HOWARD COUNTY GENERAL HOSPITAL

PATIENT TOWER ADDITION

COLUMBIA, MD

Technical Assignment 3:
Lateral System Analysis

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I. EXECUTIVE SUMMARY

This third technical report is a more in depth lateral analysis of the hospital's main lateral force resisting system. A total of 19 steel moment frames are located throughout the building at each floor to resist wind and seismic forces. These forces were obtained through a RAM model and verified by a variety of hand calculations.

It was found that wind base shear controls over seismic base shear in both the North-South and East-West directions. For wind, the base shear values from RAM were very similar to those calculated by hand. For seismic, the base shear values from RAM were lower than those calculated by hand because conservative simplifications were made to calculate the building weight, which directly effects the base shear.

Each of the 19 moment frames were modeled in STAAD and applied a 10 kip load to determine the frame deflections at each floor. From these results, relative stiffness values were calculated through which the lateral load could be distributed. My calculated values were compared to those from the RAM model. For the frames directly resisting lateral load, the distribution factors were generally within 10-20%. However, for the frames rotated at a 45 degree angle, the distribution factors had a larger discrepancy, most likely due to some simplified assumptions. Overall, I feel that my hand calculations verified the RAM results.

From the RAM model, the center of mass and rigidity were calculated for each floor. Based on eccentricities of 11 to 16.5 feet, it was determined that torsion could have a significant effect on the building. The torsional shear for each frame for each level was calculated, with results ranging from 0 to 3 kips.

Drift proved to be an issue in the building as both total wind drift and inner story displacement due to wind were well above the industry standard of H/400.

Finally, a member code check was performed in RAM that checks the interaction equation for each lateral member according to the code interaction equation. Most members were stressed below 70 percent, however five members were slightly overstressed by 1 or 2 percent. Because the overstress is so slight, this is not considered to be a huge issue. Hand calculations were performed on a typical beam and a typical column to further verify the accuracy of RAM's results, both proving to be adequately sized.

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 for a total building height of 88’-6” above grade (at the penthouse roof). 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 more thoroughly analyzes the existing lateral system. RAM Structural System was used to build a computer model of the tower. The loads applied by RAM were verified with hand calculations and compared to the forces calculated in Technical Assignment 1. The model allowed for a more exact distribution of forces and thus the building can be analyzed more effectively.

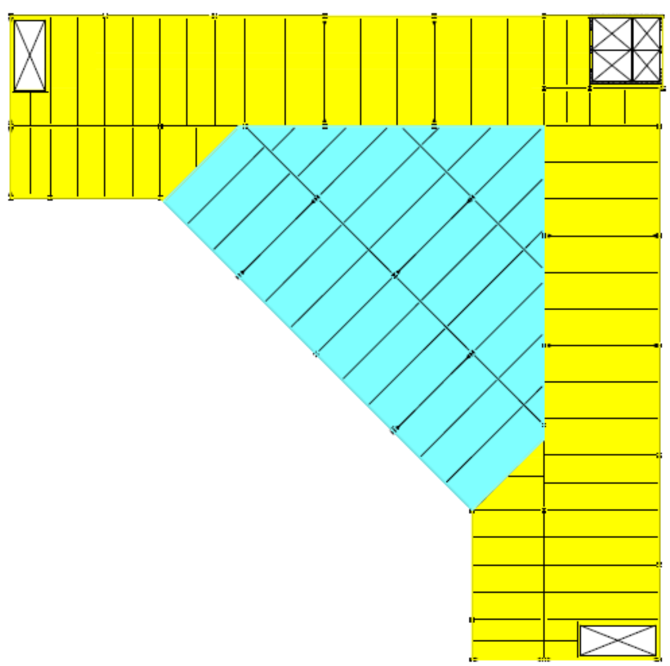
III. STRUCTURAL OVERVIEW

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 top 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.

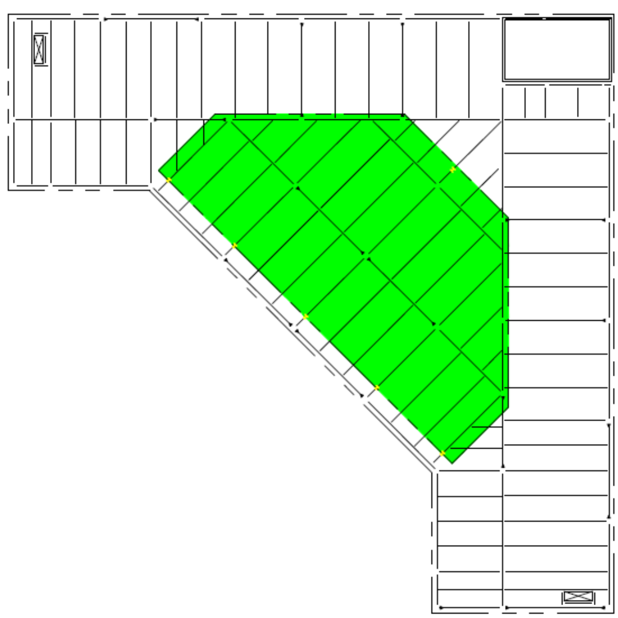
The new addition is a uniquely shaped structure, so the floors are framed in two different directions. As you can see in the figure below, the "center" floor framing (shown in blue) is rotated at a 45 degree angle from the framing along the outer "L" of the building (shown in yellow).



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.

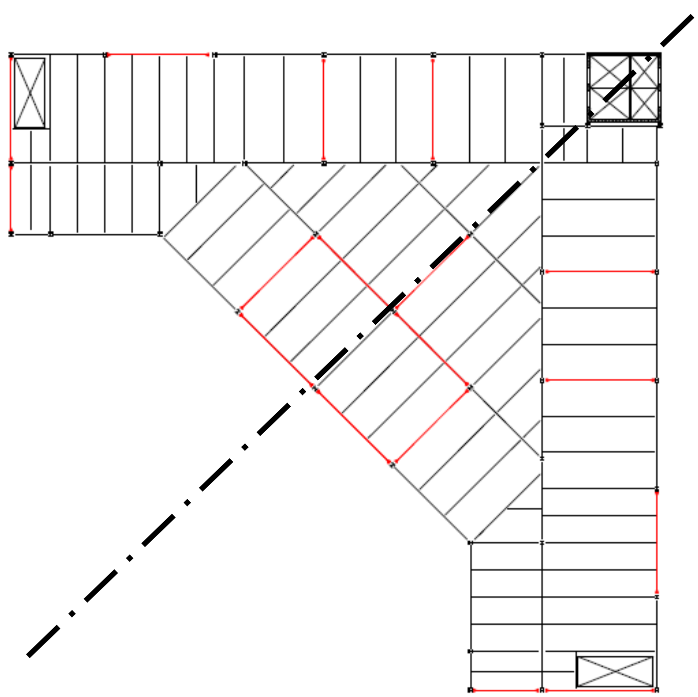
Exterior:

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.

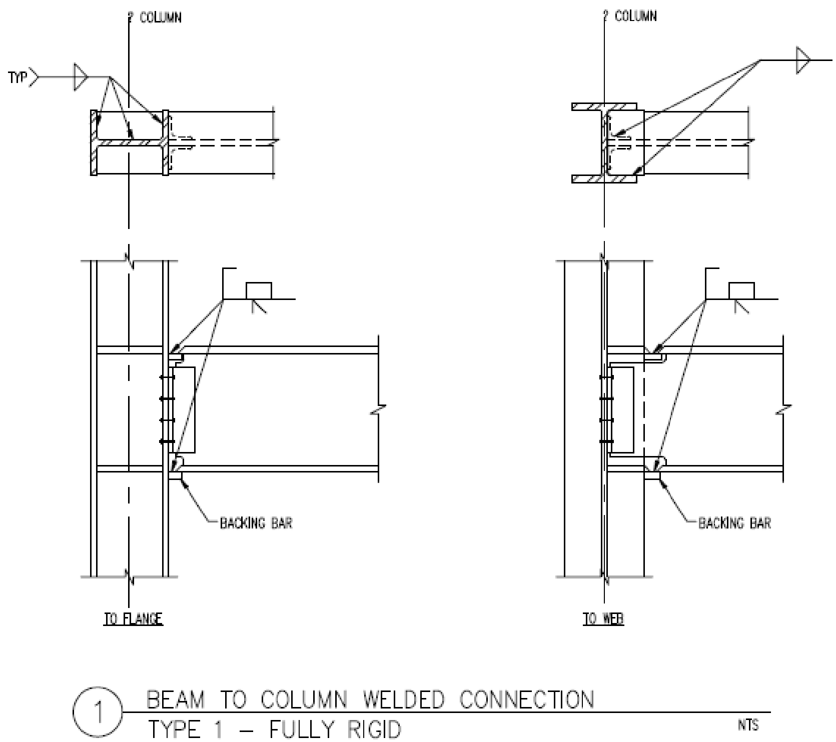
The walls that extend from the penthouse floor to the penthouse roof are composed of 6" metal studs at 16" on center with insulation. These walls have an exterior finish of "dryvit" on them for protection and aesthetics.

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 318-99 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 LRFD and ASD Combined AISC Thirteenth Edition Steel Manual, Copyright 2006.

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

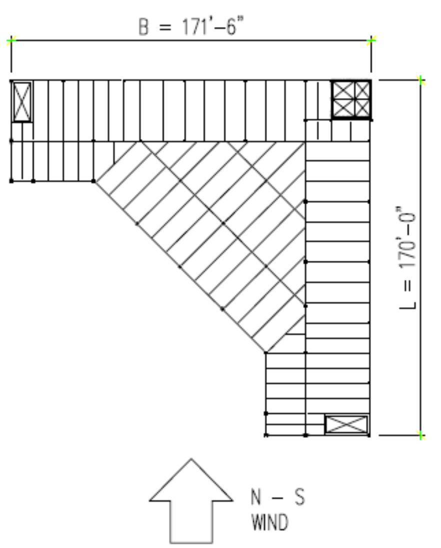
$P_{f,\,min}$ = I*20 psf = 1.1*20 psf = 22 psf

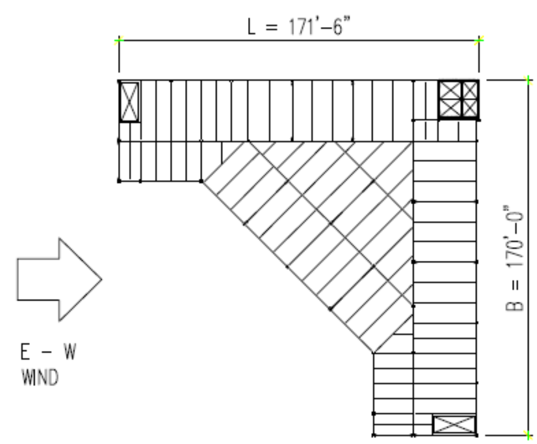
Therefore, the minimum flat roof snow load of 22 psf will be used. This will be added to the drift snow load where applicable. Otherwise, it is less than the 30 psf roof load, as expected, so the roof live load will control.

Wind Load Assumptions:

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 assumed for analysis and diagrams showing the directions of loading.

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





Both hand calculations and wind loading results based on my RAM model are included later in the report to determine the design base shear.

Seismic Load Assumptions:

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.

Some 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 partially above grade basement. This assumption was made to simplify the procedure and it results in a smaller period and therefore larger base shear, so it is conservative. The Response Modification Factor was chosen based on assuming "Ordinary Composite Moment Frames". 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
Response Modification Coefficient (R)	3.0

Both hand calculations and seismic loading results based on my RAM model are included later in the report to determine the design base shear.

VI. WIND ANALYSIS

Wind Pressures:

In accordance with ASCE 7-05, the wind pressures were calculated for each wind direction. I am assuming that there will be similar results in both directions, given that the projected lengths of the building are almost equal.

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 =	0.837					
z (ft)	**K_z**	**q_z**	**P (leeward)**	**P (windward)**	**P (sidewall)**	**+/- qGC_{pi}**
0-18	0.605	12.272	-8.097	8.219	-11.336	+/- 3.482
36	0.738	14.960	-8.097	10.019	-11.336	+/- 3.482
54	0.829	16.797	-8.097	11.250	-11.336	+/- 3.482
70.5	0.894	18.127	-8.097	12.141	-11.336	+/- 3.482
88.5	0.954	19.344	-8.097	12.956	-11.336	+/- 3.482
	q_h =	19.344				

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 =	0.838					
z (ft)	**K_z**	**q_z**	**P (leeward)**	**P (windward)**	**P (sidewall)**	**+/- qGC_{pi}**
0-18	0.605	12.272	-8.101	8.223	-11.342	+/- 3.482
36	0.738	14.960	-8.101	10.024	-11.342	+/- 3.482
54	0.829	16.797	-8.101	11.256	-11.342	+/- 3.482
70.5	0.894	18.127	-8.101	12.147	-11.342	+/- 3.482
88.5	0.954	19.344	-8.101	12.962	-11.342	+/- 3.482
	q_h =	19.344				

These pressures can be compared to those obtained from the RAM Structural System output, shown below:

WIND PRESSURES:

X-Direction:		Natural Frequency = 0.352			Structure is Flexible						
Y-Direction:		Natural Frequency = 0.330			Structure is Flexible						
CpWindward = 0.80		qLeeward (qh) = 19.34 psf									
GCpn (Parapet):		Windward = 1.80			Leeward = -1.10						
Height	Kz	Kzt	qz	Gust Factor G		CpLeeward		Pressure (psf)			
ft			psf	X	Y	X	Y	X	Y		
88.50	0.954	1.000	19.344	0.889	0.896	-0.500	-0.500	22.366	22.536		
75.00	0.910	1.000	18.450	----	----	----	----	53.506	53.506		
70.50	0.894	1.000	18.127	0.855	0.850	-0.467	-0.500	20.123	20.554		
54.00	0.829	1.000	16.797	0.855	0.850	-0.467	-0.500	19.214	19.649		
36.00	0.738	1.000	14.960	0.855	0.850	-0.467	-0.500	17.958	18.399		
18.00	0.605	1.000	12.272	0.855	0.850	-0.467	-0.500	16.120	16.571		
0.00	0.575	1.000	11.649	0.855	0.850	-0.467	-0.500	15.694	16.147		

Sample Pressure Comparison @ Height 54 feet

North-South Wind:
P_{hand} = 8.097 psf + 11.250 psf = 19.347 psf in comparison to 19.649 psf

East-West Wind:
P_{hand} = 8.101 psf + 11.256 psf = 19.357 psf in comparison to 19.214 psf

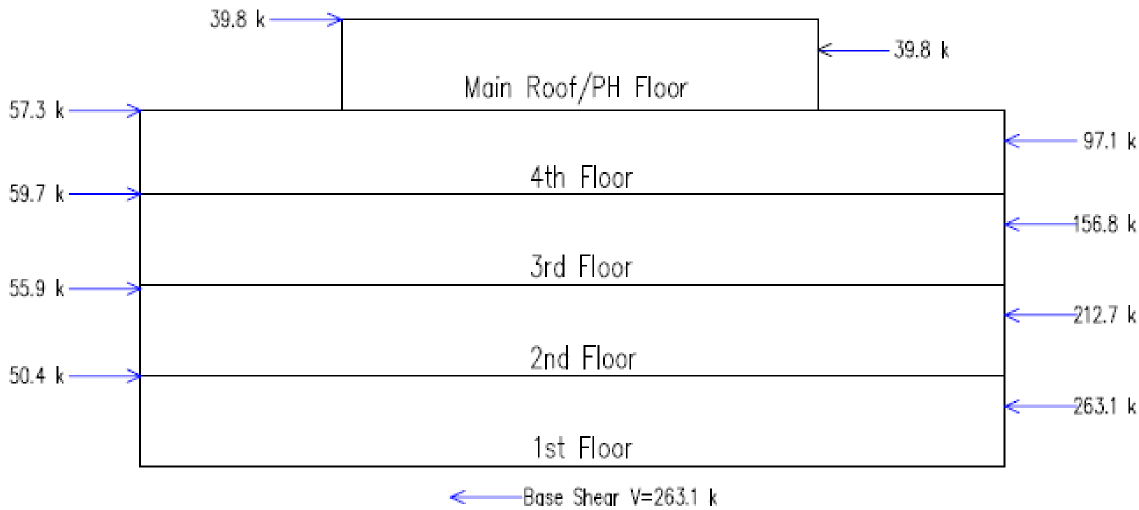
It can be seen that the RAM calculated pressures are very similar to those I calculated by hand, with the only difference seeming to be a slight variation in the gust factor. Similar to my assumption, RAM calculated comparable pressures in both wind loading directions. Based on these results, I will assume the story forces and shears from the RAM output to be correct and the RAM model to accurately demonstrate the wind loading condition.

The pressure at 75 feet is that at the parapet of the main roof. I did not calculate this in my hand calculations, so I have no pressure value to compare it to. However, it seems to be reasonable based on a quick calculation using the ASCE 7-05 equation for wind pressures at parapets.

Wind Story Forces and Story Shears:

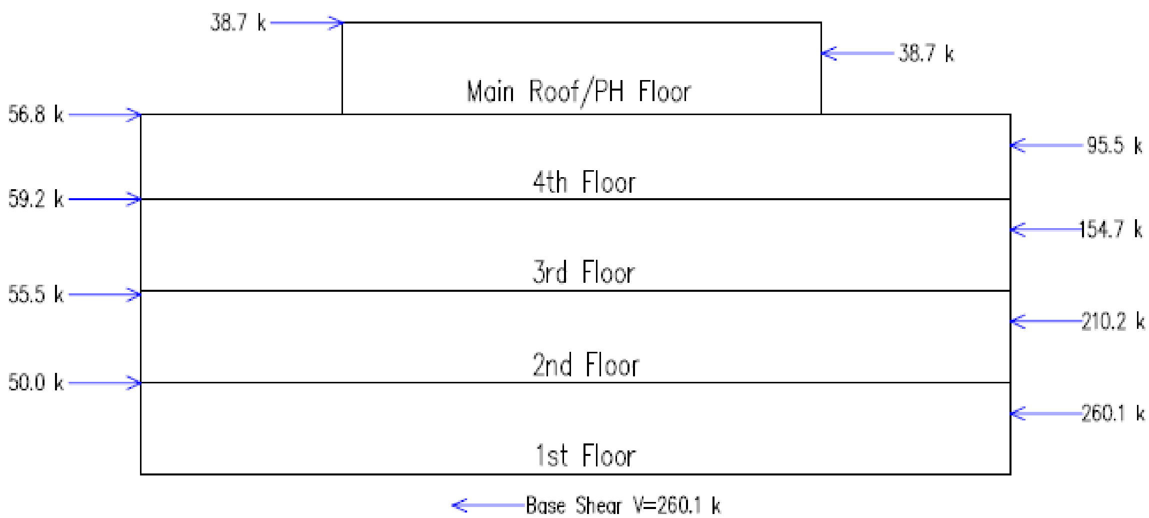
Based on the pressures calculated above, I calculated the story forces and shears. For simplification of hand calculations, the wind pressures were multiplied by the projected lengths of the building. The following story forces and shears were found:

North-South Wind:



NORTH–SOUTH WIND

East-West Wind:



EAST–WEST WIND

The story shears and forces were also obtained using the RAM model. RAM calculates the applied forces at each level for many load cases. The following results were found to control:

North-South Direction:

APPLIED STORY FORCES

Type: Wind_IBC03_1_Y

Level	Ht ft	Fx kips	Fy kips
penthouse	88.50	0.00	19.42
roof	70.50	0.00	76.67
fourth	54.00	0.00	67.55
third floor	36.00	0.00	65.90
second floor	18.00	0.00	60.51
		0.00	290.06

East-West Direction:

APPLIED STORY FORCES

Type: Wind_IBC03_1_X

Level	Ht ft	Fx kips	Fy kips
penthouse	88.50	19.19	0.00
roof	70.50	64.32	0.00
fourth	54.00	56.79	0.00
third floor	36.00	55.30	0.00
second floor	18.00	50.63	0.00
		246.22	0.00

Overturning Moment: North-South

M_{OT} = 19.42 k*88.5' + 76.67 k*70.5' + 67.55 k*54' + 65.9 k*36' + 60.51 k*18' = 14,233 ft-k

Overturning Moment: East-West

M_{OT} = 19.19 k*88.5' + 64.32 k*70.5' + 56.79 k*54' + 55.3 k*36' + 50.63 k*18' = 12,202 ft-k

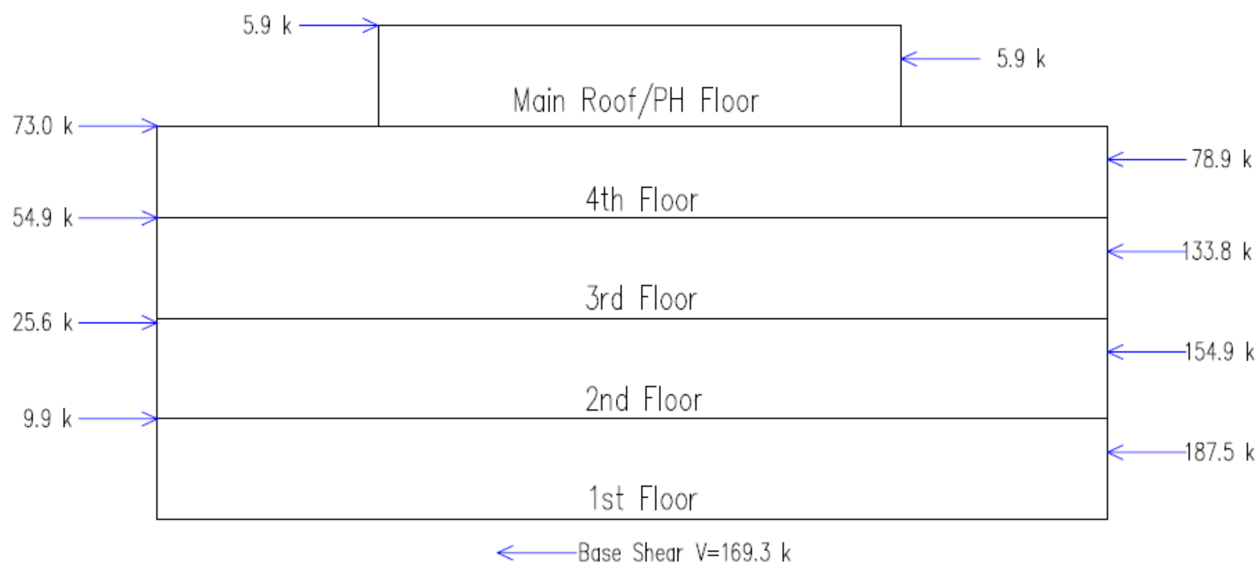
It can be concluded that the RAM model accurately models the wind loading condition. The maximum base shear due to wind is 290.06 k, which is fairly comparable to my hand calculated 263.3 k. The error is most likely due to the difference in gust factor and the actual wall area of applied wind pressure compared to my simplified calculations. This shear force will be compared to that of the seismic analysis to determine the design base shear. Overturning moment is considered for an individual frame in the member spot check calculations.

VII. SEISMIC ANALYSIS

Previously, in the loads portion of this report, the seismic variables obtained or assumed from ASCE 7-05 were listed. Based on these values, calculations for the following variables can be found in Appendix B:

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
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

The story shears and forces were then obtained, resulting in the following loading diagram:



SEISMIC LOADING

According to my hand calculations, wind will control over seismic, which makes sense considering Maryland is not in a high seismic zone. The RAM Model's applied forces verified this conclusion. The following forces were obtained from the RAM Structural System's output for the controlling seismic load case:

APPLIED STORY FORCES

Type: EQ_IBC03_X_+E_F

Level	Ht ft	Fx kips	Fy kips
penthouse	88.50	8.76	0.00
roof	70.50	45.87	0.00
fourth	54.00	37.43	0.00
third floor	36.00	19.49	0.00
second floor	18.00	6.39	0.00
		117.95	0.00

APPLIED STORY FORCES

Type: EQ_IBC03_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
penthouse	88.50	0.00	8.76
roof	70.50	0.00	45.87
fourth	54.00	0.00	37.43
third floor	36.00	0.00	19.49
second floor	18.00	0.00	6.39
		0.00	117.95

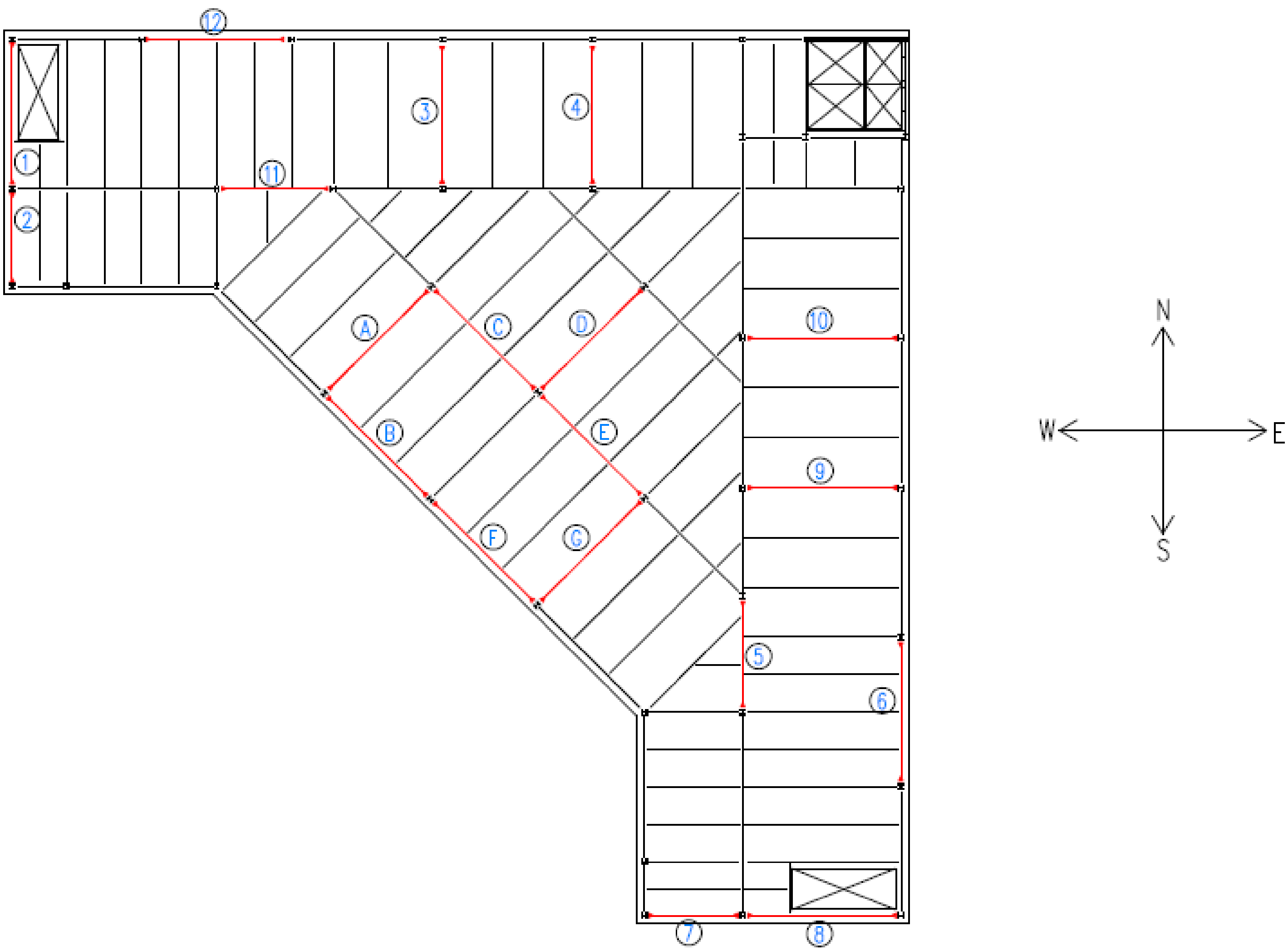
Overturning Moment:

M_{OT} = 8.76 K*88.5' + 45.87 k*70.5' + 37.43 k*54' + 19.49 k*36' + 6.39 k*18' = 6847 ft-k

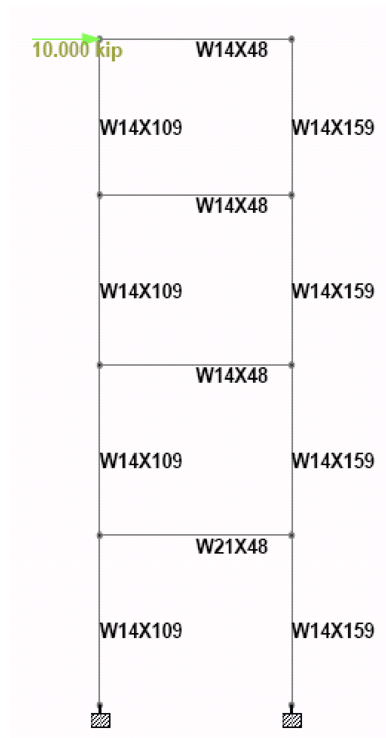
It can be seen that these forces are considerably less than those I calculated. According to the RAM Output, the building weight was calculated to be 6081.0 k compared to my 8637.2 k. This is most likely attributed to the fact that I ignored the slab depressions for the showers and ignored any floor openings. Therefore, the seismic forces calculated by the program are probably more exact. Regardless of this discrepancy, wind will still control in terms of base shear and overturning moment, so no further investigation or verification of seismic loading is necessary.

VIII. 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 which resist lateral load in both directions. The lateral elements are symmetrical about the building axis and seem to be fairly evenly spaced throughout the building. The moment frames are shown in the diagram below, numbered 1 through 6 for the North-South frames, 7 through 12 for the East-West frames, and A through G for the frames at a 45 degree angle.



Each moment frame was modeled in STAAD and subject to a 10 kip load at each level to determine the frame deflection. The inverse of this deflection was then taken as the relative stiffness for the frame at that level, by which the distribution of lateral forces could be calculated. These values could then be compared to the distribution of lateral forces modeled by RAM to verify a logical force distribution. Shown here is a sample of frame A modeled in STAAD, showing the load applied at the main roof level.



After modeling all 19 frames, distribution factors were calculated for each frame at each level. When calculating the distribution factors for the frames at a 45 degree angle, they were assumed to resist half the load that they would directly resist. In Appendix C, a spreadsheet is included that shows the method used to calculate the distribution factors based on relative stiffness. The distribution factor from RAM was obtained by dividing the load in a given frame at a given story by the total building shear at that level. Below are tables comparing these two distribution factors in each direction.

	Distribution Factors (North-South)							
	Calculated by Hand/STAAD				**Calculated by RAM**			
Frame #	Roof	4th	3rd	2nd	Roof	4th	3rd	2nd
1	0.075	0.079	0.090	0.096	0.078	0.067	0.068	0.069
2	0.088	0.092	0.092	0.094	0.078	0.081	0.081	0.076
3	0.084	0.083	0.079	0.077	0.084	0.099	0.099	0.090
4	0.085	0.085	0.082	0.080	0.090	0.101	0.106	0.097
5	0.123	0.125	0.122	0.109	0.136	0.137	0.176	0.150
6	0.117	0.117	0.107	0.102	0.127	0.176	0.087	0.070
A	0.052	0.054	0.061	0.066	0.037	0.034	0.043	0.061
B	0.063	0.061	0.058	0.056	0.072	0.051	0.057	0.058
C	0.079	0.075	0.073	0.074	0.062	0.060	0.068	0.071
D	0.043	0.045	0.046	0.050	0.041	0.034	0.036	0.053
E	0.075	0.070	0.071	0.074	0.072	0.059	0.070	0.075
F	0.063	0.061	0.058	0.056	0.074	0.068	0.060	0.061
G	0.052	0.054	0.061	0.066	0.049	0.034	0.048	0.069
Total	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

	Distribution Factors (East-West)							
	Calculated by Hand/STAAD				Calculated by RAM			
Frame #	**Roof**	**4th**	**3rd**	**2nd**	**Roof**	**4th**	**3rd**	**2nd**
7	0.090	0.093	0.092	0.094	0.102	0.102	0.109	0.100
8	0.073	0.078	0.088	0.097	0.100	0.083	0.084	0.087
9	0.086	0.086	0.085	0.081	0.073	0.105	0.104	0.095
10	0.089	0.090	0.090	0.089	0.083	0.099	0.096	0.095
11	0.111	0.108	0.099	0.087	0.124	0.132	0.119	0.095
12	0.117	0.120	0.118	0.107	0.113	0.124	0.126	0.105
A	0.053	0.055	0.061	0.066	0.048	0.035	0.040	0.057
B	0.064	0.062	0.058	0.056	0.077	0.057	0.057	0.056
C	0.080	0.076	0.073	0.075	0.067	0.061	0.062	0.065
D	0.044	0.045	0.046	0.051	0.043	0.037	0.034	0.049
E	0.076	0.071	0.071	0.075	0.055	0.065	0.064	0.069
F	0.064	0.062	0.058	0.056	0.079	0.059	0.060	0.060
G	0.053	0.055	0.061	0.066	0.037	0.040	0.046	0.066
Total	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

It can be seen that there is, in most cases, an error of about 10-20% between the hand calculated and RAM obtained distribution factors. For some frames, the discrepancy is larger, however this could be due to a number of explanations. For instance, in the North-South direction there are 4 frames in the "left wing" of the building (frames 1, 2, 3, and 4) and 2 frames in the "south wing" (frames 5 and 6). The opposite is true for the East-West direction, with 2 frames in the "left wing" (frames 11 and 12) and 4 frames in the "south wing" (frames 7, 8, 9, and 10). It therefore makes sense that frames 5, 6, 11, and 12 would take more load, due to geometry. Also, the discrepancy between the distribution factors for the frames at a 45 degree angle is significantly larger than the frames directly resisting lateral loads. The hand calculated distribution factors for these frames were based on simplified assumptions, The RAM model demonstrates the actual, and more complicated, distribution of forces to the 45 degree angle frames, providing a more accurate result.

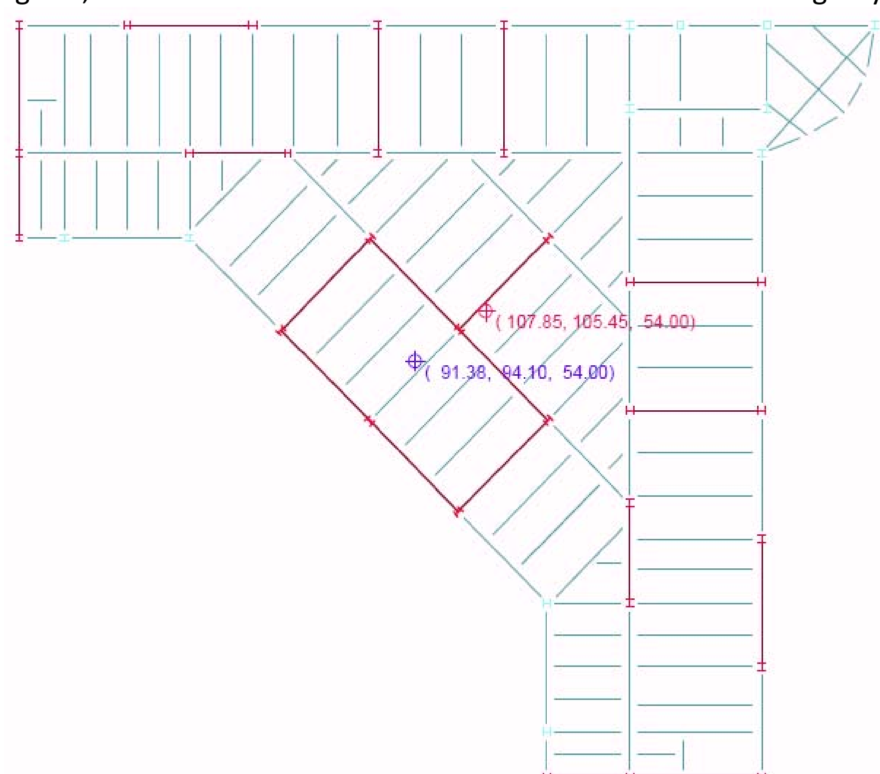
All of the above factors contribute to some discrepancy in the distribution factors, but overall my calculations seem to verify that RAM is logically distributing the lateral forces to the frames. Based on this result, RAM's forces are deemed to be more accurate and will be used later in the report for lateral member spot checks.

IX. TORSIONAL EFFECTS

Lateral loads create torsional effects on a building if the lateral resisting elements are not equally distributed throughout the building. The center of mass and rigidity must be calculated to determine if torsion from lateral forces has a significant effect on the building. These values, as well as the eccentricities were calculated using RAM Frame and are tabulated below.

	Centers of Rigidity (ft)		Centers of Mass (ft)		Eccentricities (ft)	
Level	X_R	Y_R	X_m	Y_m	E_X	E_Y
PH	100	106.05	102.21	102.23	2.21	3.82
Roof	92.5	93.84	106.82	105.45	14.32	11.61
4th	92.39	94.11	107.85	105.45	15.46	11.34
3rd	91.38	94.1	107.85	105.45	16.47	11.35
2nd	92.72	93.85	107.85	105.45	15.13	11.6
1st	109.4	108.04	109.4	108.04	0	0

For the second floor through the roof, the centers of mass and rigidity are virtually the same. In this diagram, the center of mass is shown in blue and the center of rigidity in red.



Once the centers of mass and rigidity were determined, it could be seen that the eccentricities were large enough to create considerable torsion effects. Therefore, I calculated the torsional shear on each frame at each floor. Wind load was used rather than seismic since it controlled in both directions. A representative spreadsheet of torsional effects at the 3rd floor in the North-South direction is shown below. Torsional shear calculations for the other stories can be found in Appendix D. The rigidities used in this spreadsheet were the same as those calculated by STAAD by using 1/deflection. Once again, the frames at a 45 degree angle were assumed to be half as rigid since they are not directly resisting the load. The story shears used were those calculated by RAM, which were very similar to the ones calculated by hand.

3rd FLOOR: North-South

	X	Y
C of Mass	107.85	105.45
C of Rigid	91.38	94.1
e =	16.47	11.35

Frame #	R	x_i	d_i	$R*d_i^2$	e	V (k)	Torsional Shear (k)
1	1.399	0	91.38	11682	16.47	55.3	2.79
2	1.435	0	91.38	11983	16.47	55.3	2.87
3	1.239	83	8.38	87.008	16.47	55.3	0.23
4	1.276	112	20.62	542.54	16.47	55.3	0.58
5	1.912	141	49.62	4707.6	16.47	55.3	2.07
6	1.667	171.5	80.12	10701	16.47	55.3	2.92
A	0.956	70.85	20.53	402.94	16.47	55.3	0.43
B	0.906	70.74	20.64	385.96	16.47	55.3	0.41
C	1.139	91.48	0.1	0.0114	16.47	55.3	0.00
D	0.7195	112.02	20.64	306.51	16.47	55.3	0.32
E	1.104	111.98	20.6	468.49	16.47	55.3	0.50
F	0.906	91.24	0.14	0.0178	16.47	55.3	0.00
G	0.956	111.86	20.48	400.98	16.47	55.3	0.43
			J =	41668			

It can be seen that torsional shear for the frames far away from the center of rigidity are approximately 2 to 3 kips, and torsional shear for those closer to the center of rigidity are much smaller, as expected. Therefore, these values are assumed to be satisfactorily accurate.

X. DRIFT

Wind Drift:

Drift is a serviceability requirement rather than a strength requirement, therefore it is not prescribed in the code. However, standard engineering practice over the years has employed a total building drift limit of H/400. Upon discussing this with practicing engineers, many have said that the 50 year wind, used per the code for strength requirements, is overly stringent for drift requirements. Therefore, current practice is evolving to use a reduced 10 year wind speed for drift calculations. Rather than recalculate the wind pressures based on this reduced wind speed, the conversion is that H/284 for a 50 year wind is equal to H/400 for a 10 year wind. Therefore my drift requirements will be compared to an H/284 value for the wind loads previously calculated.

Below is a spreadsheet summarizing drift due to unfactored wind loading, obtained from the RAM model. These are the total drift values at the center of mass at each level, from which story drift was also calculated. The governing wind force was in the North-South direction for all stories.

	Story Height (ft)	Total Drift (in)	Allowable H/284 (in)	Floor to Floor Height (ft)	Inner Story Displacement (in)	Allowable H_{story}/284 (in)
PH Roof	88.5	6.11	3.74	18	0.99	0.76
Main Roof	70.5	5.12	2.98	16.5	0.86	0.70
4th Floor	54	4.26	2.28	18	1.32	0.76
3rd Floor	36	2.94	1.52	18	1.64	0.76
2nd Floor	18	1.3	0.76	18	1.3	0.76

It can be seen that neither overall building drift nor inner story drift are up to the common engineering standard of H/400, even for the reduced 10 year wind speed. I verified my results with the structural engineer and no errors could be found, so this is being further examined by the engineer. This may be something to later address in my proposal.

Seismic Drift:

Seismic drift, unlike wind drift, is addressed in the code. Section 12.12 of ASCE 7-05 specifies the maximum allowable drift, story to story, based on the table below.

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a{}^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$\boxed{0.015h_{sx}}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force–resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

For this addition, a hospital without surgical facilities, the occupancy category is III per Table 1-1 of ASCE 7-05. Therefore, the maximum allowable drift for a steel moment frames, which fall under the "all other structures" category, is $0.015h_{sx}$ where h_{sx} is the story height below. Upon discussion with the structural engineer, I learned that during design, ASCE 7-05 equation 12.8-15 was used to convert the allowable drift from Table 12.12-1 into elastic drift ratio, then compared to the RAM output.

$\delta_x = (Cd*\delta_{xe})/I$
$0.015h_{sx} = (2.5*\delta_{xe})/1.25$
drift ratio $= \delta_{xe}/h_{sx} = 0.015*1.25/2.5 = 0.0075$

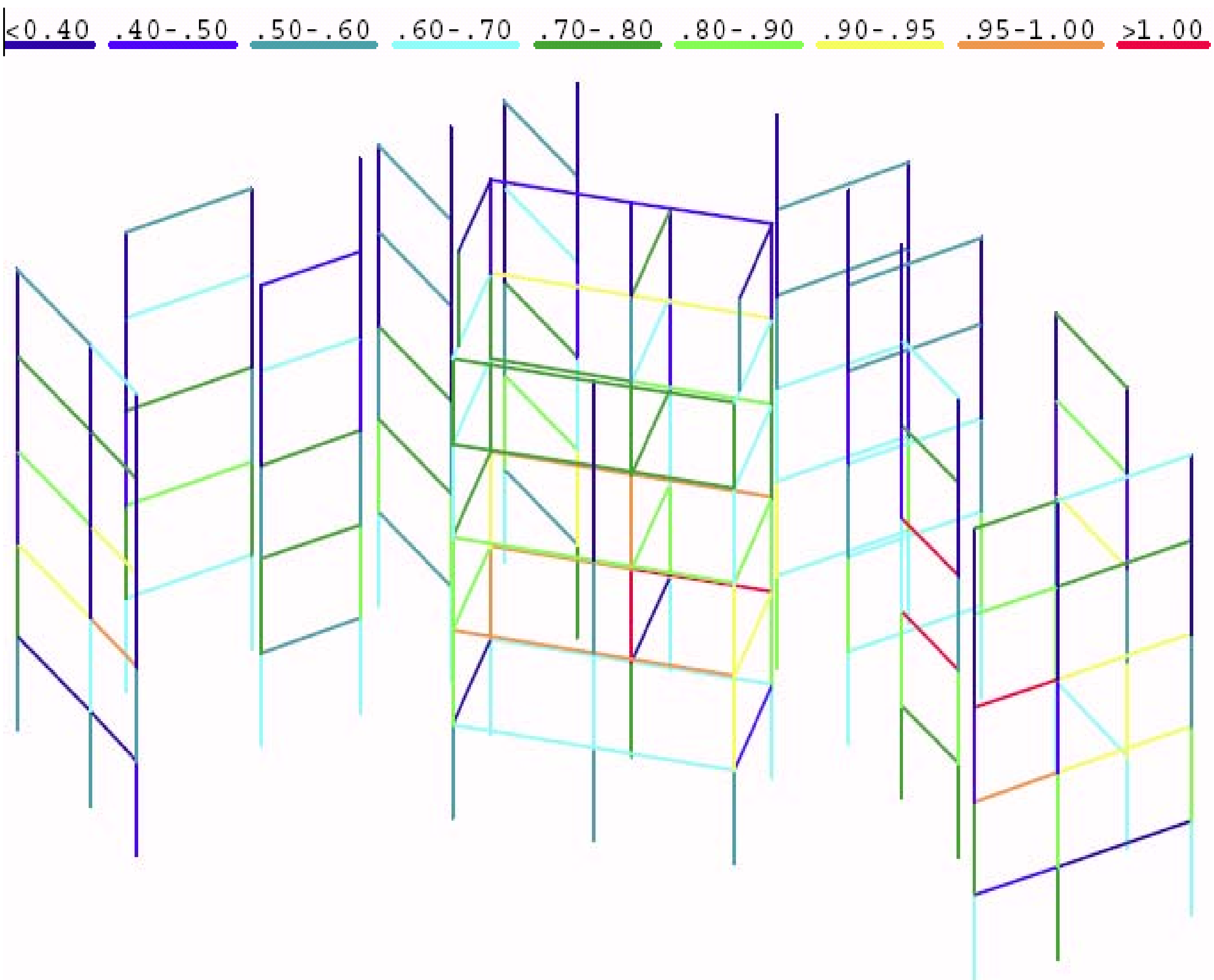
The spreadsheet below was then created with the seismic drift output from RAM. It can be concluded that seismic drift is not an issue.

	Story Height (ft)	Floor to Floor Height (ft)	Total Drift (in)	Inner Story Displacement (in)	Actual Drift Ratio	Allowable Drift Ratio
PH Roof	88.5	18	1.72	0.29	0.0013	0.0075
Main Roof	70.5	16.5	1.43	0.29	0.0015	0.0075
4th Floor	54	18	1.14	0.41	0.0019	0.0075
3rd Floor	36	18	0.73	0.43	0.0020	0.0075
2nd Floor	18	18	0.3	0.30	0.0014	0.0075

XI. MEMBER CHECKS

RAM Member Code Check:

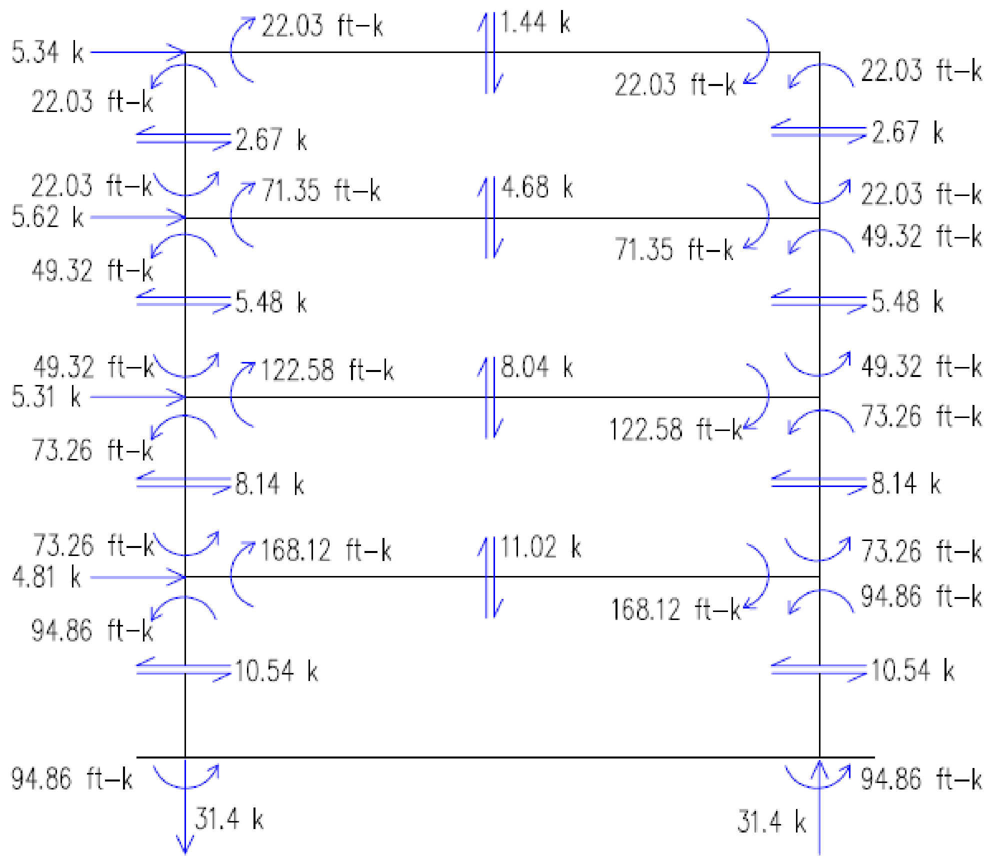
The diagram below shows an IBC 2003 LRFD code check of the lateral members from the RAM model for all load combinations. The colors represent the worst case value from the interaction equation for gravity and lateral loads, analyzed for all load combinations.



It can be seen that most members are stressed to under 70 percent of their capacity, but a few members are stressed higher. In fact, five members are colored red, which means they are overstressed. However, upon further investigation, it was found that these members produce an interaction equation value of 1.01 or 1.02, meaning they are only 1 or 2 percent overstressed. Therefore, this is not of a huge concern.

Member Spot Checks: Frame 10

Portal Analysis for 1.0W:



The first member being checked is the beam at the second floor of Frame 10, which is a W16x67. Please see Appendix _ for detailed calculations.

M_U = 359.7 ft-k

Compare to Moment from RAM: M_{max} = 337.1 ft-k

The W16x67 has plenty of capacity to carry this moment. Even without composite action the beam can resist a 488 ft-k moment. The composite action will only provide additional strength. The purpose of this spot check was to verify the results from RAM, so deflection was not checked. However, it is assumed that for a 30.5 foot span deflection controlled, requiring a larger section than necessary for flexure. Therefore, this beam is most likely not overdesigned.

These hand calculations verify the RAM calculated member loads for this beam. It is therefore assumed that the member code check shown prior is accurate for lateral beams, hence member sizes are considered adequate.

Next, a typical column will be checked. I chose to check the exterior column in Frame 10, which is a W14x109 for the whole building height. This column will be checked at the base, where the highest loading will occur. For detailed calculations, refer to Appendix _.

Check Column at Base:

P_U = 417.2 k
M_U = 197.1 ft-k

Compare to Loads from RAM: P_{max} = 415.0 k
M_{max} = 182.75 ft-k

From Column Schedule, Column is a W14x109
$KL = L_b$ = 18'

From Table 6-1 for Combined Axial and Bending,
$p*P_u + b_x*M_{ux} = (0.886e\text{-}3*417.2) + (1.30e\text{-}3*182.75) = 0.370 + 0.237 = 0.607$
0.607 < 1 therefore column is OK

It can be seen that this column is stressed to approximately 60 percent of it's capacity. The hand calculated loads were very similar to those obtained from RAM, further verifying the accuracy of the model. The member code check is now assumed to be accurate for lateral force resisting columns as well, meaning the columns are adequately sized.

XII. CONCLUSIONS

The RAM Model proved to be a very effective tool for analyzing lateral load conditions. It is important when utilizing computer program to not blindly accept the results. This report showed a variety of hand calculations through which the RAM results were verified. I feel confident that the program accurately modeled both loading conditions and member force results.

When comparing wind and seismic loads, wind controlled in both directions. Considering the location of this building, this result was expected. The base shears calculated by hand and by RAM were very similar, which was encouraging. The two directions of wind loading had similar results, as expected, considering the projected building lengths are almost equal.

The largest discrepancy between my hand calculations and RAM results occurred with the lateral load distribution factors. My hand calculated values utilized the frame deflection calculated in STAAD from a 10 kip load. This simplified model provided values of relative stiffness by which the loads could be distributed. However, the RAM model distributed the loads more accurately taking other factors like building geometry into account. Also, estimating load taken by the frames rotated at a 45 degree angle proved to be a challenge. Still, the distribution factors demonstrated similar trends in terms of which frames take the most load and which take the least. The RAM distribution factors are deemed to be more accurate since it avoids simplified assumptions like those used for hand calculations.

Due to an eccentricity between the center of mass and center of rigidity, torsional shear values of 0 to 3 kips were calculated for most lateral frames. Compared to the other loads, these values were small and not much of an issue.

Wind drift proved to be an issue as my results did not meet the engineering standard of practice. However, this is a serviceability requirement and therefore the building is still up to code. Drift will be addressed next semester and great attempts will be made to reduce it.

Overall, I feel that this report verified the building’s lateral system design. The only area of concern is the wind drift issue. Otherwise, in terms of strength and stresses, I feel confident that this building is well designed and my hand calculations validate the use of the RAM model as a design aid.

APPENDIX A – 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 =	374.98	Eqn 6-7
$\square_1$ =	0.989	$1/T_a$
N_1 =	5.544	Eqn 6-12
β =	0.010	Section 6.5.8.2

Calculate Gust Factor				
Variable	**Value**	**Fig/Table/Eqn**		
gQ =	3.4	Given		
gV =	3.4	Given		
gR =	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 =	0.048	Eqn 6-11		
I_z =	0.277	Eqn 6-5		
Q =	0.816	Eqn 6-6		
R =	0.179	Eqn 6-10		
G_f =	0.837	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 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 =	0.837					
z (ft)	**K_z**	**q_z**	**P (leeward)**	**P (windward)**	**P (sidewall)**	**+/- qGC_{pi}**
0-18	0.605	12.272	-8.097	8.219	-11.336	+/- 3.482
36	0.738	14.960	-8.097	10.019	-11.336	+/- 3.482
54	0.829	16.797	-8.097	11.250	-11.336	+/- 3.482
70.5	0.894	18.127	-8.097	12.141	-11.336	+/- 3.482
88.5	0.954	19.344	-8.097	12.956	-11.336	+/- 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	-14.850
C_p =	-0.892	Figure 6-6	44.25 to 88.5 ft	-14.445
C_p =	-0.508	Figure 6-6	88.5 to 170 ft	-8.227

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*0.837*-0.5) +/- (19.344*0.18) = -8.095 +/- 3.482

P (windward) = $q_zG_fC_p$ +/- q_hGC_{pi} = (16.797*0.837*0.8) +/- (19.344*0.18) = 11.247 +/- 3.482

P (sidewall) = $q_hG_fC_p$ +/- q_hGC_{pi} = (19.344*0.837*-0.7) +/- (19.344*0.18) = -11.334 +/- 3.482

It can be seen that these values are approximately equal to those calculated in the spreadsheet, which just a slight variation due to rounding error. This same calculation is performed in the spreadsheet at each height "z" for 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 =	374.98	Eqn 6-7
n_1 =	0.989	$1/T_a$
N_1 =	5.544	Eqn 6-12
β =	0.010	Section 6.5.8.2

Calculate Gust Factor				
Variable	**Value**	**Fig/Table/Eqn**		
gQ =	3.4	Given		
gV =	3.4	Given		
gR =	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 =	0.048	Eqn 6-11		
I_z =	0.277	Eqn 6-5		
Q =	0.817	Eqn 6-6		
R =	0.180	Eqn 6-10		
G_f =	0.838	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; $q_i = q_h$ for windward, leeward, sidewall, and roof)

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 =	0.838					
z (ft)	**K_z**	**q_z**	**P (leeward)**	**P (windward)**	**P (sidewall)**	**+/- qGC_{pi}**
0-18	0.605	12.272	-8.101	8.223	-11.342	+/- 3.482
36	0.738	14.960	-8.101	10.024	-11.342	+/- 3.482
54	0.829	16.797	-8.101	11.256	-11.342	+/- 3.482
70.5	0.894	18.127	-8.101	12.147	-11.342	+/- 3.482
88.5	0.954	19.344	-8.101	12.962	-11.342	+/- 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 wind 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	39.8	39.8	1836	38.7	38.7
Roof/PH Flr	2830	57.3	97.1	2805	56.8	95.5
4th Floor	3087	59.7	156.8	3060	59.2	154.7
3rd Floor	3087	55.9	212.7	3060	55.5	210.2
2nd Floor	3087	50.4	263.1	3060	50.0	260.1
1st/Base	-	-	263.1	-	-	260.1

APPENDIX B – 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
$0.167 \leq S_{DS} < 0.33$ and $0.067 \leq S_{D1} < 0.133$
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
$S_{D1} \leq 0.1$ 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):

1st 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

2nd 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

3rd 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

4th 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 C – LATERAL LOAD DISTRIBUTION

STAAD Model Displacements and Calculated Stiffness (1/Displacement):

	Displacement				Stiffness			
Frame #	**Roof**	**4th**	**3rd**	**2nd**	**Roof**	**4th**	**3rd**	**2nd**
1	2.621	1.589	0.715	0.176	0.382	0.629	1.399	5.682
2	2.210	1.378	0.697	0.181	0.452	0.726	1.435	5.525
3	2.328	1.522	0.807	0.221	0.430	0.657	1.239	4.525
4	2.291	1.490	0.784	0.212	0.436	0.671	1.276	4.717
5	1.595	1.013	0.523	0.155	0.627	0.987	1.912	6.452
6	1.666	1.080	0.600	0.166	0.600	0.926	1.667	6.024
7	2.210	1.379	0.697	0.182	0.452	0.725	1.435	5.495
8	2.714	1.634	0.731	0.177	0.368	0.612	1.368	5.650
9	2.310	1.477	0.758	0.210	0.433	0.677	1.319	4.762
10	2.241	1.416	0.716	0.193	0.446	0.706	1.397	5.181
11	1.793	1.186	0.650	0.197	0.558	0.843	1.538	5.076
12	1.705	1.062	0.546	0.159	0.587	0.942	1.832	6.289
A	1.875	1.158	0.523	0.129	0.533	0.864	1.912	7.752
B	1.545	1.037	0.552	0.152	0.647	0.964	1.812	6.579
C	1.245	0.845	0.439	0.114	0.803	1.183	2.278	8.772
D	2.249	1.409	0.695	0.168	0.445	0.710	1.439	5.952
E	1.309	0.899	0.453	0.114	0.764	1.112	2.208	8.772
F	1.545	1.037	0.552	0.152	0.647	0.964	1.812	6.579
G	1.875	1.158	0.523	0.129	0.533	0.864	1.912	7.752
				N-S Total	5.114	7.927	15.612	59.003
				E-W Total	5.031	7.836	15.574	58.532

Distribution Factor Comparisons:

	Distribution Factors (North-South)							
	Calculated by Hand/STAAD				Calculated by RAM			
Frame #	Roof	4th	3rd	2nd	Roof	4th	3rd	2nd
1	0.075	0.079	0.090	0.096	0.078	0.067	0.068	0.069
2	0.088	0.092	0.092	0.094	0.078	0.081	0.081	0.076
3	0.084	0.083	0.079	0.077	0.084	0.099	0.099	0.090
4	0.085	0.085	0.082	0.080	0.090	0.101	0.106	0.097
5	0.123	0.125	0.122	0.109	0.136	0.137	0.176	0.150
6	0.117	0.117	0.107	0.102	0.127	0.176	0.087	0.070
A	0.052	0.054	0.061	0.066	0.037	0.034	0.043	0.061
B	0.063	0.061	0.058	0.056	0.072	0.051	0.057	0.058
C	0.079	0.075	0.073	0.074	0.062	0.060	0.068	0.071
D	0.043	0.045	0.046	0.050	0.041	0.034	0.036	0.053
E	0.075	0.070	0.071	0.074	0.072	0.059	0.070	0.075
F	0.063	0.061	0.058	0.056	0.074	0.068	0.060	0.061
G	0.052	0.054	0.061	0.066	0.049	0.034	0.048	0.069
Total	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000

	Distribution Factors (East-West)							
	Calculated by Hand/STAAD				Calculated by RAM			
Frame #	Roof	4th	3rd	2nd	Roof	4th	3rd	2nd
7	0.090	0.093	0.092	0.094	0.102	0.102	0.109	0.100
8	0.073	0.078	0.088	0.097	0.100	0.083	0.084	0.087
9	0.086	0.086	0.085	0.081	0.073	0.105	0.104	0.095
10	0.089	0.090	0.090	0.089	0.083	0.099	0.096	0.095
11	0.111	0.108	0.099	0.087	0.124	0.132	0.119	0.095
12	0.117	0.120	0.118	0.107	0.113	0.124	0.126	0.105
A	0.053	0.055	0.061	0.066	0.048	0.035	0.040	0.057
B	1.000	0.062	0.058	0.056	0.077	0.057	0.057	0.056
C	0.080	0.076	0.073	0.075	0.067	0.061	0.062	0.065
D	0.044	0.045	0.046	0.051	0.043	0.037	0.034	0.049
E	0.076	0.071	0.071	0.075	0.055	0.065	0.064	0.069
F	0.064	0.062	0.058	0.056	0.079	0.059	0.060	0.060
G	0.053	0.055	0.061	0.066	0.037	0.040	0.046	0.066
Total	1.000	1.000	1.000	1.000	1.000	1.000	1.000	

APPENDIX D – TORSION

MAIN ROOF: North-South

	X	Y					
C of Mass	106.82	105.45					
C of Rigid	92.5	93.84					
e =	14.32	11.61					**Torsional**
Frame #	**R**	**x_i**	**d_i**	**$R*d_i^2$**	**e**	**V (k)**	**Shear (k)**
1	0.382	0	92.5	3268.5	14.32	64.32	2.47
2	0.452	0	92.5	3867.4	14.32	64.32	2.92
3	0.43	83	9.5	38.808	14.32	64.32	0.29
4	0.436	112	19.5	165.79	14.32	64.32	0.59
5	0.627	141	48.5	1474.9	14.32	64.32	2.13
6	0.6	171.5	79	3744.6	14.32	64.32	3.32
A	0.2665	70.85	21.65	124.91	14.32	64.32	0.40
B	0.3235	70.74	21.76	153.18	14.32	64.32	0.49
C	0.4015	91.48	1.02	0.4177	14.32	64.32	0.03
D	0.2225	112.02	19.52	84.779	14.32	64.32	0.30
E	0.382	111.98	19.48	144.96	14.32	64.32	0.52
F	0.3235	91.24	1.26	0.5136	14.32	64.32	0.03
G	0.2665	111.86	19.36	99.887	14.32	64.32	0.36
			J =	13169			

MAIN ROOF: East-West

	X	Y					
C of Mass	106.82	105.45					
C of Rigid	92.5	93.84					
e =	14.32	11.61					**Torsional**
Frame #	**R**	**y_i**	**d_i**	**$R*d_i^2$**	**e**	**V (k)**	**Shear (k)**
7	0.452	0	93.84	3980.29	11.61	76.67	2.87
8	0.368	0	93.84	3240.59	11.61	76.67	2.33
9	0.433	83	10.84	50.8799	11.61	76.67	0.32
10	0.446	112	18.16	147.084	11.61	76.67	0.55
11	0.558	141	47.16	1241.03	11.61	76.67	1.78
12	0.587	170	76.16	3404.8	11.61	76.67	3.02
A	0.2665	111.49	17.65	83.0207	11.61	76.67	0.32
B	0.3235	90.87	2.97	2.85356	11.61	76.67	0.06
C	0.4015	111.61	17.77	126.783	11.61	76.67	0.48
D	0.2225	111.64	17.8	70.4969	11.61	76.67	0.27
E	0.382	91.1	2.74	2.8679	11.61	76.67	0.07
F	0.3235	70.36	23.48	178.349	11.61	76.67	0.51
G	0.2665	70.47	23.37	145.551	11.61	76.67	0.42
			J =	12674.6			

4th FLOOR: North-South

	X	Y
C of Mass	107.85	105.45
C of Rigid	92.39	94.11
e =	15.46	11.34

Frame #	R	x_i	d_i	$R*d_i^2$	e	V (k)	Torsional Shear (k)
1	0.629	0	92.39	5369.09	15.46	56.79	2.44
2	0.726	0	92.39	6197.07	15.46	56.79	2.81
3	0.657	83	9.39	57.9291	15.46	56.79	0.26
4	0.671	112	19.61	258.034	15.46	56.79	0.55
5	0.987	141	48.61	2332.21	15.46	56.79	2.01
6	0.926	171.5	79.11	5795.27	15.46	56.79	3.07
A	0.432	70.85	21.54	200.436	15.46	56.79	0.39
B	0.482	70.74	21.65	225.924	15.46	56.79	0.44
C	0.5915	91.48	0.91	0.48982	15.46	56.79	0.02
D	0.355	112.02	19.63	136.795	15.46	56.79	0.29
E	0.556	111.98	19.59	213.375	15.46	56.79	0.46
F	0.482	91.24	1.15	0.63745	15.46	56.79	0.02
G	0.432	111.86	19.47	163.763	15.46	56.79	0.35
			J =	20951			

4th FLOOR: East-West

	X	Y
C of Mass	107.85	105.45
C of Rigid	92.39	94.11
e =	15.46	11.34

Frame #	R	y_i	d_i	$R*d_i^2$	e	V (k)	Torsional Shear (k)
7	0.725	0	94.11	6421.102	11.34	67.55	2.49
8	0.612	0	94.11	5420.296	11.34	67.55	2.11
9	0.677	83	11.11	83.56353	11.34	67.55	0.28
10	0.706	112	17.89	225.9568	11.34	67.55	0.46
11	0.843	141	46.89	1853.481	11.34	67.55	1.45
12	0.942	170	75.89	5425.253	11.34	67.55	2.61
A	0.432	111.49	17.38	130.4918	11.34	67.55	0.27
B	0.482	90.87	3.24	5.059843	11.34	67.55	0.06
C	0.5915	111.61	17.5	181.1469	11.34	67.55	0.38
D	0.355	111.64	17.53	109.0918	11.34	67.55	0.23
E	0.556	91.1	3.01	5.037416	11.34	67.55	0.06
F	0.482	70.36	23.75	271.8781	11.34	67.55	0.42
G	0.432	70.47	23.64	241.423	11.34	67.55	0.37
			J =	20373.78			

3rd FLOOR: North-South

	X	Y					
C of Mass	107.85	105.45					
C of Rigid	91.38	94.1					
e =	16.47	11.35					**Torsional**
Frame #	**R**	**x_i**	**d_i**	**$R*d_i^2$**	**e**	**V (k)**	**Shear (k)**
1	1.399	0	91.38	11682.1	16.47	55.3	2.79
2	1.435	0	91.38	11982.7	16.47	55.3	2.87
3	1.239	83	8.38	87.008	16.47	55.3	0.23
4	1.276	112	20.62	542.535	16.47	55.3	0.58
5	1.912	141	49.62	4707.62	16.47	55.3	2.07
6	1.667	171.5	80.12	10700.8	16.47	55.3	2.92
A	0.956	70.85	20.53	402.936	16.47	55.3	0.43
B	0.906	70.74	20.64	385.965	16.47	55.3	0.41
C	1.139	91.48	0.1	0.01139	16.47	55.3	0.00
D	0.7195	112.02	20.64	306.514	16.47	55.3	0.32
E	1.104	111.98	20.6	468.493	16.47	55.3	0.50
F	0.906	91.24	0.14	0.01776	16.47	55.3	0.00
G	0.956	111.86	20.48	400.975	16.47	55.3	0.43
			J =	41667.7			

3rd FLOOR: East-West

	X	Y					
C of Mass	107.85	105.45					
C of Rigid	91.38	94.1					
e =	16.47	11.35					**Torsional**
Frame #	**R**	**y_i**	**d_i**	**$R*d_i^2$**	**e**	**V (k)**	**Shear (k)**
7	1.435	0	94.1	12706.65	11.35	65.9	2.42
8	1.368	0	94.1	12113.38	11.35	65.9	2.31
9	1.319	83	11.1	162.514	11.35	65.9	0.26
10	1.397	112	17.9	447.6128	11.35	65.9	0.45
11	1.538	141	46.9	3383	11.35	65.9	1.29
12	1.832	170	75.9	10553.8	11.35	65.9	2.50
A	0.956	111.49	17.39	289.106	11.35	65.9	0.30
B	0.906	90.87	3.23	9.452207	11.35	65.9	0.05
C	1.139	111.61	17.51	349.2175	11.35	65.9	0.36
D	0.7195	111.64	17.54	221.3553	11.35	65.9	0.23
E	1.104	91.1	3	9.936	11.35	65.9	0.06
F	0.906	70.36	23.74	510.6104	11.35	65.9	0.39
G	0.956	70.47	23.63	533.8083	11.35	65.9	0.41
			J =	41290.45			

2nd FLOOR: North-South

	X	Y
C of Mass	107.85	105.45
C of Rigid	92.72	93.85
e =	15.13	11.6

Frame #	R	x_i	d_i	$R*d_i^2$	e	V (k)	Torsional Shear (k)
1	5.682	0	92.72	48848.1	15.13	55.3	2.78
2	5.525	0	92.72	47498.4	15.13	55.3	2.70
3	4.525	83	9.72	427.515	15.13	55.3	0.23
4	4.717	112	19.28	1753.4	15.13	55.3	0.48
5	6.452	141	48.28	15039.3	15.13	55.3	1.64
6	6.024	171.5	78.78	37386.7	15.13	55.3	2.51
A	3.876	70.85	21.87	1853.88	15.13	55.3	0.45
B	3.2895	70.74	21.98	1589.22	15.13	55.3	0.38
C	4.386	91.48	1.24	6.74391	15.13	55.3	0.03
D	2.796	112.02	19.3	1041.48	15.13	55.3	0.28
E	4.386	111.98	19.26	1626.98	15.13	55.3	0.45
F	3.2895	91.24	1.48	7.20532	15.13	55.3	0.03
G	3.876	111.86	19.14	1419.93	15.13	55.3	0.39
			J =	158499			

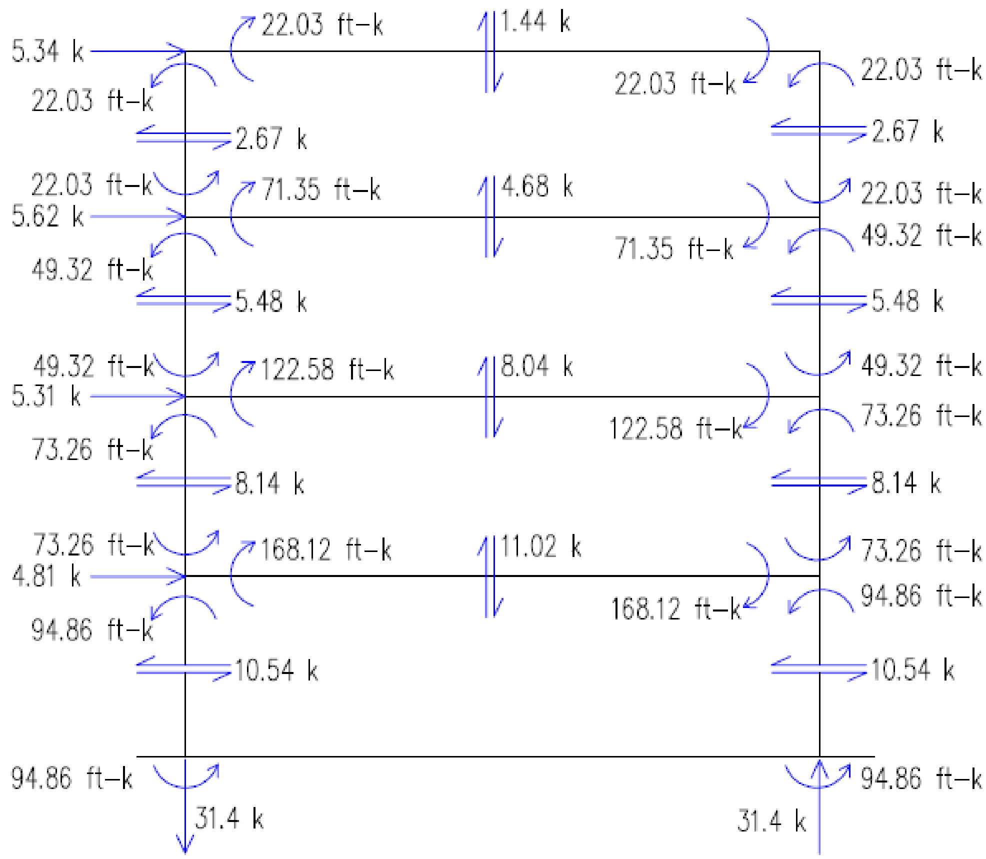
2nd FLOOR: East-West

	X	Y
C of Mass	107.85	105.45
C of Rigid	92.72	93.85
e =	15.13	11.6

Frame #	R	y_i	d_i	$R*d_i^2$	e	V (k)	Torsional Shear (k)
7	5.495	0	93.85	48398.98	11.6	65.9	2.49
8	5.65	0	93.85	49764.2	11.6	65.9	2.56
9	4.762	83	10.85	560.5945	11.6	65.9	0.25
10	5.181	112	18.15	1706.738	11.6	65.9	0.45
11	5.076	141	47.15	11284.57	11.6	65.9	1.15
12	6.289	170	76.15	36468.79	11.6	65.9	2.31
A	3.876	111.49	17.64	1206.093	11.6	65.9	0.33
B	3.2895	90.87	2.98	29.21208	11.6	65.9	0.05
C	4.386	111.61	17.76	1383.422	11.6	65.9	0.38
D	2.796	111.64	17.79	884.8895	11.6	65.9	0.24
E	4.386	91.1	2.75	33.16913	11.6	65.9	0.06
F	3.2895	70.36	23.49	1815.081	11.6	65.9	0.37
G	3.876	70.47	23.38	2118.716	11.6	65.9	0.44
			J =	155654.5			

APPENDIX E – FRAMING SPOT CHECK CALCULATIONS

Portal Analysis for 1.0W:



Check Beam at Second Floor:

Dead Load:
w_D = 65 psf*9.67' = 0.629 k/ft

Live Load:
w_L = 100 psf*9.67'*[0.25 + 15/(2*30.5*9.67)$^{1/2}$] = 0.839 k/ft

Factored Load:
$1.2w_D + 0.5w_L$ = 1.2*0.629 + 0.5*0.839 = 1.17 k/ft

Moments:
$M_{1.2D+0.5L}$ = (1.17*30.5^2)/12 = 90.7 ft-k
$M_{1.6W}$ = 1.6*168.12 = 269.0 ft-k
M_{max} = 90.7 + 269.0 = 359.7 ft-k

Compare to Moment from RAM: M_{max} = 337.1 ft-k

Check Column at Base:

Live Load:
A_T = 443 per floor*4 floors = 1329 SF
A_I = 4*1329 = 5316 SF
L_R = 100*[0.25 + 15/(5316)$^{1/2}$]= 45.6 > 0.4*100 = 40 therefore, use L_R = 45.6 psf
P_L = (4 floors*443 SF*45.6 psf) + (443 SF*30 psf) = 94.1 k

Dead Load:
Assume exterior average load of 60 psf
P_D = (5 floors*443 SF*65 psf) + (60 psf*29'*70.5') = 266.6 k

Wind Load:
P_W = 31.4 k from Overturning Moment

Factored Load:
P_U = 1.2*266.6 + 0.5*94.1 + 1.6*31.4 = 417.2 k

Moments:
Assume ½ of $M_{1.2D+0.5L}$ from Beam into Column Below = 45.35 ft-k
$M_{1.6W}$ = 1.6*94.86 = 151.78 ft-k
M_{max} = 45.35 + 151.78 = 197.1 ft-k

Compare to Loads from RAM: P_{max} = 415.0 k
M_{max} = 182.75 ft-k

From Column Schedule, Column is a W14x109
KL = L_b = 18'

From Table 6-1 for Combined Axial and Bending,
p*P_u + b_x*M_{ux} = (0.886e-3*417.2) + (1.30e-3*182.75) = 0.370 + 0.237 = 0.607
0.607 < 1 therefore column is OK