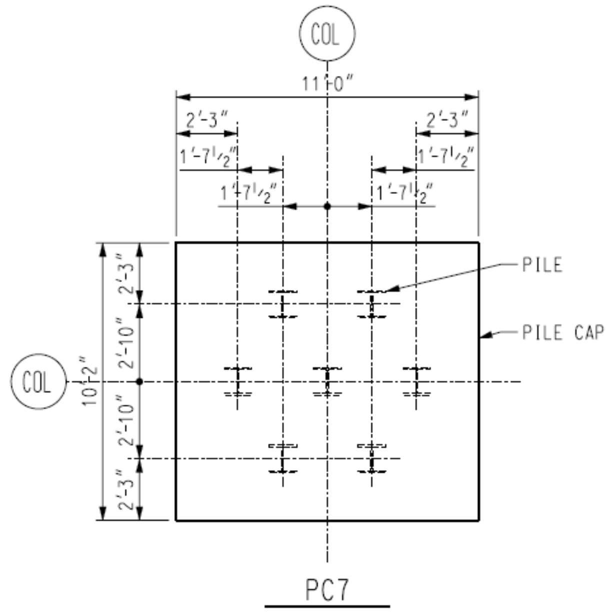
Main Wind Force Resisting System (Lateral System)

The main wind force resisting system consists of 4 concrete shear walls arranged in a core box at the center of the building. The walls travel the height of the building. They are 12" thick and range in length from 25 to 36 feet. The typical wall reinforcing is #4 bars each way in each face at 12" on center. All of the shear walls have at least one or two openings in them per floor for doors and windows. In addition to the typical reinforcing two #9 bars travel the height of the building on each side of any opening. At the edge of each shear wall four #11 bars travel vertically through the structure. (see shear wall diagram for more information)



Foundation

The building's columns rest on the foundation system consisting of H-piles and 4,000psi concrete pile caps. The pile caps range in size, shape, and the number of piles they sit on. The H-piles are H12x74s and are grouped in bunches of 2, 3, 4, 5, 6, 7, and 8 piles. The pile cap sizes range from 6'x 8'x 43" for the grouping of 2 piles to 11'x 10'x 60" for the areas with 8 piles. The shear walls also rest on strip pile caps topping H-piles. The edge of the slab on grade rests on grade beams which span the pile caps.

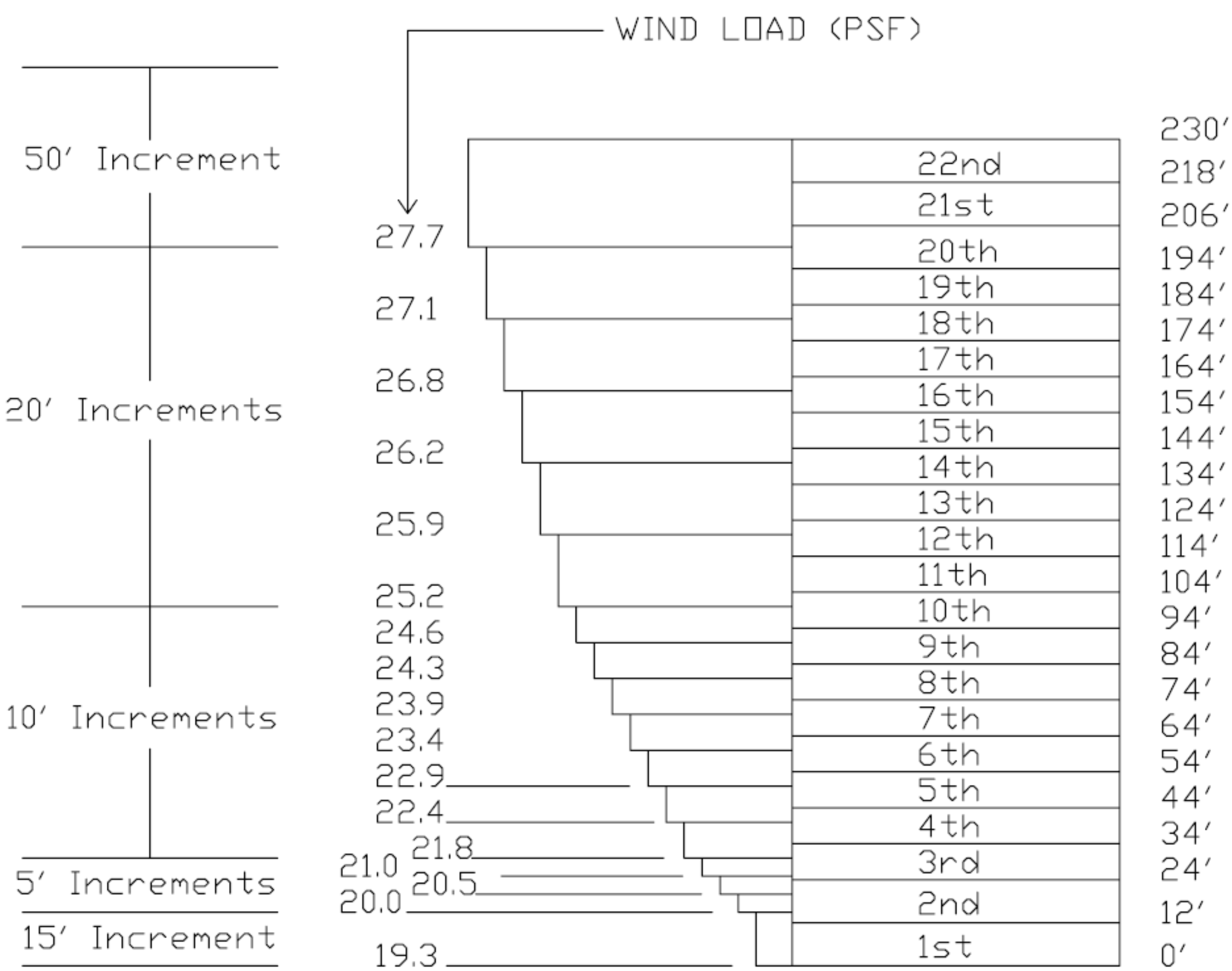


Design Loads

•110 psf Dead Load
•15 psf Miscellaneous Dead Load
•15 psf Snow Load
•40psf Live Load Typical Floor
•30 psf Live Load Roof
•100 psf Live Load Public Space
•150 psf Live Load Mechanical Floor

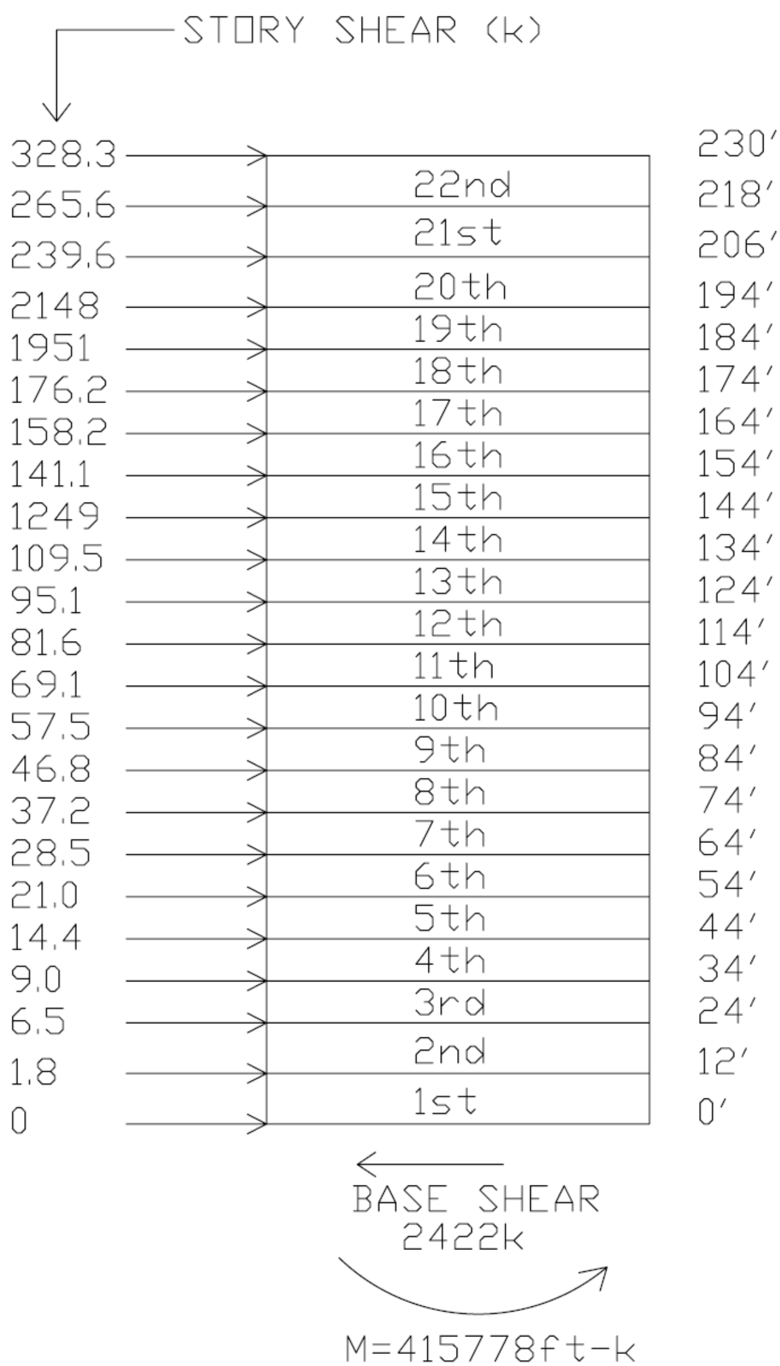
Wind Load

The following image is a wind loading diagram for the apartment tower. For the calculations I estimated the building to be a 91'x157' rectangle. These dimensions are conservative and provide the loading for the worst case scenario pressures on the structure. In order to calculate the building pressures I used method 2 for high rise buildings from ASCE7. It was also determined that the tower was not able to be classified as a rigid structure and therefore a gust factor needed to be found. Other relevant information used in the wind loading calculations includes an importance factor of 1 and a wind exposure of class "C". The total base shear on the building due to this loading case is 894k and the total resisting moment at the base of the structure is 108,792 ft-k. All of the information presented here is generated from calculations and spreadsheets in the appendix.



Seismic Load

For the seismic calculations I also estimated the shape of the building as a 91’x157’ rectangle. Items to note include: seismic use group I, importance factor 1.0, soil site class E, and an R value of 6.0 for specially reinforce concrete shear walls. All other design values, calculations, and spreadsheets are given in the appendix.



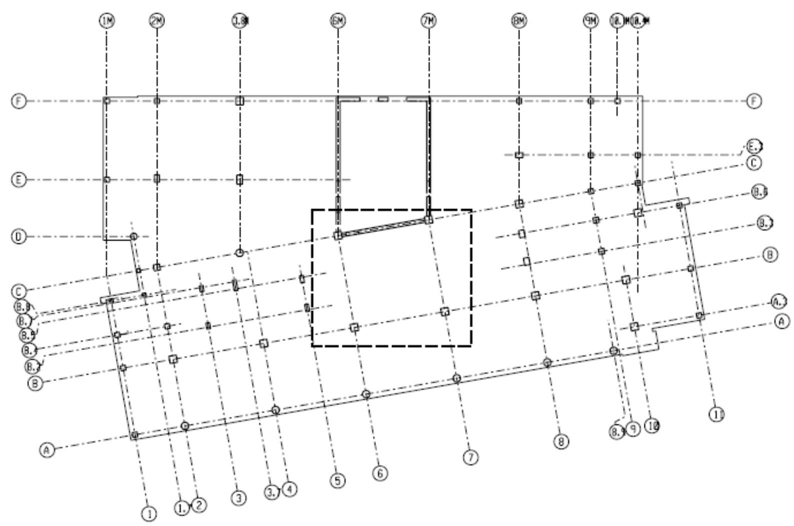
Snow Load

The ground snow load in Wilmington Delaware can be conservatively assumed to be 25psf. Being in an open area the exposure factor for the building is 0.9. The building has the typical 1.0 thermal and importance factors. After multiplying the ground load by exposure, thermal, and importance factors the roof snow load is 15.75psf.

Spot Checks

2-way slab

For the 2-way slab check I used an average interior bay 25'x25'6". It can be seen in the diagram below enclosed by column lines B,C,6, and 7. This bay is typical of floors 3-20. However the calculations I did only apply to floors 15-20 where the compressive strength of the concrete is 4,500 psi. The slab analyzed is an 8" flat slab with #6 bars at 10" on center each way in the top, and #4 bars at 10" on center each way in the bottom. I used 40psf live load and assumed 120psf total dead load on the panel. I checked the bay for the long span direction of 25'6". The following table gives my moments.

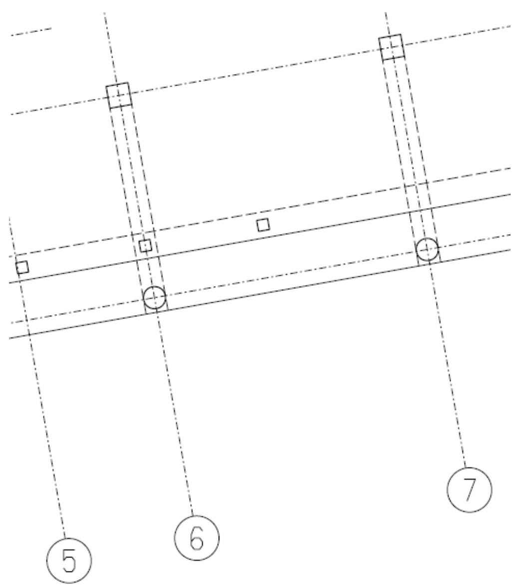


	Column Strip	Middle Strip
Support	175.1ft-k	58.4ft-k
Midspan	75.4ft-k	50.3ft-k

I checked the top steel using the 175.1ft-k moment as the worst case and found that the slab is sufficient to withstand this bending moment. For the bottom steel I used the 75.4ft-k moment and also found that the #4 bars were sufficient. See appendix for all other calculations and assumptions.

Beam Check

For the beam spot check I used a reinforced concrete beam spanning between columns A6 and B6 on the 21st floor. The beam is 24" wide and 16" deep, with five #8 bars in the top, and four #8 bars in the bottom. The beam is in an exterior bay running perpendicular to the edge of slab. It is supported by a 24" diameter round column on one side and a 24" square column on the other. The beam has a 101ft-k positive moment and a 157.3ft-k negative moment. I found that both the top and bottom steel were sufficient to withstand these moments. See appendix for all other calculations and assumptions.

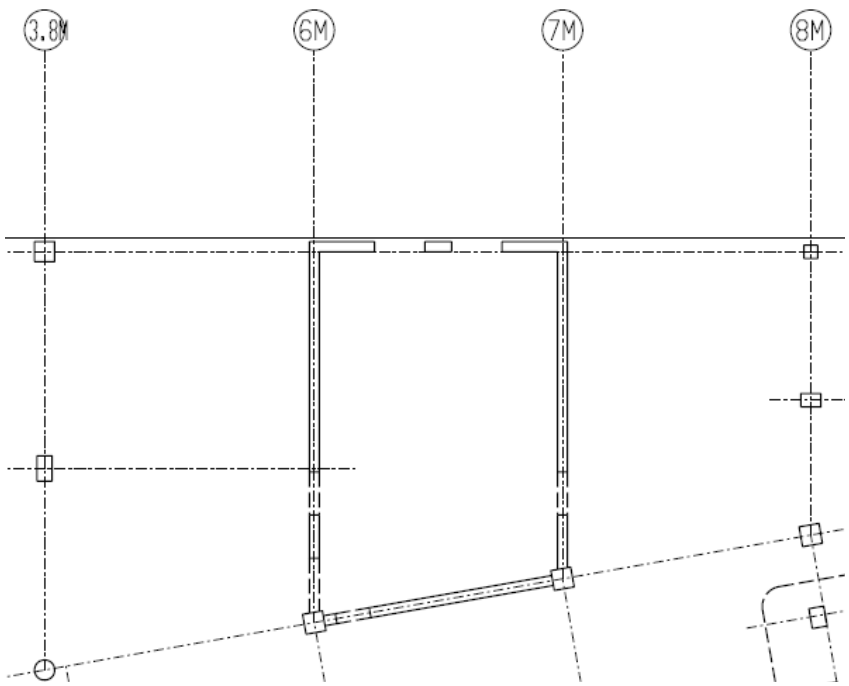


Column Check

For the column check I used column B7 (upper right most column in above figure). It is a 24" square column with ten #11 bars for reinforcement. I analyzed the column between level 15 and 16. The column has a tributary area of 550sqft per floor above. Using a live load reduction the total live load can be lowered from 40psf to 16psf. On completion of the column calculations I found that it was sufficient to carry the load applied. See appendix for all other calculations and assumptions.

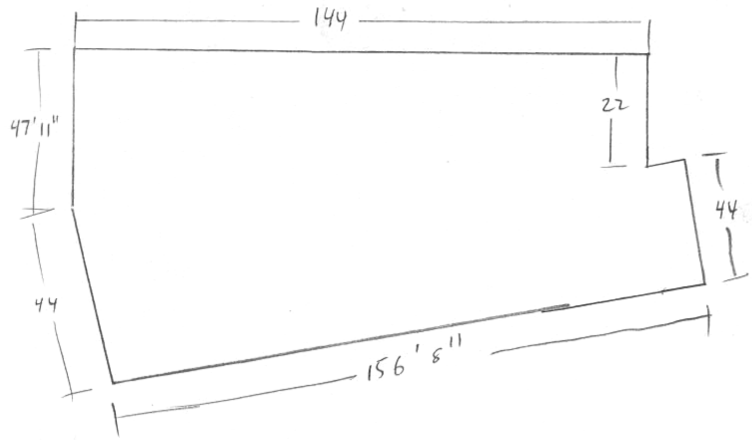
Shear Wall Check

For the shear walls I checked the controlling wall for the worst case shear load. I assumed the distribution to be equal between each of the shear walls in each direction. The worst case shear was for seismic loading at 2422k, or 1211k per wall. I determined the maximum load able to be resisted by the wall to be 1865k and therefore conclude the shear wall system to be adequate. See appendix for all other calculations and assumptions.

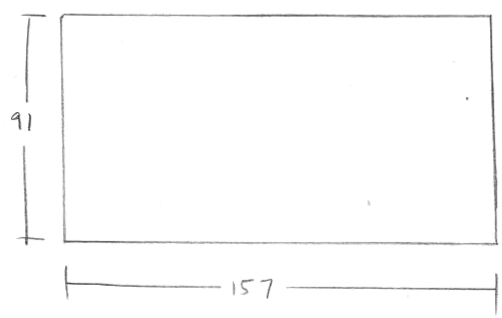


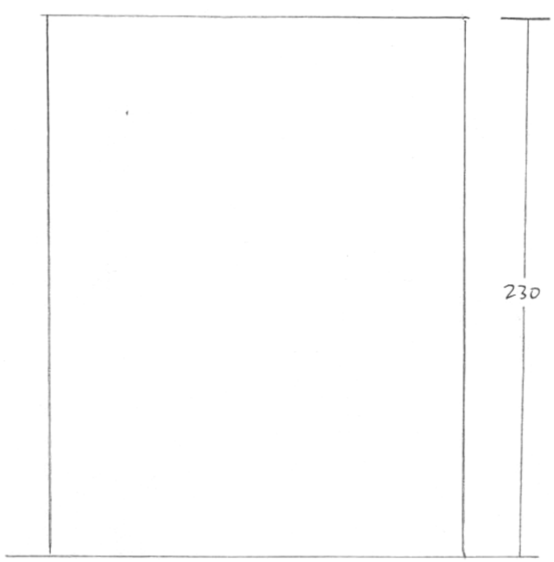
Appendix

Wind Calculations



assume





floors	1+2	12'0"
floors	3-19	10'6"
floors	20-21	12'0"
floor	22	12'0"
total		230'0"

BASIC WIND SPEED: 75 mph (1609.3) use 90 mph

IMPORTANCE FACTOR I = 1.05 (1609.5)

WIND EXPOSURE "C" OPEN TERRAIN

① find V 6.5.4
find Kd

② find I 6.5.5

③ find K_z, K_h 6.5.6

④ find K_{zt} 6.5.7

⑤ find G or Gf 6.5.8

⑥ enclosure classification 6.5.9

⑦ internal pressure coeff. GC_{pi} 6.5.11.1

⑧ external pressure coeff 6.5.11.2,3
C_p or GC_{pf} C_f

⑨ velocity pressure q_z or q_h 6.5.10

⑩ design wind load p or F 6.5.12

① V = 90 mph
K_d = .85

② Building category II
I = 1.0

③ K_z, K_h = 1.50

④ K_{zt} = 1.0 flat ground

⑤ C_t = .016 x = .9

$T = .016(230)^{.9} = 2.14$

$n_1 = \frac{1}{2.14} = .467$ FLEXIBLE

Gust factor calcs.

$$G = .925\left(\frac{1+1.7\,I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7\,g_v I_{\bar{z}}}\right)$$

$g_v = g_Q = 3.4$

$$g_R = \sqrt{2\ln[(3600)(.467)]} + \frac{.577}{\sqrt{2\ln[3600(.467)]}} = 4.00$$

n_1 n_1

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)}$$

$z_{min} = 15$ ft

$c = .20$

$\bar{z} = .6(230) = 138$ ft

$\ell = 500$ ft

$\bar{\epsilon} = 1/5.0$

$$L_{\bar{z}} = 500\left(\frac{138}{33}\right)^{1/5} = 665.6$$

$\bar{b} = .65$

$\bar{\alpha} = 1/6.5$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = .65\left(\frac{138}{33}\right)^{1/6.5} 90\left(\frac{88}{60}\right) = 106.9$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}} = \frac{.467\,(665.6)}{106.9} = 2.91$$

$$R_n = \frac{7.47\,N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(2.91)}{[1+10.3(2.91)]^{5/3}} = .0712$$

$$n_h = 4.6\,n_1\left(h/\bar{V}_{\bar{z}}\right) = 4.6(.467)\left(\frac{230}{106.9}\right) = 4.62$$

$$n_B = 4.6\left(\frac{n_1}{\bar{V}_{\bar{z}}}\right) = 4.6\left(\frac{.467}{106.9}\right) = .020$$

$$n_L = 15.4\,n_1\left(\frac{L}{\bar{V}_{\bar{z}}}\right) = 15.4(.467)\left(\frac{157}{106.9}\right) = 10.56$$

$$R_\ell = \frac{1}{n} - \frac{1}{2n^2}\left(1-e^{-2n}\right)$$

$$R_h = \frac{1}{4.62} - \frac{1}{2(4.62)^2}\left(1-e^{-2(4.62)}\right) = .193$$

$$R_B = \frac{1}{.02} - \frac{1}{2(.02)^2}\left(1-e^{-2(.02)}\right) = .987$$

$$R_L = \frac{1}{10.56} - \frac{1}{2(10.56)^2}\left(1-e^{-2(10.56)}\right) = .090$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B(.53+.47R_L)} = \sqrt{\left(\frac{1}{.05}\right).0712(.193)(.987)\left[.53+.47(.09)\right]}$$

$R = .394$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .2\left(\frac{33}{138}\right)^{1/6} = .158$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}} = \sqrt{\frac{1}{1+.63\left(\frac{91+230}{665.6}\right)^{.63}}} = .846$$

$$G_f = .925\left[\frac{1+1.7(.158)\sqrt{3.4^2(.846)^2+(4)^2(.394)^2}}{1+1.7(3.4)(.158)}\right] = .909$$

Velocity Pressure

$$q_z = .00256\, K_z K_{zt} K_d V^2 I$$

see spreadsheet

$$p = \overset{q_z}{q}\, G_f C_p - \overset{q_h}{q_i}(G\,C_{pi})$$

$G C_{pi} = .18$

$C_p = .8$ for windward

$-.35$ for leeward E-W

$-.5$ for leeward N-S

WIND CALCULATIONS (see calcs. for additional info.)

Kzt=	1
Kd=	0.85
V=	90
I=	1
Gf=	0.909
Gcpi=	0.18
Cp windward=	0.8
Cp leeward=	-0.35

Height	Kz	qz	p(windward)	p(leeward)	pressure (psf)
0-15	0.85	14.98176	6.129478656	-13.18784935	19.317328
20	0.9	15.86304	6.770345472	-13.18784935	19.95819482
25	0.94	16.568064	7.283038925	-13.18784935	20.47088827
30	0.98	17.273088	7.795732378	-13.18784935	20.98358172
40	1.04	18.330624	8.564772557	-13.18784935	21.7526219
50	1.09	19.211904	9.205639373	-13.18784935	22.39348872
60	1.13	19.916928	9.718332826	-13.18784935	22.90618217
70	1.17	20.621952	10.23102628	-13.18784935	23.41887562
80	1.21	21.326976	10.74371973	-13.18784935	23.93156908
90	1.24	21.855744	11.12823982	-13.18784935	24.31608917
100	1.26	22.208256	11.38458655	-13.18784935	24.57243589
120	1.31	23.089536	12.02545336	-13.18784935	25.21330271
140	1.36	23.970816	12.66632018	-13.18784935	25.85416952
160	1.39	24.499584	13.05084027	-13.18784935	26.23868961
180	1.43	25.204608	13.56353372	-13.18784935	26.75138307
200	1.46	25.733376	13.94805381	-13.18784935	27.13590316
250	1.53	26.967168	14.84526735	-13.18784935	28.0331167
230	1.502	26.4736512	14.48638194	-13.18784935	27.67423128

story	elev.	trib. H below	trib. H above	trib. range	V(lb)	V(k)	M(ft*k)
ground	0		6	0-6	18196.92298	18.19692298	0
1	12	6	6	6-18	36695.69422	36.69569422	440.3483307
2	24	6	5	18-29	35514.20979	35.51420979	852.3410349
3	34	5	5	29-39	34030.87708	34.03087708	1157.049821
4	44	5	5	39-49	35057.1612	35.0571612	1542.515093
5	54	5	5	49-59	35882.21314	35.88221314	1937.639509
6	64	5	5	59-69	36687.14186	36.68714186	2347.977079
7	74	5	5	69-79	37492.07058	37.49207058	2774.413223
8	84	5	5	79-89	38115.89034	38.11589034	3201.734788
9	94	5	5	89-99	38538.47792	38.53847792	3622.616924
10	104	5	5	99-109	39484.26916	39.48426916	4106.363993
11	114	5	5	109-119	39584.88525	39.58488525	4512.676919
12	124	5	5	119-129	40490.43006	40.49043006	5020.813328
13	134	5	5	129-139	40591.04615	40.59104615	5439.200185
14	144	5	5	139-149	41134.37304	41.13437304	5923.349718
15	154	5	5	149-159	41194.74269	41.19474269	6343.990375
16	164	5	5	159-169	41919.17854	41.91917854	6874.745281
17	174	5	5	169-179	41999.67142	41.99967142	7307.942826
18	184	5	5	179-189	42542.9983	42.5429983	7827.911687
19	194	5	6	189-200	46863.70475	46.86370475	9091.558722
20	206	6	6	200-212	52814.39186	52.81439186	10879.76472
21	218	6	6	212-224	52814.39186	52.81439186	11513.53743
22	230	6	0	224-230	26407.19593	26.40719593	6073.655064
						894.0519381	108792.146

Base Shear=	894 k
Base Resisting Moment=	108792 ft*k

Seismic Calcuations

Seismic

Seismic use group I

$I = 1.0$

$1.0s = 7.5 \quad S_1 = .075$

$.2s = 30 \quad S_s = .3$

$R = 6.0$ special reinforced conc. shear walls

Soil Site Class E

$F_a = 2.34$

$F_v = 3.5$

$S_{DS} = \frac{2}{3} F_a S_s = \frac{2}{3}(2.34)(.3) = .468$

$S_{D1} = \frac{2}{3} F_v S_1 = \frac{2}{3}(3.5)(.075) = .175$

$T_0 = .2 \frac{S_{D1}}{S_{DS}} = .2\left(\frac{.175}{.468}\right) = .075$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{.175}{.468} = .374$

$T = 2.14$

$S_a = \frac{S_{D1}}{T} = \frac{.175}{2.14} = .0818$

seismic design cat. C

SEISMIC DISTRIBUTION (see Seismic cals. for additional info.)

k= 1.82
v= 2421.9

	Story	Story Load (k)	Story Height(ft)	wx*hx^k	Cvx	Story Shear (k)	Story Mom.(ft*k)
	roof	2643	230	52534163.9	0.135568	328.3328146	75516.54735
	22	2357	218	42496229.71	0.109665	265.5968169	57900.10608
	21	2357	206	38335216.79	0.098927	239.5909384	49355.73331
	20	2357	194	34368353.84	0.08869	214.7984761	41670.90436
	19	2357	184	31212461.55	0.080546	195.0744923	35893.70658
	18	2357	174	28194163.53	0.072757	176.2104577	30660.61964
	17	2357	164	25314851.61	0.065327	158.2150711	25947.27167
	16	2357	154	22576015.49	0.058259	141.0976431	21729.03704
	15	2357	144	19979256.3	0.051558	124.8681804	17981.01797
	14	2357	134	17526303	0.045228	109.5374888	14678.0235
	13	2357	124	15219032.68	0.039274	95.11730011	11794.54521
	12	2357	114	13059495.86	0.033701	81.6204297	9304.728986
	11	2357	104	11049948.66	0.028515	69.06097811	7182.341723
	10	2357	94	9192894.5	0.023723	57.45459147	5400.731599
	9	2357	84	7491139.475	0.019331	46.81880753	3932.779832
	8	2357	74	5947867.873	0.015349	37.17352775	2750.841053
	7	2357	64	4566748.925	0.011785	28.54168443	1826.667804
	6	2357	54	3352094.283	0.00865	20.95022494	1131.312147
	5	2357	44	2309103.788	0.005959	14.4316477	634.992499
	4	2357	34	1444279.382	0.003727	9.026589164	306.9040316
	3	3215	24	1045118.264	0.002697	6.531875556	156.7650134
	2	3215	12	295999.4098	0.000764	1.84996414	22.19956968
	1	0	0	0	0	0	0
Totals		53856		387510738.8	1	2421.9	415777.777

Base Shear= 2422 k
Base Resisting Moment= 415778 ft*k

Snow Load Calculations

SNOW LOAD 15 psf
ROOF LOAD 30 psf
FLOOR LL 40 psf
PUBLIC LL 100 psf
AVG. FLOOR DL 125 psf

FLOOR 1 + 2	3215 k
FLOOR 3 – 22	2357 k
ROOF	2643 k

$k = 1.82$

$$C_s = \frac{S_{DS}}{R/I} = \frac{.468}{6} = .078$$

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{.175}{2.14\,(6)} = .014$$

$$C_s = .044(S_{DS})(I) = .044(1.468) = .021$$

$$C_s = \frac{.5(.075)}{6} = .00625 \leftarrow \text{controls}$$

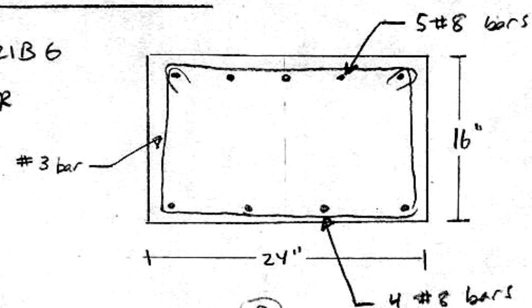
$$V = 387510(.00625) = 2421.9\ k$$

Spot Checks

SPOT CHECK BEAM

BEAM 21B6

21st FLOOR

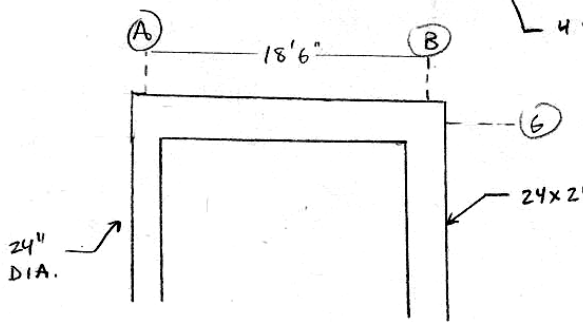


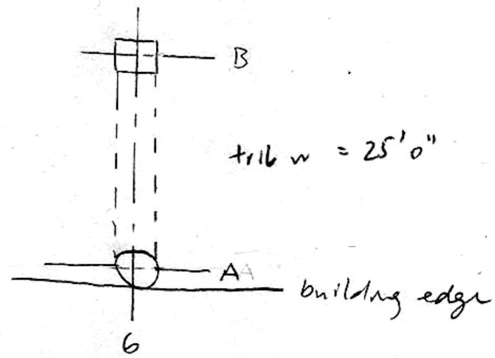
3" clr. bott.

2" clr. side

1.5" clr. top

assume $f'c = 4500psi$





$w_u = 208psf\,(25') = 5.2\ klf$

discontinuous end integral w/ support $\frac{w_u l_n^2}{14}$ positive moment

negative moment $\frac{w_u l_n^2}{9}$

$l_n = 16'6"$

$$\frac{w_u l_n^2}{14} = \frac{5.2\,(16.5)^2}{14} = 101.1\ ft{\cdot}k$$

$$\frac{w_u l_n^2}{9} = 157.3\ ft{\cdot}k$$

check bottom steel (pos. moment)

$$a = \frac{A_s f_y}{.85 f'c\, b} = \frac{3.16\,(60)}{.85\,(4.5)(24)} = 2.07 \qquad d = 16 - 3 - 3/8 - 1/2 = 12.125$$

$$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 3.16(60)\left(12.125 - \frac{2.07}{2}\right) = 2102.7 = 175.2\ ft{\cdot}k$$

$$\phi M_n = .9\,(175.2) = 157.7\ ft{\cdot}k \geq 101.1\ ft{\cdot}k \quad \underline{OK}$$

check top steel (neg moment)

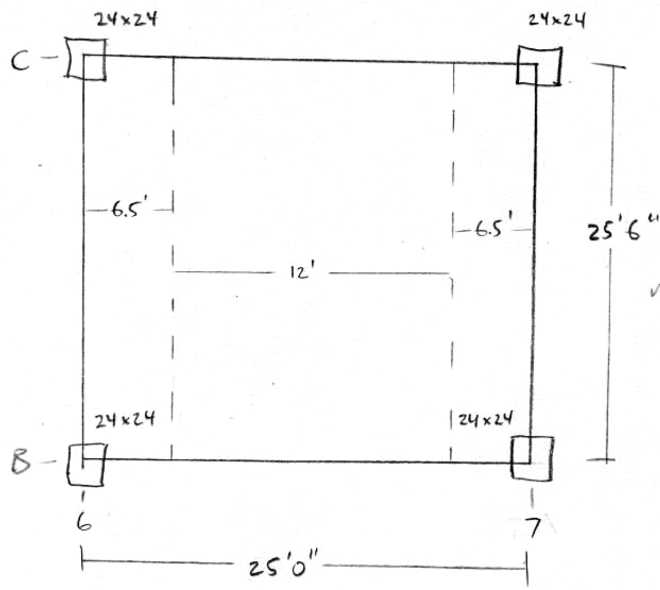
$a = \frac{A_s f_y}{.85 f'_c b} = \frac{3.95(60)}{.85(4.5)(24)} = 2.58$ $d = 16 - 1.5 - 3/8 - .5 = 13.625$

$M_n = A_s f_y \left(d - \frac{a}{2}\right) = 3.95(60)\left(13.625 - \frac{2.58}{2}\right) = 2923.4 = 243.6$ ft·k

$\phi M_n = .9(243.6) = 219$ ft·k ≥ 157.3 ft·k <u>OK</u>

2 WAY SLAB CHECK

AVG. INTERIOR BAY



ACTUAL DESIGN

8" slab w/ #6 E.W. TOP @ 10"
w/ #4 E.W. BOTTOM @ 1

LL = 40 psf

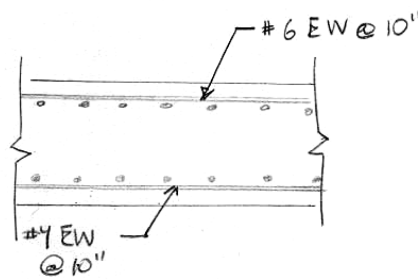
misc: Super imposed DL assume = 20 psf

$SW = \frac{8}{12}(150) = 100\,psf$

$w_u = 1.2(120) + 1.6(40) = 208\,psf$

$M_o = \frac{.208\,(25)(23.5)^2}{8} = 359\ ft\cdot k$

		TOTAL Mu	Width	Moment / Width
support	C.S. 75%	175.1	13	13.5
233.4 65%	M.S. 25%	58.4	12	4.9
midspan	C.S. 60%	75.4	13	5.8
125.7 35%	M.S. 40%	50.3	12	4.2



$1\#6 \Rightarrow A_s = .44$ As per ft $= .44\left(\frac{12}{10}\right) = .528\ in^2$

$d = 8 - .75 - .75 - \frac{.75}{2} = 6.125$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{.528(60)}{.85(4.5)(12)} = .690$

worst case neg. moment, check top reinf.

$M_n = .528(60)\left(6.125 - \frac{.690}{2}\right) = 183.1\ in\cdot k = 15.3\ ft\cdot k$

$\phi M_n = .9(15.3) = 13.7\ ft\cdot k \geq 13.5\ ft\cdot k$ OK

$1\#4 \Rightarrow A_s = .20 \quad A_s \text{ per ft} = .20\left(\frac{12}{10}\right) = .24\,in^2$

$d = 8.0 - .75 - .5 - .25 = 6.5\ in$

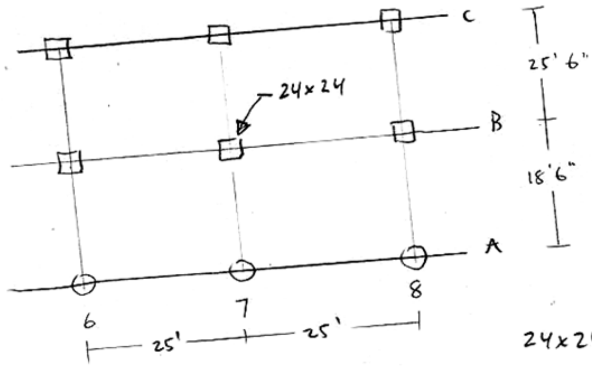
$$a = \frac{.24(60)}{.85(4.5)(12)} = .314$$

$$M_n = .24(60)\left(6.5 - \frac{.314}{2}\right) = 91.3\ in\cdot k = 7.6\ ft\cdot k$$

$\phi M_n = .9(7.6) = 6.8\ ft\cdot k \geq 5.8\ ft\cdot k$ OK

COLUMN CHECK

B7 @ LEVEL 15



24x24 w/ 18-#1

tributary area

$25'\left(\frac{25.5+18.5}{2}\right) = 550\ ft^2$

snow load = 16 psf

self weight = 125 psf

live load = 40 psf

roof load = 30 psf ← assume 40 for this

$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$ $K_{LL} = 4$

$L = 40\left(.25 + \frac{15}{\sqrt{4\,[8(550)]}}\right) = 14.5$ can't be less than $.4L_o = 16$

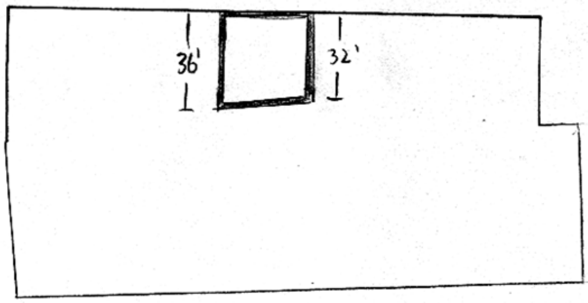
use LL = 16 psf

load per floor = $1.2(125) + 1.6(16) = 175.6$ psf

total load = $175.6\,(8)(550) = 772.6$ k

$P_n = 4000\,psi\,(24)^2 = 2304\,k \geq 772.6\,k$ OK

SHEAR WALL

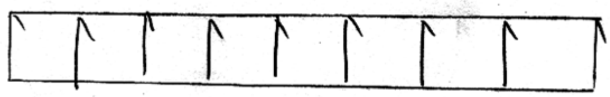


seismic base shear controls

V) 2422 k

M) 415778 ft k

32' section controls



Assume each take half the load

$V_n = 10\sqrt{f'_c}\, h d$ $\quad d = .8\ell = .8(32) = 25.6$

$h = 12"$

$V_n = 10\sqrt{4000}\,(12)(25.6)(12) = 2331\ k$

$\phi V_n = .8(2331) = 1865\ k$

$V_u = \frac{2422}{2} = 1211\ k$

$\phi V_n = 1865\ k \geq 1211\ k$ OK