The Towers at Greenville Place

Tower 'B'

Wilmington, DE

Technical Report #1

Shawn Brandt
Structural Option
Consultant: Dr Behr
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Table of Contents

Executive Summary

The objective of this report is to analyze the original design of Tower 'B' of The Towers at Greenville Place. The building is located in Wilmington, Delaware. This is a seven story mid-rise upscale apartment building consisting of 89 units. It was completed in July 2007 and is one of four nearly identical towers. This report consists of an overview of the structural systems, material strength, applicable codes, and design loads. The documents for this building unfortunately did not provide loading information with which to compare the results of my wind and seismic calculations.

The wind analysis in this report references ASCE 7-05, which is the same standard the original design used. The east/west was determined to have the greatest wind load. When comparing the surface area of the east/west direction to the surface area of the north/south direction, it is expected that the east/west direction would control due to the fact that it is larger. Though the building does have a parapet, it was determined that its effective force would be negligible when compared to other elements.

The seismic analysis, too, references the ASCE 7-05 design standard. This was found to be the overall controlling lateral force when comparing base shears of wind and seismic. Since detailed information was not provided in the design documents, assumptions and simplifications about the building materials and weights were made so that a total building weight could be calculated for use in finding seismic loads. Should there have been values to compare to, this would possibly account for any discrepancies.

Spot checks were performed for the hollow core concrete plank, reinforced CMU shear walls, and reinforced CMU load bearing walls. Once more, lack of information made it impossible to compare these findings. The building specifications listed three possible hollow core concrete planks that may have been used during construction. It was assumed for calculation purposes that an 8" by 4' plank from Nitterhouse was used. Spot checks can be found in Appendix B.

Despite lack of information to compare calculations to, the analyses performed for Tower 'B' provided for a better understanding of its structural systems.

Introduction

Tower 'B' of The Towers at Greenville Place is one of three virtually identical buildings. The towers, 'A', 'B', and 'C', are all directly neighboring upscale apartment buildings in Wilmington, Delaware. The project was complete in July of 2007 at an overall cost of $11.5 Million by a Design-Bid-Build delivery method. It is owned and managed by Pettinaro Real Estate Development Company.

The 180,000 square foot building consists of 89 different apartment units. One level is partially below grade and, on top of that, there are seven. The partially below grade ground floor is 12' and houses the lobby, exercise room, game room/café, storage, housekeeping, and electrical room. The ground floor lobby entrance opens to ground level, where as the opposite side of the building is nearly entirely below grade. The first floor is 10' and begins the typical apartment unit layout. Floors two through seven are also typical in layout, but only rise 9 feet and 4 inches each. The roof, though accessible, is virtually bare and houses no mechanical equipment.

Figure 1: North-west view of Tower 'B', showing canopy entrance.

Structural System Overview

Foundation

Foundations were designed according to recommendations on the geotechnical engineer's reports prepared by Advanced Geoservices Corp. The building's foundation is made up a combination of spread and continuous reinforced cast-in-place concrete footings. The design was based on an allowable soil bearing capacity of 3000 psf and calls for 3000 psi concrete.

The ground floor slab is 4 inch slab on grade laid on 4 mil poly vapor barrier and 4 inches of crushed stone. It is reinforced with 6x6 W1.4xW1.4 welded wire fabric (WWF). The slab on grade id designed to have a strength of 3500 psi.

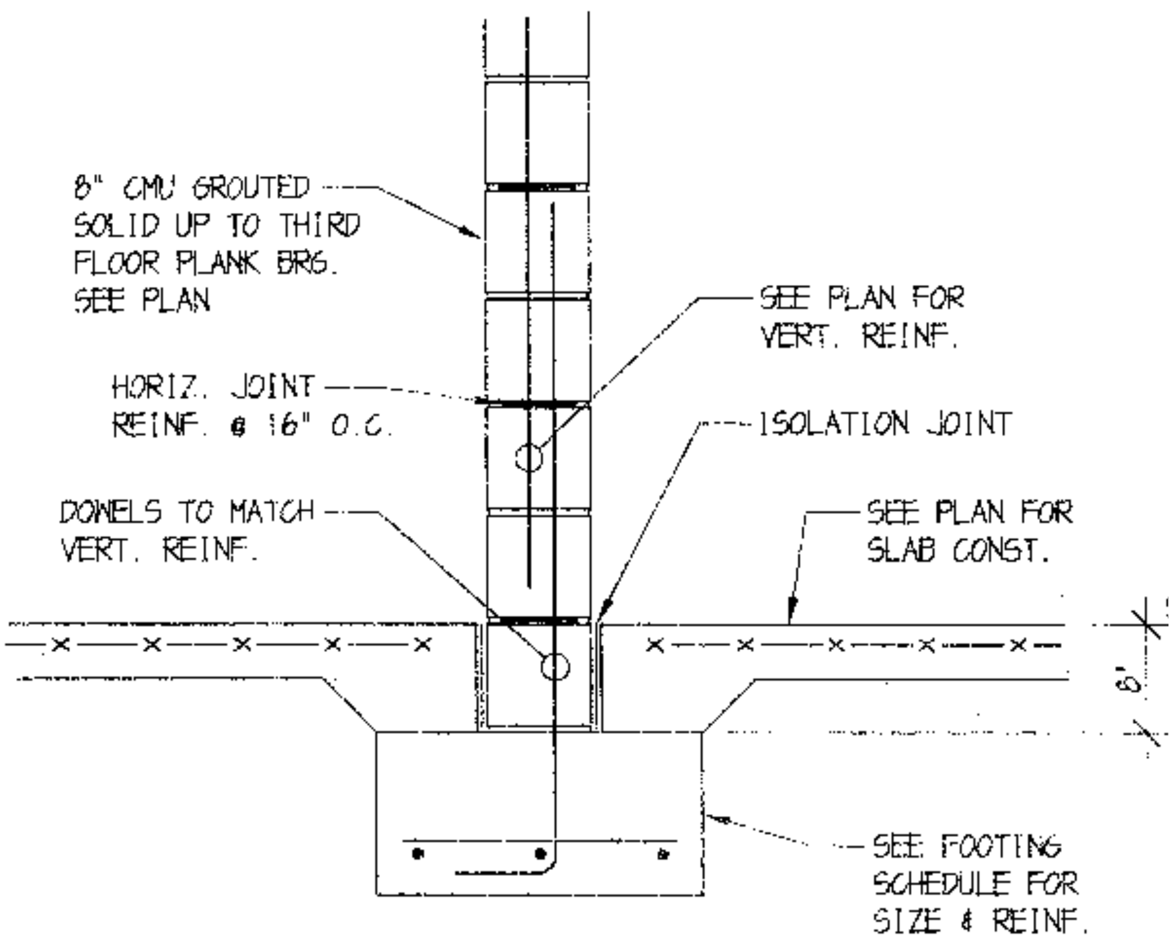


Figure 2: Typical interior foundation sections.

Shear Walls

The shear walls are 8 inch CMU with reinforced grouted cells that go all the way down to the foundation. Tower 'B' has three different strengths of shear walls. Each shear wall is essentially laid out the same, only differing slightly by the size and spacing of steel reinforcing used, depending on which level they reside. These walls each have two different spacing criteria. As you can see in Figure 3, the reinforcing at the ends of the walls are spaced more tightly than that compared to the middle portion.

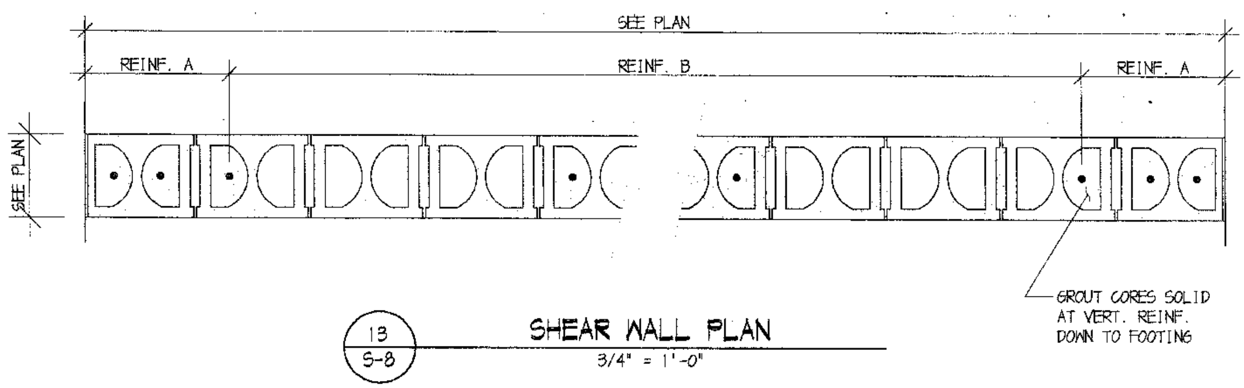


Figure 3: Typical shear wall plan.

Typical Wall

Nearly every wall in Tower 'B' contributes to the supporting the gravity loads. With the exception of cast in place concrete on the partially below grade ground floor, every wall is CMU. Figure 4 shows all load bearing CMU walls have regularly spaced reinforcing in grouted cells. Walls on floors 1 through 3 call for #4 reinforcing bar spaced at 32 inches on center. Walls on floors 4 through 7 call for #4 reinforcing bars spaced at 48 inches on center. Window and door opening are supported by precast concrete lintels, as can been see in plan in figure 4 and in detail in figure 5.

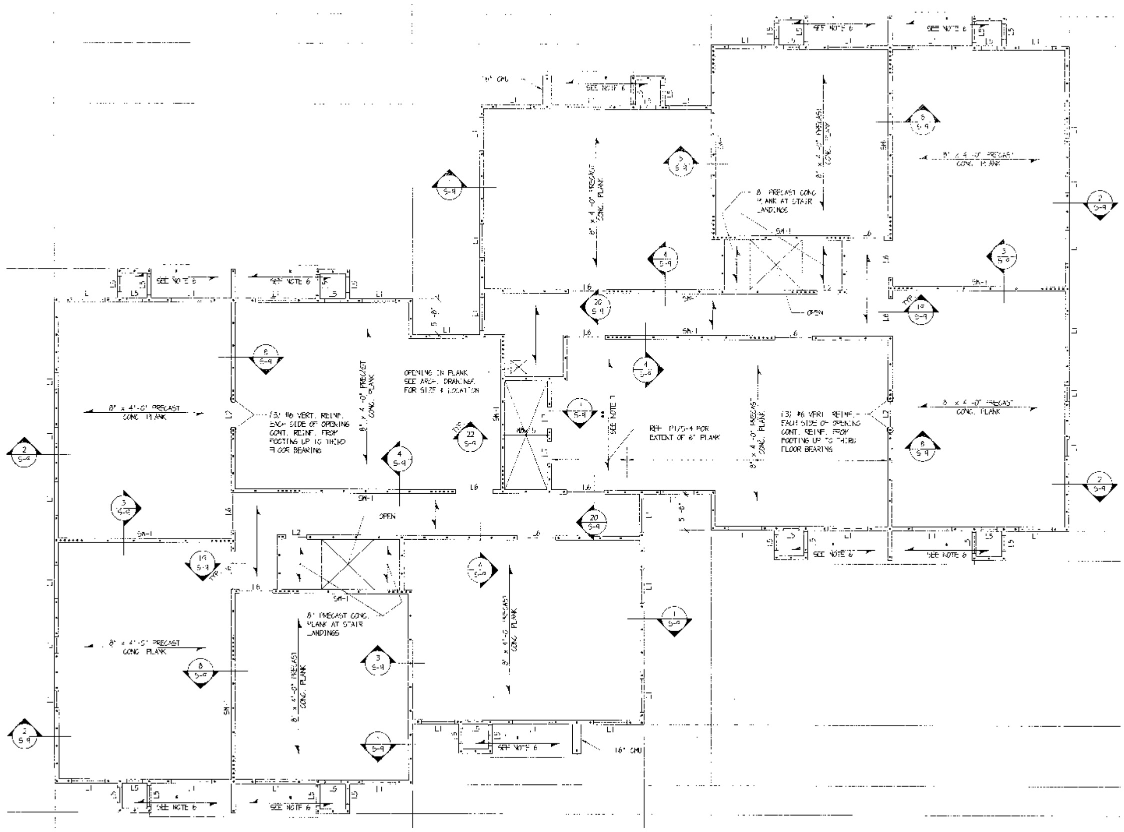


Figure 4: Typical plan layout showing reinforced CMU walls.

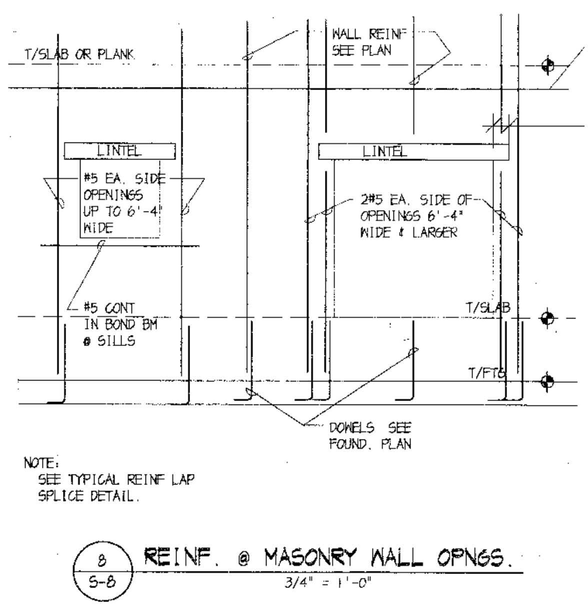


Figure 5: Typical wall openings supported by lintels.

Floor System

The floors of Tower 'B' are precast hollow core concrete plank. The corridor floors are 6 inch plank and all others are 8 inch plank. Referring back to figure 4, the planks span one direction each, but alternate per floor section. Special attention was given to certain plank joints due to the camber and direction of the planks. Said joints were off level where mid spans met perpendicularly with plank ends. Joints and levels corrections were filled solid with 300 psi flowable grout.

The support for the floor planks, as stated before, comes from the CMU walls. At the top of each level's CMU wall is a CMU bond beam with one continuous #5 reinforcing bar. The planks sit directly on a 3 inch bearing strip on the top of the wall. The floors are tied in using #4 reinforcing bars spaced at 48 inches and bent to suit each locations condition. Figures 6 through 8 display a variety of floor plank to wall connection conditions.

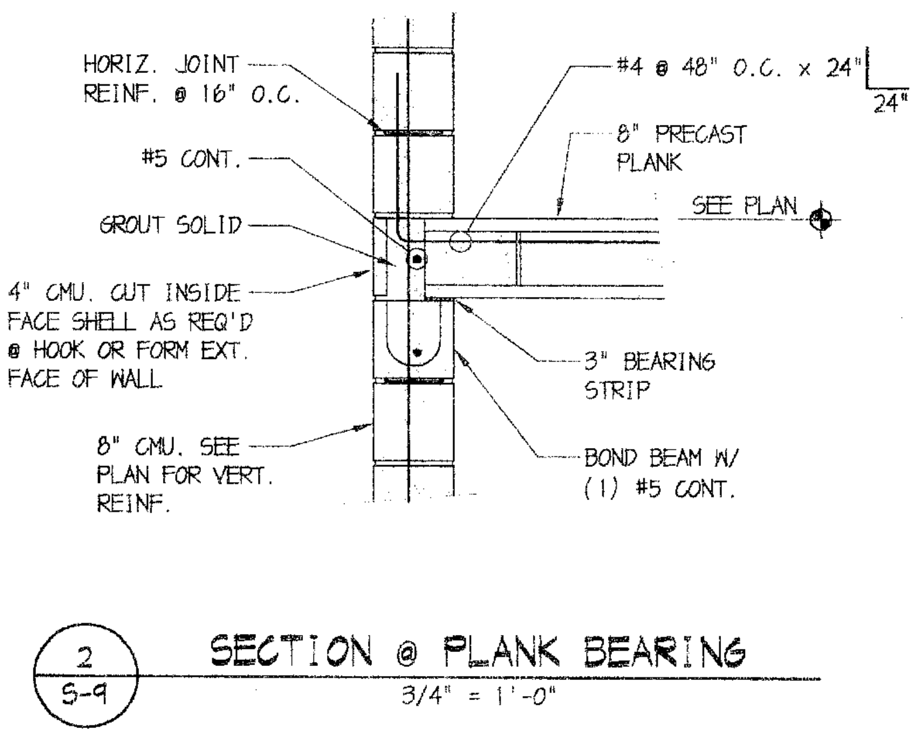


Figure 6: Detail of floor plank bearing on CMU wall.

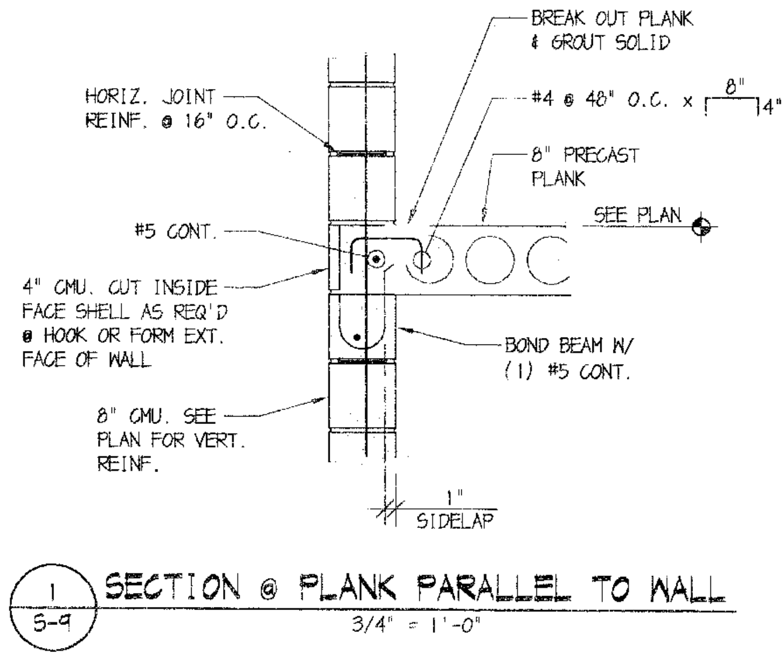


Figure 7: Detail of floor plank running parallel to wall connection.

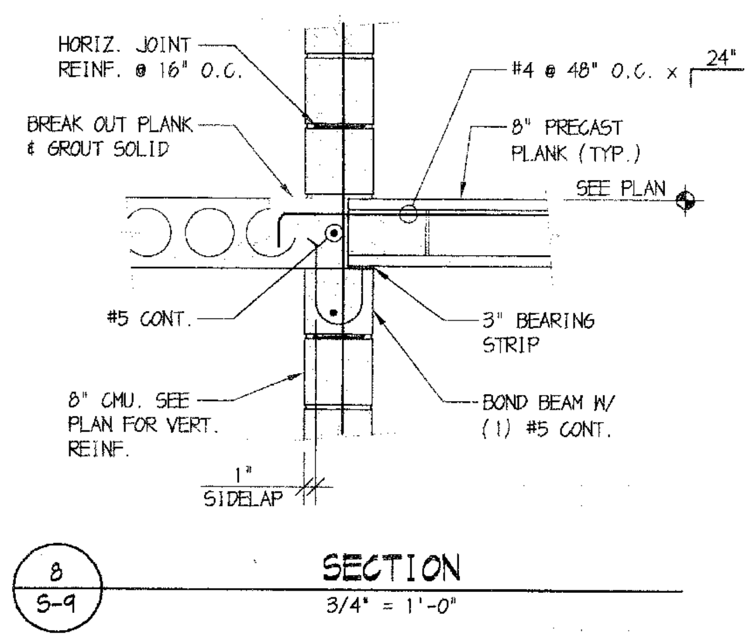


Figure 8: Detail of both bearing and parallel connection conditions.

Roof System

The roof of Tower 'B' is the same basic design as the typical floor system. It is accessible but the layout is mostly empty. Much like the other floors, the roof consists of 8 inch plank throughout except over the corridors where it is 6 inch plank and bears on the CMU wall. Joints, again, are filled solid with 3000 psi flowable grout. Figures 9 and 10 show two connection conditions.

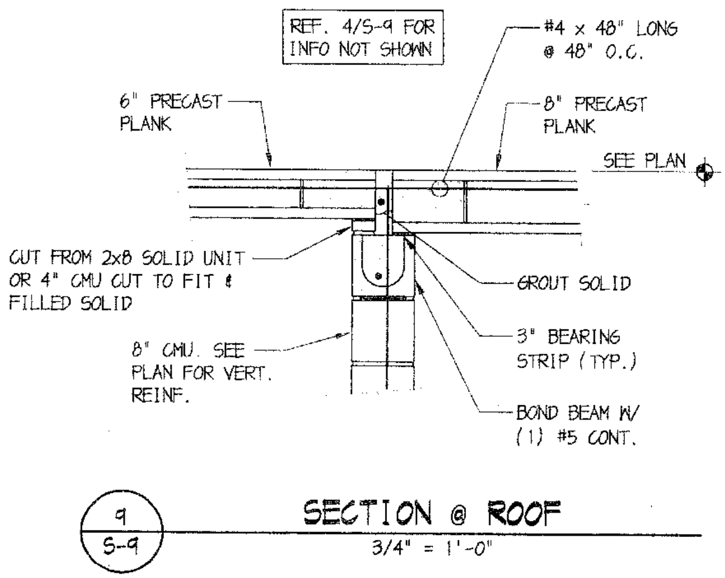


Figure 9: Detail of roof floor connection.

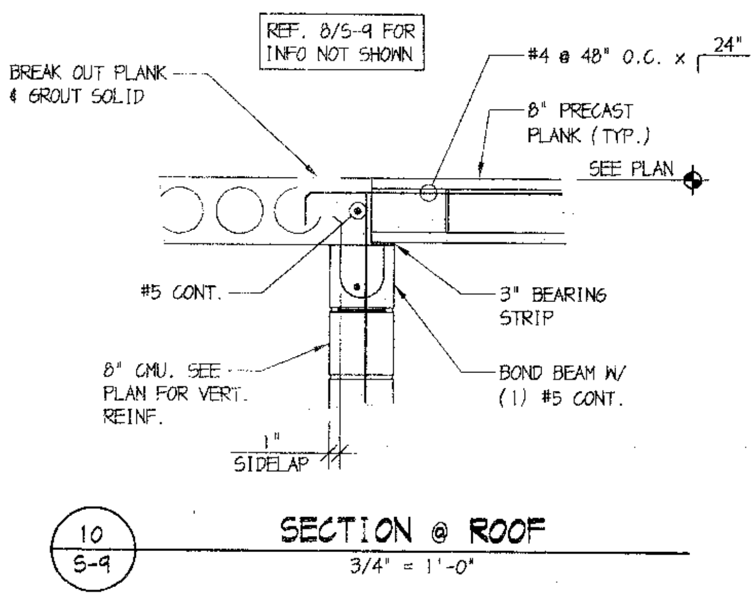


Figure 10: Detail of roof floor connection.

Material Strengths

Masonry

8" CMU – ASTM C90 Grade N	1900 psi
Core Grout	3000 psi
Bond Bean Grout	2500 psi

Precast

8" x 4' Hollow Core Plank	5000 psi
Joint Grout	3000 psi

Concrete

Foundation Wall	3000 psi
Slab on Grade	3500 psi
Footings	3000 psi
Reinforcement	60 ksi (A615)

Cold Formed Steel Framing

12, 14, & 16 Gage Studs	50 ksi (A653)
18 & 20 Gage Studs	33 ksi (A653)

Applicable Codes

Original Design Codes

- International Building Code (IBC), 2003 edition
    - With Amendments adopted by New Castle County (DE)
- American Concrete Institute (ACI)
    - Building Code Commentary 318-02
- American Institute of Steel Construction (AISC)
    - Steel Construction Manual

Additional References Used for Thesis

- American Society of Civil Engineers (ASCE)
    - ASCE 7 – 05
- Precast/Prestressed Concrete Design Handbook
    - PCI Manual for the Design of Hollow Core Slabs
- National Concrete and Masonry Association (NCMA) TEK
    - TEK 14-5A (2006)

Design Loads

The building design loads were determined by referencing ASCE 7-05. The live loads were then compared with the loads determined by the designer. However, the designed dead loads were not specified in the documents, therefore, the values determined by this analysis could not be compared to actual design dead loads. Tables 1 and 2 show this data. Snow loads were also calculated and can be found in Appendix A.

Live Loads		
Area	Actual Design	Thesis Design
Lobbies	100 psf	100 psf
1st Floor Corridor	100 psf	100 psf
Upper Corridors	40 psf	40 psf
Apartment	40 psf	40 psf
Balconies	60 psf	60 psf
Roof	30 psf	20 psf

Table 1: Live Loads

Dead Loads		
Type	Load (psf)	Total Load (Kips)
Hollow Core Concrete Plank	61.25	6362.65
4 inch Slab On Grade	50	649.25
Load Bearing CMU Walls	42	3438.92
MEP	10	1038.80
Ceiling Finish	0.75	77.91
Partitions	8	831.04
Floor Finish	1	103.88
EPDM Roof Assembly	2	25.97
Misc. (Storage LL for Seismic)		81.31
Total Dead Load (W), (Kips)		12609.73

Table 2: Dead Loads

Wind

Wind loads were calculated referencing ASCE 7-05 and flowcharts describing Method 2 for the main wind-force resisting system (MWFRS). According to ASCE 7-05, the structure was found to be rigid. For this preliminary analysis, the shape of Tower 'B' was simplified into a solid rectangular shape. The overall dimensions of the building footprint were used for this basic shape. The effects of the parapets, due to their size, were negligible. The calculated values take into account the effects of internal pressure and were done on a worst case scenario basis. Refer to Appendix A for a list of values and calculations. Refer to tables 3 through 7 and figures 11 through 14 for a detailed breakdown of wind loads.

Pressure (psf) North/South			
Level	Height Above Ground (ft)	Windward	Leeward
Top	78.67	13.99	-8.46
7	68.67	13.57	-8.46
6	59.33	13.13	-8.46
5	50.00	12.64	-8.46
4	40.67	12.09	-8.46
3	31.33	11.43	-8.46
2	22.00	10.61	-8.46
1	12.00	9.82	-8.46
Ground	0.00	9.82	-8.46
L/B = 160.67/127.33 = 1.262			

Table 3: Wind Pressure Acting in the North/South Direction

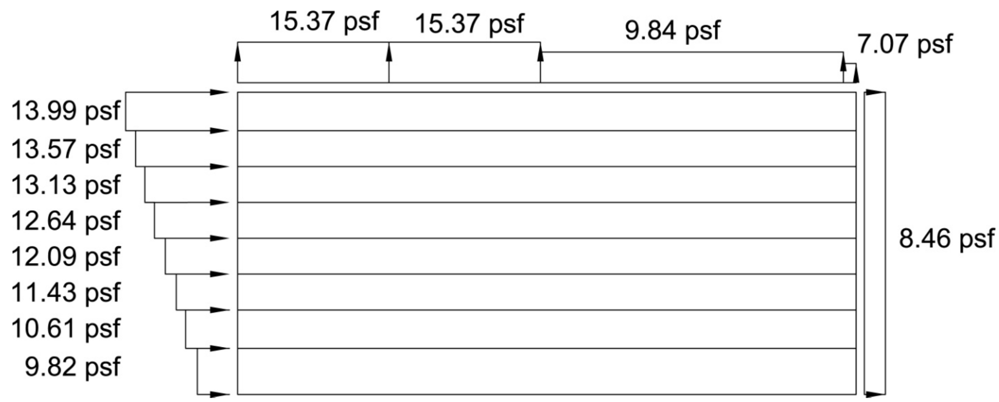


Figure 11: Diagram of Wind Pressure Acting in the North/South Direction

Pressure (psf) East/West			
Level	Height Above Ground (ft)	Windward	Leeward
Top	78.67	13.99	-9.84
7	68.67	13.57	-9.84
6	59.33	13.13	-9.84
5	50.00	12.64	-9.84
4	40.67	12.09	-9.84
3	31.33	11.43	-9.84
2	22.00	10.61	-9.84
1	12.00	9.39	-9.84
Ground	0.00	2.93	-9.84
L/B = 127.33/160.67 = .792			

Table 4: Wind Pressure Acting in the East/West Direction

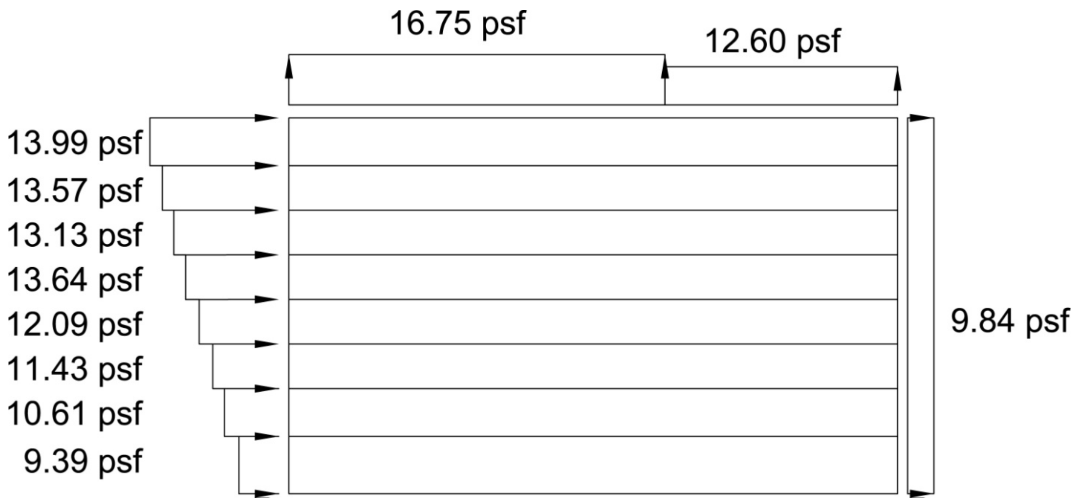


Figure 12: Diagram of Wind Pressure Acting in the East/West Direction

Roof N/S			
Horiz Dist. From WW edge (ft)	Cp Roof	Pressure (psf)	Force (kips)
0 to 39.335	-0.9	-15.37	34.26
39.335 to 78.67	-0.9	-15.37	34.26
78.67 to 157.34	-0.5	-9.84	43.87
157.34 to 160.67	-0.3	-7.07	1.33
Roof E/W			
Horiz Dist. From WW Edge	Cp Roof	Pressure (psf)	Force (kips)
0 to 39.335	-1.0	-16.75	50.96
39.335 to 127.33	-0.7	-12.60	85.77

Table 5: Wind Pressure and Force Acting on the Roof

Force (kips) North/South		
Level	Height Above Ground (ft)	Level Force
Top	78.67	28.43
7	68.67	26.07
6	59.33	25.53
5	50.00	24.94
4	40.67	24.25
3	31.33	23.42
2	22.00	23.13
1	12.00	25.59
Ground	0.00	
Total Force		201.36

Table 6: Wind Forces Acting on the North/South Direction

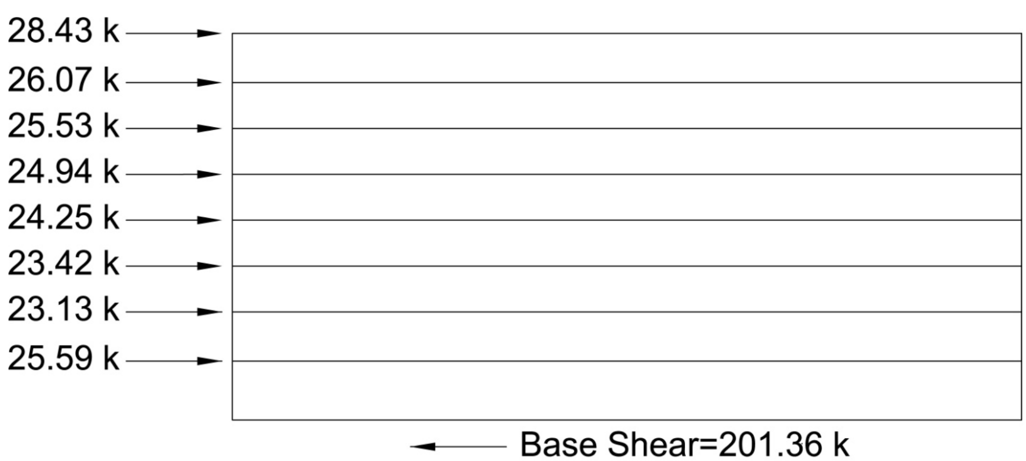


Figure 13: Diagram of Wind Forces Acting in the North/South Direction

Force (kips) East/West		
Level	Height Above Ground (ft)	Level Force
Top	78.67	38.10
7	68.67	34.97
6	59.33	34.29
5	50.00	33.54
4	40.67	32.67
3	31.33	31.63
2	22.00	31.34
1	12.00	32.87
Ground	0.00	
Total Force		269.40

Table 7: Wind Forces Acting on the East/West Direction

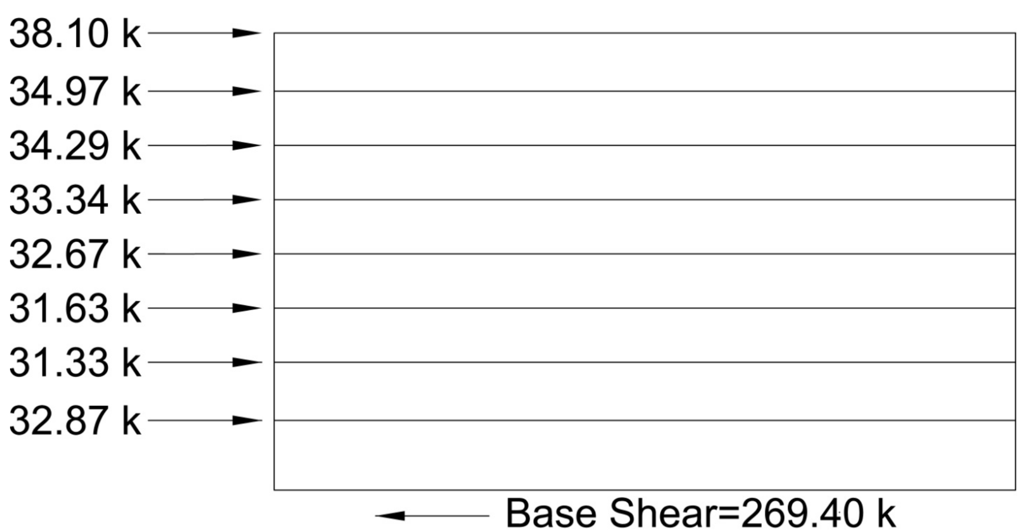


Figure 14: Diagram of Wind Forces Acting on the East/West Direction

Seismic

Seismic loads were calculated referencing ASCE 7-05 and flowchart 6.8. These values are detailed in table 8. It was assumed that the structure is rigid. The total building weight of Tower 'B' used to calculate seismic loads is detailed in table 2. Refer to Appendix A for a list of values and calculations. Table 9 shows a comparison of the lateral forces.

Seismic Design Story Shear					
Level	Height	Wx (Kips)	wxhx^k	Fx (Kips)	Vx (Kips)
Roof	0	821.30	129910	66.31	-
7	10	1463.24	197681	100.91	66.31
6	9.33	1463.24	166845	85.17	167.22
5	9.33	1463.24	136811	69.84	252.39
4	9.33	1463.24	126299	64.47	322.22
3	9.33	1463.24	79548	40.61	386.69
2	9.33	1463.24	52787	26.95	427.30
1	10	1492.79	26659	13.61	454.24
Ground	12	1516.21	0	0.00	467.85
					Base Shear (kips) = 467.85

Table 8: Seismic Design Story Shear Forces

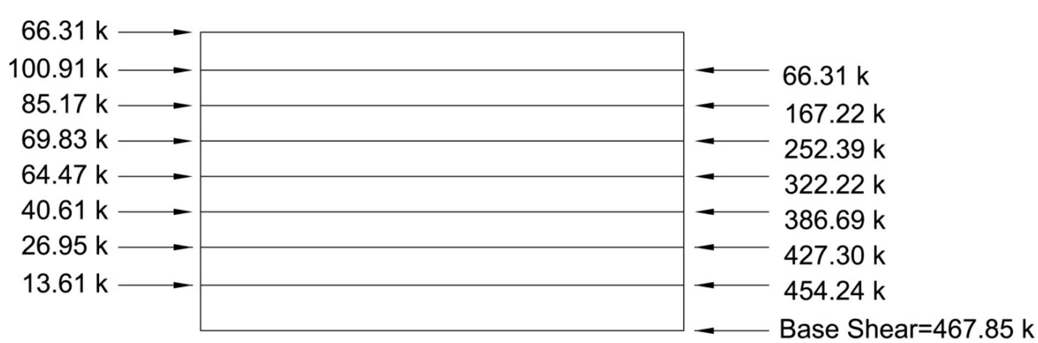


Figure 15: Diagram of Seismic Design Story Shear Forces

Worst Case Lateral Loads (Base Shear)	
Wind North/South	201.36 k
Wind East/West	269.40 k
Seismic	467.85 k

Table 9: Worst Case Base Shears

Appendix A - Calculations

Wind Calculations

Wind Calculations

$V = 90$ mph

$K_d = 0.85$

$I = 1.0$

exposure Category B

$K_{zt} = 1.0$

q_z → excel sheet $\quad q_z = 0.00256\, K_z K_{zt} K_d V^2 I$

q_h → excel sheet $\quad q_h = 0.0256\, K_z K_{zt} K_d V^2 I$

Structure is rigid

$G = 0.85$

$P_p =$ GC_{pn} windward $= 24.396$

GC_{pn} Leeward $= -16.264$

EAST / WEST

$L/B = \frac{127.33}{160.67} = .792$

WW $C_p = 0.8$

LW $C_p = -0.5$

Side $C_p = -0.7$

North / south

$L/B = \frac{160.67}{127.33} = 1.262$

WW $C_p = 0.8$

LW $C_p = -0.5$

Side $C_p = -0.7$

$GC_{pi} = \pm\ 0.18$

Roof: $\frac{h}{L} = \frac{78.17}{160.67} = .48$ E/W $\quad C_p$ → refer to excel chart

$\frac{h}{L} = \frac{78.67}{127.37} = .67$ N/S $\quad C_p$ → refer to excel chart

Seismic Calculations

Seismic Calcs.

Seismic Importance factor 1.0

Seismic use group I

Seismic design Category B

Site class C

Occupancy Category II

Lat. 39.7773092

Long. -75.546922

S_s & S_1 from USGS Ground motion Parameter Calculator.

$S_s = 0.263$

$S_1 = 0.058$

$S_{m_s} = F_a S_s = (1.2)(.263) = 0.316$

$S_{m_1} = F_v S_1 = (1.7)(0.058) = 0.099$

$S_{D_s} = \frac{2}{3} S_{m_s} = \frac{2}{3}(.316) = .210$

$S_{D_1} = \frac{2}{3} S_{m_1} = \frac{2}{3}(0.099) = 0.066$

$T_a = C_t h_n^x$ $\qquad C_t = 0.016$

$h_n = 78.67$ $\qquad x = .9$

$T \approx T_a = 0.016\,(78.69)^{.9} = .813$ sec

$S_{D_1} \le 0.1 \therefore C_u = 1.7$ $\qquad C_u T_a = 1.7(.813) = 1.383$

$T_L = 6\text{sec} > T$

$C_s = \dfrac{S_{D_1}}{T\,\frac{R}{I}} \le \dfrac{S_{DS}}{\frac{R}{I}}$ $\qquad T = .813$, $R = 2$, $I = 1.0$

$C_s = \dfrac{0.066}{.813\left(\frac{2}{1.0}\right)} = 0.0406 \le \dfrac{.210}{\left(\frac{2}{1}\right)}\ .105$

Base shear V

$V = C_s W = 0.0406(11523.44) = 467.85$ kip $\qquad k = 1.16$

Refer to Excel

Snow Load Calculations

SNOW LOAD

Thermal factor 1.0 C_t
Importance factor 1.0 I
Ground snow load 25 psf
Flat roof snow load 20 psf

Exposure = 0.9 = C_e
slope factor
$C_s = 1.0$

$p_g = 25 psf$

$\gamma = 0.13\, p_g + 14 \le 30$ pcf

$0.13(25) + 14 = 17.25 \le 30$

$\gamma = 17.3$

Category II

$p_f = 0.7\,(C_e)(C_t)(I)(p_g) = 0.7(0.9)(1.0)(1.0)(25) = 15.75$

$p_{f_{min}} = 20$ psf $> 15.75 \therefore$ $p_f = 20$ psf $= p_s$

Excel Spreadsheet Wind Calculations

L/B=160.67/127.33=1.262

Pressure (psf) North/South

Level	Height Above Ground Z(ft)	Kz	qz (psf)	qh (psf)	Gcpi	Cp WW	Cp LW	Cp Side	G	Windward	Leeward	Sidewall	Width (ft)	Story Heigl	Area (sf)	Force (Pou	Force (Kips	Story Force	Overturning Moment (ft-k)
Top	78.67	0.922762	16.26424	16.26423	0.18	0.8	-0.4	-0.7	0.85	13.98724	-8.4574	-12.6048	127.33	5	636.65	14289.38	14.28938	28.42988	2236.578
	73	0.90325	15.92032	16.26423	0.18	0.8	-0.4	-0.7	0.85	13.75338	-8.4574	-12.6048	127.33	5	636.65	14140.49	14.14049		
7	68.67	0.887607	15.6446	16.26423	0.18	0.8	-0.4	-0.7	0.85	13.56589	-8.4574	-12.6048	127.33	4.67	594.6311	13095.73	13.09573	26.06545	1789.914
	64	0.869924	15.33294	16.26423	0.18	0.8	-0.4	-0.7	0.85	13.35396	-8.4574	-12.6048	127.33	4.67	594.6311	12969.71	12.96971		
6	59.33	0.851294	15.00458	16.26423	0.18	0.8	-0.4	-0.7	0.85	13.13067	-8.4574	-12.6048	127.33	4.67	594.6311	12836.94	12.83694	25.53373	1514.916
	54.67	0.831629	14.65796	16.26423	0.18	0.8	-0.4	-0.7	0.85	12.89498	-8.4574	-12.6048	127.33	4.67	594.6311	12696.79	12.69679		
5	50	0.810681	14.28874	16.26423	0.18	0.8	-0.4	-0.7	0.85	12.6439	-8.4574	-12.6048	127.33	4.67	594.6311	12547.49	12.54749	24.93537	1246.768
	45.33	0.788285	13.89399	16.26423	0.18	0.8	-0.4	-0.7	0.85	12.37547	-8.4574	-12.6048	127.33	4.67	594.6311	12387.88	12.38788		
4	40.67	0.764227	13.46997	16.26423	0.18	0.8	-0.4	-0.7	0.85	12.08714	-8.4574	-12.6048	127.33	4.67	594.6311	12216.42	12.21642	24.24631	986.0973
	36	0.738054	13.00864	16.26423	0.18	0.8	-0.4	-0.7	0.85	11.77343	-8.4574	-12.6048	127.33	4.67	594.6311	12029.88	12.02988		
3	31.33	0.709328	12.50234	16.26423	0.18	0.8	-0.4	-0.7	0.85	11.42915	-8.4574	-12.6048	127.33	4.67	594.6311	11825.16	11.82516	23.423	733.8425
	26.67	0.677431	11.94013	16.26423	0.18	0.8	-0.4	-0.7	0.85	11.04685	-8.4574	-12.6048	127.33	4.67	594.6311	11597.83	11.59783		
2	22	0.641179	11.30117	16.26423	0.18	0.8	-0.4	-0.7	0.85	10.61235	-8.4574	-12.6048	127.33	4.67	594.6311	11339.47	11.33947	23.13278	508.9211
	17	0.595644	10.49858	16.26423	0.18	0.8	-0.4	-0.7	0.85	10.0666	-8.4574	-12.6048	127.33	5	636.65	11793.31	11.79331		
1	12	0.57472	10.12978	16.26423	0.18	0.8	-0.4	-0.7	0.85	9.815812	-8.4574	-12.6048	127.33	5	636.65	11633.64	11.63364	25.59401	307.1281
	6	0.57472	10.12978	16.26423	0.18	0.8	-0.4	-0.7	0.85	9.815812	-8.4574	-12.6048	127.33	6	763.98	13960.37	13.96037		
Ground	0	0.57472	10.12978	16.26423	0.18	0.8	-0.4	-0.7	0.85	9.815812	-8.4574	-12.6048	127.33	0	0	0	0		
																		201.3605	9324.166

V=90
Kd=0.85
I=1.0
Exposure Category B
Kzt=1.0
alpha=7.0
Zg=1200
Kz=2.01(z/Zg)^(2/alpha)
n1=100/H=100/78.67=1.27 (rigid structure)
G=0.85
Pp: WW=24.39636 LW=-16.26424
Gcpi= +/- 0.18

Excel Spreadsheet wind Calculations

L/B=127.33/160.67=.792

Pressure (psf)East/West

Level	Height Above Ground Z(ft)	Kz	qz (psf)	qh (psf)	Gcpi	Cp WW	Cp LW	Cp Side	G	Windward	Leeward	Sidewall	Width	Story Heigl	Area (sf)	Force (Pou	Force (Kips	Story Force	Overturning Moment (ft-k)
Top	78.67	0.922762	16.26424	16.26423	0.18	0.8	-0.5	-0.7	0.85	13.98724	-9.83986	-12.6048	160.67	5	803.35	19141.5	19.1415	38.09514	2996.944
	73	0.90325	15.92032	16.26423	0.18	0.8	-0.5	-0.7	0.85	13.75338	-9.83986	-12.6048	160.67	5	803.35	18953.63	18.95363		
7	68.67	0.887607	15.6446	16.26423	0.18	0.8	-0.5	-0.7	0.85	13.56589	-9.83986	-12.6048	160.67	4.67	750.3289	17562.01	17.56201	34.96501	2401.047
	64	0.869924	15.33294	16.26423	0.18	0.8	-0.5	-0.7	0.85	13.35396	-9.83986	-12.6048	160.67	4.67	750.3289	17402.99	17.40299		
6	59.33	0.851294	15.00458	16.26423	0.18	0.8	-0.5	-0.7	0.85	13.13067	-9.83986	-12.6048	160.67	4.67	750.3289	17235.46	17.23546	34.29406	2034.667
	54.67	0.831629	14.65796	16.26423	0.18	0.8	-0.5	-0.7	0.85	12.89498	-9.83986	-12.6048	160.67	4.67	750.3289	17058.61	17.05861		
5	50	0.810681	14.28874	16.26423	0.18	0.8	-0.5	-0.7	0.85	12.6439	-9.83986	-12.6048	160.67	4.67	750.3289	16870.22	16.87022	33.53903	1676.951
	45.33	0.788285	13.89399	16.26423	0.18	0.8	-0.5	-0.7	0.85	12.37547	-9.83986	-12.6048	160.67	4.67	750.3289	16668.81	16.66881		
4	40.67	0.764227	13.46997	16.26423	0.18	0.8	-0.5	-0.7	0.85	12.08714	-9.83986	-12.6048	160.67	4.67	750.3289	16452.46	16.45246	32.66954	1328.67
	36	0.738054	13.00864	16.26423	0.18	0.8	-0.5	-0.7	0.85	11.77343	-9.83986	-12.6048	160.67	4.67	750.3289	16217.08	16.21708		
3	31.33	0.709328	12.50234	16.26423	0.18	0.8	-0.5	-0.7	0.85	11.42915	-9.83986	-12.6048	160.67	4.67	750.3289	15958.75	15.95875	31.63066	990.9885
	26.67	0.677431	11.94013	16.26423	0.18	0.8	-0.5	-0.7	0.85	11.04685	-9.83986	-12.6048	160.67	4.67	750.3289	15671.9	15.6719		
2	22	0.641179	11.30117	16.26423	0.18	0.8	-0.5	-0.7	0.85	10.61235	-9.83986	-12.6048	160.67	4.67	750.3289	15345.89	15.34589	31.33774	689.4304
	17	0.595644	10.49858	16.26423	0.18	0.8	-0.5	-0.7	0.85	10.0666	-9.83986	-12.6048	160.67	5	803.35	15991.86	15.99186		
1	12	0.539222	9.504109	16.26423	0.18	0.8	-0.5	-0.7	0.85	9.390356	-9.83986	-12.6048	160.67	5	803.35	15448.6	15.4486	32.86755	394.4106
	6	0.442343	7.796556	16.26423	0.18	0.8	-0.5	-0.7	0.85	8.22922	-9.83986	-12.6048	160.67	6	964.02	17418.96	17.41896		
Ground	0	0	0	16.26423	0.18	0.8	-0.5	-0.7	0.85	2.927562	-9.83986	-12.6048	160.67	0	0	0	0		
																		269.3987	12513.11

Roof N/S

Horiz Dist. From	Effective Width		Resultant F	Cp Roof	qh	G	Gcpi	Pressure (p	Force
0 to 39.335	56.67	39.335	19.67	-0.9	16.26423	0.85	0.18	-15.3697	34.26082
39.335 to 78.67	56.67	39.335	59.0025	-0.9	16.26423	0.85	0.18	-15.3697	34.26082
78.67 to 157.34	56.67	78.67	118.005	-0.5	16.26423	0.85	0.18	-9.83986	43.86835
157.34 to 160.6	56.67	3.32	158.97	-0.3	16.26423	0.85	0.18	-7.07494	1.331111

Roof E/W

Horiz Dist. From	Effective Width		Resultant F	Cp Roof	qh	G	Gcpi	Pressure (p	Force
0 to 39.335	77.33	39.335	19.6675	-1	16.26423	0.85	0.18	-16.7522	50.95631
39.335 to 127.3	77.33	87.998	83.334	-0.7	16.26423	0.85	0.18	-12.6048	85.77409

Excel Spreadsheet Seismic Calculations

Level		Height	Floor System (Kips)	Load Bearing CMU Walls (psf)	MEP	Ceiling Finish	Partitions	Floor Finish	Misc. (Storage LL for Seismic)	Wx (Kips)
Roof		0	795.33125	0	0	0	0	25.97	0	821.30125
	7	10	795.33125	411.453	129.85	9.73875	103.88	12.985	0	1463.238
	6	9.33	795.33125	411.453	129.85	9.73875	103.88	12.985	0	1463.238
	5	9.33	795.33125	411.453	129.85	9.73875	103.88	12.985	0	1463.238
	4	9.33	795.33125	411.453	129.85	9.73875	103.88	12.985	0	1463.238
	3	9.33	795.33125	411.453	129.85	9.73875	103.88	12.985	0	1463.238
	2	9.33	795.33125	411.453	129.85	9.73875	103.88	12.985	0	1463.238
	1	10	795.33125	441	129.85	9.73875	103.88	12.985	0	1492.785
Ground		12	649.25	529.2	129.85	9.73875	103.88	12.985	81.31	1516.21375
									Total Load (Kips)	12609.728

	Load (psf)	Total Load (Kips)
Hollow Core Concrete Plank	61.25	6362.65
4 inch Slab On Grade	50	649.25
Load Bearing CMU Walls	42	3438.918
MEP	10	1038.8
Ceiling Finish	0.75	77.91
Partitions	8	831.04
Floor Finish	1	103.88
EPDM Roof Assembly	2	25.97
Misc. (Storage LL for Seismic)		81.31
Total Dead Load (W), (Kips)		12609.728

Level		hx	Wx	wxhx^k	Cvx	V	Fx (Kips)	Vx (Kips)
Roof		78.67	821.30125	129909.5559	0.14174	467.851664	66.31296	-
	7	68.67	1463.238	197680.9166	0.21568	467.851664	100.9072	66.31295601
	6	59.33	1463.238	166845.0073	0.18204	467.851664	85.16683	167.2201282
	5	50	1463.238	136810.735	0.14927	467.851664	69.8357	252.3869626
	4	46.67	1463.238	126298.6825	0.1378	467.851664	64.46977	322.2226586
	3	31.33	1463.238	79547.9568	0.08679	467.851664	40.60564	386.692427
	2	22	1463.238	52786.79338	0.05759	467.851664	26.94527	427.2980632
	1	12	1492.785	26659.22425	0.02909	467.851664	13.60833	454.2433351
Ground		0	1516.21375	0	0	467.851664	0	467.851664
				916538.8717			467.8517	

Appendix B – Spot Checks

Load Bearing Masonry Wall Spot Check

Load Bearing Masonry Wall spot check (exterior wall 4th floor)

spans 38'-8"

DL = CMU = 42 psf => $[(9.33)(2)+10](42) = 1203.72$ lb/ft

plank = 61.25 psf = $\frac{28}{2}(61.25)(4) = 3430$ lb/ft

Ceiling, MEP, Partitions, floor = $\frac{28}{2}(.75+8+1+10)(4) = 1106$ lb/ft

Roof = $\frac{28}{2}(2) = 28$ lb/ft

LL = $[40 \text{ psf} \times 3 \text{ floors} + 20 \text{ psf roof}]\frac{28}{2} = 1960$ lb/ft

$w = 13.64$ psf

$y = H/2 = 4.67$

assume $e = 3/4$ inch

$M_{wind} = \frac{13.64(10)^2}{8} = 170.5$ ft-lb/ft

$M_{PD} = P_D e(y/H) = [1203.72 + 3430 + 1106 + 28](.75)(4.67/9.33) = 1716.6$

$M_{PL} = P_L e(y/H) = 1960(.75)(\frac{4.67}{9.33}) = 735.79$

$M_{max} = .75(170.5 + 1716.6 + 735.79) = 1967.2$ ft-lb/ft

Walls are reinforced with No. 4 @ 32" o.c.

According to an extrapolation of an interaction diagram from NCMA TEK 14-5A (2006), the load bearing wall system is adequate.

Precast Hollow Core Concrete Plank Spot Check

Precast Hollow core Plank Spot check

Span = 28.33 ft   8"x4' (typ)

LL = 40 psf
DL = 10 MEP (psf)
.75 ceiling (psf)
1 floor finish (psf)
8 partitions (psf)
total = 19.75 psf

$1.2D + 1.6L = 1.2(19.75) + 1.6(40) = 87.7$

according to Nitterhouse Concrete Products description an 8"x4' Hollow core plank can be safely loaded to 107 psf for a span of 29 feet, when using this load combination. Therefore plank are OK.

Masonry Shear Wall Spot Check

Masonry Shear wall spot check

Wall on 7th Level

One of Two shear walls on 7th Level => assume it will take 50% of shear from that level.

worst case shear caused by seismic 66.31^{K}

$V = .5(66.31) = 33.2^{K}$

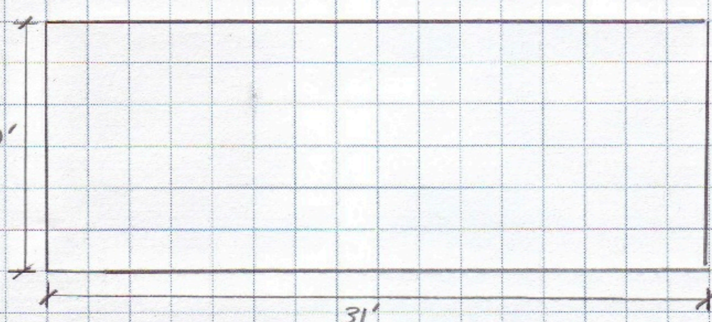
8" CMU

Assume $f'_m = 1350$ psi for ASTM C90

height = 10'

length = 31'

reinforcement => #4 @ 48" oc.



$$f_v = \frac{V}{t d_v} = \frac{33200}{7.62 \times 31 \times 12} = 11.71 \text{ psi}$$

$$\frac{M}{V d_v} = \frac{h}{2 d_v} = \frac{10' \times 12''}{2 \times 31 \times 12} = 0.161 < 1.0 \therefore$$

$$F_{vm} = \frac{1}{3}\left[4 - \frac{M}{Vd}\right]\sqrt{f'_m} = \frac{1}{3}\left[4 - 0.161\right](\sqrt{1350}) = 47.02 \text{ psi}$$

$$\leq \left(120 - \frac{45M}{Vd}\right) = \left(120 - \frac{45(10)(12)}{2 \times 31 \times 12}\right) = 112.74 \text{ psi} \quad \therefore \text{OK} \checkmark$$

Appendix C – Additional Photographs


18 13:20


STORAGE
18 13:16


10 13:07


10 13:07