HOWARD COUNTY GENERAL HOSPITAL

PATIENT TOWER ADDITION

COLUMBIA, MD

Technical Assignment 1:
Structural Concepts and Existing Conditions Report

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October 5, 2007

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I. EXECUTIVE SUMMARY

This first technical report explores the structural design for the new patient tower addition to the Howard County General Hospital. It helped me to better understand the building and it's various structural systems. A general description of the addition's size and function is included in the Introduction portion of this report.

The body of this report includes both written descriptions and hand calculations that describe and explore the structural makeup of the building. The Structural Overview section explains the typical floor, roof, foundation, and lateral systems used in the patient tower addition. The Codes and Material Properties used for design as well as those I've used for analysis are listed in the next section of the report. The Loads section of this report identifies the live loads, dead loads, snow loads, wind loads, and seismic loads from which the building was designed. This portion of the report includes several loading diagrams for further explanation of the various loading cases. Spot checks of the typical framing elements were performed using these loads and then compared to the structural engineer's design to check the members' adequacy. Finally, an appendix is included at the end that consists of various calculations and spreadsheets from which the other sections' results were obtained.

It was found that the designer's member selections were appropriate for the explored loads. There were a few variations in my results from those of the existing design. These are in most cases explained by differing assumptions, my simplification of the design procedures, or the use of different code editions. Regardless, the discrepancies are minimal and seem to be reasonable. Given this preliminary analysis, I believe that this report verifies the structural engineer's existing design.

II. INTRODUCTION TO BUILDING

Howard County Hospital is a member of Johns Hopkins Medicine located in Columbia, MD. It has been serving the surrounding community for over thirty years and grown significantly in the last decade. The most recent expansion, the 114,261 square foot patient tower, began construction in April 2007. This tower consists of one level partially below grade, four levels above grade, and a generously sized penthouse. The basement level consists mainly of offices for the hospital staff, storage areas, and mechanical/electrical rooms. The first floor is made up of a large gym along with cardio pulmonary and physical therapy areas. Patient rooms comprise the upper three levels, with each of the three floors providing thirty new beds for surgical or other medical patients.

The patient tower addition is part of a larger allover expansion known as the "Campus Development Plan." It is located on the south west side of the existing south building, close to Cedar Lane. Currently, the site consists of asphalt paved driveways and parking areas as well as a small landscape area. The topography gently slopes towards the west with an overall change in elevation of about 12 feet. The façade was selected to be horizontal bands of precast concrete, glass, and aluminum panels, similar to the existing hospital's exterior.

This expansion of the hospital was designed with large column bays and a 100 psf live load for flexibility in case of future renovations. Other portions of the hospital are currently undergoing renovations, demonstrating that designing for flexibility is a legitimate issue as the hospital grows and changes. This need for flexibility also contributed to the selection of moment frames as opposed to braced frames or another lateral system.

This report begins to explain the designed structural system and verify that the selected members are capable of carrying the loads. Some simplified analysis and spot checks were used along with some more in-depth detailed analysis.

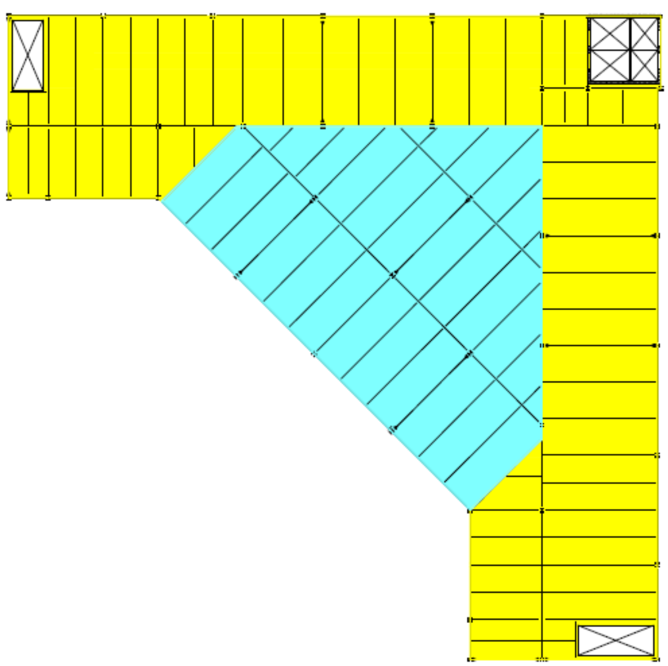
III. STRUCTURAL OVERVIEW

Floor System:

The typical floor framing system is 3 ¼" lightweight concrete on 2" deep 18 gage composite metal deck for a total depth 5 ¼". Composite action is achieved with ¾" diameter by 4" shear studs evenly spaced along the length of supported beams. This total floor system attains a fire rating of two hours, according to the United Steel Deck catalog. There are three typical infill beam sizes – W12x19, W14x22, and W16x26. These beams vary from 19 feet to 30 ½ feet in length and are usually spaced at 7'-3" or 9'-8". In addition to the standard composite slab, additional reinforcing of 5 foot long #4 bars are specified at 16" on center over all interior girders.

The first floor has a small 1-story extension on the north side of the building that connects to the existing hospital. This area is framed with W10x12 and W14x22 infill beams. The composite slab in this area is the same 5 ¼" thickness as the main addition.

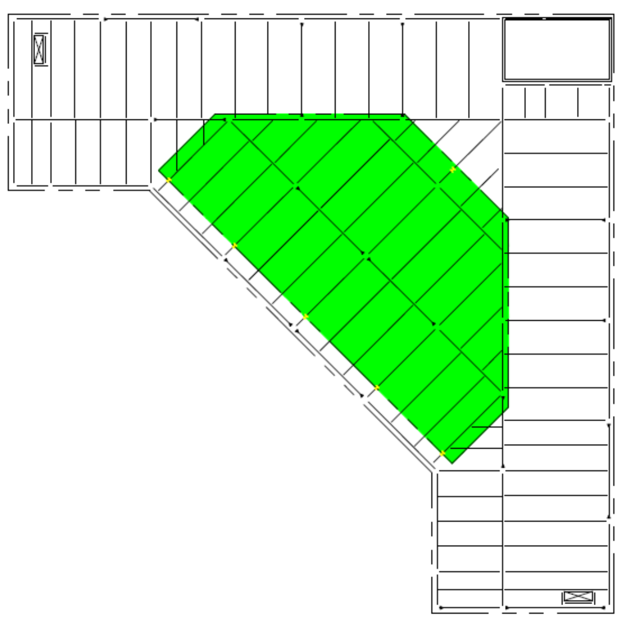
The new addition is a uniquely shaped structure, so the floors are framed in two different directions. As you can see in the figure below, the "center" floor framing (shown in blue) is rotated at a 45 degree angle from the framing along the outer "L" of the building (shown in yellow).



The second, third, and fourth floors required 2" depressed slabs in the patient rooms for prefabricated "stall-less" showers. The depressions are framed out with W12x19 beams in each of the thirty patient rooms on each of the three floors. This irregularity in the floor system resulted in additional members and some increased beam sizes from the typical framing.

Roof System:

The main roof is also a composite system since a considerable portion of it is occupied for the mechanical penthouse floor. This roof/floor system is composed of the same 3 ¼" lightweight concrete on 2" metal deck as the typical floors are. Infill beam sizes and lengths are similar to those mentioned above in the typical floor system. Transfer girders are also required at this level for 6 new columns that extend from the roof/penthouse floor up to the penthouse roof. You can see the portion of this level that is roof, shown in white below, and the portion that is penthouse, shown in green below.



The penthouse roof is the only floor system that varies from the typical system as it is 1 ½" wide rib 20 gage metal roof deck. The infill beams are typically either 24'-9" long W10x19s or 35'-4" long W16x36s. The framing at the penthouse roof is at a forty-five degree angle, the same direction as that in the "center" framing area of the typical floors.

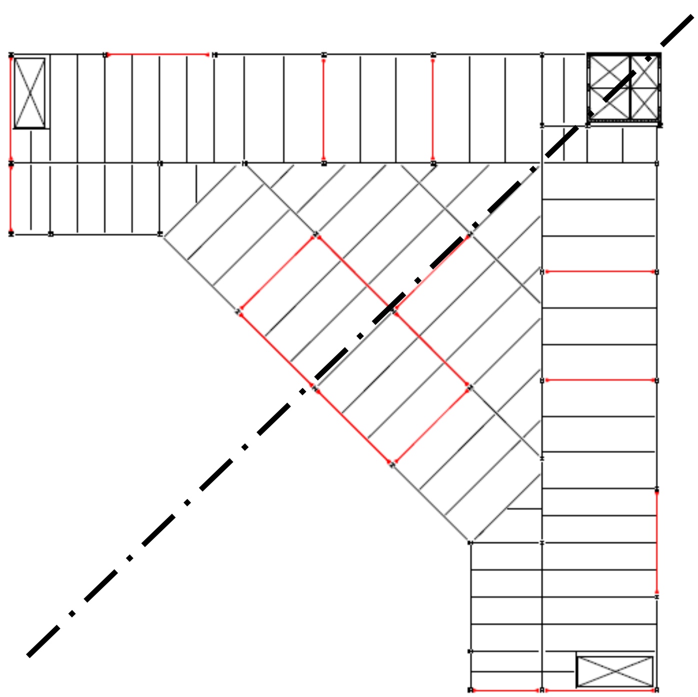
Exterior:

The exterior of the building is typically precast, metal and glass panels. The precast panels are 8" thick. At the first floor on the east side of the building, a curtain wall system is used similar to the curtain wall used on the existing hospital. The only variation to the precast, metal, and glass striping pattern is that the 39.5' true south and true north walls are made up of almost exclusively precast with a few punched out windows.

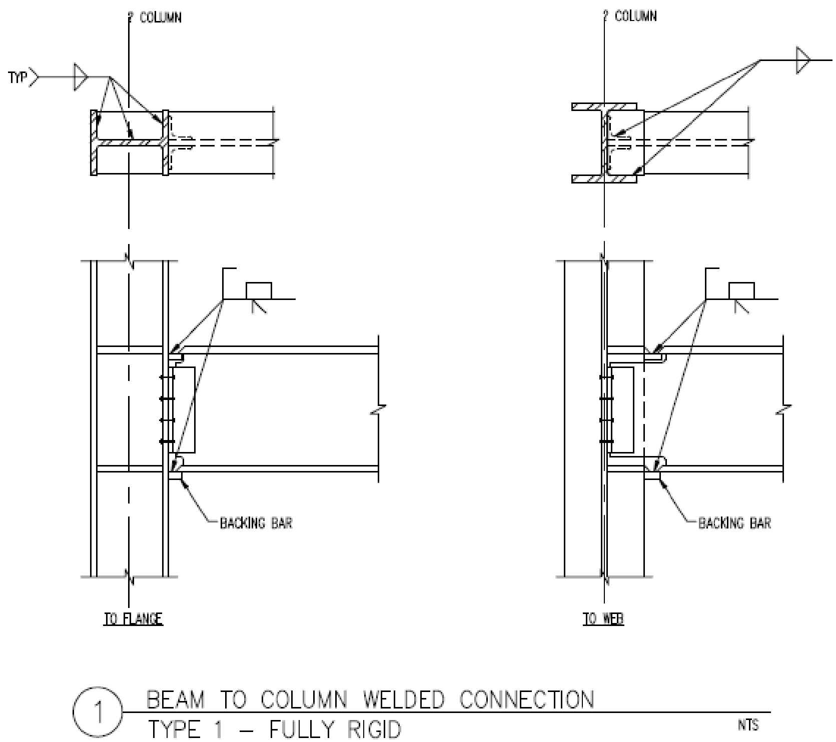
The walls that extend from the penthouse floor to the penthouse roof are composed of 6" metal studs at 16" on center with insulation. These walls have an exterior finish of "dryvit" on them for protection and aesthetics.

Lateral Load Resisting System:

Steel moment frames were used at each level to resist lateral loads. Each floor contains 19 moment frames, 8 of which are along the perimeter of the building and 11 are interior beams. The moment frames are symmetrical about the same diagonal axis that the building is. These lateral force-resisting beams are highlighted in red in the diagram below with the axis of symmetry shown as a dashed line.



At each of these moment frames, end beam reactions are called out on the plans for which the moment connections can be designed. According to the detail a double angle connection is used to connect the beam web to the column with the angle welded to the beam and bolted to the column. Stiffener plates are then added to the column at the same elevation and thickness as the beam flanges. Backing bars are then welded to connect the beam flanges to the face of the column or the column stiffeners, depending on the orientation of the column. The following diagram is the detail provided for the moment connections.



Moment frames were used to allow for floor plan flexibility. With the hospital constantly growing and the changing demands various branches (i.e. surgery, physical therapy, rehabilitation, etc.), the space initially designed for patient rooms could have an alternate use sometime down the road. If trusses or braced frames were used, the location of these braces would reduce the flexibility of the space.

Foundation System:

Five soil test borings were taken at the site of the new patient tower. They were drilled to a depth of about 30 feet each according to ASTM D 1586 standards. It was found that the top layer of soil was fill soil consisting of sand and silt, but the basement floor elevation should generally fall below this layer of soil. Therefore, a new allowable bearing pressure of 6,000 psf was used to design the foundations.

The footing sizes of the main addition vary from 8 foot by 8 foot to 11 foot by 11 foot square footings along with a few rectangular footings. Smaller 4 and 5 foot square footings occur at columns located in the one-story extension to the north of the main tower. Along the north wall of the building, there is an existing retaining wall footing. This footing is to be field verified and any portions that interfere with the new footings are to be removed.

A 14" thick concrete foundation wall surrounds that building at the basement level. The wall is reinforced with #4 bars at 12" vertical on each face and #5 bars at 12" horizontal. Concrete piers protrude from the wall at the location of exterior columns from which steel columns extend from the first floor up.

The slab on grade is 5" thick reinforced with 6x6" WWF on a vapor retarder over a minimum 4" layer of clean, well graded gravel or crushed stone. There is a small area, approximately 20 by 40 feet, where the top of slab elevation is depressed one foot.

The photo below shows the excavation for the basement, which is partially above grade. The soldier piles and wood lagging that were installed can be seen as well.

IV. CODES AND MATERIAL PROPERTIES

Codes and Standards:

Rathgeber & Goss Associates designed the Howard County General Hospital patient tower, which began design in 2004, according to the 2000 International Building Code and ASCE 7-98. Concrete design specifically references ACI 318-98 while steel design followed the AISC Load and Resistance Factor Design, Third Edition 2001.

My report will utilize the more recent versions of the building codes, the 2006 International Building Code, which references ASCE 7-05. For concrete analysis and design, I will be using ACI 318-05 and for steel design, I will be using the Load and Resistance Factor Design portion of the AISC Thirteenth Edition Steel Manual.

Material Strengths:

Concrete

Application	f'c @ 28 days	Weight (pcf)
Slabs-on-grade	3000 psi	145
Fill on Metal Deck	3500 psi	110
Footings	3000 psi	145
Precast Units	5000 psi	145
Piers	4000 psi	145

Steel

Materials	Fy (ksi)
Wide-Flange Shapes	50
Channels, Angles, and Plates	36
Structural Pipe	35
Round HSS Shapes	42
Square/Rectangular HSS Shapes	46
Reinforcing Steel	60

V. LOADS

Dead Loads:

The majority of the floor dead load is composed of the composite slab on metal deck system. This load was found in the deck catalog for 20 gage deck with normal weight concrete and a total slab depth of 5 ¼”, as specified on the plans. Other loads, such as MEP dead load, were assumed based on accepted practice values.

The exterior dead load at the building perimeter is mainly the precast panel dead load listed below. The only exception is on the east side of the tower at the first floor, were the curtain wall system is present. A 10 psf dead load was assumed for the glass and aluminum panels.

The roof dead load is not included in the table below, but is estimated to be 15 psf for deck, framing, and other miscellaneous roofing materials. This occurs at the main roof where the penthouse is not located, as well as the penthouse roof.

Floor Material Dead Loads

Material	Load
5 1/4" Composite Deck/Slab	41 psf
Framing	7 psf
MEP	10 psf
Miscellaneous	7 psf
Total	65 psf

Exterior Wall Dead Loads

Precast Panels (8” thick)..........0.10 ksf
150 pcf*(8”/12) = 100 psf = 0.10 ksf
Glass/Aluminum..........0.01 ksf
Curtain Wall (18’ tall)..........0.36 klf
Metal Stud Wall @ 16” oc..........0.015 klf

Live Loads:

Most of the design live loads were included on the structural general notes and were verified with the newer code, ASCE 7-05. Any live loads not listed in the structural general notes were taken from chapter 4 of ASCE 7-05. A live load of 100 was used for the hospital, though not required, for future flexibility reasons.

Location	Load	Comments
Framed Floor Areas	100 psf	80LL + 20 for Partitions
Lobbies/Stairs	100 psf	
Storage	125 psf	Unreducible
Penthouse	125 psf	Unreducible
Roof	30 psf	Unreducible

Snow Load:

Snow load is not typically greater than the 30 psf roof live load in the Mid-Atlantic area where the hospital is located. In this case, the ground snow load is 25 psf while the calculated flat roof snow load is 22 psf.

There are a few locations in which snow drift must be considered from higher roofs. There is a small projection on the north side of the tower addition where snow drift will occur. Also, portions of the main roof could experience snow drift from the higher penthouse roof and elevator area. Refer to Appendix D for these special case snow drift diagrams and calculations. It was determined that leeward snow drift controlled for all three of the drift conditions

Ground Snow Load (P_g)........25 psf
Snow Exposure Factor (C_e)........1.0
Importance Factor (I_s)........1.1
Thermal Factor (C_t)........1.0
Flat Roof Snow Load (P_f)........19.25 psf

The flat roof snow load is less than the code minimum of

$P_{f,\,min}$ = I*20 psf = 1.1*20 psf = 22 psf

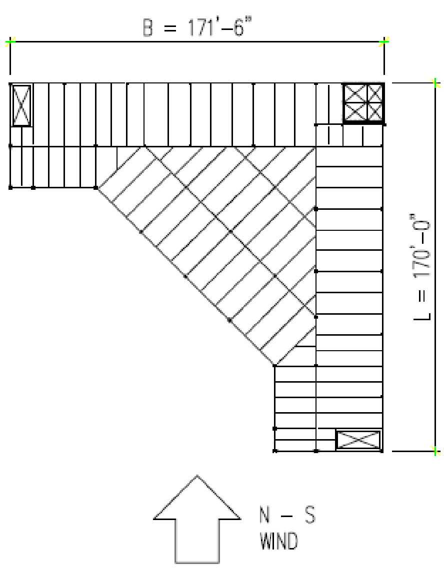
Therefore, the minimum flat roof snow load of 22 psf will be used. This will be added to the drift snow load where applicable. Otherwise, it is less than the 30 psf roof load, as expected, so the roof live load will control.

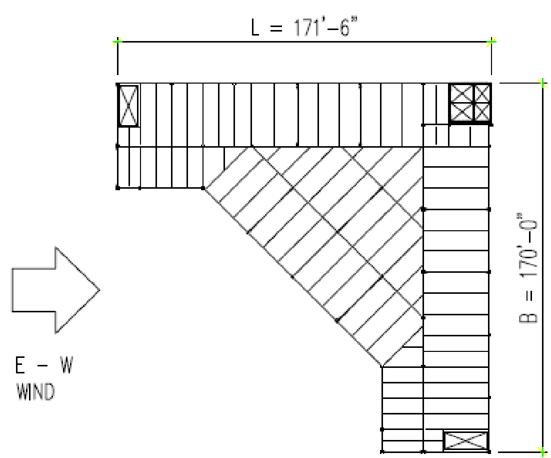
Wind Loads:

Wind loads were determined in accordance with ASCE 7-05 and with the assumptions listed below. The building is enclosed and cannot use the simplified design procedure outline in ASCE 7-05 because the mean roof height is over 60 feet. Therefore, the more extensive analytical procedure must be used. Below are the factors that are constant for both the North-South wind direction and the East-West wind direction.

Basic Wind Speed (V)	90 mph
Importance Factor (I)	1.15
Wind Directionality Factor (K_d)	0.85
Exposure Category	B
Topographic Factor (K_{zt})	1.0
Enclosure Classification	Enclosed
Internal Pressure Coefficient (GC_{pi})	+/- 0.18

Below you can see the direction of wind based on building orientation.





Calculations were performed in both wind directions shown above, however very few factors differed since the projected length and width of the building are almost equal. Below are the calculated wind pressures for each wind direction.

NORTH – SOUTH WIND

Calculate Pressures using Eqn						
C_p =	0.8	Figure 6-6	(Windward)			
C_p =	-0.5	Figure 6-6	(Leeward)			
C_p =	-0.7	Figure 6-6	(Sidewall)			
G_f =	0.787					
z (ft)	**K_z**	**q_z**	**P (leeward)**	**P (windward)**	**P (sidewall)**	**+/- qGC_{pi}**
0-18	0.605	12.272	-7.613	7.728	-10.659	+/- 3.482
36	0.738	14.960	-7.613	9.421	-10.659	+/- 3.482
54	0.829	16.797	-7.613	10.578	-10.659	+/- 3.482
70.5	0.894	18.127	-7.613	11.415	-10.659	+/- 3.482
88.5	0.954	19.344	-7.613	12.181	-10.659	+/- 3.482
	q_h =	19.344				

Calculate Roof Pressure				
H/L =	0.521	requires interpolation		**P (roof)**
C_p =	-0.917	Figure 6-6	0 to 44.25 ft	-13.963
C_p =	-0.892	Figure 6-6	44.25 to 88.5 ft	-13.582
C_p =	-0.508	Figure 6-6	88.5 to 170 ft	-7.735

EAST – WEST WIND

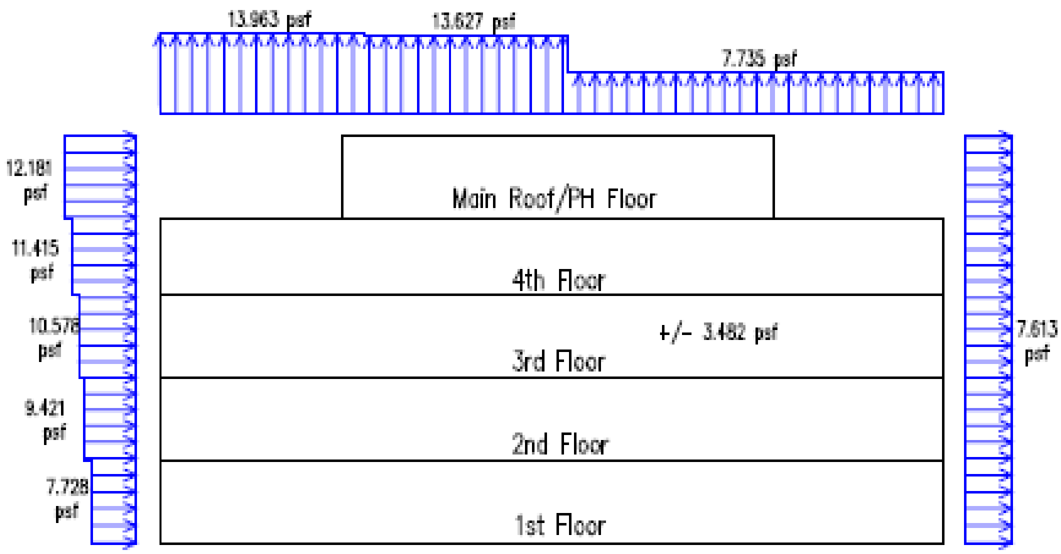
Calculate Pressures using Eqn						
C_p =	0.8	Figure 6-6	(Windward)			
C_p =	-0.5	Figure 6-6	(Leeward)			
C_p =	-0.7	Figure 6-6	(Sidewall)			
G_f =	0.788					
z (ft)	**K_z**	**q_z**	**P (leeward)**	**P (windward)**	**P (sidewall)**	**+/- qGC_{pi}**
0-18	0.605	12.272	-7.621	7.736	-10.670	+/- 3.482
36	0.738	14.960	-7.621	9.431	-10.670	+/- 3.482
54	0.829	16.797	-7.621	10.589	-10.670	+/- 3.482
70.5	0.894	18.127	-7.621	11.427	-10.670	+/- 3.482
88.5	0.954	19.344	-7.621	12.194	-10.670	+/- 3.482
	q_h =	19.344				

Calculate Roof Pressure				
H/L =	0.516	requires interpolation		**P (roof)**
C_p =	-0.913	Figure 6-6	0 to 44.25 ft	-13.917
C_p =	-0.894	Figure 6-6	44.25 to 88.5 ft	-13.627
C_p =	-0.506	Figure 6-6	88.5 to 171.5 ft	-7.713

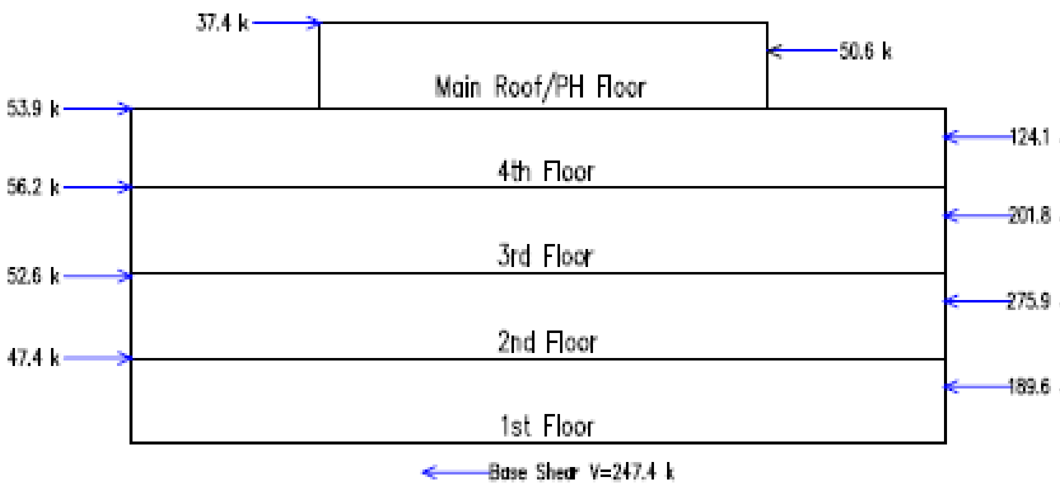
You can see that the pressures for the wind loading in the two directions are very similar, as expected, because of the building's projected dimensions. For complete calculations and references for all variables used, see Appendix B.

Below are the loading diagrams showing wind pressures, story forces, and story shears at each level. Internal pressures are shown on the pressure diagrams but excluded in the story forces and shear diagrams. Refer to Appendix B for the spreadsheet conversion from pressures to story forces and story shears shown in the diagrams.

NORTH – SOUTH WIND DIAGRAMS

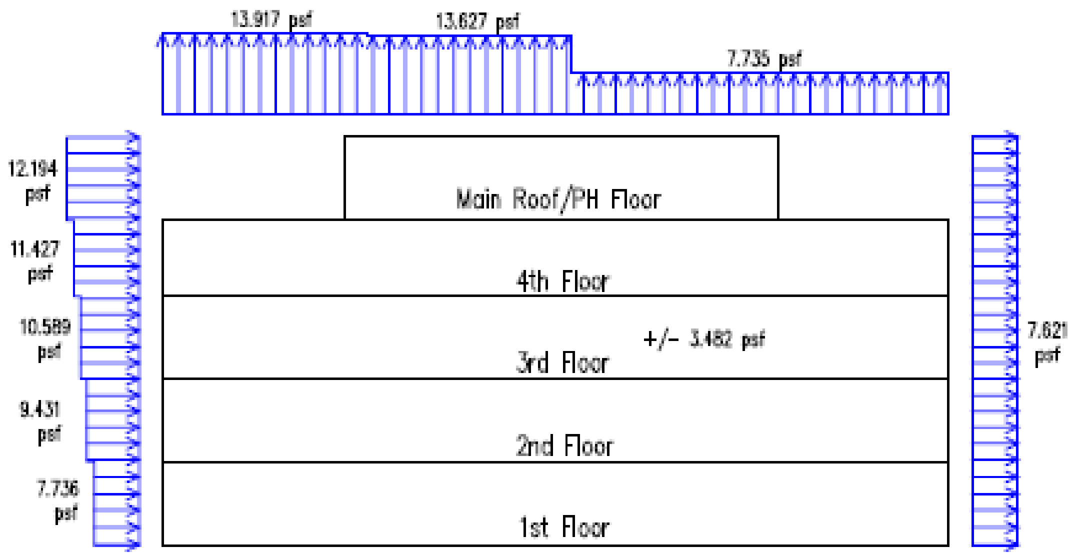


NORTH–SOUTH WIND

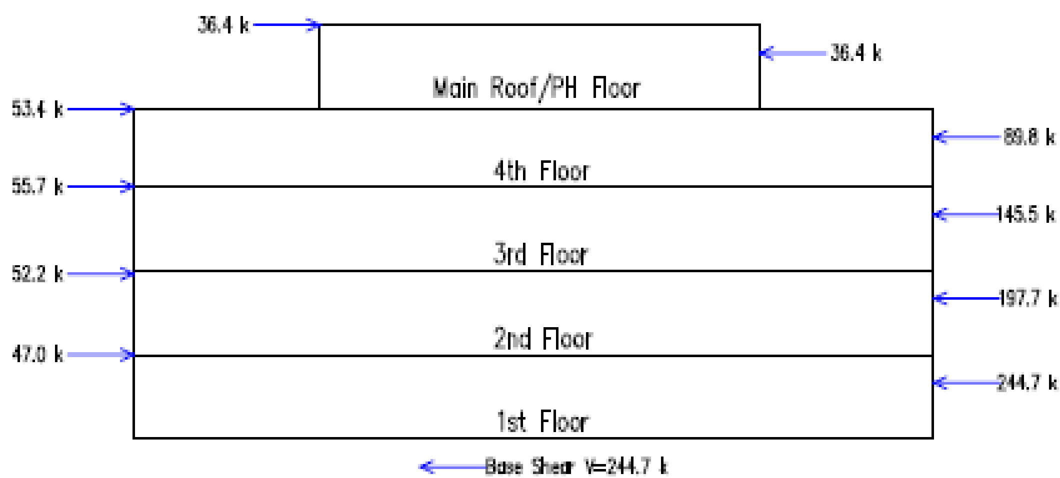


NORTH–SOUTH WIND

EAST – WEST WIND



EAST–WEST WIND



EAST–WEST WIND

From these values, you can see that the North-South base shear of V= 247.4 k controls when comparing these two wind loadings.

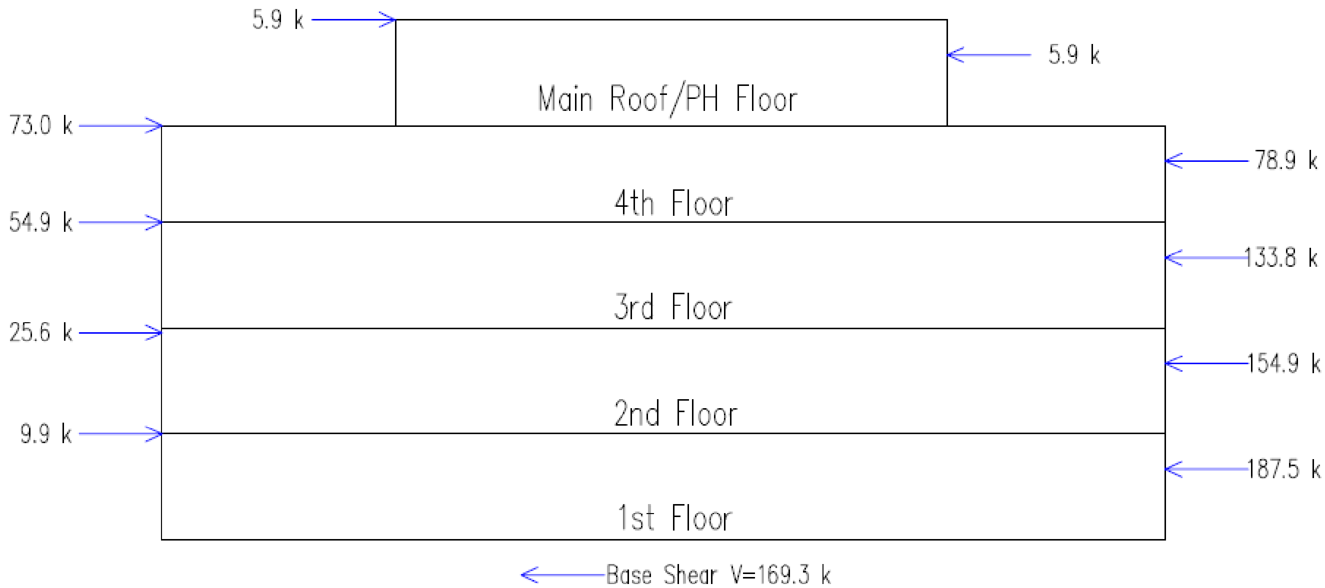
Seismic Loads:

A seismic analysis of the building was performed to determine the total base shear as well as the shear distribution at each floor. The spectral response accelerations S_S and S_1 were obtained from the United States Government Seismic Design Values for Buildings (http://earthquake.usgs.gov/research/hazmaps/design) using the latitude and longitude of the Howard County General Hospital. The seismic loads were calculated using the equivalent lateral force method in accordance with ASCE 7-05. To determine the response coefficient, the seismic force system used was "Steel Systems Not Specifically Detailed for Seismic Resistance" as specified in the structural general notes.

A few important assumptions and/or decisions should be noted. The building is classified as Seismic Use Group III rather than IV because no surgery facilities are located within the new tower addition. This results in the importance factor of 1.25 rather than 1.5, which is what the designer used as well. Also, the total above grade height was considered to be 88.5 feet, which includes the penthouse, but does not take into consideration the basement which is partially above grade on some sides. This assumption was made to simplify the procedure and it results in a smaller period and therefore larger base shear, so it is conservative. Finally, the weight of the building was calculated excluding the slab depression for the shower stalls for the sake of simplicity.

Mapped Spectral Response Accelerations	S_S = 0.160 g
	S_1 = 0.050 g
Site Class	D
Seismic Use Group	III
Importance Factor (I)	1.25
Site Class Factors	Fa = 1.6
	Fv = 2.4
Adjusted Spectral Response Accelerations	S_{MS} = 0.256
	S_{M1} = 0.12
Design Spectral Response Accelerations	S_{DS} = 0.171
	S_{D1} = 0.08
Seismic Design Category	B
Response Modification Coefficient (R)	3.0
Approximate Fundamental Period (T_a)	1.011
Fundamental Period (T)	1.719
Seismic Response Coefficient (C_s)	0.0194
Effective Seismic Weight (W)	8637.2 k

Using these values and the interpolated value of k = 1.61, and the application of the vertical distribution of seismic forces, the following loads can be obtained:



SEISMIC LOADING

From these story loads, a total overturning moment of M = 9734.9 ft-k can be obtained. You can see the base shear due to seismic loading is V=169.3 k which is less than the wind load base shear, therefore wind controls.

Lateral Load Distribution:

As specified above, wind load controls over seismic in both the North-South and East-West direction. There are 19 moment frames at each floor, 6 in the North-South direction, 6 in the East-West direction, and 7 at a 45 degree angle. The lateral elements are symmetrical about the building axis and seem to be fairly evenly spaced throughout the building, so it is reasonable to assume a simplified distribution of forces. I am assuming that each of the moment frames at a 45 degree angle will take half the amount of force that the moment frames in the direction of analysis will take. Therefore, in the North-South direction, I am assuming the load gets distributed between a “theoretical” 9 moment frames:

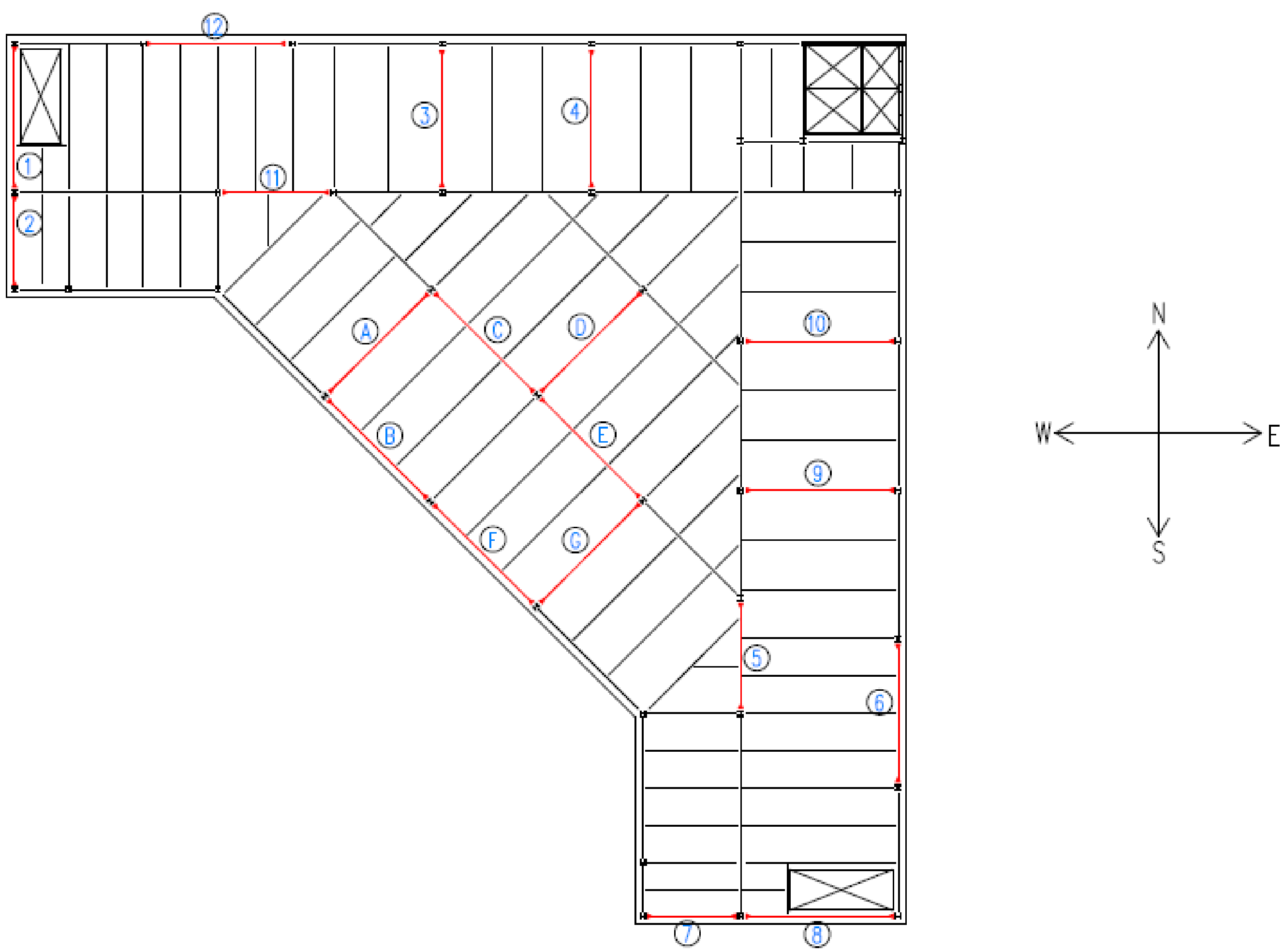
6 N-S frames + ½*7 45 degree frames = 9.5 frames total

And in the East-West direction, I am assuming that the load gets distributed between a "theoretical" 10 moment frames:

6 E-W frames + ½*7 45 degree frames = 9.5 frames total

Also, I am assuming that the exterior moment frames will take more load than the interior moment frames, so I am applying a factor of 1.2 to the exterior moment frames' distribution factors.

For simplicity, I numbered the frames in the North-South direction 1-6 and the frames in the East-West direction 7-12 while the frames at a 45 degree angle will get assigned a letter A-G. This diagram shows the frame labels:



With the simplified distribution of load explained above, the following spreadsheet can be obtained which calculates the distribution factors for each moment frame:

NORTH – SOUTH

Frame #	Dist. Factor	*1.2?	Final DF
1	0.105	Y	0.126
2	0.105	Y	0.126
3	0.105	N	0.105
4	0.105	N	0.105
5	0.105	N	0.105
6	0.105	Y	0.126
A	0.053	N	0.053
B	0.053	Y	0.063
C	0.053	N	0.053
D	0.053	N	0.053
E	0.053	N	0.053
F	0.053	Y	0.063
G	0.053	N	0.053

EAST – WEST

Frame #	Dist. Factor	*1.1 Ext	Final DF
7	0.105	Y	0.126
8	0.105	Y	0.126
9	0.105	N	0.105
10	0.105	N	0.105
11	0.105	N	0.105
12	0.105	Y	0.126
A	0.053	N	0.053
B	0.053	Y	0.063
C	0.053	N	0.053
D	0.053	N	0.053
E	0.053	N	0.053
F	0.053	Y	0.063
G	0.053	N	0.053

These distribution factors can then be multiplied by the shear forces at each floor shown in the wind load diagrams (since wind controls) to obtain the lateral load on each moment frame at each floor. For a sample calculation, see the lateral member spot check section.

VI. TYPICAL FRAMING SIZE SPOT CHECKS

Deck Capacity:

From the structural general notes and plans, I concluded that the floor system was designed with 2" 18 gage Lok-Floor Deck by USD with 3 ¼" lightweight concrete for a total thickness of 5 ¼". The following table came from the USD website:

18 gage	4.50	62.08	32.6	0.292	34	1.53	5.4	42.99	4560	9.20	11.33	11.71	0.023
	5.00	72.04	37.5	0.333	38	1.81	7.3	50.72	5240	8.75	10.84	11.20	0.027
	5.25	77.02	40.0	0.354	41	1.95	8.3	54.72	5590	8.54	10.62	10.97	0.029
	5.50	82.00	42.6	0.375	43	2.10	9.5	58.78	5950	8.35	10.41	10.76	0.032
	6.00	91.95	48.0	0.417	48	2.39	12.1	67.07	6530	8.01	10.02	10.36	0.036
	6.25	96.93	50.8	0.438	50	2.54	13.6	71.29	6730	7.86	9.84	10.17	0.038
	6.50	101.91	53.6	0.458	53	2.69	15.2	75.55	6920	7.71	9.68	10.00	0.041
	7.00	111.87	59.5	0.500	58	3.00	18.8	84.17	7340	7.44	9.36	9.67	0.045
	7.25	116.85	61.9	0.521	60	3.16	20.7	88.52	7500	7.32	9.21	9.52	0.047
	7.50	121.83	64.3	0.542	62	3.31	22.8	92.91	7670	7.24	9.07	9.38	0.050

The red box highlights the maximum unshored spans, in feet, for 1, 2, or 3 span conditions. In the tower framing system, with a 5 ¼" total slab depth (far left column), the typical centerline to centerline beam spacing is 9.67'. You can see that the deck can span 10.62' and 10.97' for 2 or 3 span conditions, respectively. The one span condition does not apply to this building, so the deck fits this requirement.

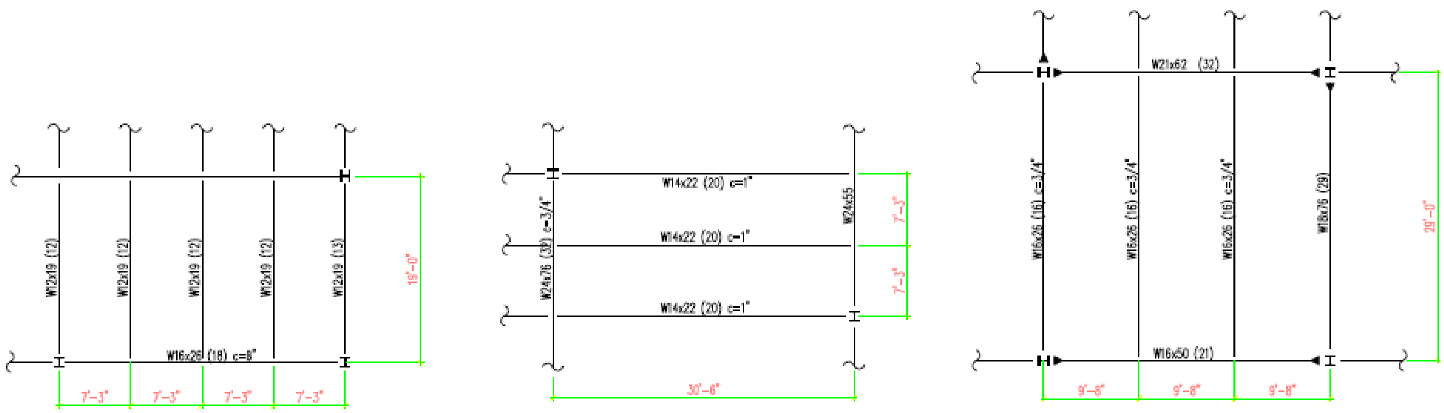
Next, one must verify whether the composite deck can carry the floor loads. The typical hospital loading is a dead load of 65 psf and a live load of 100 psf (see Loads section). The deck tables are concerned only with live load, as you can see in this table:

			L, Uniform Live Loads, psf *											
	Slab Depth	φMn in.k	6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
18 gage	4.50	62.08	400	400	400	400	375	330	290	260	230	205	180	155	135
	5.00	72.04	400	400	400	400	400	385	340	300	270	240	220	195	180
	5.25	77.02	400	400	400	400	400	400	365	325	290	260	235	210	190
	5.50	82.00	400	400	400	400	400	400	390	345	305	275	250	225	205
	6.00	91.95	400	400	400	400	400	400	400	385	345	310	280	250	230
	6.25	96.93	400	400	400	400	400	400	400	400	365	325	295	265	240
	6.50	101.91	400	400	400	400	400	400	400	400	385	345	310	280	255
	7.00	111.87	400	400	400	400	400	400	400	400	400	380	340	310	280

Rounding the 9.67’ span up to 10.00’, for 5 ¼” total thickness the capacity is 290 psf, which is much greater than the 100 psf live load for the typical office condition, or even the 125 psf live load for storage areas and the penthouse.

Typical Composite Beams:

There are three main infill beam sizes, a W12x19, W14x22, and W16x26. These beams are used in different locations based on span and spacing. You can see below typical framing areas for each of these beams:



CASE A CASE B CASE C

On the second, third, and fourth floors, there are some variations in members because a 2” slab step down is required where the prefabricated shower units are being installed in the patient rooms. In these cases, the depression must be framed out with additional infill beams and some of the sizes increased.

For this analysis, I am going to exclude this special condition, even though it is a recurring circumstance. Instead, I will examine the three beam sizes shown in the framing plans above as they are the typical cases on the first floor framing and framing areas without slab depressions on the upper floors. If the analysis of these three cases confirms the designer’s beam sizes, I will then conclude that the beams framing the slab depressions were most likely also sized correctly.

It must be verified that each of these beams are designed properly for the necessary loads which requires checking bending, number of shear studs, and deflection of the composite slab. My calculations for the three typical composite beams are in Appendix A.

For Case A, the W12x19 beam, I found that bending, number of studs, and deflection requirements were easily met. Most likely the designer chose this beam as a minimum size for the building, because its full capacity is not utilized.

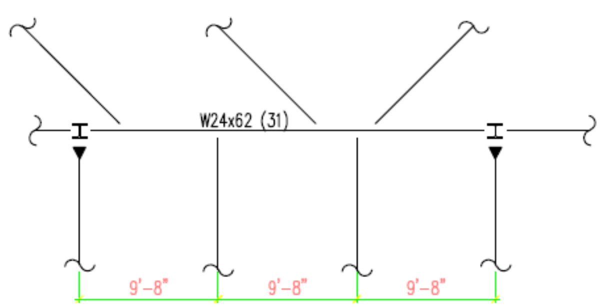
The typical W14x22 also met all requirements, however the deflection was close to the L/240 limit. I rounded Y2 down to be more conservative because linear interpolation is not valid for Table 3-20, so the I_{LB} I used is slightly lower than the actual I_{LB}. Also, I made an assumption on the plastic neutral axis location for each beam during my analysis. The structural engineer designed these beams using a RAM Structural System model, so the program could make alternate assumptions to those I used.

Similarly to the W14x22, the W16x26 was close to the deflection limit. Once again, I think my deflection calculations were somewhat conservative and the exact deflection is probably further below the L/240 limit. However, I also calculated that 23 shear studs are required for this beam, and 20 were provided per the plans. This discrepancy is easily explained because the calculation for the number of shear studs is directly related to the plastic neutral location, which is an assumption made by the designer.

Typical Composite Girder:

The girder sizes in the typical floor plan vary somewhat considerably between W16s and W27s. This variation becomes even greater on the upper floors where the slab is depressed for the shower stalls. However, the most typical sizes seem to be W21s and W24s with smaller sizes occurring only where the spans are short and larger sizes occurring only at unusually long spans.

The points at which the girders are loaded vary because of the change in framing direction due to the building's unusual shape. I performed calculations on a interior 29 foot W24x62 that seemed to be a median sized girder with the typical condition of 3 beams framing into it on each side. The framing on each side of the girder at this location are in opposite directions, which is a fairly common condition for the interior girders in this building. Below is the framing at this girder:



Upon my analysis of this girder, I found that in order to provide to required moment capacity, more than 31 shear studs would be required. However, this discrepancy between my calculations and the designer's could be the result of a few assumptions and simplified procedures. First, I calculated each of the five point loads using tributary area. For the three beams framing in at an angle, I may have slightly overestimated the tributary area in an attempt to be conservative. Even just a small increase in tributary area could have a somewhat significant effect on the point load. Also, I know the beams framing in at 90 degrees apply loads at the third points of the girder, but the exact location of the other point loads are not as easily determined. Applying the load at a location even just inches offset from the actual location would change the maximum moment.

The designer used a RAM model of the structure to size the beams, so the computer's calculation of point load values, locations, and inherently the maximum moment would be more exact than my method of using simplifications like tributary area. I made sure to err on the conservative side, so the fact that my design resulted in more shear studs is not surprising.

Typical Column:

The column sizes used in the addition are a variety of wide flange shapes ranging in size from W12x40 to W14x159. The typical column bay is 29 feet by 29 feet, for a typical total tributary area of 841 square feet per column per floor. The live loads at the penthouse and roof are not reducible, as noted in the loads section, per the code. I am not going to analyze any specific column, rather a typical condition considering the standard bay size, and then I will compare that to the overall column size range.

Live load reduction per the code provides a new live load of:

A_T = 841 per floor*4 floors = 3364 SF
A_I = 4*3364 = 13,456 SF (assuming an interior column)
$L_R = 100*[0.25 + 15/(13456)^{1/2}] = 38 < 0.4*100 = 40$ therefore, use L_R = 40 psf

According to live load reduction for the four typical floors calculated above, and the unreduced floor loading required for the roof, the total design live load for a typical column in a typical bay is:

P_L = 4*841*40 psf + 841*30 psf = 159.8 k

The typical dead load consisting of the composite slab, structure, mechanical, and miscellaneous load, is 65 psf. The main roof is composed of the same structure as the typical floor because it is also utilized as the penthouse floor. This results in a total design dead load of:

$P_D = 5*841*65 \text{ psf} = 273.3 \text{ k}$

Therefore, the total factored design load for a typical column that extends up to the main roof is:

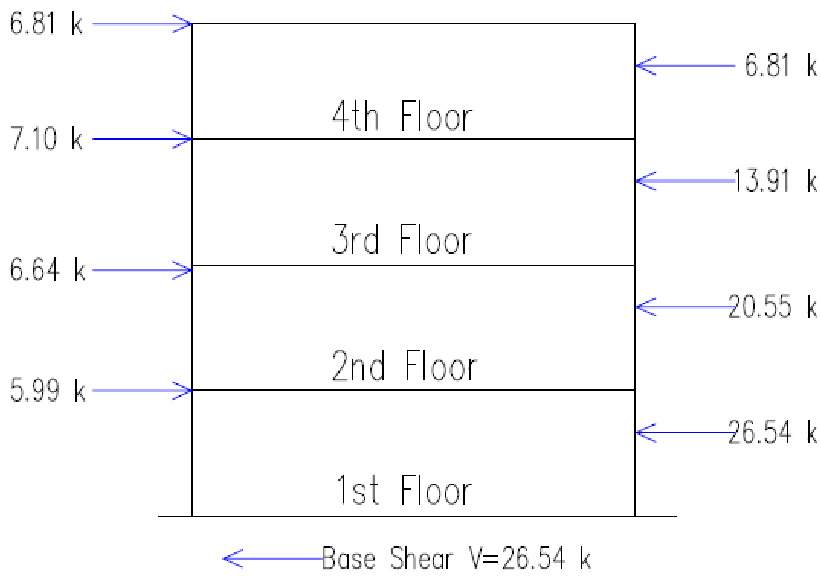
$1.2*P_D + 1.6*P_L = 1.2*159.8 + 1.6*273.3 = 629 \text{ k}$

The typical floor-to-floor height for the addition is 18 feet, so that will be used as the column unbraced length. Assuming that the column is pinned-pinned and therefore K=1.0, the required column size from Table 4-1 of the Steel Manual is a W12x72. This column size falls within the range of the designed columns. Though it is somewhat on the low end of the W12x40 to W14x159 range, that is to be expected because the load was calculated assuming a column that extends to the main roof only. Other columns that extend to the penthouse roof accumulate additional dead load from the roof as well as a considerable amount of unreducible live load. These columns would most likely have to be significantly larger. Also, I analyzed the column as a gravity column only, without taking any moment. The columns that are moment connected to the beams will also receive a portion of the moment which would increase the required size. See the lateral member analysis for a check of combined axial and bending in a lateral column.

From this check of a typical column, the designer's column sizes are assumed to be appropriate.

Lateral Member:

Assumptions for how the lateral load is distributed to each moment frame are explained in the lateral load distribution portion of the Loads section. I am going to analyze moment frame 6, which resists lateral loads in the North-South direction. This moment frame occurs where the building extends to the main roof only, so the penthouse roof loads will be excluded from this analysis. The distribution factor for this moment frame was previously calculated to be 0.126. After applying this factor to the wind forces calculated previously, the following loading diagram can be obtained:



After performing a portal frame analysis on this moment frame to obtain the column moments, axial load due to lateral forces, and performing a load take down to obtain the gravity axial load, I found that the columns work in combined axial and bending. My results show that the columns are easily below the 1.0 limit. This could mean that the designer calculated a higher lateral load than I did, which would have resulted in a larger moment. It could be due to the differing code editions used for analysis. The designer could have also just been designing the lateral members conservatively. Either way, the discrepancy is of no concern because the current design is proven to be more conservative than my calculations require and the lateral column clearly works.

APPENDIX A – FRAMING SPOT CHECK CALCULATIONS

Typical Composite Beams:

(A) W12x19 with 12 shear studs

w_u = (1.2*65 + 1.6*100)*7.25' = 1726 lb/ft = 1.726 k/ft
M_u = (1.726*19^2)/8 = 77.9 ft-k
V_u = (1.726*19)/2 = 16.4 k

- Check Bending:
  Assume PNA at location 6 to reduce number of shear studs because available strength seems to be >> required strength
  $\sum Q_n$ = 104 k @ PNA 6 from Table 3-19
  b_{eff} = minimum of: ¼ span = ¼ *19 = $\boxed{\text{4.75 ft}}$
  spacing = 7.25 ft
  a = $\sum Q_n/(0.85*f'c*b_{eff})$ = 104/(0.85*3.5*4.75*12) = 0.61"
  Y2 = 5.25" – 0.61"/2 = 4.95"
  Use Y2 = 4.5" because rounding down is conservative
  ϕMn = 162 ft-k > 77.9 ft-k, therefore **OK**

- Check Number of Shear Studs:
  Assume $\sum Q_n$ = 17.2 k from Table 3-21 (LW concrete, ¾" diameter studs, 1 stud/rib)
  # of studs required = (104/17.2)*2 = 12.1
  12 studs provided, therefore **OK**

- Check Deflection:
  Use Y2 = 4.5" because rounding down is conservative
  I_{LB} = 300 in^3 from Table 3-20
  Δ_{max} = (5*1.726*19^4*1728)/(384*29000*300) = 0.582"
  0.582" = L/392 < L/240 = 0.95", therefore **OK**

(B) W14x22 with 20 shear studs and 1" camber

w_u = (1.2*65 + 1.6*100)*7.25' = 1726 lb/ft = 1.726 k/ft
M_u = (1.726*30.5^2)/8 = 200.7 ft-k
V_u = (1.726*30.5)/2 = 26.3 k

- Check Bending:
  Assume Y2 = 4.5", which requires PNA @ BFL
  $\sum Q_n$ = 157 k @ BFL from Table 3-19
  b_{eff} = minimum of: ¼ span = ¼ *30.5 = 7.63 ft
  spacing = $\boxed{7.25 \text{ ft}}$
  a = $\sum Q_n$/(0.85*f'c*b_{eff}) = 157/(0.85*3.5*7.25*12) = 0.61"
  Y2 = 5.25" – 0.61"/2 = 4.95"
  Use Y2 = 4.5" because rounding down is conservative
  ϕM_n = 218 ft-k > 200.7 ft-k, therefore **OK**

- Check Number of Shear Studs:
  Assume $\sum Q_n$ = 17.2 k from Table 3-21 (LW concrete, ¾" diameter studs, 1 stud/rib)
  # of studs required = (157/17.2)*2 = 18.3
  20 studs provided, therefore **OK**

- Check Deflection:
  Use Y2 = 4.5" because rounding down is conservative
  I_{LB} = 473 in^3 from Table 3-20
  Δ_{max} = (5*1.726*30.5^4*1728)/(384*29000*473) = 2.45"
  2.45" – 1" camber = 1.45" = L/252 < L/240 = 1.525", therefore **OK**

(C) W16x26 with 20 shear studs and ¾" camber

w_u = (1.2*65 + 1.6*100)*9.67' = 2301 lb/ft = 2.301 k/ft
M_u = (2.301*30.5^2)/8 = 267.6 ft-k
V_u = (2.301*30.5)/2 = 35.1 k

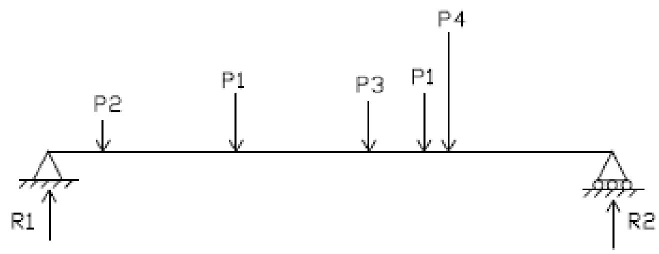
- Check Bending:
  Assume Y2 = 4.5", which requires PNA @ BFL
  $\sum Q_n$ = 194 k @ BFL from Table 3-19
  b_{eff} = minimum of: ¼ span = ¼ *30.5 = $\boxed{7.63 \text{ ft}}$
  spacing = 9.67 ft
  a = $\sum Q_n$/(0.85*f'c*b_{eff}) = 194/(0.85*3.5*7.63*12) = 0.71"
  Y2 = 5.25" – 0.53"/2 = 4.89"
  Use Y2 = 4.5" because rounding down is conservative
  ϕM_n = 289 ft-k > 267.6 ft-k, therefore **OK**

- Check Number of Shear Studs:
  Assume $\sum Q_n$ = 17.2 k from Table 3-21 (LW concrete, ¾" diameter studs, 1 stud/rib)
  # of studs required = (194/17.2)*2 = 22.6
  20 studs provided, therefore **NOT OK**, however they are close and I could have made different assumptions than the structural engineer

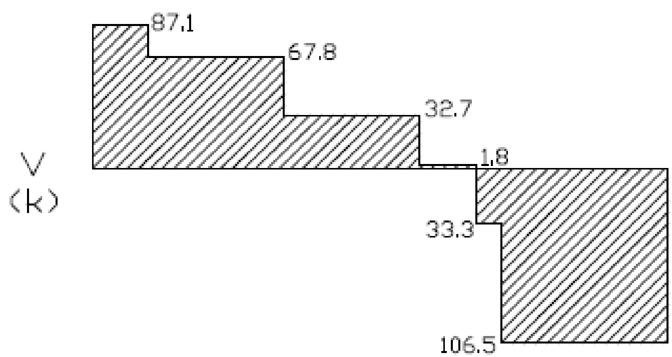
- Check Deflection:
  Use Y2 = 4.5"
  I_{LB} = 694 in^3 from Table 3-20
  Δ_{max} = (5*2.301*30.5^4*1728)/(384*29000*694) = 2.23"
  2.23" – ¾" camber = 1.48" = L/247 < L/240 = 1.525", therefore **OK**

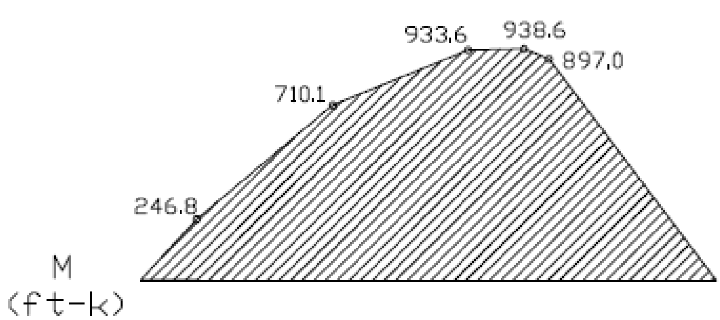
Typical Composite Girder:

P_1 = 35.1 k (from typical W16x26 beam above)
P_2 = 2.301*16.75'/2 = 19.3 k
P_3 = 2.301*26.83'/2 = 30.9 k
P_4 = [(1.2*65+1.6*100)*46'/2]*26.75'/2 = 73.2 k



R_1 = [19.3*2.83'+35.1*(9.67'+19.3')+30.9*16.5'+73.2*20.58']/29' = 106.5 k
R_2 = 193.6 k – 106.5 k = 87.1 k





M_{max} = 938.6 ft-k

Assume Y2 = 4.0”, which requires PNA @ BFL
$\sum Q_n$ = 496 k @ BFL from Table 3-19
b_{eff} = minimum of: ¼ span = ¼ *19 = $\boxed{7.25\text{ ft}}$
spacing of girders >> ¼ span
a = $\sum Q_n/(0.85*f'c*b_{eff})$ = 496/(0.85*3.5*7.25*12) = 1.92”
Y2 = 5.25” – 1.92”/2 = 4.29”
Use Y2 = 4.0” because rounding down is conservative
ϕM_n = ft-k > 938.6 ft-k, therefore **OK**

Lateral Framing Member:

Moment Frame 6: Distribution Factor = 1.26 from spreadsheet

Story Forces/Shears @ Each Floor:

Story	Load	Story Force	Story Shear
Roof	53.9	6.81	6.81
4th	56.2	7.10	13.91
3rd	52.6	6.64	20.55
2nd	47.4	5.99	26.54
1st/Base	-	-	26.54

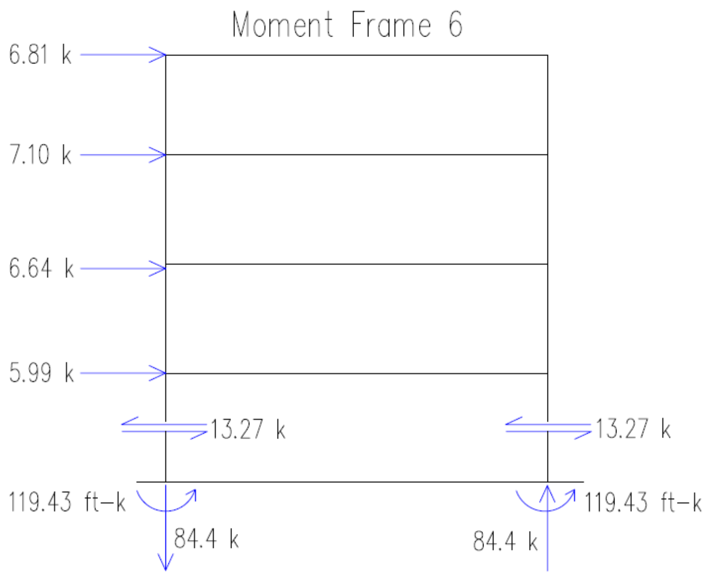
Portal Frame Analysis @ Base:

Total Base Shear for Moment Frame 6 = 26.54 k
Shear in each column = 26.54/2 = 13.27

Overturning Moment for Frame 6=
26.54*18’ + 20.55*36’ + 13.91*54’ + 6.81*70.5’ = 2448.8 ft-k

Axial Load due to lateral forces = 2448.8/29’ = 84.4 k

Moment @ Base of Column = 119.43 ft-k



Axial Load @ Column:

Live Load:

A_T = 443 per floor*4 floors = 1772 SF

A_I = 3*1772 = 5316 SF

$L_R = 100*[0.25 + 15/(5316)^{1/2}] = 46 > 0.4*100 = 40$ therefore, use L_R = 46 psf

P_L = (4 floors*443 SF*46 psf) + (443 SF*30 psf) = 94.8 k

Dead Load:

Assume exterior average load of 70 psf

P_D = (5 floors*443 SF*65 psf) + (70.5'*29'*70 psf) = 287.1 k

Factored Load:

1.2D + 1.6L controls

$P_U = 1.2*P_D + 1.6*P_L = 1.2*287.1 + 1.6*94.8 = 496.2$ k

Check that Column is OK in Combined Axial and Bending:

From Column Schedule, Column is a W14x145

$KL = L_b$ = 18'

From Table 6-1 for Combined Axial and Bending,

$p*P_u + b_x*M_{ux}$ = (0.646e-3*496.2) + (0.949e-3*119.43) = 0.321 + 0.114 = 0.435

0.435 < 1 therefore column is OK

APPENDIX B – WIND CALCULATIONS

NORTH-SOUTH WIND

Dimensions and Period

H = 88.5
L = 170
B = 171.5
L/B = 0.99
T_a = 1.01 >1 therefore flexible
*for calculation of Ta see Seismic calcs

Variable	Value	Fig/Table/Eqn
V =	90	Figure 6-1
I =	1.15	Table 6-1
K_{zt} =	1	Eqn 6-3
K_d =	0.85	Table 6-4
GC_{pi} =	0.18	Figure 6-5
α =	7.00	Table 6-2
z_g =	1200	Table 6-2
â =	0.14	Table 6-2
b hat =	0.84	Table 6-2
α bar =	0.25	Table 6-2
b bar =	0.45	Table 6-2
c =	0.30	Table 6-2
l =	320	Table 6-2
Є bar =	0.33	Table 6-2
z bar =	53.10	0.6*h
L_z =	199.21	Eqn 6-7
n_1 =	0.989	$1/T_a$
N_1 =	2.945	Eqn 6-12
β =	0.500	Section 6.5.8.2

Calculate Gust Factor				
Variable	**Value**	**Fig/Table/Eqn**		
g_Q =	3.4	Given		
g_V =	3.4	Given		
g_R =	4.19	Eqn 6-9		
V_z hat =	66.9	Eqn 6-14		
R_h =	0.152	Eqn 6-13a	**n =**	6.02
R_B =	0.082	Eqn 6-13a	**n =**	11.66
R_L =	0.026	Eqn 6-13a	**n =**	38.71
R_N =	0.071	Eqn 6-11		
I_z =	0.277	Eqn 6-5		
Q =	0.757	Eqn 6-6		
R =	0.031	Eqn 6-10		
G_f =	0.787	Eqn 6-8		

The following equations were used in the spreadsheets below:

$K_z = 2.01*(z/z_g)^{2/\alpha}$

$q_z = 0.00256* K_z*K_{zt}*K_d*V^2*I$

$P = q*G_f*C_p +/- q_i*GC_{pi}$ ($q = q_z$ for windward; $q = q_h$ for leeward, sidewall, and roof; $q_i = q_h$ for windward, leeward, sidewall, and roof)

Calculate Pressures using Equations Above						
C_p =	0.8	Figure 6-6	(Windward)			
C_p =	-0.5	Figure 6-6	(Leeward)			
C_p =	-0.7	Figure 6-6	(Sidewall)			
G_f =	0.787					
z (ft)	**K_z**	**q_z**	**P (leeward)**	**P (windward)**	**P (sidewall)**	**+/- qGC_{pi}**
0-18	0.605	12.272	-7.613	7.728	-10.659	+/- 3.482
36	0.738	14.960	-7.613	9.421	-10.659	+/- 3.482
54	0.829	16.797	-7.613	10.578	-10.659	+/- 3.482
70.5	0.894	18.127	-7.613	11.415	-10.659	+/- 3.482
88.5	0.954	19.344	-7.613	12.181	-10.659	+/- 3.482
	q_h =	19.344				

Calculate Roof Pressure				
H/L =	0.521	requires interpolation		**P (roof)**
C_p =	-0.917	Figure 6-6	0 to 44.25 ft	-13.963
C_p =	-0.892	Figure 6-6	44.25 to 88.5 ft	-13.582
C_p =	-0.508	Figure 6-6	88.5 to 170 ft	-7.735

These spreadsheet were designed to reference the cells in the other spreadsheets where the variables were defined. However, below you can see a sample calculation of how the pressures were obtained.

Find P @ h = 54'

$K_z = 2.01*(z/z_g)^{2/\alpha} = 2.01*(54/1200)^{2/7.0} = 0.829$
$q_z = 0.00256* K_z*K_{zt}*K_d*V^2*I = 0.00256*0.829*1*0.85* 90^2*1.15 = 16.797$

P (leeward) = $q_hG_fC_p$ +/- q_hGC_{pi} = (19.344*0.787*-0.5) +/- (19.344*0.18) = -7.612 +/- 3.482

P (windward) = $q_zG_fC_p$ +/- q_hGC_{pi} = (16.797*0.787*0.8) +/- (19.344*0.18) = 10.575 +/- 3.482

P (sidewall) = $q_hG_fC_p$ +/- q_hGC_{pi} = (19.344*0.787*-0.7) +/- (19.344*0.18) = -10.657 +/- 3.482

It can be seen that these values are equal to those calculated in the spreadsheet. This same calculation is performed using the spreadsheet for each height "z" in both the North-South and East-West directions.

EAST-WEST WIND

Dimensions and Period

H = 88.5
L = 171.5
B = 170
L/B = 1.01
T_a = 1.01 >1 therefore flexible
*for calculation of Ta see Seismic calcs

Variable	Value	Fig/Table/Eqn
V =	90	Figure 6-1
I =	1.15	Table 6-1
K_{zt} =	1	Eqn 6-3
K_d =	0.85	Table 6-4
GC_{pi} =	0.18	Figure 6-5
α =	7.00	Table 6-2
z_g =	1200	Table 6-2
â =	0.14	Table 6-2
b hat =	0.84	Table 6-2
α bar =	0.25	Table 6-2
b bar =	0.45	Table 6-2
c =	0.30	Table 6-2
l =	320	Table 6-2
Є bar =	0.33	Table 6-2
z bar =	53.10	0.6*h
L_z =	200.97	Eqn 6-7
n_1 =	0.989	$1/T_a$
N_1 =	2.971	Eqn 6-12
β =	0.500	Section 6.5.8.2

Calculate Gust Factor				
Variable	**Value**	**Fig/Table/Eqn**		
g_Q =	3.4	Given		
g_V =	3.4	Given		
g_R =	4.19	Eqn 6-9		
V_z hat =	66.9	Eqn 6-14		
R_h =	0.152	Eqn 6-13a	n =	6.02
R_B =	0.083	Eqn 6-13a	n =	11.56
R_L =	0.025	Eqn 6-13a	n =	39.05
R_N =	0.070	Eqn 6-11		
I_z =	0.277	Eqn 6-5		
Q =	0.758	Eqn 6-6		
R =	0.031	Eqn 6-10		
G_f =	0.788	Eqn 6-8		

The following equations were used in the spreadsheets below:

$K_z = 2.01*(z/z_g)^{2/\alpha}$

$q_z = 0.00256* K_z*K_{zt}*K_d*V^2*I$

$P = qG_fC_p +/- q_iGC_{pi}$ ($q = q_z$ for windward; $q = q_h$ for leeward, sidewall, and roof)

Calculate Pressures using Equations Above						
C_p =	0.8	Figure 6-6	(Windward)			
C_p =	-0.5	Figure 6-6	(Leeward)			
C_p =	-0.7	Figure 6-6	(Sidewall)			
G_f =	0.788					
z (ft)	**K_z**	**q_z**	**P (leeward)**	**P (windward)**	**P (sidewall)**	**+/- qGC_{pi}**
0-18	0.605	12.272	-7.621	7.736	-10.670	+/- 3.482
36	0.738	14.960	-7.621	9.431	-10.670	+/- 3.482
54	0.829	16.797	-7.621	10.589	-10.670	+/- 3.482
70.5	0.894	18.127	-7.621	11.427	-10.670	+/- 3.482
88.5	0.954	19.344	-7.621	12.194	-10.670	+/- 3.482
	q_h =	19.344				

Finally, the pressures above were converted to story forces, which can be added to obtain the story shears. Story forces were obtained by multiplying by the square footage of the building face. Below are these calculations for both directions. The loading diagrams can be found in the main report wind load section.

	North - South Wind			East - West Wind		
Story	**Area (SF)**	**Story Force (k)**	**Story Shear (k)**	**Area (SF)**	**Story Force (k)**	**Story Shear (k)**
PH Roof	1890	37.4	37.4	1836	36.4	36.4
Roof/PH Flr	2830	53.9	91.3	2805	53.4	89.8
4th Floor	3087	56.2	147.4	3060	55.7	145.5
3rd Floor	3087	52.6	200.0	3060	52.2	197.7
2nd Floor	3087	47.4	247.4	3060	47.0	244.7
1st/Base	-	-	247.4	-	-	244.7

APPENDIX C – SEISMIC CALCULATIONS

Seismic Factors/Coefficients:

Mapped Spectral Response Accelerations: $S_S = 0.160$ g
$S_1 = 0.050$ g
Site Class: D
Seismic Use Group: III
Importance Factor (I): 1.25
Response Modification Coefficient (R): 3.0
Site Class Factors: $F_a = 1.6$
$F_v = 2.4$
Adjusted Spectral Response Accelerations: $S_{MS} = 0.256$
$S_{M1} = 0.12$

$S_{MS} = F_a*S_S = 0.160*1.6 = 0.256$
$S_{M1} = F_v*S_1 = 0.050*2.4 = 0.12$

Design Spectral Response Accelerations: $S_{DS} = 0.171$
$S_{D1} = 0.08$

$S_{DS} = (2/3)* S_{MS} = (2/3)*0.256 = 0.171$
$S_{D1} = (2/3)* S_{M1} = (2/3)*0.12 = 0.08$

Seismic Design Category: B

$0.167 \le S_{DS} < 0.33$ and $0.067 \le S_{D1} < 0.133$
Therefore, Seismic Design Category B

Approximate Period (T_a): 1.011

Steel Moment Resisting Frames
Therefore C = 0.028 and x = 0.8
$T_a = C_t*h_n^x = 0.028*(88.5')^{0.8} = 1.011$

Fundamental Period (T): 1.719

$S_{D1} \le 0.1$ Therefore, $C_u = 1.7$
$T = C_u*T_a = 1.7*0.011 = 1.719$

Seismic Response Coefficient (C_s): 0.0194

$$C_S = \min \left| \begin{array}{l} S_{DS}/(R/I) = 0.171/(3.0/1.25) = 0.0713 \\ S_{D1}/[T*(R/I)] = 0.08/[1.719*(3.0/1.25)] = 0.0194 \leftarrow \text{controls} \\ (S_{D1}*T_L)/[T^2*(R/I)] = (0.08*8)/[1.719*(3.0/1.25)] = 0.0902 \end{array} \right.$$

Seismic Weight of the Building (W):

1^{st} floor: Floor Load = 18056 sf*0.065 ksf = 1173.6 k
CMU Wall Load = 70'*18'*0.04 ksf = 50.4 k
Exterior Wall Load = (10'*214.2' + 3'*171.5' + 18'*79')*0.1 ksf + (170'*0.36 klf) + (8'*556.7'*0.01 ksf) = 513.6 k
Total Load = 1173.6 + 50.4 + 513.6 = 1737.6 k

2^{nd} floor: Floor Load = 18056 sf*0.065 ksf = 1173.6 k
CMU Wall Load = 70'*18'*0.04 ksf = 50.4 k
Exterior Wall Load = (9'*556.7'+18'*79')*0.1 ksf + 9'*556.7'*0.01 ksf = 693.3 k
Total Load = 1173.6 + 50.4 + 693.3 = 1917.3 k

3^{rd} floor: Floor Load = 18056 sf*0.065 ksf = 1173.6 k
CMU Wall Load = 70'*18'*0.04 ksf = 50.4 k
Exterior Wall Load = (3'*556.7' + 18'*79')*0.1 ksf + 15'*556.7'*0.01 ksf = 392.7 k
Total Load = 1173.6 + 50.4 + 392.7 = 1616.7 k

4^{th} floor: Floor Load = 18056 sf*0.065 ksf = 1173.6 k
CMU Wall Load = 70'*16.5'*0.04 ksf = 46.2 k
Exterior Wall Load = (7'*556.7 + 18'*79')*0.1 ksf + 9.5'*556.7'*0.01 ksf = 584.8 k
Total Load = 1173.6 + 46.2 + 584.8 = 1804.6 k

PH/Main Roof: Floor/Roof Load = 18056 sf*0.065 ksf = 1173.6 k
CMU Wall Load = 70*18*0.04 ksf = 50.4 k
Exterior Wall Load = 4'*635.7'*0.1 ksf + 18'*317'*0.015 ksf = 339.9 k
Total Load = 1173.6 + 50.4 + 339.9 = 1563.9 k

PH Roof: Roof Load = 5875 sf*0.015 psf = 88.1 k

Total Weight (W) = 1737.6 + 1917.3 + 1616.7 + 1804.6 + 1563.9 + 88.1 = 8728.2 k

Seismic Base Shear and Shear Load Distribution:

$V = C_s*W = 0.0194*8728.2 = 169.3$ k

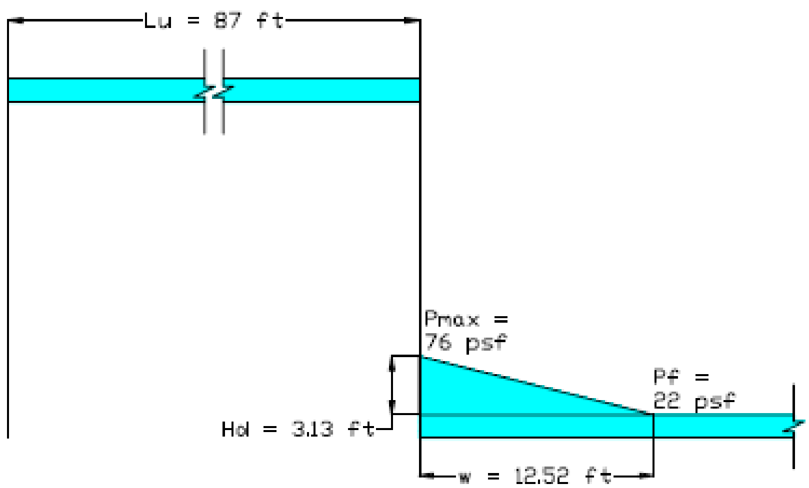
Story	h_x (ft)	Floor Load (k)	CMU wall	Ext. Load (k)	Total W (k)	$h_x^k*W_x$	C_{vx}	$Fx = C_{vx}*V$	V_x (k)	M_x (ft-k)
PH Roof	88.5	88.1		0	88.1	120103.2	0.035	5.9	0	524.997
Roof/PH	70.5	1173.6	50.4	339.9	1563.9	1478405	0.431	73.0	5.9	5148.036
4	54	1173.6	46.2	584.8	1804.6	1110542	0.324	54.9	79.0	2962.018
3	36	1173.6	50.4	392.7	1616.7	517936.4	0.151	25.6	133.8	920.954
2	18	1173.6	50.4	693.3	1917.3	201223.5	0.059	9.9	159.4	178.9
1		1173.6	50.4	513.6	1737.6	0	0.000	0.0	169.3	0
				Totals =	8728.2	3428210	1	169.3		9734.9

APPENDIX D – SNOW DRIFT CALCULATIONS

Snow Drift Calculations:

Case A – Drift from Penthouse Roof to Main Roof

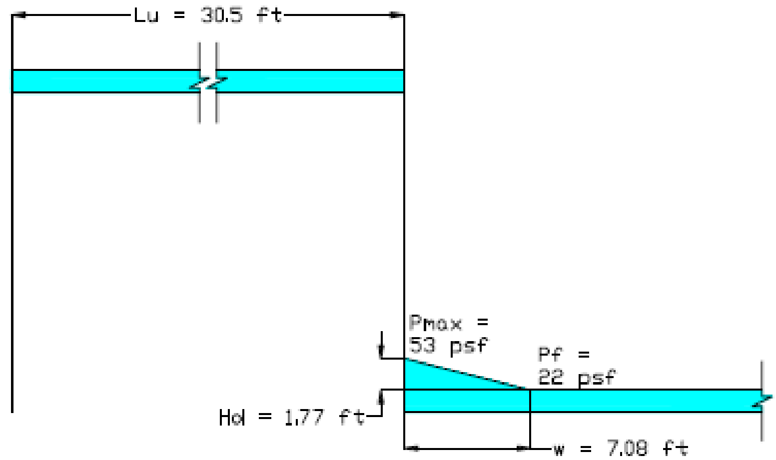
$l_u = 87$ ft
$h_c = 18$ ft
$\gamma = 0.13p_g + 14 = 0.13*25 + 14 = 17.25$ pcf
$h_d = 0.43* l_u^{1/3}*(p_g +10)^{1/4} – 1.5 = 0.43*(87)^{1/3}*(25+10)^{1/4} – 1.5 = 3.13$ ft
$w = 4*h_d = 4*3.13 = 12.52$ ft
$p_d = h_d*\gamma = 3.13*17.25 = 54$ psf
$p_{max} = p_d + p_f = 54 + 22 = 76$ psf



CASE A

Case B – Drift from Elevator to Main Roof

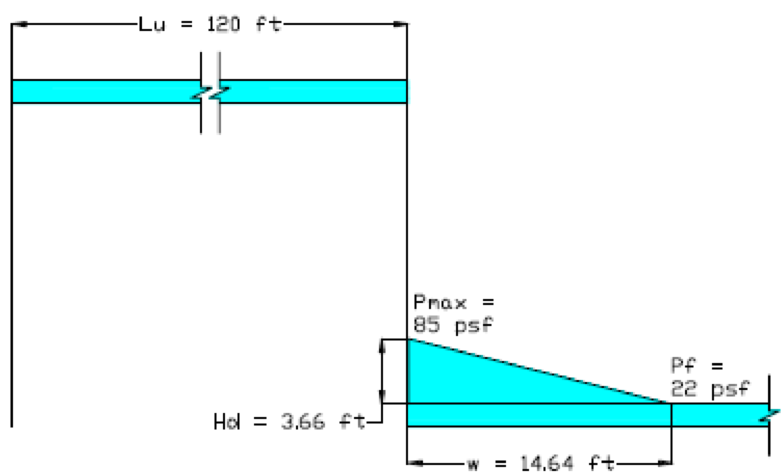
$l_u = 30.5$ ft
$h_c = 18$ ft
$\gamma = 0.13p_g + 14 = 0.13*25 + 14 = 17.25$ pcf
$h_d = 0.43* l_u^{1/3}*(p_g +10)^{1/4} – 1.5 = 0.43*(30.5)^{1/3}*(25+10)^{1/4} – 1.5 = 1.77$ ft
$w = 4*h_d = 4*1.77 = 7.08$ ft
$p_d = h_d*\gamma = 1.77*17.25 = 31$ psf
$p_{max} = p_d + p_f = 31 + 22 = 53$ psf



CASE B

Case C – Drift from Main Roof to Low Roof

$l_u = 120$ ft
$h_c = 54$ ft
$\gamma = 0.13p_g + 14 = 0.13*25 + 14 = 17.25$ pcf
$h_d = 0.43* l_u^{1/3}*(p_g+10)^{1/4} - 1.5 = 0.43*(120)^{1/3}*(25+10)^{1/4} - 1.5 = 3.66$ ft
$w = 4*h_d = 4*3.66 = 14.64$ ft
$p_d = h_d*\gamma = 3.66*17.25 = 63$ psf
$p_{max} = p_d + p_f = 63 + 22 = 85$ psf



CASE C