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Technical Report 3
Bed Tower Addition at Appleton Medical Center
Appleton, WI

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Senior Thesis Studio

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Table of Contents

Executive Summary

This technical report encompasses a wide range of subjects related to lateral system analysis. The focus of this report was to understand the importance of how a lateral system works. To accomplish this, the Bedtower Addition at Appleton Medical Center was analyzed.

Throughout the report, topics such as relative stiffness, center of rigidity, load cases, load combinations, and drifts were looked at closely. These topics are the basic essentials to a lateral system analysis.

During research, it was determined that certain load cases controlled under applied loads. Wind and seismic loads were closely compared. Once it was determined that wind pressures would control design, the overall controlling load combination was found. Through the use of RAM Structural, a computer modeling program, all cases and load combinations were analyzed. Sorting out which load combinations would affect the structure was a daunting task but it was easily helped by knowing wind would control. After applying load combinations to the structure, the controlling load combination ended up being: 1.2(Dead) + 1.0(Live) + (0.5)Snow + 1.6(Wind)

RAM Structural was also helpful in determining story drifts, displacements, and story shears. Drifts were calculated and compared to acceptable industry standards. Wind load drifts had to pass H/400 and seismic load drifts were to pass $0.01h_{sx}$. These calculations were based on serviceability, meaning unfactored loads were not taken into account. Displacements were used in determining the controlling load cases and combinations. Story shears were used when understanding the distribution of lateral forces to the braced frames.

Lateral member spot checks were also analyzed to determine if each member held the adequate strength capacity. All checked members were found to be acceptable by AISC standards. However, all members passed with more than enough strength concluding there might have been error in the hand calculations or computer modeling input.

In conclusion, the entire system was determined to resist lateral forces successfully.

Introduction

Bed Tower Addition at Appleton Medical Center, owned by ThedaCare is located in Appleton, Wisconsin approximately two hours (~106 miles) northeast from Madison, Wisconsin. The building was measured at a height of 107 ft and 3 in. above grade to the highest occupied floor, which entails 9 stories including a basement. The total size of the addition is 152,330 sq ft. This includes renovation done to the existing hospital plus the new addition itself.

Picture 1: *Bird's eye view of Appleton Medical Center*

Reason for the need of bedtower addition was to accommodate more patients for the hospital. Because of its size, it stands out amongst the rest of the complex. It has a unique triangular shape layout which is carried throughout all the floors of the building. The horizontal streaks of CMU along the exterior make the addition look very sleek and long. Accommodating the long streaks are large areas of glass. Both materials work together to show floor separation and this gives the perception that the addition is taller than it actually is.

The first floor is the lobby area which consists of the registration and waiting area along with a mini coffee shop. Offices are located on the second floor area which is a very large space and has movable partitions. Third through eighth floors consist of patient rooms, waiting rooms, and floor manager offices. The second to fourth floors connect to the

Picture 2: *Perspective view of Bed Tower Addition entrance*

original hospital with the fourth floor extended into the original building, which is the emergency and surgery center.

On the exterior of the building, the façade consisted of two essential components which are a stone façade and large areas of glazing. Limestone and Cast Stone, architectural concrete building unit used to simulate natural cut stone, make up the entire exterior. Limestone makes up the crown running along the bottom of building. Cast stone is what is seen throughout the rest of the exterior which makes up the vertical façade.

Picture 3: *Bed Tower Addition*

Glazing makes up the other half of the exterior. There are three kinds of glazing. They are: 1) Clear Vision Glass; 2) Tinted Vision Glass; and 3) Spandrel Glass. The clear vision glass is used on the first floor where the lobby is located to allow the most daylight and energy. The tinted vision glass and spandrel glass work together to shade the patient rooms and stairwells and they don't transmit as much sunlight or energy as the clear vision glass.

Structurally, the addition is made up of a system of steel framing and composite deck. The foundation is a mat padding. On top of the roof, there is a large penthouse which holds the mechanical equipment which is all supported by the steel framing of the building. For lateral loads, cross bracing is integrated within the frame.

Picture 4: *Construction of the addition*

Code

International Code

- 2006 International Building Code
    - Live load reduction used for typical floor loads and corridors above the first floor.

Design Codes

- ASTM International
    - Concrete and testing of masonry
- ACI 318-08
    - Reinforced concrete design and construction
- AISC 360-05
    - Structural steel - Designed for "in place" loads
- SDI
    - Steel roof decking
    - Steel composite floor deck - Designed as unshored
- OSHA Safety Standards
    - Steel erection
    - Steel joist erection
    - Metal decking erection
- ASCE 7-05
    - Wind loads

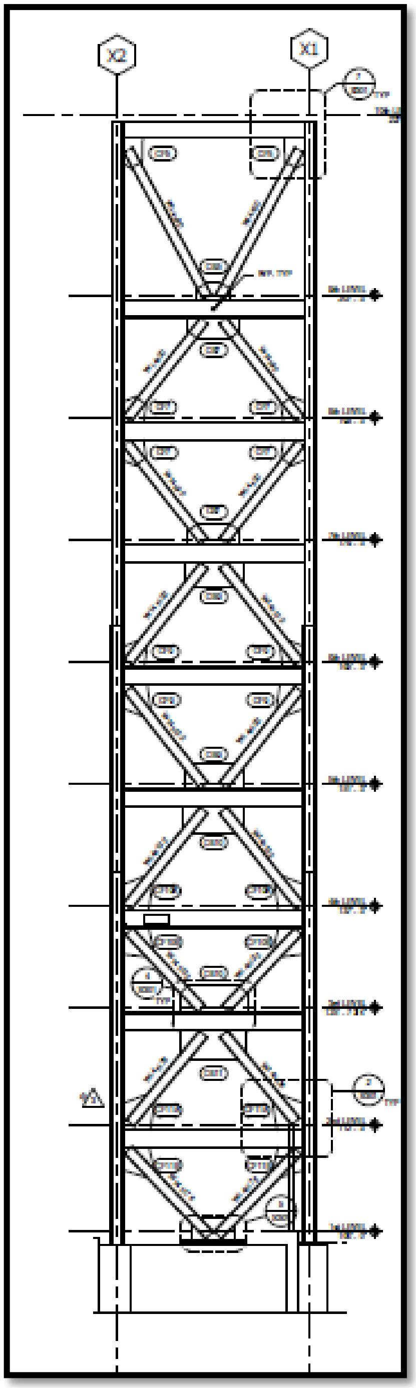


Figure 1: *Elevation of a braced frame system*

Courtesy of HGA

Structural System

The overall lateral system is a rigid frame with cross bracing. Rigid frames are commonly used when there is a need to provide unobstructed interior space with total adaptability. For the case of the Appleton Medical Center, a rigid frame was the best decision. It allowed the architects to create large spaces without being hindered by the structural system. This is very important because work space is more efficient and user friendly.

Bracing

Concentrically steel braced frames in each direction resist the lateral loads while the concrete slabs on metal deck act as the diaphragm which transfers the loads to the braced frames. There are 8 sections where the braced frames run vertically throughout the building. The typical frame runs from the top of the foundation to the top of the 9th level penthouse roof. Two others run to the top of the 9th level and the last one runs just between the 9th and 10th level. Shown on the previous page is a typical braced frame in Figure 1.

Connection to the mat foundation, explained later in the foundation section, help transfer the lateral loads to the base. The braced beams are connected to the columns and floor beams by gusset plates for ease of construction and transfer of loads. Close-up of the braced frames are pictured on the left in Figure 2.

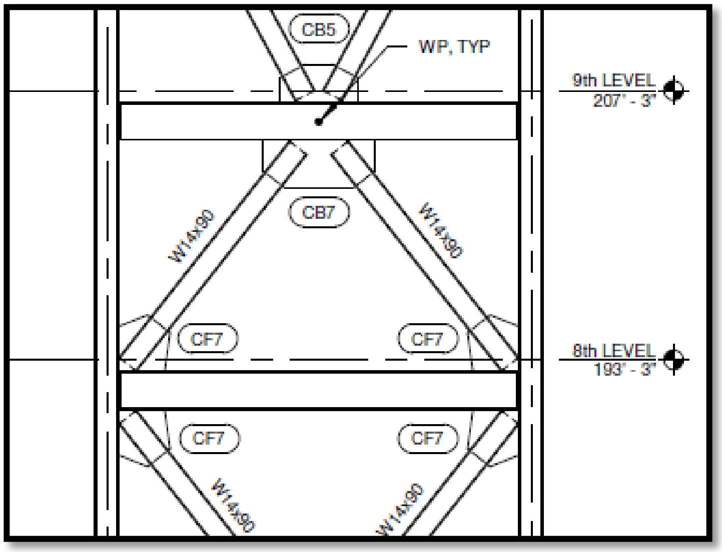


Figure 2: *Close-up of the braced frame system*

Figure 3 (Above): *Close-up of gusset plate construction for the braced frame*

To the right are construction photos of the gusset plates used and connection to the foundation for the braced frames in Figures 3 and 4, respectively.

All 3 Figures Courtesy of HGA

Figure 4: *Picture of a typical column connection to the foundation using a base plate*

Foundation

The geotechnical report was completed by River Valley Testing Corporation. Originally, the foundation was designed with spread footing in mind, but after investigation by RVT they recommended three alternatives which included the currently used mat foundation. Tests indicated that the natural soils on the site were able to hold bearing pressures ranging from 1,500 psf to more than 6,000 psf. The footings were then designed for a maximum soil bearing pressure of 3500 psf for just gravity loads and 4200 psf for gravity plus lateral loads. Spread footings range from 6 ft x 6 ft to 9 ft x 9 ft with depths being 1 to 2 ft. Maximum allowable interior column loads were to be 1,500 kips and the maximum allowable perimeter wall load was 3 kips per lineal foot.

Typical reinforcement for the mat slab includes the use of #7, #9, and #11 bars. The thickness of the mat slab is 3 ft 6 in. throughout the entire foundation under the triangular side of the addition. The area where the addition connects to the original part of the building has various thicknesses with 12 in. being the typical.

Most importantly, the braced frames are connected to the foundation to resist overturning moment. Typical thicknesses of these are 4 ft and run as long as the column spacing. Columns are connected to the bases by steel plates that are connected to the top of the concrete by 6 #6 hooks. The bases are reinforced by 5 #5 bars running horizontally and #5 bars running vertically spaced at 12 in. O.C. Pictured below in Figure 5 is a section and elevation of the braced frame to foundation connection with reinforcement.

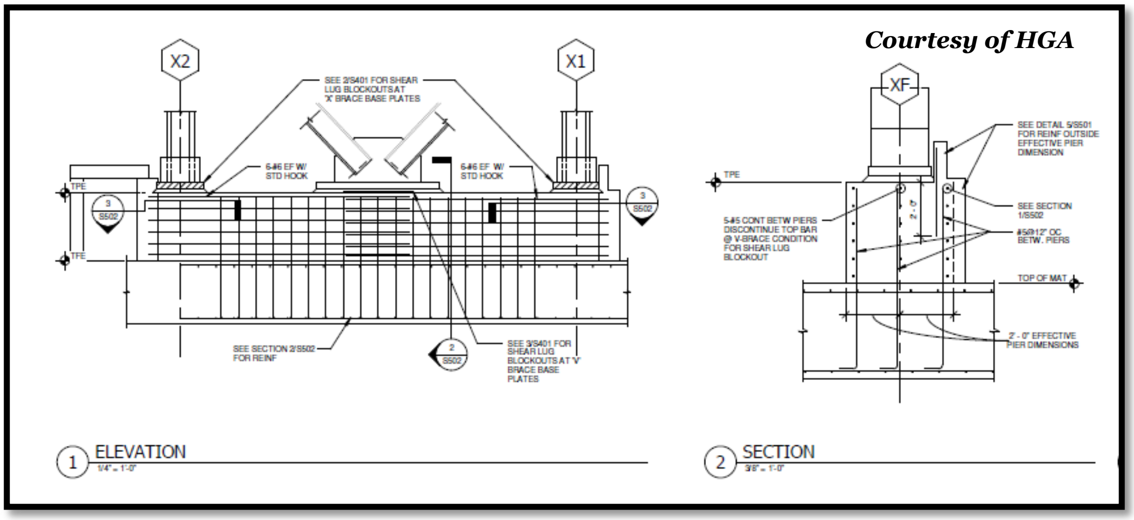


Figure 6 shows where the braced frames are connected at the foundation level in green. There is one more braced frame, but as stated earlier in the bracing section, this one is located on the top level.

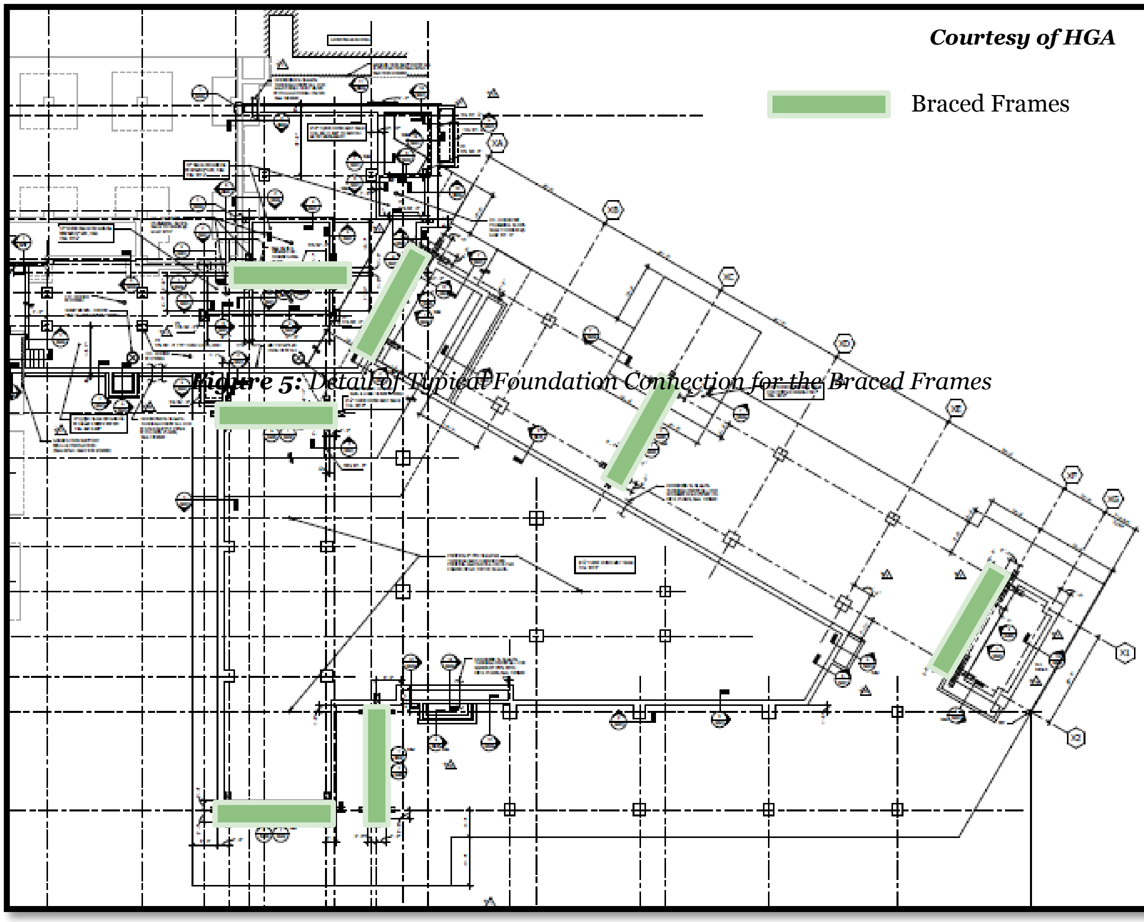


Figure 6: *Location of braced frames*

Floor Construction

Typical floor construction for the addition included the use of 4 types of "deck." Most floors were constructed of 3 in., 18 gage galvanized steel deck with a 4-½ in. normal weight concrete topping, making it a total thickness of 7-½ in. reinforced with 6x6 WWF. One floor was a combination of two decks. One "deck" was a 10 in. light weight concrete slab which was reinforced with #4 @ 18 in. O.C. running longitudinally. The other deck was a 2 in., 18 gage galvanized steel deck with a 3-½ in. light weight concrete topping making it a total thickness of 5-½ in. and reinforced with 6x6 WWF. Both the galvanized decks are composite and require a stud length of 5 in. for the

7-½ in. deck and 4 in. for the 5-½ in. deck. The roof deck was just a 1-½ in. 20 gage galvanized steel decking.

Bay sizes were typically set at 30 ft, especially on the outer spans of the building where the patient rooms are located. But, due to the irregular shape of the addition and use of the interior space, column lines were placed where columns were to not interfere with the working space of the interior. Bays of the interior ranged in various lengths. Decking typically spanned 10 ft and was supported by beams ranging from W14's to W21's with the typical being W16's. Lengths of the beams were typically 22' and were supported by girders ranging from W18's to W24's, but some exterior girders were W30's. Below in Figure 7 is a typical floor plan.

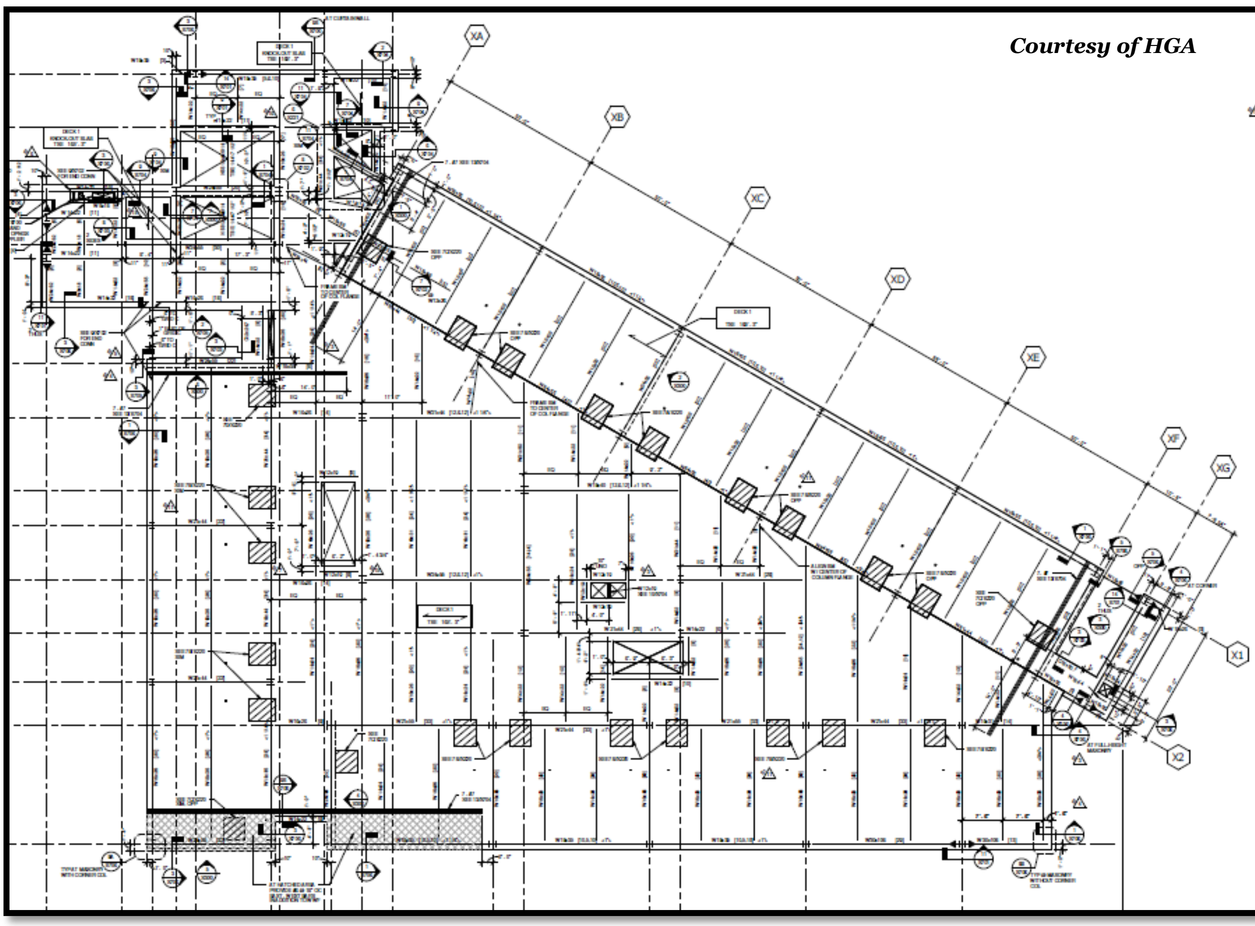


Figure 7: *Typical Floor Plan*

Construction Materials and Building Loads

Materials used in construction were specified in the general structural notes on Sheet S001. More information on the materials was found on the floor plans and detailed sections and elevations as well.

Figures provided by J. Elliott

Dead Loads	
Material	Load (psf)
Superimposed	30
Composite Deck	
7.5" Thick 3" Steel	75
5.5" Thick 2" Steel	57
Roof	2.14
10" Slab	120 pcf

Figure 8: *Dead Loads*

Properties of Materials		
Material		*Strength*
Concrete	**Weight**	**f'c (psi)**
Composite Deck	145	3500
All other concrete	115	4000
Steel	**Grade**	**fy (ksi)**
Reinforcing Bars	A615	60
W Shapes	A992	50
Other Shapes	A36	36
Rectangular HSS	A500 - B	46
Round HSS	A500 - B	42
Bolts	A325/A490	60
Studs	A108	50

Figure 9: *Properties of Materials*

Dead loads used for calculations were found in various ways. The composite deck and roof deck were found using the Vulcraft Roof and Steel Deck manual. The weight of the 10 in. light weight concrete slab was known and it was then assumed a superimposed dead load of 30 psf was used.

Live Loads		
Occupancy	Design (psf)	Thesis (psf)
Typ. Hosp. Floor	80	80
Corridors (Above 1st Floor)	80	80
Corridors (1st Floor)	100	100
Lobby Floor	100	100
Stair and Exits	100	100
Storage	125	125
Mechanical Room	125	125
Snow Load	34	34

Figure 10: *Live Loads*

Live loads were found using ASCE 7-05 there is just a quick note on them. When doing research, typical hospital floors for patient rooms were found to be 40 psf but it is believed that 80 psf was used because corridors (above 1st floor) with a load of 80 psf controlled. Because the patient rooms were found above the 1st floor, 80 psf was used for ease of calculations, although it is a conservative approach to this design.

Building Weight

In Technical Report 1, the total building weight was hand calculated. This process was very tedious and many human errors could have occurred. For this technical report, the total building weight was calculated with the assistance of a computer modeling program, to be explained later. The computer modeling program took into account self-weight of the steel beams, girders and columns as well as slab, deck, and superimposed dead load. Façade weight was added to these calculations and differences between the hand calculations and computer calculations were relatively close. However, because there was a change in building weight, seismic story forces were recalculated for this report.

Floor Weights				
Level	Previous Weight (k)	Model Weight (k)	Façade (k)	Current Weight (k)
2	2275	1846	292	2137
3	2402	2253	277	2530
4	2192	1930	290	2220
5	2385	2229	317	2546
6	2373	2151	294	2445
7	2328	2132	294	2426
8	2323	2133	294	2427
9	2532	2077	363	2440
10	1840	314	198	512
Total	20651			19682

Figure 11: *Building Weight Comparison*
Figure provided by J. Elliott

Lateral Loads

Wind Load Design

Chapter 6 of ASCE 7-05 was used to determine the wind load pressures. For simplicity of analysis, the addition was modeled as a rectangular box. Parameters for the box spanned between the furthest reaching corners of the building in both x and y directions. In Figure 12 below, is the rectangular box and dimensions used for the calculating wind load pressures.

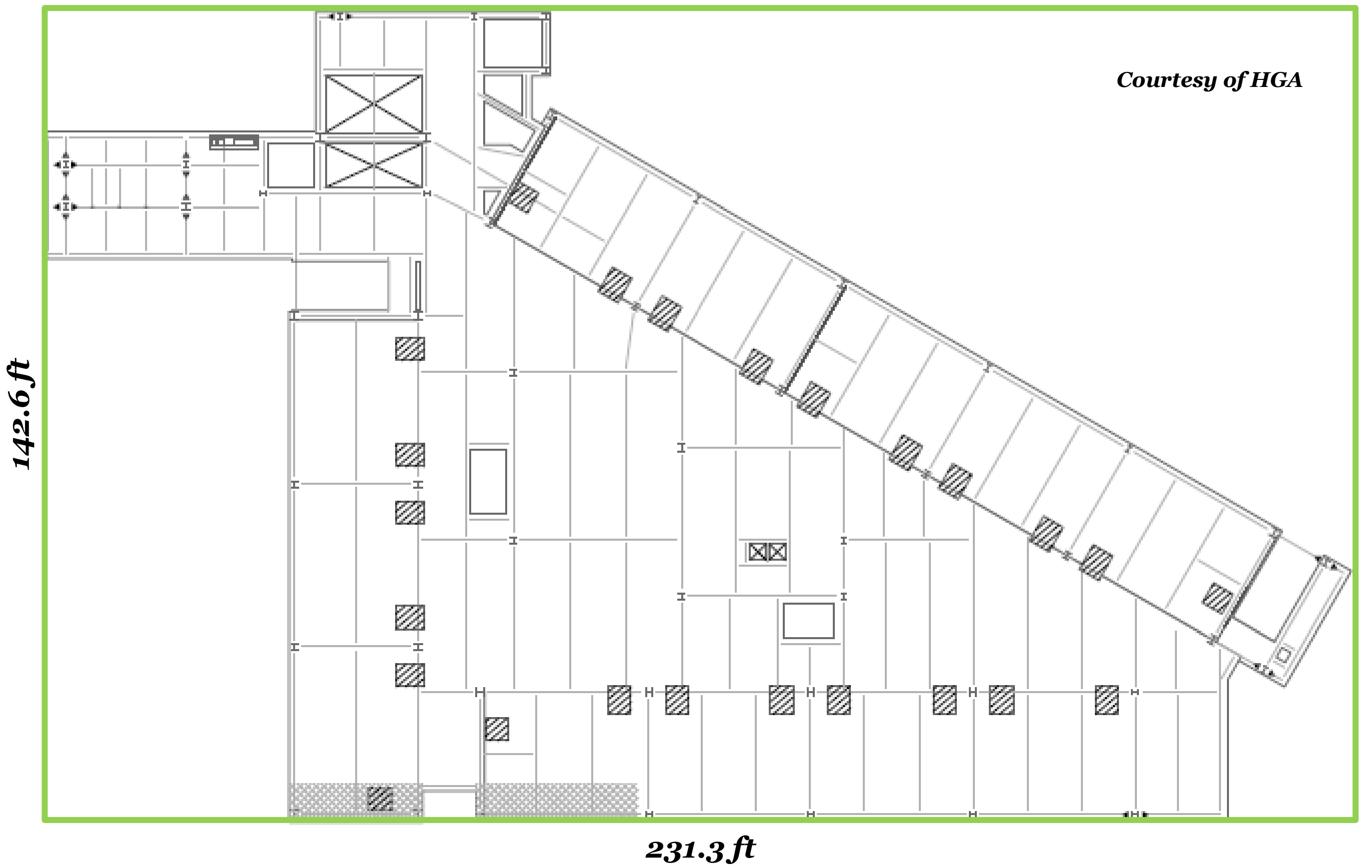


Figure 12: *Wind Load Parameters*

In Technical Report 1, the parameters were much smaller than the ones above. For this technical report, wind load pressures were recalculated for the adjusted parameters and the results are listed on the following page. Figures 13 & 14 show the applied story pressures, forces, leeward pressure, total base shear and overturning moment for the East/West and South/North directions respectively. Work done for these calculations can be found in Appendix A.

West to East				
Level	Ht. (ft)	Windward (psf)	Windward (k)	M (k-ft)
1	0	0.00	0	0
2	12.25	11.65	21.30	260.87
3	25.646	12.97	23.10	592.31
4	37.25	14.05	25.62	954.48
5	51.25	15.03	29.99	1536.86
6	65.25	15.79	31.50	2055.37
7	79.25	16.62	33.15	2627.11
8	93.25	17.17	34.25	3193.78
9	107.25	17.72	43.56	4671.34
10	127.75	18.41	26.89	3434.88
Base Shear		269.35	**Overturning M**	19327.0

South to North				
Level	Ht. (ft)	Windward (psf)	Windward (k)	M (k-ft)
1	0	0.00	0	0
2	12.25	11.65	34.56	423.35
3	25.646	12.97	37.48	961.21
4	37.25	14.05	41.58	1548.94
5	51.25	15.03	48.66	2494.03
6	65.25	15.79	51.12	3335.46
7	79.25	16.62	53.80	4263.30
8	93.25	17.17	55.58	5182.88
9	107.25	17.72	70.68	7580.69
10	127.75	18.41	43.63	5574.14
Base Shear		437.10	**Overturning M**	31364.0

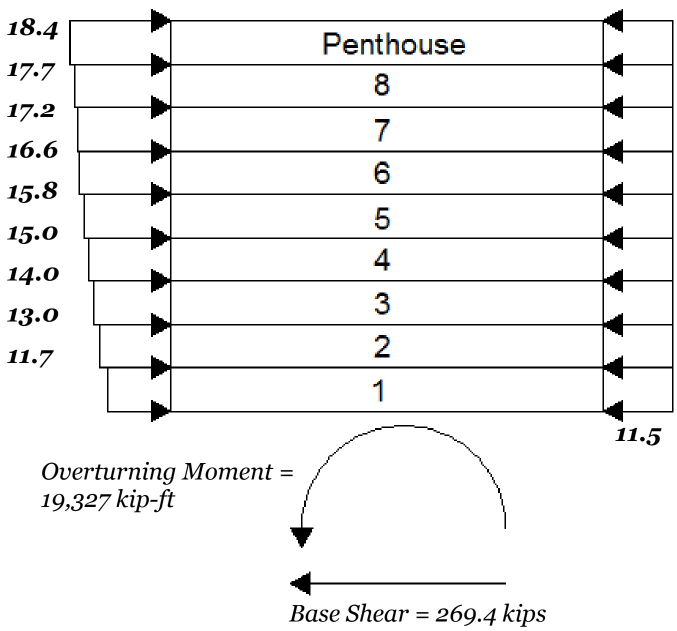


Figure 13: *West to East Wind Loading in pounds per square foot (PSF)*
Figure provided by J. Elliott

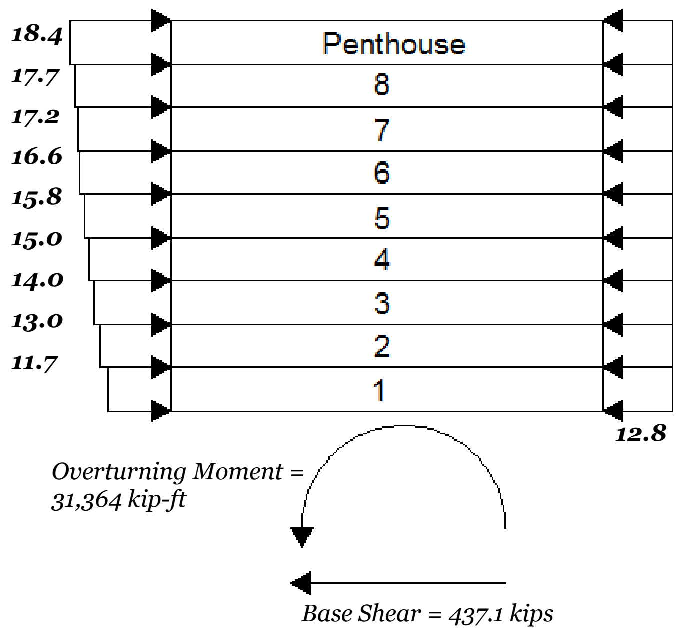


Figure 14: *South to North Wind Loading in pounds per square foot (PSF)*
Figure provided by J. Elliott

From the figures provided, the base shear in the West/East direction was 269.4 kips and 437.1 kips in the South/North direction. Overturning moments were found to be 19,327 k-ft and 31,364 k-ft in the West/East and South/North directions respectively. These numbers are reasonable. The reason for a larger base shear and overturning moment in the South/North direction is because of the larger width (231.3 ft) perpendicular to the wind pressure loads. When comparing numbers, the longer length of 231.3 ft was 1.6 times longer than the shorter width of 142.6 ft. The larger base shear was in fact 1.6 times bigger than the smaller one supporting this reasoning.

Seismic Design

Chapter 11 and Chapter 12 of ASCE 7-05 were used for seismic design. Appleton, Wisconsin is a low risk seismic area. Earthquakes would very rarely affect this structure over a long period of time and it reasonable to assume wind loads would control. After checking seismic design criteria, it was determined that the structure was to be designed for SDC A. Chapter 11 of ASCE 7-05 states SDC A structures need only to comply with section 11.7 thus avoiding the use of the equivalent lateral force method. Section 11.7 states that story forces will be found by:

$$F_x = 0.01W_x \quad \text{where,}$$

Fx = Story force at story x
Wx = Weight of story at story x

After using the above equation and help of excel, the base shear for the structure was 191.7 kips and the overturning moment was 12,094 kip-ft. Comparing these values to the wind load properties provided previously, it is supported that wind loads indeed control. Check for seismic design criteria was done by hand and this can be found in Appendix B. Below, Figure 15 shows the calculations for the story forces, base shear and overturning moment.

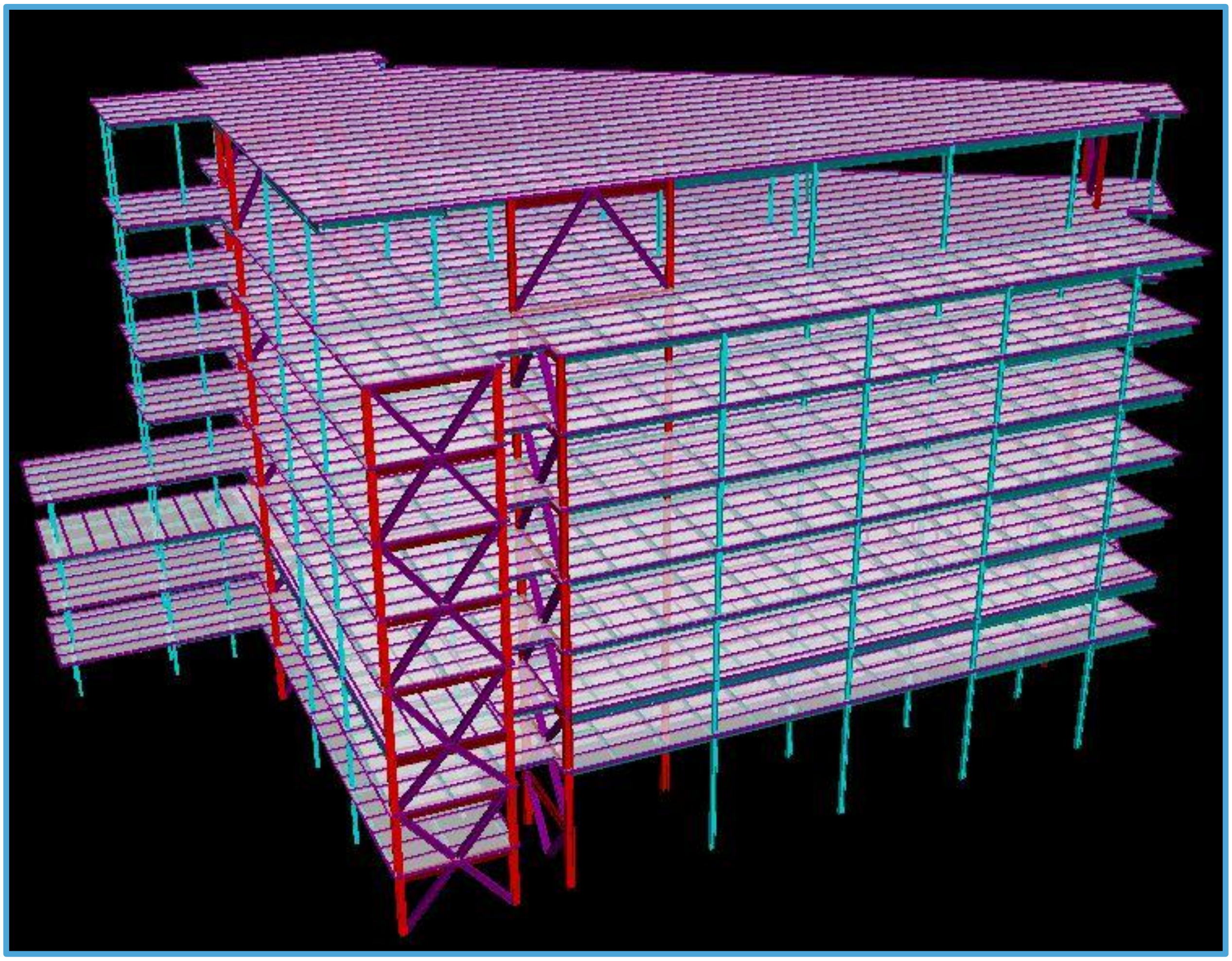
Seismic Load Calculations					
Level	Ht. (ft)	Weight (k)		Fx	M (k-ft)
1	0	0		0	0
2	12.25	2137.20		21.37	261.81
3	25.646	2529.80		25.30	648.79
4	37.25	2219.60		22.20	826.80
5	51.25	2545.80		25.46	1304.72
6	65.25	2445.40		24.45	1595.62
7	79.25	2426.00		24.26	1922.61
8	93.25	2427.10		24.27	2263.27
9	107.25	2439.70		24.40	2616.58
10	127.75	511.60		5.12	653.57
Base Shear		191.71	**Overturning M**		12093.8

Figure 15: *Seismic Load Calculations*
Figure provided by J. Elliott

Lateral Analysis

For the purpose of this report, a computer model was used to determine the controlling load cases and lateral system properties. Properties included lateral drift, periods, lateral forces, and torsion. RAM Structural, a computer modeling program developed by Bentley, was used for the analysis.

During the analysis, the column connection to the foundation was modeled as a pin. All other connections were modeled as fixed. Self-weight of the building was self-calculated by RAM. This included weight of each structural member such as steel, frame, and deck elements. A superimposed dead load of 30 psf and live load of 80 psf was applied as surface loads to the diaphragm of each floor.



Because of the advantages RAM Structural provides, time was taken in trying to model all gravity and lateral members as accurately as possible. This included assigning each individual member. Also a rigid diaphragm was assigned to each floor which outlined the entire perimeter.

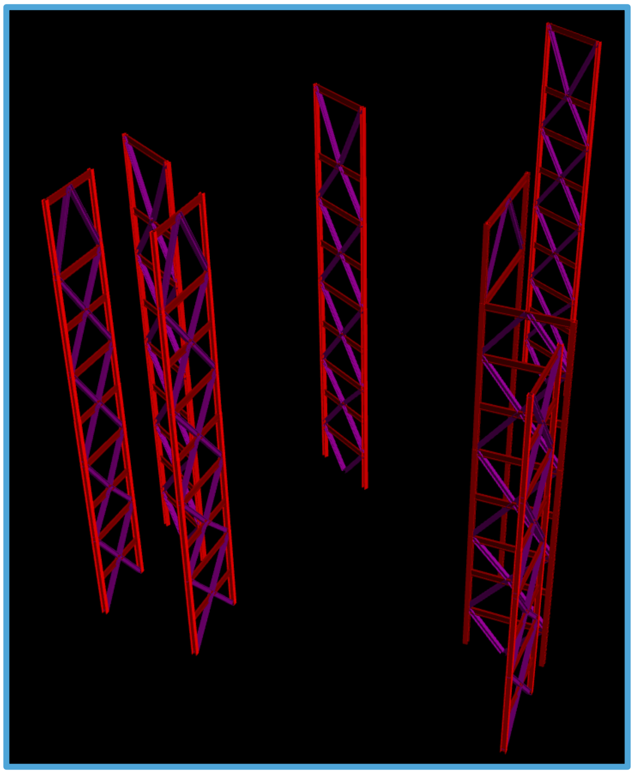


Figure 16B: *3-D RAM Model of lateral system*

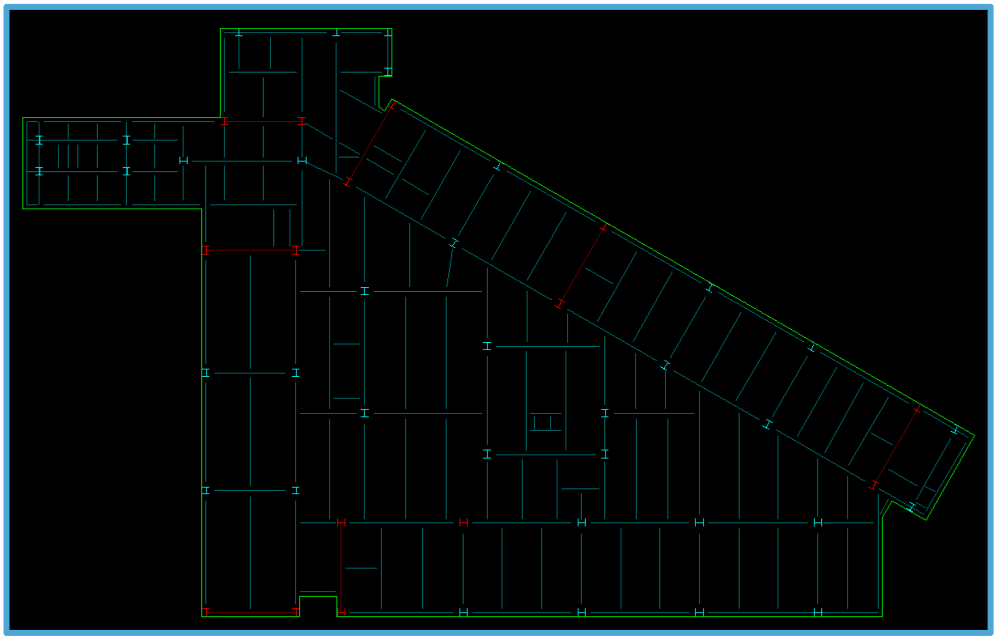


Figure 16C: *2-D Model of a typical floor plan*

There were a few things that were observed after the model was analyzed. Modal analysis provided a first mode period of 1.76 seconds. This was approximately 1 second larger than the approximate period of 0.757 seconds found during wind design. The period found from the RAM model indicated that the structure was actually flexible instead of rigid. This is a possibility because there were few braced frames within the building to resist the forces. Also, flexibility of the building could be an advantage to a hospital setting. Forces wouldn't be felt as normal, but rather a sway would occur thus un-disturbing the presence in the hospital.

Trying to recreate a structural system which has already been completed was a difficult task but the results alone were not helpful enough to understand how a lateral system works. In order to accomplish this, several aspects of a structural system were studied.

Relative Stiffness

One aspect focused on was the relative stiffness of the lateral braced frames within the structure. Relative stiffness is looking at the distribution of the forces within a diaphragm to the lateral systems. To further understand this concept, the stiffness of each frame was found with the help of RISA – 2D. Each braced frame was modeled with a 1 kip load applied to the top and columns modeled as pinned connections to the base. Figures on the following page demonstrates this concept. Displacements were found for each braced frame and plugged into the equation:

$$K_f = \frac{P}{\Delta}$$

Once the stiffness of each frame was determined, the contribution of each frame to the overall system in its respective direction was found, also known as its relative stiffness.

Hand calculations to find the relative stiffness can be found in Appendix C.

There are a few quick notes about the relative stiffness calculations. For a typical diaphragm there were three braced frames in the x-drection, one braced frame in the y-direction and three braced frames running diagonally. Each diagonal frame was broken up into its x and y components in order to calculate relative stiffness in each direction.

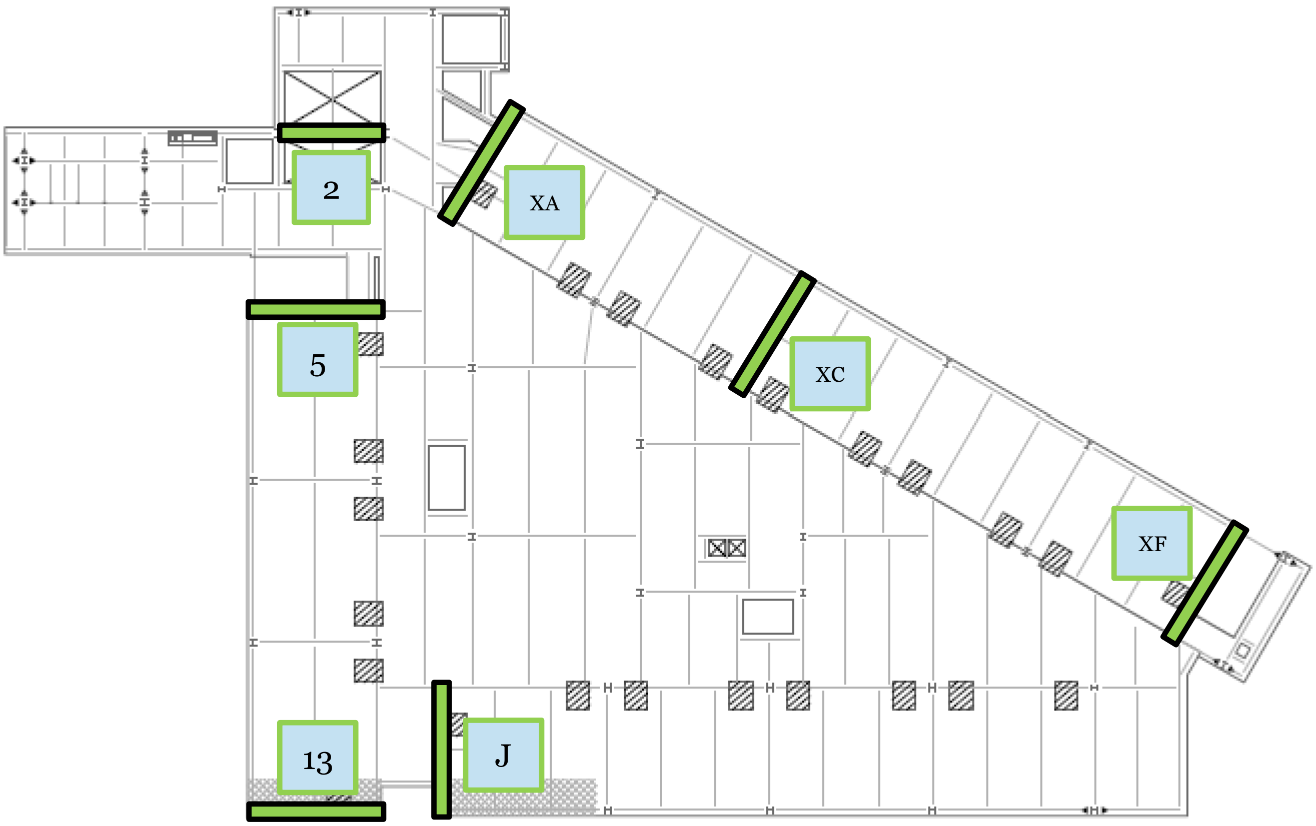
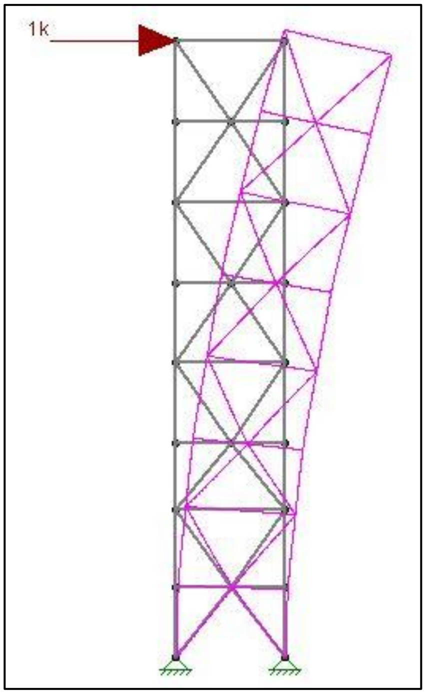
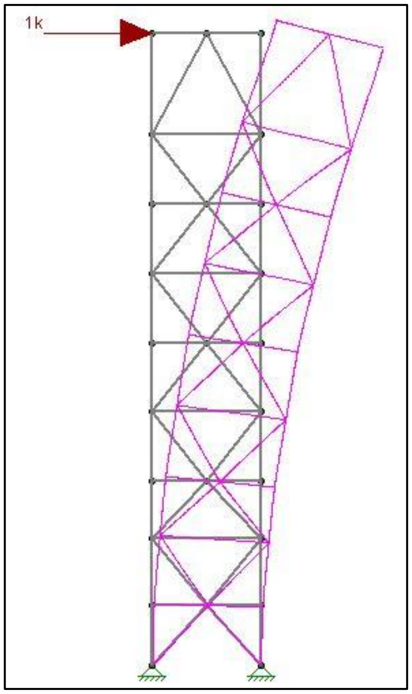


Figure 17: *Location and labels of each braced frame*
Plan courtesy of HGA – Figure provided by J. Elliott

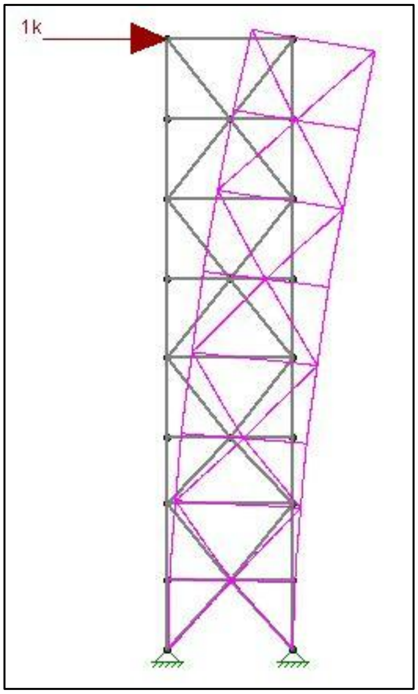
Frames running in the West/East direction



Frame 2



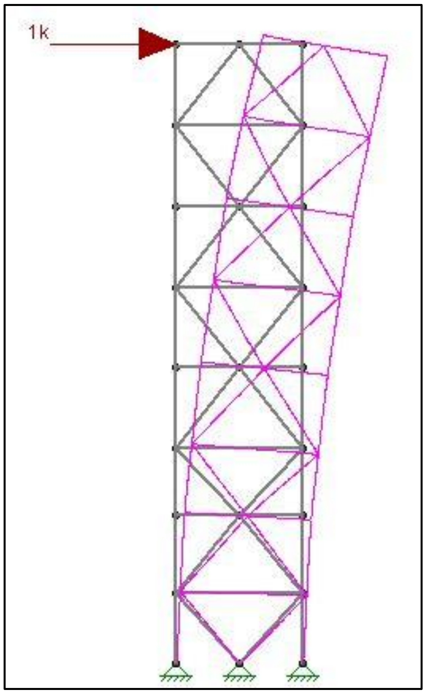
Frame 5



Frame 13

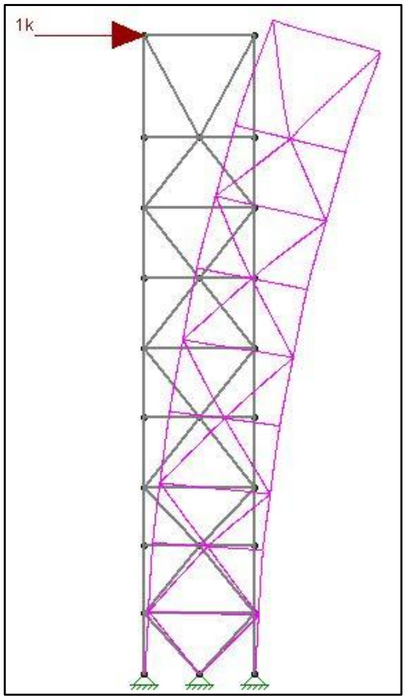
Figure 18: *Deflected shape of each braced frame*
Figure provided by J. Elliott

Frame running in the South/North direction



Frame J

Typical Frame running diagonally



Frames XA, XC, XF

Frame Stiffness			
Frame	Force (k)	Δmax (in)	k (k/in)
XA	1	0.015	66.7
XC	1	0.015	66.7
XF	1	0.015	66.7
J	1	0.009	111.1
13	1	0.009	111.1
5	1	0.015	66.7
2	1	0.019	52.6

In the West/East Direction		
Frame	Stiffness (k/in)	Relative Stiffness
2	52.6	13.04%
5	66.7	16.51%
13	111.1	27.52%
XAy	57.8	14.31%
XCy	57.8	14.31%
XFy	57.8	14.31%
Total	403.7	100.00%

In the South/North Direction		
Frame	Stiffness (k/in)	Relative Stiffness
J	111.1	52.62%
XAy	33.4	15.79%
XCy	33.4	15.79%
XFy	33.4	15.79%
Total	211.2	100.00%

Figure 19: *Relative Stiffness Tables*
Figure provided by J. Elliott

The figure above shows the relative stiffness of each braced frame in their respective directions. Because of their individual stiffness's, it can be seen which frames will take a majority of the loads. In the West/East direction, frame 13 will take 27.52% of the total load while frame J in the South/North direction will take 52.62% of the total load.

Now that the relative stiffness of each a typical diaphragm has been completed, to help understand where the exact location of each load will be applied, the center of mass and center of rigidity will be the next focus

Center of Rigidity

The center of rigidity and center of mass vary for each diaphragm. Loads applied to each diaphragm will be applied to the center of mass and if there happens to be an eccentricity between the center of mass and center of rigidity, torsion will occur.

Due to the irregular shape of the addition, it was assumed there would be some torsion. In order to confirm this, the RAM model was checked to see where the center of rigidity and center of mass was on each floor. RAM concluded that both points did not lie on top of each other meaning there was an eccentricity and torsion would occur. To double check that the RAM model was setup correctly, the center of rigidity was calculated by hand using the stiffness's found earlier. Hand calculations can be found in Appendix D. Important equations were:

$$X_r = \frac{\sum R_i x_i}{\sum R_i} \qquad Y_r = \frac{\sum R_i y_i}{\sum R_i}$$

Xr is the distance to the center of rigidity in the x-direction
Yr is the distance to the center of rigidity in the y-direction

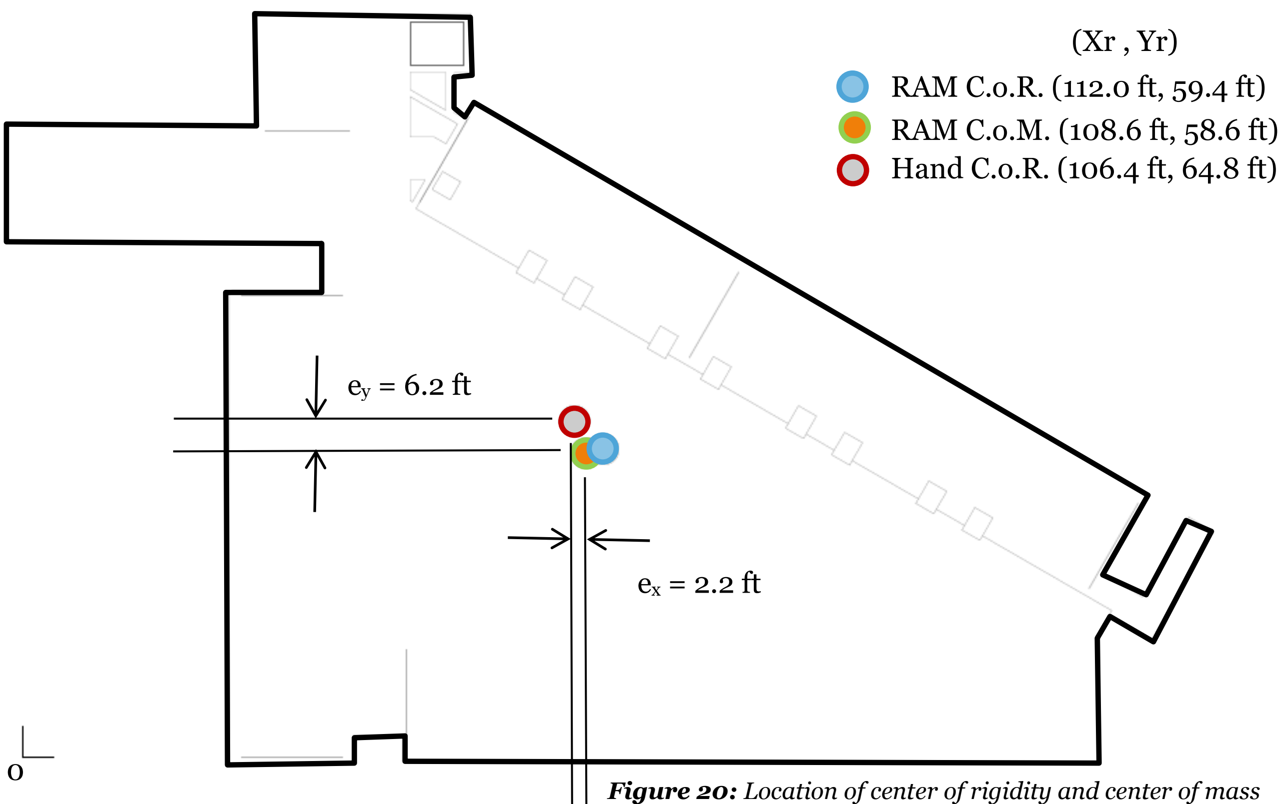


Figure 20: *Location of center of rigidity and center of mass*
Figure provided by J. Elliott

Now that the center of rigidity and center of mass have been located, these two locations will be crucial when identifying the controlling load cases on the structure. Loads produced by the wind will be applied to the center of mass or in relation to. This will create torsion. Seismic loads will create deflections which will be measured from the center of rigidity and discussed later in report.

Below in Figure 21 are tables showing the torsional rigidities, direct shears, torsional shears, and total shears for each braced frame in their respective directions. Sample Calculations can be found in Appendix E.

Total Story Distribution in x-direction (in kips)						
Level	Vxa	Vxc	Vxf	V2	V5	V13
2	58.66	58.93	65.69	47.00	62.82	97.78
3	54.26	54.52	60.77	43.48	58.11	90.46
4	49.18	49.40	55.07	39.40	52.66	81.98
5	43.57	43.77	48.79	34.91	46.66	72.63
6	36.78	36.95	41.18	29.47	39.38	61.31
7	29.44	29.57	32.97	23.59	31.52	49.07
8	21.86	21.96	24.48	17.51	23.41	36.44
9	13.94	14.01	15.62	11.17	14.93	23.25
10	4.86	4.89	5.45	3.90	5.21	8.11

Total Story Distribution in y-direction (in kips)				
Level	Vxa	Vxc	Vxf	VJ
2	147.77	148.70	150.09	238.87
3	134.32	135.17	136.42	217.12
4	120.54	121.30	122.43	194.85
5	105.71	106.38	107.37	170.88
6	88.61	89.17	90.00	143.24
7	71.91	72.37	73.04	116.24
8	54.27	54.61	55.12	87.72
9	36.41	36.64	36.98	58.85
10	13.40	13.49	13.61	21.67

Figure 21: *Total distribution to braced frames for each story*
Figure provided by J. Elliott

Wind Load Analysis

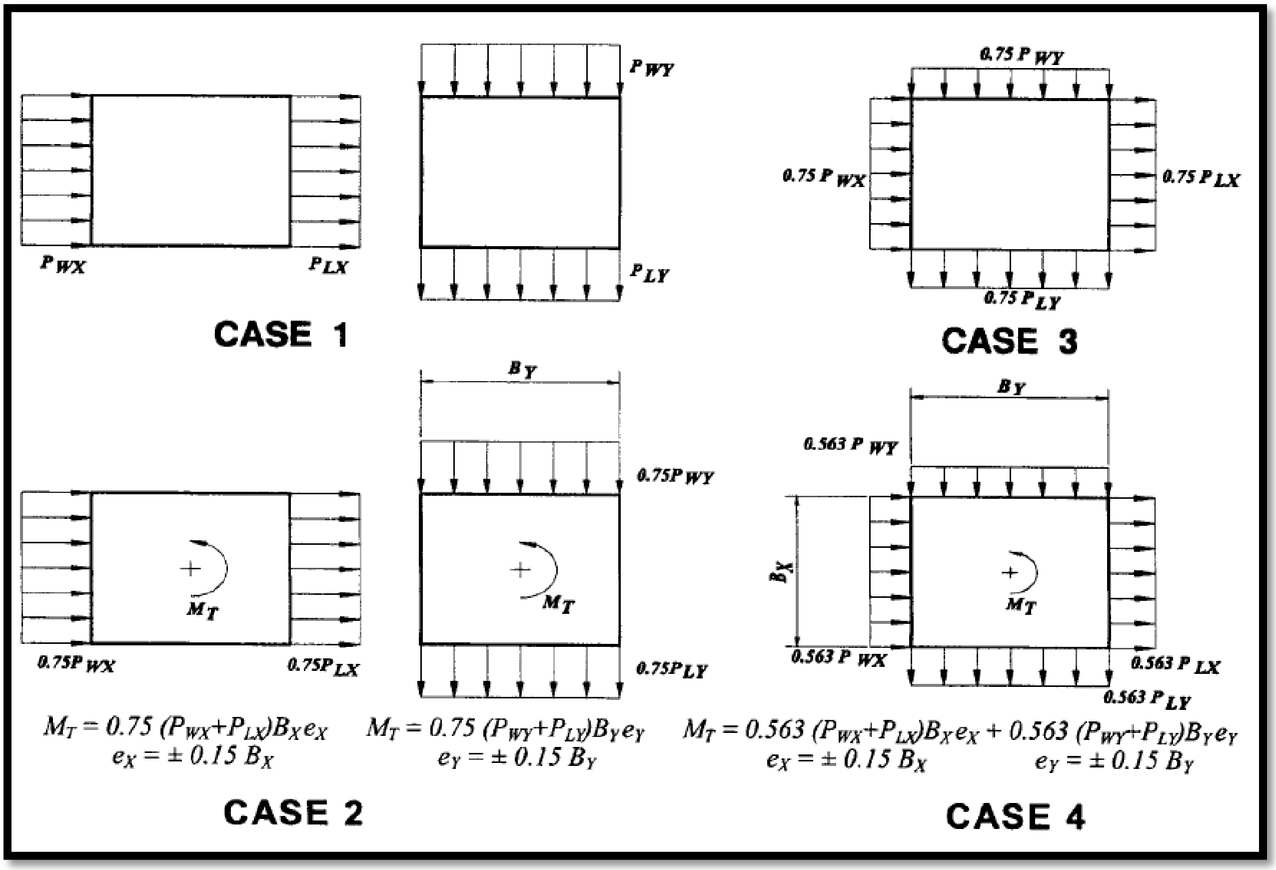


Figure 22: *Wind load cases from ASCE 7-05 (Figure 6-9)*

In Figure 22 above are the load cases used for the wind load analysis. A summary of each case is provided below:

Case 1 – 100% of the wind pressure is applied in each the East/West direction and the North/South direction.

Case 2 – 75% of the wind pressure is applied in each the East/West direction, and the North/South direction. In addition, an eccentricity equal to 15% of the building width perpendicular to the wind pressure is taken into account creating torsion.

Case 3 – 75% of the wind pressure is applied simultaneously in both the East/West and North/South directions.

Case 4 – 56.3% of the wind pressure is applied simultaneously in both East/West and North/South directions. In addition, an eccentricity equal to 15% of the building width perpendicular to the wind pressure is taken into account creating torsion.

Cases 1 and 3 apply the forces to the center of mass while Cases 2 and 4 need to take into account an eccentricity. Because eccentricity could be positive or negative, this had to be taken into account when checking for the controlling load case.

After looking at the distances from the center of rigidity to each cases eccentricity, there were a few things to expect. Case 2 in the West/East and South/North direction will be controlled by a negative eccentricity. Due to that assumption, Case 4 will be controlled when both forces are acting simultaneously with the negative eccentricity as well.

Case 2 and 4 West/East (+e)					
Xcom	Ycom	Ly (ft)	e_{xcom}	Ycor	e_{xcor}
108.6	58.62	142.6	21.4	59.4	20.6
Case 2 and 4 South/North (+e)					
Xcom	Ycom	Lx (ft)	e_{ycom}	Xcor	e_{ycor}
108.6	58.62	231.3	34.7	112.0	31.3

Case 2 and 4 West/East (-e)					
Xcom	Ycom	Ly (ft)	e_{xcom}	Ycor	e_{xcor}
108.6	58.62	142.6	21.4	59.4	22.2
Case 2 and 4 South/North (-e)					
Xcom	Ycom	Lx (ft)	e_{ycom}	Xcor	e_{ycor}
108.6	58.62	231.3	34.7	112.0	38.1

Figure 23: *Tables showing the eccentricities for Cases 2 and 4*
Figure provided by J. Elliott

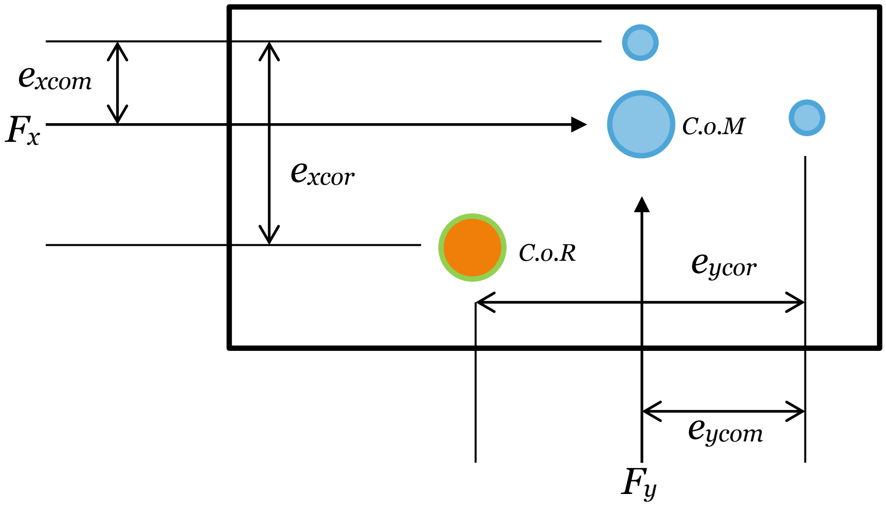


Figure 24: *Diagram of variables for Figure 23. Shows positive eccentricity*
Figure provided by J. Elliott

The eccentricities computed on the previous page were then taken into account when finding which of the wind load cases would control overall. These were inserted into the RAM model and the results are below in Figure 25.

From the table on the right, it can be see that two different load cases controlled the both directions. In the West/East direction, Case 3, where forces from both directions acted simultaneously, had the largest displacement with 1.04 in. In the South/North direction, Case 1, where forces acted just in the y-direction, had the largest displacement with 1.57 in. These two cases were then just used in the load combinations when determining which overall load combination would control the design of the structure.

Controlling Wind Load Case			
Case	Direction	Δx (in)	Δy (in)
1	X	0.784	-0.302
	Y	-0.606	**1.569**
2	X+e	0.611	-0.219
	X-e	0.565	-0.234
	Y+e	-0.514	1.157
	Y-e	-0.396	1.196
3	X+Y	0.133	0.95
	X-Y	**1.043**	-1.403
4	X+Y CW	0.162	0.732
	X+Y CCW	0.038	0.692
	X-Y CW	0.844	-1.032
	X-Y CCW	0.721	-1.072

Figure 25: *Displacements due to wind load cases*
Figure provided by J. Elliott

Seismic Load Analysis

Just as the controlling wind load case was determined, seismic was done as well. For seismic, the design criteria found from ASCE 7-05 earlier, was applied to the RAM program. Four load cases were compared. Case 1 and 2 were seismic forces in the x-direction with a positive and negative 5% accidental eccentricity applied, respectively. Case 3 and 4 was the same with seismic forces in the y-direction being the exception.

After analyzing the cases in RAM, the results concluded Case 1 controlled the x-direction with a 0.327 in. displacement while case 4 controlled the y-direction with a 0.388 in. displacement. Both displacements were much smaller than the wind concluding that wind controls the design and should a factor in controlling the load combination.

Controlling Seismic Load Case			
Case	Direction	Δx (in)	Δy (in)
1	X+e	**0.327**	-0.128
2	X-e	0.318	-0.131
3	Y+e	-0.148	0.384
4	Y-e	-0.134	**0.388**

Figure 26: *Displacements due to seismic load cases*
Figure provided by J. Elliott

Load Combinations

To determine the controlling load combination, the worst case scenario was taken into account. These load combinations from Chapter 2 of ASCE 7 -05 were taken into account:

1. 1.4D
2. 1.2D + 1.6L + 0.5S
3. 1.2D + 1.6S + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5S
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.6W
7. 0.9D + 1.0E

RAM was very useful in determining the controlling load combination because there was a large number available due to the various number of wind and seismic load cases. However, after determining which wind and seismic load cases controlled, it was easy to eliminate many combinations. After RAM analyzed these applicable load combinations, it was determined that the controlling load combination in both directions turned out to be combination 4 (1.2D + 1.0L + 0.5S + 1.6W). The controlling factor was the included wind load case 3, one of the controlling wind load cases found previously. Figure 27 shows the max displacements from the roof for each combination.

The controlling load combination will be very important when gravity members are checked because dead, live and snow loads have been included. Therefore load combination 7 from Figure 27 controls:

$$1.2(\text{Dead}) + 1.0(\text{Live}) + 0.5(\text{Snow}) + 1.6(\text{Wind}_{3(x\text{-}y)})$$

Controlling Load Combination			
#	Combination	Δx (in)	Δy (in)
1	1.4D	0.166	-0.305
2	1.2D + 1.6L	0.268	-0.513
3	1.2D + 1.6L + 0.5S	0.269	-0.52
4	1.2D + 1.6S + 0.8W1y	-0.337	0.972
5	1.2D + 1.6S + 0.8W3(x-y)	0.983	-1.406
6	1.2D + 1.0L + 0.5S + 1.6W1y	-0.748	2.084
7	1.2D + 1.0L + 0.5S + 1.6W3(x-y)	**1.891**	**-2.671**
8	1.2D + 1.0L + 1.6W1y	-0.749	2.09
9	1.2D + 1.0L + 1.6W3(x-y)	1.889	-2.664
10	1.2D + 1.6W1y	-0.828	2.249
11	1.2D + 1.6W3(x-y)	1.811	-2.506
12	0.9D + 1.6W1y	-0.863	2.314
13	0.9D + 1.6W3(x-y)	1.775	-2.441
14	0.9D + 1.0E1	0.434	-0.327
15	0.9D + 1.0E4	-0.028	0.192
16	1.2D + 1.0L + 1.0E1 + 0.2S	0.549	-0.552
17	1.2D + 1.0L + 1.0E4 + 0.2S	0.087	-0.034

Figure 27: *Displacements due to load combinations*
Figure provided by J. Elliott

Drift

Drift checks were evaluated to prevent damage to structural and non-structural components. These drifts were calculated under the controlling loads found earlier because this is a serviceability check, which means factored loads were not used. Wind and seismic drifts were compared with their allowable drifts. According to industry standard, H/400 is the allowable drift for wind. According to ASCE 7-05 the allowable drift for seismic for occupancy category IV is $0.01h_{sx}$. The following page shows the results concluding that all story drifts passed, thus acceptable.

Wind Drift Comparison: East/West Direction				
Level	Height (ft)	Δ (in)	Drift (in)	Allowable (in)
2	12.25	0.04	0.04	0.37
3	25.65	0.11	0.07	0.40
4	37.25	0.18	0.07	0.35
5	51.25	0.30	0.12	0.42
6	65.25	0.42	0.12	0.42
7	79.25	0.55	0.13	0.42
8	93.25	0.69	0.14	0.42
9	107.25	0.83	0.14	0.42
Roof	127.25	1.04	0.22	0.60
Drifts based on Wind Load Case 3 (x-y)				
Allowable Drifts based on h/400 (in)				

Seismic Drift Comparison: East/West Direction				
Level	Height (ft)	Δ (in)	Drift (in)	Allowable (in)
2	12.25	0.01	0.014	0.12
3	25.65	0.04	0.027	0.13
4	37.25	0.07	0.027	0.12
5	51.25	0.11	0.040	0.14
6	65.25	0.15	0.037	0.14
7	79.25	0.19	0.047	0.14
8	93.25	0.24	0.045	0.14
9	107.25	0.28	0.042	0.14
Roof	127.25	0.33	0.048	0.20
Drifts based on Seismic Load Case 1 (x+e)				
Allowable drifts based on $0.010h_{sx}$				

Wind Drift Comparison: North/South Direction				
Level	Height (ft)	Δ (in)	Drift (in)	Allowable (in)
2	12.25	0.08	0.08	0.37
3	25.65	0.18	0.10	0.40
4	37.25	0.30	0.12	0.35
5	51.25	0.46	0.16	0.42
6	65.25	0.64	0.18	0.42
7	79.25	0.83	0.19	0.42
8	93.25	1.04	0.21	0.42
9	107.25	1.25	0.21	0.42
Roof	127.25	1.57	0.32	0.60
Drifts based on Wind Load Case 1 (y)				
Allowable Drifts based on h/400 (in)				

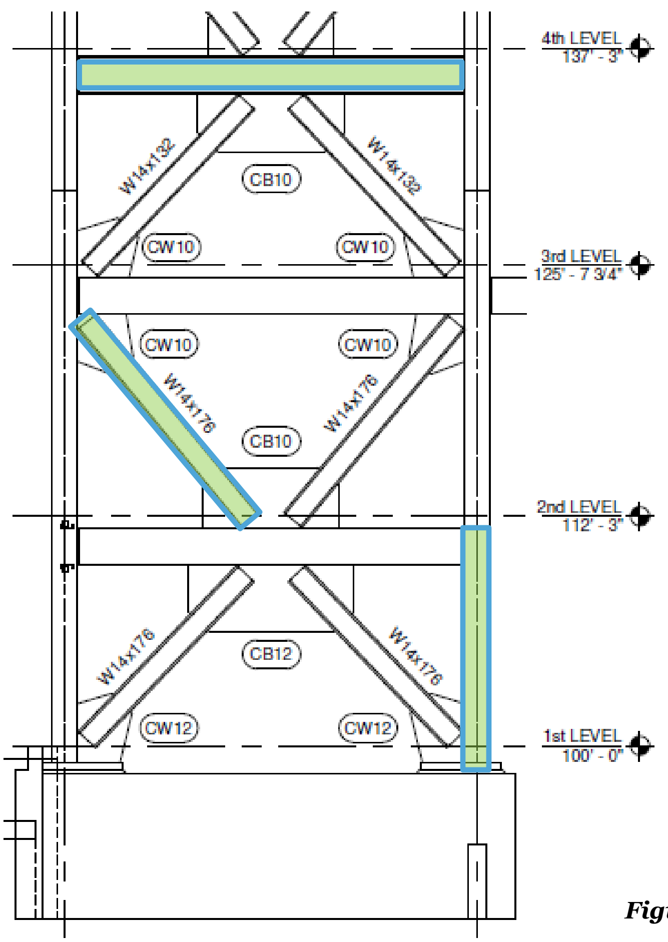
Seismic Drift Comparison: North/South Direction				
Level	Height (ft)	Δ (in)	Drift (in)	Allowable (in)
2	12.25	0.03	0.025	0.12
3	25.65	0.05	0.028	0.13
4	37.25	0.08	0.031	0.12
5	51.25	0.13	0.043	0.14
6	65.25	0.17	0.046	0.14
7	79.25	0.22	0.047	0.14
8	93.25	0.27	0.050	0.14
9	107.25	0.32	0.049	0.14
Roof	127.25	0.39	0.069	0.20
Drifts based on Wind Load Case 4 (y-e)				
Allowable drifts based on $0.010h_{sx}$				

Figure 27: *Allowable drift comparison*
Figure provided by J. Elliott

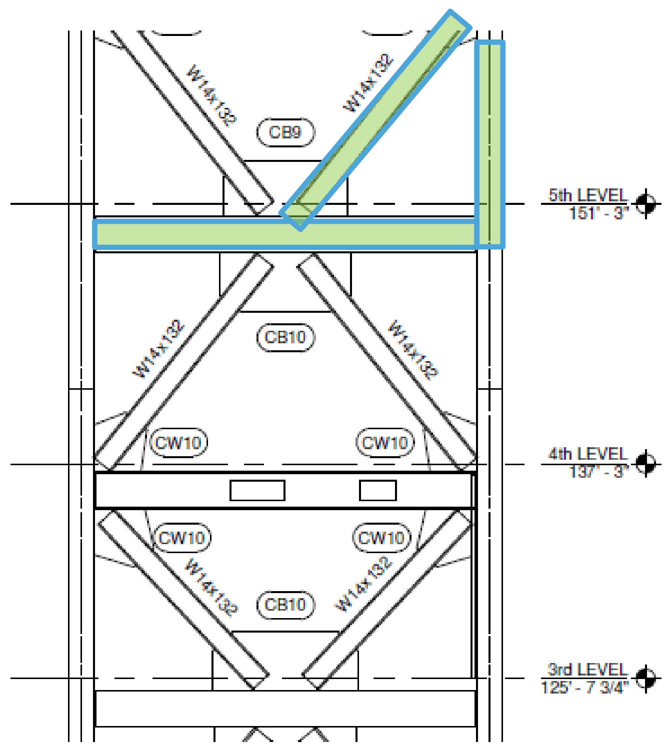
Lateral Member Spot Checks

Members of braced frames 5 and J were analyzed because each run in the opposite direction. For frame J, members were selected from a small proximity which was the fifth floor area. For frame 5, arbitrary members were picked which were not close to each other. Figure 28 shows which members of each frame were selected.

To analyze each individual member, combined loading effects were taken into account. From AE 401, it was learned that Chapter 6 of AISC 360 provided tables in regards to combined loading members. Because diagonal bracing only took axial loading, Chapter 4 of AISC 360 was used to check against failure. All members performed and passed. Hand calculations can be found in Appendix F.



Frame 5



Frame J

Figure 28: *Lateral members checked*
Figure provided by J. Elliott
Elevation courtesy of HGA

Conclusion

After analyzing all essential parts of the Bedtower Addition's lateral structural system, it has been determined that it would be successful in resisting lateral loads.

Use of this report proves Technical Report 1's theory that wind load cases would determine design of the lateral system. When comparing wind load and seismic load cases, wind load cases resulted in larger displacements and drifts. Because wind controlled, all load combinations available which included wind were analyzed. It was then determined the overall controlling load combination was:

$$1.2(\text{Dead}) + 1.0(\text{Live}) + 0.5(\text{Snow}) + 1.6(\text{Wind})$$

During the process of analyzing the lateral system, it was also concluded that wind load pressures applied in the South/North direction would affect the structure more than the West/East direction. However, it was observed that there were more braced frames along the x-direction. This was due to the architecture of the building. When checking the relative stiffness of the braced frames, the y-direction was less stiff resulting in the larger deflections.

Once the center of rigidity and center of mass were found, it was also determined that there would be torsional effects on the building. These however did not control. After all drifts were determined, they were found to pass acceptable industry standards.

Lastly, several members of the lateral system were checked to see if they would be able to hold the adequate strength capacity. All members were determined to pass. However, questions arose when checking the members because low moments. This could be caused by the braced frames taking much of the axial load thus not carrying over much moment to the columns of the braced system. Another reason is because the beam lateral beams and crossing bracing were modeled as pins.

It was also concluded that the structure was flexible due to a period higher than the one previously calculated. During Technical Report 1, an approximate period of 0.757 seconds was found but during modal analysis, a period of 1.76 seconds was calculated. This could be due to the change in stiffness in both directions.

Overall, the lateral system was found to be designed to hold adequate strength for lateral forces. The strength it holds is more than enough capacity to not bring it down for a while.

Appendix

Appendix A: Wind Load Calculations

Important Factors					
G	0.85	W. Cp	0.8		
		L. Cp	-0.5	<-------	S-N direction
GCpi	-0.18		-0.432	<-------	W-E direction
	0.18		-0.364	<-------	NE-SW direction
		S. Cp	-0.7		
Key:	G =	Gust Effect Factor			
	Gcpi =	Internal Pressure Coeff			
	W =	Windward			
	L =	Leeward			
	S =	Side			

West to East

Floor	Elev. (ft)	z	Floor Ht. (ft)	kz	qz	qh	Wall Length (ft)	Windward (psf)	Windward (k)	Leeward (psf)	Leeward (k)
1	100.00	0	12.25	0.57	11.55	21.1	142.5	0.00	0	0.00	0
2	112.25	12.25	13.40	0.57	11.55	21.1	142.5	11.65	21.30	-11.55	-21.10
3	125.65	25.65	11.60	0.67	13.48	21.1	142.5	12.97	23.10	-11.55	-20.57
4	137.25	37.25	14.00	0.74	15.07	21.1	142.5	14.05	25.62	-11.55	-21.06
5	151.25	51.25	14.00	0.82	16.52	21.1	142.5	15.03	29.99	-11.55	-23.03
6	165.25	65.25	14.00	0.87	17.63	21.1	142.5	15.79	31.50	-11.55	-23.03
7	179.25	79.25	14.00	0.93	18.85	21.1	142.5	16.62	33.15	-11.55	-23.03
8	193.25	93.25	14.00	0.97	19.66	21.1	142.5	17.17	34.25	-11.55	-23.03
9	207.25	107.25	20.50	1.01	20.47	21.1	142.5	17.72	43.56	-11.55	-28.38
10	227.75	127.75	0.00	1.06	21.49	21.1	142.5	18.41	26.89	-11.55	-16.86

South to North

Floor	Elev. (ft)	z	Floor Ht. (ft)	kz	qz	qh	Wall Length (ft)	Windward (psf)	Windward (k)	Leeward (psf)	Leeward (k)
1	100.00	0	12.25	0.57	11.55	21.1	231.3	0.00	0	0.00	0
2	112.25	12.25	13.40	0.57	11.55	21.1	231.3	11.65	34.56	-12.77	-37.85
3	125.65	25.65	11.60	0.67	13.48	21.1	231.3	12.97	37.48	-12.77	-36.90
4	137.25	37.25	14.00	0.74	15.07	21.1	231.3	14.05	41.58	-12.77	-37.79
5	151.25	51.25	14.00	0.82	16.52	21.1	231.3	15.03	48.66	-12.77	-41.33
6	165.25	65.25	14.00	0.87	17.63	21.1	231.3	15.79	51.12	-12.77	-41.33
7	179.25	79.25	14.00	0.93	18.85	21.1	231.3	16.62	53.80	-12.77	-41.33
8	193.25	93.25	14.00	0.97	19.66	21.1	231.3	17.17	55.58	-12.77	-41.33
9	207.25	107.25	20.50	1.01	20.47	21.1	231.3	17.72	70.68	-12.77	-50.92
10	227.75	127.75	0.00	1.06	21.49	21.1	231.3	18.41	43.63	-12.77	-30.26

AE SENIOR THESIS | WIND LOAD CALC. | JESSEL ELLIOTT

USING ASCE 7-05:

BASIC WIND SPEED V (FIG 6-1): 90 MPH (40 m/s)
IMPORTANCE FACTOR I (TAB. 6-1): 1.15
OCCUPANCY CATEGORY (TAB 1-1): IV
EXPOSURE CATEGORY (F/ PLANS): B
DIRECTIONALITY FACTOR Kd (TAB 6-4): 0.85
TOPOGRAPHIC FACTOR Kzt : 1.0
VEL PRESSURE EXP. COEF. Kz (TAB 6-2): 1.04

VELOCITY PRESSURE

$qz = 0.00256\, K_z K_{zt} K_d V^2 I$ SAMPLE CALCULATION

$= 0.00256(1.04)(1.0)(0.85)(90)^2(1.15) = 21.1$ psf

$q_h = 21.1$ psf FOR $h = 127'$

GUST FACTOR

RIGID IF $n_1 > 1.0$ HZ ; FLEXIBLE IF $n_1 < 1.0$ HZ

$n_1 = \frac{1}{T}$ T = NATURAL PERIOD $T = C_t h_n^{\,x}$

CONSIDERED ALL OTHER STRUCTURAL SYSTEMS

$h_n = 127'$ $C_t = .02$, $x = 0.75$

$T = 0.02\,(127)^{0.75} = 0.757$ sec.

$n_1 = 1/0.757 = 1.3 > 1.0$ SO RIGID STRUCTURE

$\longrightarrow G = 0.85$

DESIGN WIND PRESSURE

$p = q G C_p - q_i (GC_{pi})$ w/ $GC_{pi} = \pm 0.18$

for K_z 25 – 0.66 25.65
30 – 0.70

$$\left(\frac{30-25}{0.70-0.66}\right) = \left(\frac{30-25.65}{0.70-x}\right)$$

$$x = 0.70 - \frac{(30-25.65)(0.70-0.66)}{(30-25)}$$

EXTERNAL PRESSURE COEFFICIENTS

WINDWARD WALL $C_p = 0.8$ USE w/ q_z

LEEWARD WALL W-E DIREC. $L/B = \frac{167.67}{126.25} = 1.34$ $C_p = -0.432$
USE w/ q_h S-N DIRECT. $L/B = \frac{126.25}{167.67} = 0.75$ $C_p = -0.5$
NE-SW DIRECT. $L/B = \frac{165.167}{98.33} = 1.68$ $C_p = -0.364$

SIDE WALL $C_p = -0.7$ USE w/ q_h

Appendix B: Seismic Design Criteria

REVISED SEISMIC ANALYSIS | TECH REPORT #3 | JESSEL ELLIOTT | 1/1

FROM SHEET S001, ADDITION WAS LISTED AS SDC A.

ANALYSIS WILL BE DETERMINED BY:

$$F_x = 0.01\, w_x$$

CHECK SEISMIC DESIGN CRITERIA

FIGURE 22-1 SPECTRAL RESPONSE ACCEL., $S_s = 0.068$ OK
22-2 SPECTRAL RESPONSE ACCEL, $S_1 = 0.033$ OK
TABLE 11.5-1 SEISMIC IMPORTANCE FACTOR, $I = 1.5$ OK
11.4-1 $F_a = 0.8$
11.4-2 $F_v = 0.8$

$$S_{MS} = F_a S_s = 0.8(.068) = .0544$$
$$S_{M1} = F_v S_1 = 0.8(.033) = .0264$$

$$S_{DS} = 2/3\, S_{MS} = 2/3(.0544) = .0363 < .167 \text{ (TABLE 11.6-1)}$$
$$S_{D1} = 2/3\, S_{M1} = 2/3(.0264) = .0176 < .067 \text{ (TABLE 11.6-2)}$$

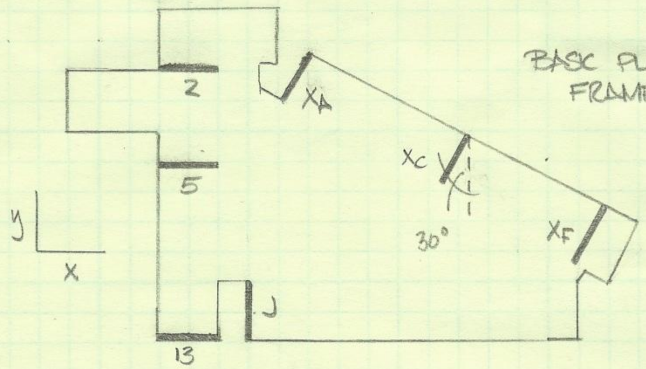
BOTH VALUES FALL IN CATEGORY A FOR DESIGN REQUIREMENTS SO STRUCTURE IS SDC A.

ALSO ACCORDING TO 11.4.1
$S_1 = 0.033 < .04$ and $S_s = 0.068 < .15$
THIS ALSO CLARIFIES SDC A

BECAUSE STRUCTURE IS SDC A, CHAPTER 11.7.1 OF ASCE 7-05 STATES IT NEEDS ONLY TO COMPLY WITH CHAPTER 11.7 THUS AVOIDING THE EQUIVALENT LATERAL FORCE METHOD.

Appendix C: Relative Stiffness

RELATIVE STIFFNESS | TECH REPORT #3 | JESSEL ELLIOTT | 1/1



BASIC PLAN OF FRAME MEMBERS (TYPICAL)

FROM RISA, STIFFNESSES (K/in):

XA = 66.7
XC = 66.7
XF = 66.7
J = 111.1
13 = 111.1
5 = 66.7
2 = 52.6

ALL DUE TO A 1K LOAD

X/y COMPONENTS OF XA, XC, XF

$\cos 30° = \frac{x}{K_{xc}} \Rightarrow x = 57.76$

$\sin 30° = \frac{y}{K_{xc}} \Rightarrow y = 33.35$

IN THE X-DIRECTION

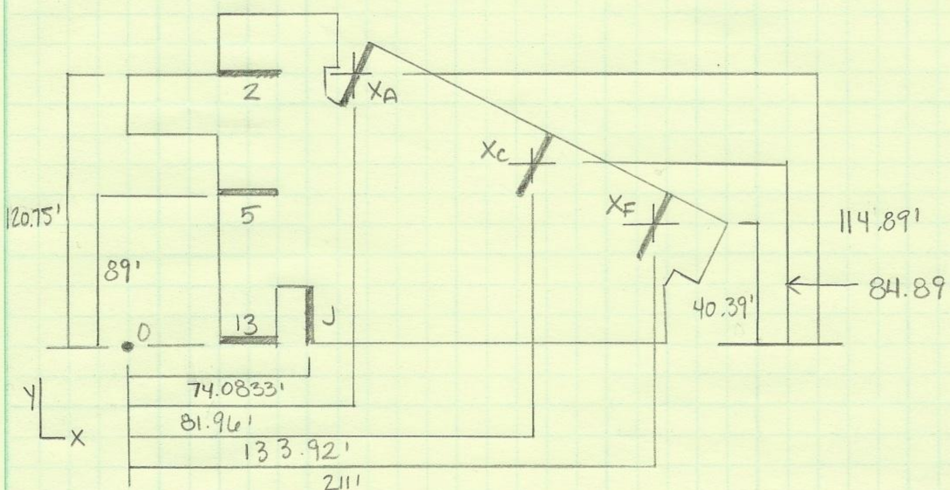
FRAME	STIFFNESS	RELATIVE STIFFNESS
2	52.6	13.03%
5	66.7	16.52%
13	111.1	27.52%
XA_x	57.76	14.31%
XC_x	57.76	14.31%
XF_x	57.76	14.31%
TOTAL:	403.68	100%

IN THE y-DIRECTION

FRAME	STIFFNESS	RELATIVE STIFFNESS
J	111.1	52.62%
XA_y	33.35	15.79%
XC_y	33.35	15.79%
XF_y	33.35	15.79%
TOTAL:	211.15	100%

Appendix D: Center of Rigidity Calculations

CENTER OF RIGIDITY | TECH REPORT #3 | JESSEL ELLIOTT | 1/1



FROM RELATIVE STIFFNESS CALCULATIONS:

STIFFNESSES:

2 = 52.6 k/in
5 = 66.7 k/in
13 = 111.1 k/in
J = 111.1 k/in

$X_{Ax} = 57.76$
$X_{Ay} = 33.35$
$X_{Cx} = 57.76$
$X_{Cy} = 33.35$
$X_{Fx} = 57.76$
$X_{Fy} = 33.35$

$$X_R = \frac{\Sigma R_i X_i}{\Sigma R_i} \qquad Y_R = \frac{\Sigma R_i Y_i}{\Sigma R_i}$$

$$X_R = \frac{(111.1)(74.0833) + (33.35)[81.96 + 133.92 + 211]}{111.1 + 3(33.35)}$$

$$= 106.4'$$

$$Y_R = \frac{(66.7)(89) + (52.6)(120.75) + (57.76)[40.39 + 84.89 + 114.89]}{111.1 + 66.7 + 52.6 + 57.76(3)}$$

$$= 64.8'$$

FROM RAM – $X_R = 111.97$
$Y_R = 59.38$

DUE TO THE DIFFICULTY OF THE ANGLED FRAMES X_A, X_C, X_F SEEMS PLAUSIBLE.

CENTER OF MASS FROM RAM: $X_M = 108.6$ $Y_M = 58.62$

ECCENTRICITY: $e_x = X_R - X_M = 106.4 - 108.6 = -2.2'$
$e_y = Y_R - Y_M = 64.8 - 58.62 = 6.18'$

Appendix E: Total Shear Distribution Calculations

Torsional Rigidity				
Frame	Ri (k/in)	di (ft)	di^2 (ft^2)	$R_i d_i^2$
XAx	57.76	24.4	595.4	34388
XAy	33.35	50.1	2510.0	83709
XCx	57.76	27.5	756.3	43681
XCy	33.35	20.09	403.6	13460
XFx	57.76	104.6	10941.2	631961
XFy	33.35	24.4	595.4	19855
J	111.1	32.3	1043.3	115910
13	111.1	64.8	4199.0	466513
5	66.7	24.2	585.6	39062
2	52.6	56.0	3136.0	164954
		$J = \Sigma R_i d_i^2$	Total	1613493.466
				$(k/in)*ft^2$

Direct Shear in x-direction			
Frame	Ri (k/in)	Story Shear (k)	Direct Shear (k)
Vxa(d)	57.76	395	56.52
Vxc(d)	57.76	395	56.52
Vxf(d)	57.76	395	56.52
V2(d)	52.6	395	51.47
V5(d)	66.7	395	65.27
V13(d)	111.1	395	108.71
Total	403.68		

Direct Shear in y-direction			
Frame	Ri (k/in)	Story Shear (k)	Direct Shear (k)
Vxa(d)	33.35	683.51	149.33
Vxc(d)	33.35	683.51	149.33
Vxf(d)	33.35	683.51	149.33
VJ(d)	52.6	683.51	235.52
Total	152.65		

Torsional Shear from x-direction loading					
Frame	Ri (k/in)	di (ft)	e (ft)	Story Shear (k)	Torsional Shear (k)
XAx	57.76	24.4	6.2	395	2.14
XAy	33.35	50.1	2.2	395	0.90
XCx	57.76	27.5	6.2	395	2.41
XCy	33.35	20.09	2.2	395	0.36
XFx	57.76	104.6	6.2	395	9.17
XFy	33.35	24.4	2.2	395	0.44
J	111.1	32.3	2.2	395	1.93
13	111.1	64.8	6.2	395	10.93
5	66.7	24.2	6.2	395	2.45
2	52.6	56.0	6.2	395	4.47
	J = ΣRidi2	Total	1613493.466		
			(k/in)*ft2		

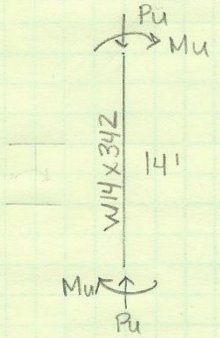
Torsional Shear from y-direction Loading					
Frame	Ri (k/in)	di (ft)	e (ft)	Story Shear (k)	Torsional Shear (k)
XAx	57.76	24.4	6.2	683.51	3.70
XAy	33.35	50.1	2.2	683.51	1.56
XCx	57.76	27.5	6.2	683.51	4.17
XCy	33.35	20.09	2.2	683.51	0.62
XFx	57.76	104.6	6.2	683.51	15.87
XFy	33.35	24.4	2.2	683.51	0.76
J	111.1	32.3	2.2	683.51	3.34
13	111.1	64.8	6.2	683.51	18.91
5	66.7	24.2	6.2	683.51	4.24
2	52.6	56.0	6.2	683.51	7.74
	J = ΣRidi2	Total	1613493.466		
			(k/in)*ft2		

Appendix F: Spot Check Calculations

SPOT CHECKS | TECH REPORT #3 | JESSEL ELLIOTT | 1/2

FOR FRAME J (IN y-DIRECTION) @ 5TH FLOOR

COLUMN W14 x 342



$P_u = 567.87$ k $\quad M_{uy} = 5.34$ k·ft $\quad M_{ux} = 3.17$ k·ft

$L_b = 14 \quad L_b = 14(1.65) = 23.1'$ use 24'

$p \times 10^3 = 0.247 \quad b_y \times 10^3 = 0.353 \quad b_x \times 10^3 = 0.463$

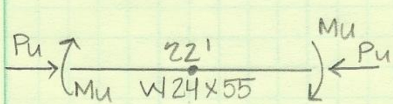
$pP_u = 0.247(567.87) = 140.26 < 200$

use $\frac{1}{2} pP_r + \frac{9}{8}(b_x M_{rx} + b_y M_{ry}) \leq 1000$

$\frac{1}{2}(140.26) + \frac{9}{8}(.353(5.34) + .463(3.17))$

$= 73.9 \ll 1000$ OK

BEAM W24 x 55



$P_u = 22.14$ k

$M_{ux} = 29.56$ k' $\quad L_b = 11'$

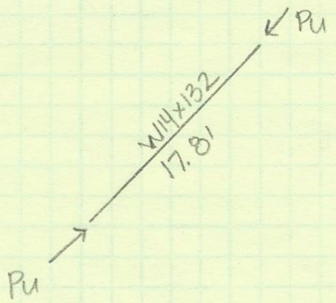
$p \times 10^3 = 2.81 \quad b_x \times 10^3 = 2.45$

$\frac{1}{2} pP_r + \frac{9}{8}(b_y \cancel{M_{ry}}^{0} + b_x M_{rx}) < 1000$

$\frac{1}{2}(2.81(22.14)) + \frac{9}{8}(2.45(29.56))$

$= 112.6 < 1000$ OK

CROSS BRACING W14 x 132



$P_u = 242.76$ k

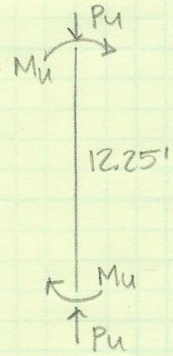
$L_b = 17.8'$

$\phi_c P_n = 1370 > 242.76$ OK

SPOT CHECKS | TECH REPORT #3 | JESSEL ELLIOTT | 2/2

FOR FRAME 5 (IN X-DIRECTION)

COLUMN W14 x 398 @ 1ST FLOOR



$P_u = 1169\ K \quad M_{uy} = 20.05\ K\cdot ft \quad M_{ux} = 16.98\ K\cdot ft$

$L_b = 12.25' \qquad L_b = 12.25(1.66) = 20.3 \approx 22$

$p \times 10^3 = 0.292 \quad b_y \times 10^3 = .353 \quad b_x \times 10^3 = .361$

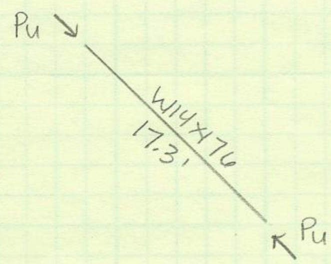
$pP_u = (.292)(1169) = 341.3 > 200$ use

$pP_r + b_y M_{ry} + b_x M_{rx} \leq 1000$

$341.3 + .353(20.05) + .361(16.98)$

$= 354.5 < 1000$ OK

CROSS BRACE W14x176 @ 2nd FLOOR

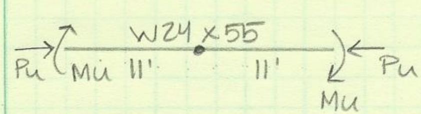


$P_u = 299.1\ K$

$L_b = 17.31$

$\phi_c P_c = 1890\ K \gg 299.1\ K$ OK

BEAM W24x55 @ 4TH LEVEL



$P_u = 19.21\ K$

$M_{ux} = 22.03\ K' \quad L_b = 11'$

$p \times 10^3 = 2.81 \quad b_x \times 10^3 = 2.45$

$\frac{1}{2} pP_r + 9/8 (\cancel{b_y M_{ry}} + b_x M_{rx}) < 1000$

$\frac{1}{2}(2.81)(19.21) + 9/8(2.45(22.03))$

$= 87.7 \ll 1000$ OK