Temple University

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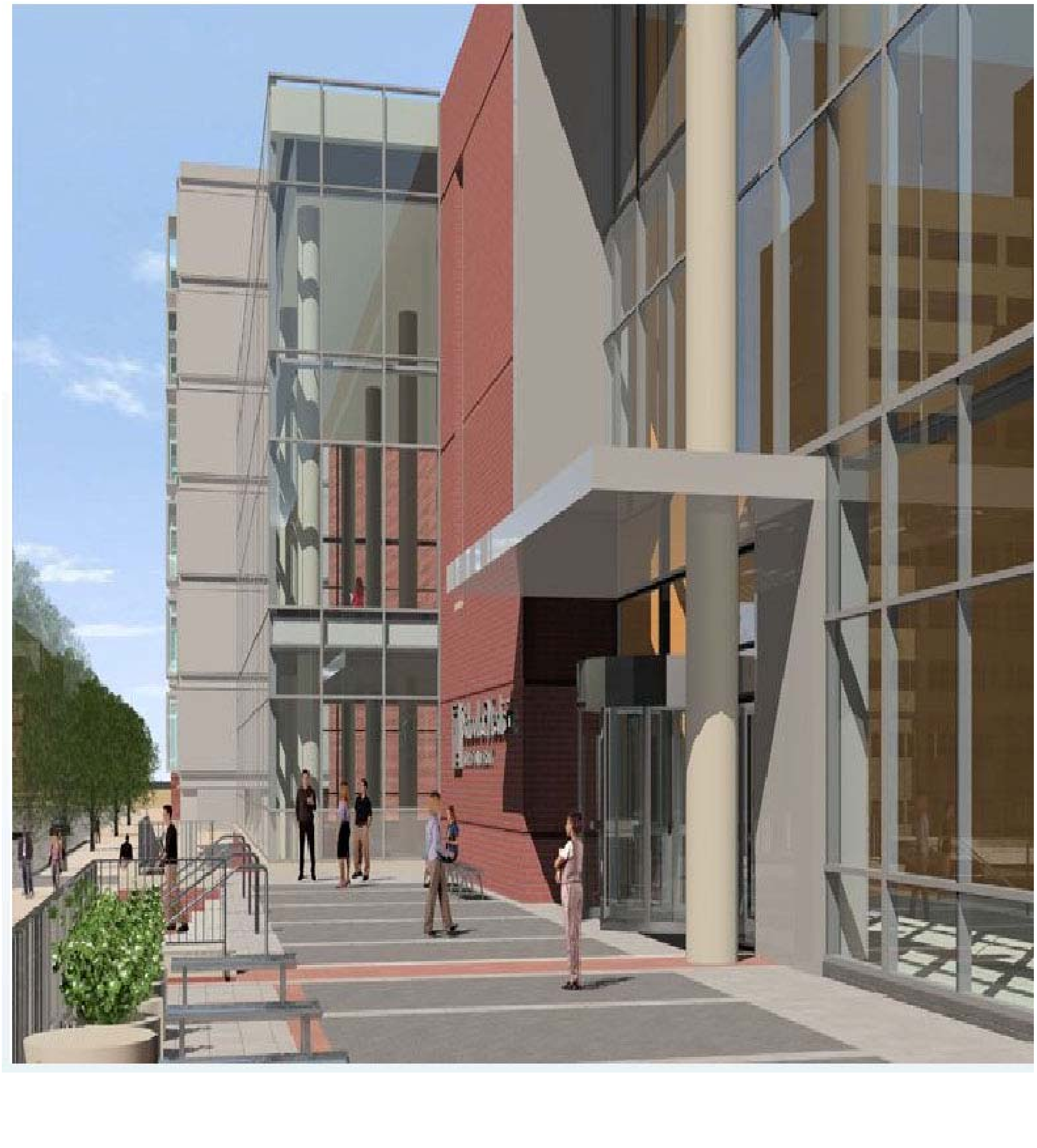
Multipurpose Health Science Center



[TECHNICAL ASSIGNMENT 3]

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Executive Summary

The purpose of this report is to investigate the lateral system of the Temple University Multipurpose Health Science Center. Wind and seismic forces in various load combinations were applied to a RAM Structural System model – including hand back-up calculation – in order to check the lateral system strength, drift, and shear, and impacts on the foundations including overturning.

The health science center is a new 480,000 square foot, $150 million addition to Temple University's medical campus north of Center City Philadelphia, Pennsylvania. The 13 story building contains offices, a café, and a large library, with spaces primarily allocated to laboratories, classrooms, and their support facilities. The architectural features are governed by the concept of expressing internal functions externally, where the curved glass façade of the east elevation indicating the office areas with the brick rectangle behind it expressing the laboratories and classrooms, and the oval tower expressing student meeting and studying spaces.

Built on a previous parking lot, the steel framed building will connect to an existing building via a bridge and tunnel. Foundations consist of 40% footings and 60% caissons which terminate at solid bedrock, present at 30' to 50' depths. The lateral system consists of concentric braced frames in the E-W direction, with moment frames in the N-S direction, all of which are supported by caissons. The floor system consists of composite steel decking.

The lateral analysis of this building includes load determination, distribution, and calculation of torsional shears, with checks for strength, overturning moment, and story drift. Lateral load analysis yielded a wind base shear of 1525 kips in the East-West direction and 506 kips in the North-South direction, while seismic analysis resulted in a 1001 kips base shear acting in both directions. These loads were then manually distributed vertically, and then laterally distributed using a RAM Structural System model. Wind load combinations in the East-West direction and seismic load combinations in the North-South direction controlled the distribution. Relative stiffness was checked manually, since it is the guiding concept behind these distributions. After the relative stiffness of the frames was calculated, the story shear given by the RAM output could be manually checked. Significant discrepancies were most likely due to the simplified analysis method used to determine the relative stiffness of the frames.

An analysis of torsional shear was also included using center of mass and geometry, which was verified by hand. It was found that torsional shear was greatest for both controlling load conditions due to the asymmetrical base of the building, but the seismic torsional shear was especially high due to the size of the base shear in the North-South direction, which was much higher than the wind load combinations in the same direction.

Since member sizes were included in the computer model, the output member forces were compared to the capacities listed in the design documents, yielding satisfactory results. Additionally, an overturning moment of nearly 40,000 k-ft was calculated using story shear data and compared to bearing capacities derived from the design documents. Both tension capacity (3077 kips) and compressions bearing capacities (3014 kips) proved adequate to resist the overturning moment reactions of 1035 kips of a typical frame. Lastly, a story drift analysis yielded wind load drifts under the allowable 6", but seismic loads did not meet restrictions with drifts around 13". This is most likely due to a combination of analysis error and the fact that the building was not specifically designed for seismic loading.

Introduction

Building Description

The Temple University Multipurpose Health Science Center is a new 480,000 square foot, $150 million addition to Temple University's medical campus north of Center City Philadelphia, Pennsylvania. The 13 story building contains offices, a café, and a large library, with spaces primarily allocated to laboratories, classrooms, and their support facilities. The architectural features are governed by the concept of expressing internal functions externally, where the curved glass façade of the east elevation indicating the office areas with the brick rectangle behind it expressing the laboratories and classrooms, and the oval tower expressing student meeting and studying spaces.

Figure 1: Temple University Multipurpose Health Science Center. Image from www.temple.edu/medicine/

Report Topics

This report will continue with a description of the loads and load cases, as well as which had the most significant impact on the lateral system. This will be followed by a description of how these loads were distributed to the lateral system via hand calculations and a computer model. Strength, overturning moment, and drift are checks included in the analysis as well. Lastly, a conclusion draws together the finding of this report. This is followed with the appendix which includes additional calculations.

Structural Description

Soils & Foundations

Built on a previous parking lot, the steel framed building will connect to an existing building via a bridge and tunnel. The geotechnical survey indicated solid bedrock at 30’ to 50’ depths, compact micaceous silty fines to coarse sand gravel at slightly shallower depths, and a similar, but less compact soil near the surface.

Foundations consist of 40% footings and 60% caissons which terminate at solid bedrock. Footing thickness ranges from 1’4” to 4’4”, with sizes generally ranging from 4’x4’ to 9’x9’. The deep foundation system consists of steel columns sitting on concrete piers, caps and caissons. These vary in diameter from 36” to 96” and bear down to bedrock. Basements slabs are located at 15’ to 25’ depths, have welded wire reinforcement, and are typically 6” thick, with 8”-12” thick slabs in areas exposed to weather or with heavier loading.

The concrete used is 28-day, normal weight concrete at f’c=4000 psi for most areas, with the primary exception being concrete exposed to weather-for example, the truck ramp-which is also air-entrained, normal weight at f’c=5000. Reinforcing is grade 60.

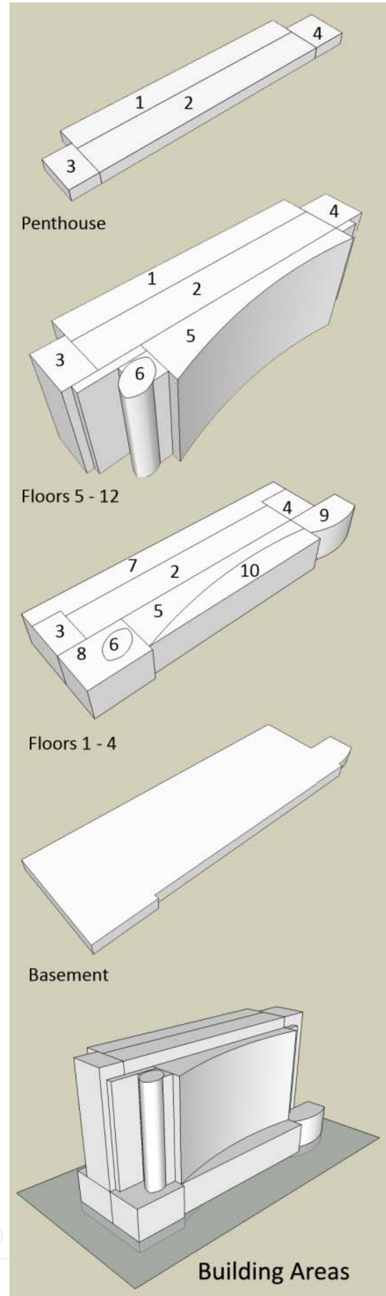

Figure 3: Building Areas

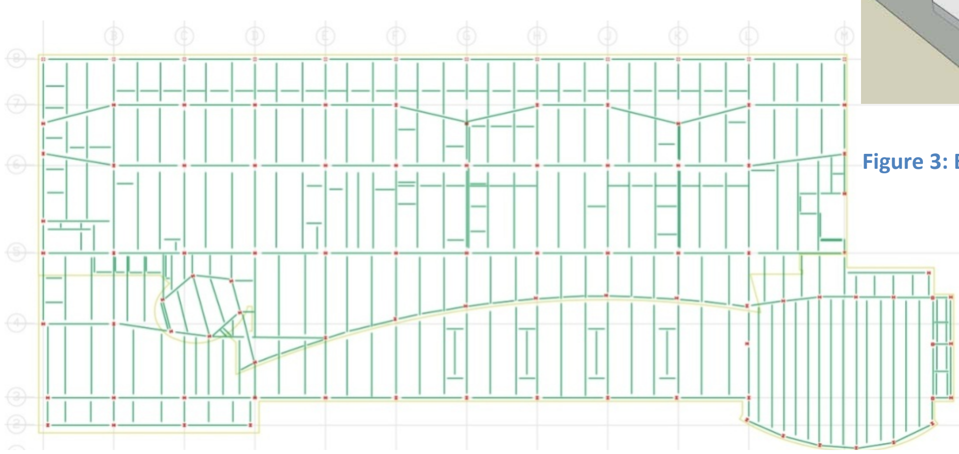

Figure 2: Framing Layout

Steel Framing

Due to the irregularities in the building form the framing layout varies significantly within a floor, but remains similar vertically. See figures 2 3, and 4 for a graphic representation.

The primary lateral system shown in figure 4 consists of five concentrically braced frames labeled 0 through 4in the E-W direction (short direction), with two large moment frames labeled 6 and 7in the N-S direction (long direction). At the penthouse levels, braced frames are solely used for both directions, connecting to the primary frames. These frames consist mostly of W-shapes, with L-shapes used in the penthouse levels. All of the frames bear down on caissons with approximately 3,100 kip capacities.

The gravity system consists of composite steel decking running in the N-S direction, perpendicularly to the beams. Slabs are typically 2.5”, f’c=4,000 psi, NWC on 3”deep, 20 gage, galvanized composite steel deck, with welded wire reinforcement. Floor loads are carried to from beams, to girders to columns, which are usually two levels in height. Special framing occurs in the library (area 8), the three story atrium space (area 10), and the auditorium (area 9) due to the large open spaces.

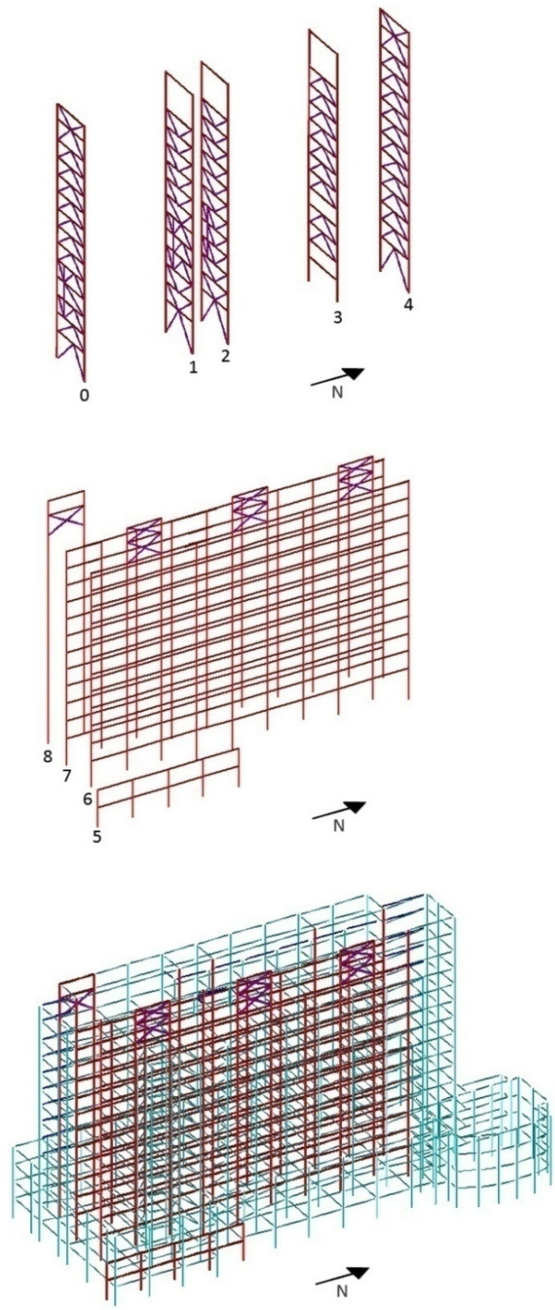

Figure 4: From top to bottom: E-W lateral system, N-S lateral system, overall framing

This building also has three transfer trusses which take column point loads from above and redistribute them to offset columns at a lower level. Two of these trusses are located between the first and second floors, are 15’4” deep, and span 46.5’ in order to clear space for the loading dock below. A third truss is located between the 5th and 6th floors, is 14’8” deep, and spans 62’ in order to relocate columns for corridors on lower levels.

Applicable Codes

Below are listed the codes used by the original designers.

- IBC 2003 (Philadelphia building code)
- ASCE7-02
- Concrete:
  - ACI 318 "Building Code Requirements for Structural Concrete"
  - ACI 316 "Manual of Standard Practice for Detailing Concrete Structures"
  - ACI 301, 302, 304, 305, 306, 308, 311, 318, 347
- Steel:
  - AISC "Specifications for Design, Fabrication and Erection of Structural Steel for Buildings"
  - AISC "Code of Standard Practice for Steel Buildings and Bridges"
  - American Welding Society (AWS) D1.1 "Structural Welding Code – Steel."
  - American Welding Society (AWS) D1.1 "Structural Welding Code – Steel."
  - ASTM A6 " General Requirements for Rolled Steel Plates, Shapes, Sheet Piling, and Bars for Structural Use."
  - ASTM A325 "Specifications for Structural Joints"
  - Steel Deck Institute "Design Manual for Composite Decks, Form Decks, and Roof Decks"

For this design and analysis IBC 2006 and ASCE7-05 will be used

Loads and Load Cases

Live & Dead Loads

The loads in tables 1 and 2 were determined by reviewing the building documents and noting the loads used by the original designers, who based their loading off of the IBC 2003, the adopted building code of Philadelphia, Pennsylvania.

Design dead loads, found in table 3 were not presented in the building documents, so material unit weights and ASCE 7-05 Minimum Design Dead Loads were used to make dead load assumptions.

Table 1: Loads	
Live Loads	
Area	Load (psf)
Slab on Grade	150
Truck Drive Aisle	300
High Density Storage Area	300
Elevated Frame Slabs	150
Office/corridor	100
Library	150
Roof	30
Penthouse	150
Dead Loads	
	Load (psf)
Decking	50.1
Girders & Beams	7
Mech/Elec	20
Total	77.1
Snow Loads	
Flat-roof snow load	22 psf
Snow Exposure Factor	0.9
Snow Load Importance Factor	1.1
Thermal Factor	1

Wind Loads

Wind lateral loads were based off of the ASCE7-05-6.5 Analytical Procedure. Basic assumptions about the building and site –listed below and presented in full detail in the appendix – were used to determine the gust factor G_f, internal pressure coefficients C_{pi}, and velocity pressure q_z. These were then used to determine windward and leeward pressures for distinct elevations, as seen in table 2 below. Distribution was accomplished by taking the wind pressures at each level and the respective portion of the individual story height affected by the wind pressure (See figure 5 in the Load Distribution section). These were then summed and multiplied by the length of the floor level to obtain a story shear. This was then done for each level. Lastly the individual story shears were summed to obtain a base shear.

The base shear was 1525 kips in the East-West direction to the large area of those elevations, while the base shear for the North-South direction was only 506 kips since the area of these elevations is so much smaller. Full calculations can be found in the appendix.

Basic wind speed	-	90mph	Topographic factor	-	1
Exposure category	-	B	G_f	-	0.803
Importance factor	-	1.15	C_{pi}	-	±0.18
Occupancy type	-	III			

Table 2: Windward Pressure (psf)

Height	kz	qz=20.269*kz	$p=q*G_f*C_p$	± qi*(GCpi)	=	p
0-15	0.57	11.55	7.42	0	=	7.42
20	0.62	12.57	8.07	0	=	8.07
25	0.66	13.38	8.59	0	=	8.59
30	0.7	14.19	9.11	0	=	9.11
40	0.76	15.40	9.90	0	=	9.90
50	0.81	16.42	10.55	0	=	10.55
60	0.85	17.23	11.07	0	=	11.07
70	0.89	18.04	11.59	0	=	11.59
80	0.93	18.85	12.11	0	=	12.11
90	0.96	19.46	12.50	0	=	12.50
100	0.99	20.07	12.89	0	=	12.89
120	1.04	21.08	13.54	0	=	13.54
140	1.09	22.09	14.19	0	=	14.19
160	1.13	22.90	14.71	0	=	14.71
180	1.17	23.71	15.23	0	=	15.23
200	1.2	24.32	15.62	0	=	15.62

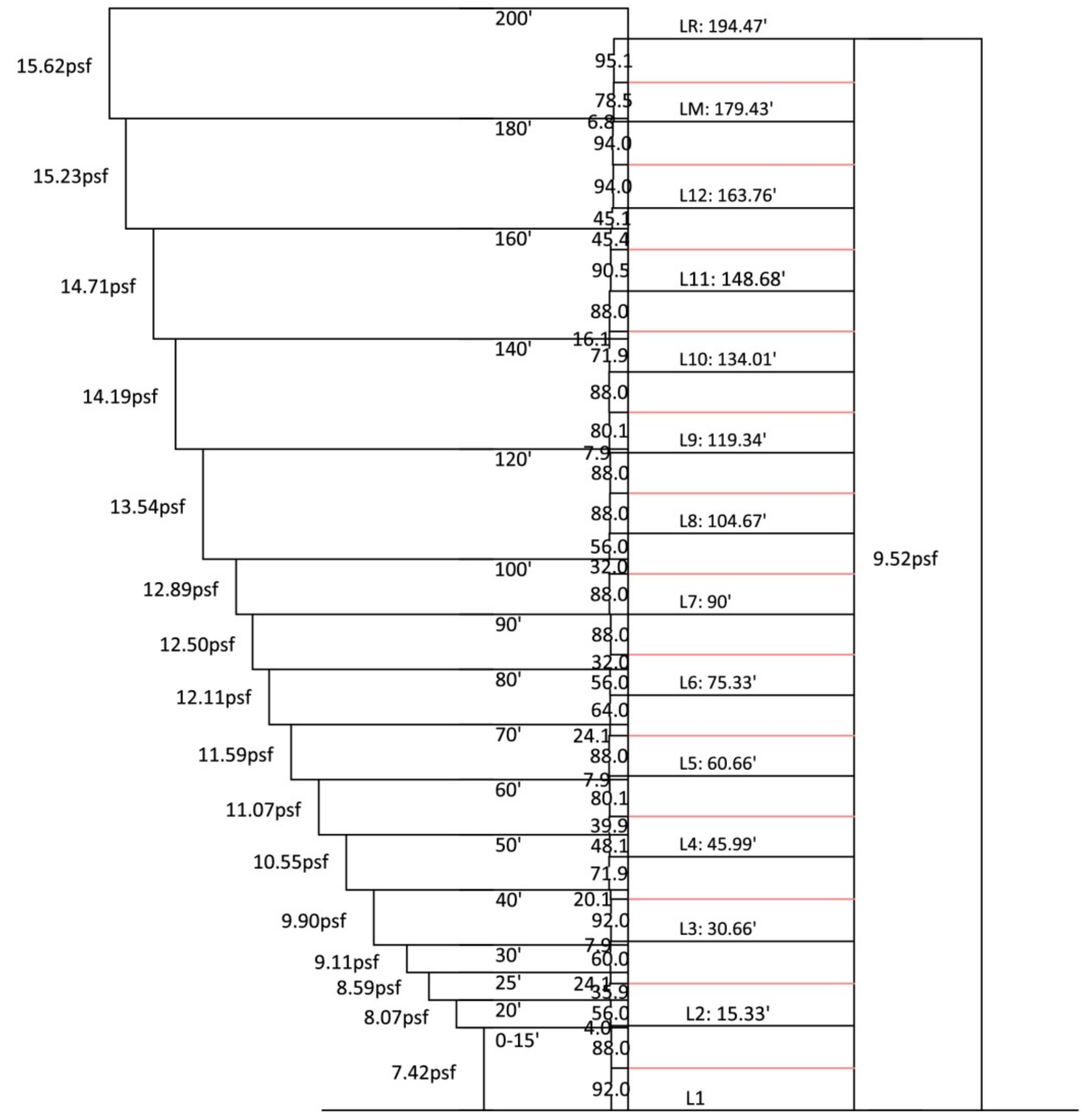

Figure 5: Wind load diagram

Seismic Loads

The ASCE7-05 code was used to investigate the seismic loads for the building which were expected to be relatively low, given the building's location in Philadelphia, Pennsylvania. The Equivalent lateral Force Procedure (ASCE 12.8.2) was used to obtain a base sheer for the building. A direct comparison with the structural notes within the design documents was not possible since the base shear was not indicated in the notes or specifications; however, the notes did state that structural system was not specifically detailed for seismic loads. It also provided various seismic data which were used for this analysis including the site class, C, and the spectral response coefficients: S_{DS}=0.219, S_{D1}=0.068. The estimated dead weight of 77.1psf was used to calculate the building base shear of 1001 kips.

Below appears the ASCE equation for determining the vertical distribution of seismic loads, which is found in table 3, of the Lateral Force Distribution Section. This was accomplished by multiplying the base shear by the appropriate C_{vx}, which is a function of floor area, height, building weight, and the seismic response coefficient.

$$Cvx = \frac{(w_x * H_x^K)}{(\sum wx * Hx^K)}$$

Table 3: Vertical Seismic Distribution

Level	Area (SF)	w_x (K)= A * DL(77.1psf) * C_s(.028)	H_x	K	$w_x * H_x^K$	Cvx = $\frac{(w_x * H_x^K)}{(\sum wx * Hx^K)}$	F_x (K)= C_{vx} * V(1001.25k)
1st	57000	123.05	0.00	1.272	0.00	0.00	0.00
2nd	51800	111.83	15.53	1.272	3661.95	0.01	13.29
3rd	45000	97.15	30.67	1.272	7560.05	0.03	27.44
4th	57000	123.05	46.00	1.272	16036.71	0.06	58.20
5th	28000	60.45	60.67	1.272	11202.46	0.04	40.66
6th	28000	60.45	75.33	1.272	14752.77	0.05	53.54
7th	28000	60.45	90.00	1.272	18499.79	0.07	67.14
8th	28000	60.45	104.67	1.272	22417.35	0.08	81.36
9th	28000	60.45	119.33	1.272	26484.76	0.10	96.12
10th	28000	60.45	134.00	1.272	30693.59	0.11	111.39
11th	28000	60.45	148.67	1.272	35029.87	0.13	127.13
Pent.	28000	60.45	163.75	1.272	39610.39	0.14	143.75
Mez.	7500	16.19	181.50	1.272	12093.85	0.04	43.89
Roof	21500	46.41	194.46	1.272	37848.00	0.14	137.36
	463800	1001.25			275891.56		1001.25

Load Combinations

The following IBC 2006 load combinations were analyzed using RAM Structural System. The second and third groups of Load combinations had the most significant impact on the lateral system, while the bold underlined load combinations controlled.

$1.4D$
$1.2D + 1.6L_r$
$1.2D + 1.6S$
$1.2D + 1.6L_r + 0.5S$
$1.2D + 1.6S + 0.5L_r$
$1.2D + 1.6L_r \pm 0.8W$
$1.2D + 1.6S \pm 0.8W$

$1.2D + 0.5S + 0.5L_r \pm 1.6W$
$1.2D + 0.5S \pm 1.6W$
$0.9D \pm 1.6W$
$1.2D \pm 1.6W$
$\underline{1.2D + 0.5L_r \pm 1.6W}$

$0.9D \pm 1.0E$
$\underline{1.2D \pm 1.0E}$

Lateral Force Distribution

Lateral loads in the Temple University Multipurpose Health Science Center are resisted through a combination of braced frames and moment frames. The loads acting on these frames were determined through a combination of hand calculations and computer modeling. The vertical lateral distributions were manually imputed into a RAM Structural System model, shown below. The floors of the building were modeled as rigid diaphragms which allowed the relative stiffness of the different frames to be determined by hand. The relative stiffness of the frames was then used by the RAM model with various dead, live, wind and seismic load combinations to determine individual story shears, and frame loads.

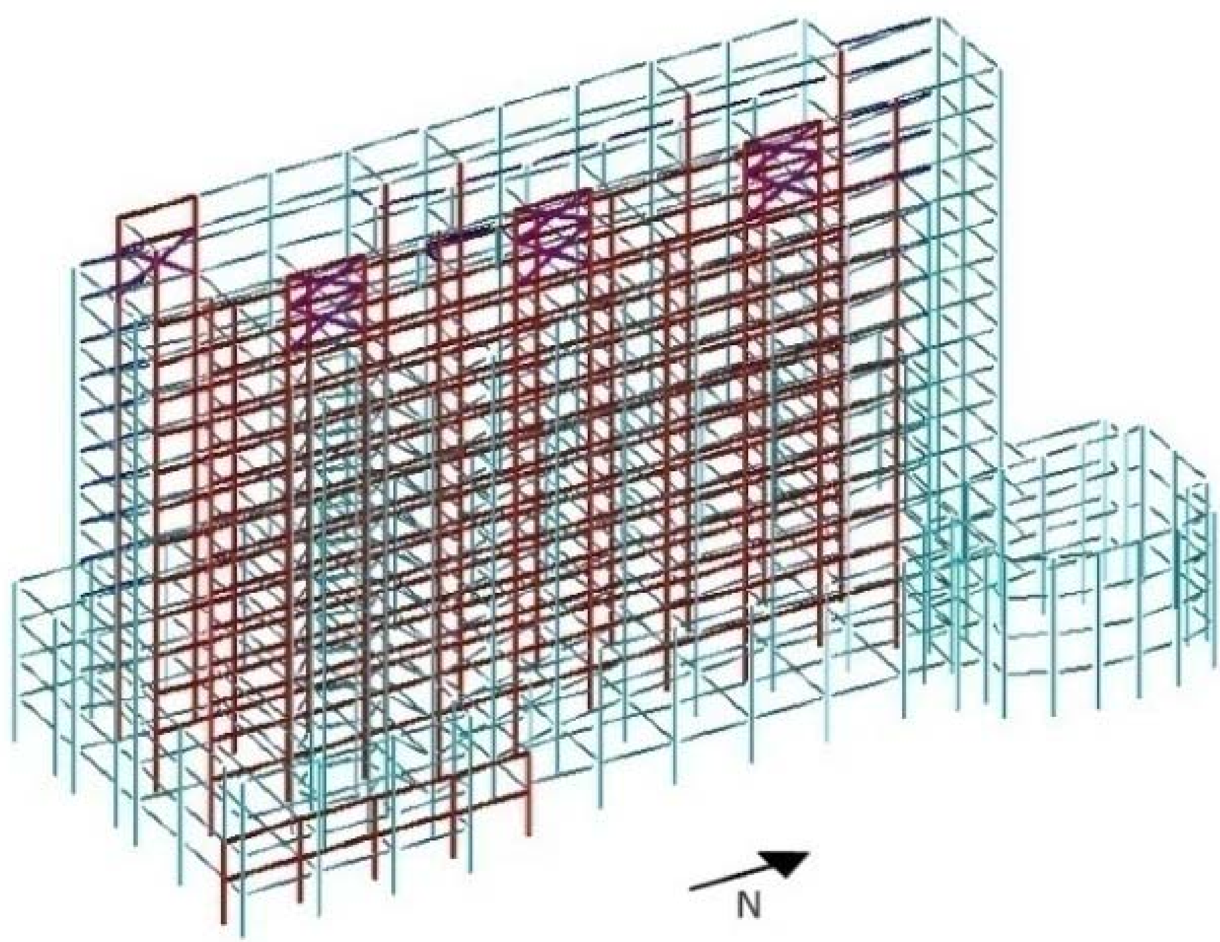

Figure 6: RAM Framing Model

A detailed analysis of the lateral resisting system was made possible by creating a RAM Structural System model. Five concentrically braced frames - labeled 0 through 4 - provide the primary resistance to East-West lateral loading, while two primary moment frames - labeled 6 and 7 - provide resistance to North-South loading. There are also two other frames in this direction - labeled 5 and 8 – which provide lateral resistance for the two story library and penthouse, respectively. It is assumed that all of the lateral loads are transferred to these frames.

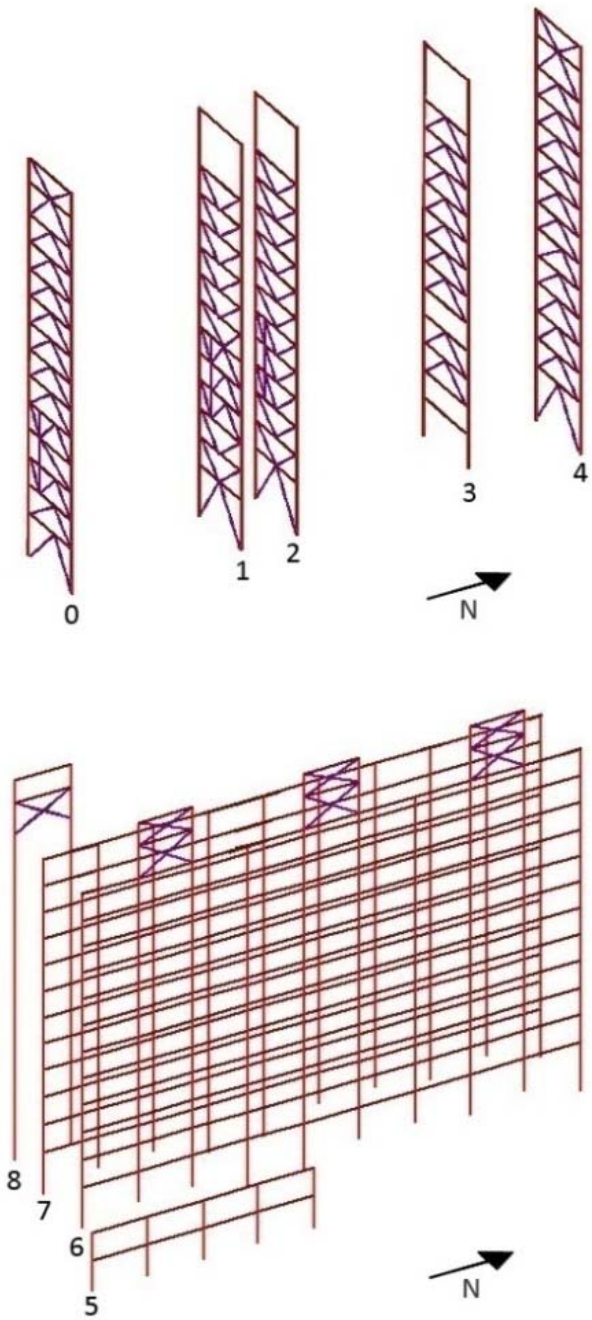

Figure 7: Above: E-W braced frames. Below: N-S moment frames

The floors of the building were modeled as rigid diaphragms, which is an acceptable assumption for a composite steel deck floor system. Forces applied to these rigid diaphragms are then transferred through the floor to the lateral elements via relative stiffness. Relative stiffness for this building was determined through a separate computer model of the lateral forms. Unit loads were applied to each individual story level of each frame, resulting in displacements. The inverse of these displacements results in the stiffness of each frame, so that each frame can be compared to the others on a floor by floor basis. One floor two, for example, the stiffness of frame # 1 is higher than the rest, indicating that it will take a higher percentage of the lateral loading, dependent on its distance to the center of rigidity (see table below). This topic is further discussed in the Torsion section.

Table 4: Relative Stiffness

	E-W Frames									
	Frame #0		Frame #1		Frame #2		Frame #3		Frame #4	
Level	Disp Y	1/Disp	Disp Y	1/Disp	Disp Y	1/Disp	Disp Y	1/Disp	Disp Y	1/Disp
Roof	0.00198	505.05	0.05601	17.85	0.05558	17.99	0.05518	18.12	0.00198	505.05
Mez.	0.00225	444.44	0.06413	15.59	0.0634	15.77	0.06869	14.56	0.00223	448.43
Pent.	0.00123	813.01	0.00105	952.38	0.00109	917.43	0.00107	934.58	0.00097	1030.93
11th	0.00117	854.70	0.00115	869.57	0.00119	840.34	0.00116	862.07	0.00114	877.19
10th	0.00115	869.57	0.00113	884.96	0.00116	862.07	0.00114	877.19	0.00112	892.86
9th	0.00107	934.58	0.00111	900.90	0.00114	877.19	0.00112	892.86	0.00109	917.43
8th	0.001	1000.00	0.00109	917.43	0.00111	900.90	0.00109	917.43	0.00102	980.39
7th	0.00094	1063.83	0.00108	925.93	0.00109	917.43	0.00103	970.87	0.00092	1086.96
6th	0.00078	1282.05	0.00096	1041.67	0.00105	952.38	0.00096	1041.67	0.00084	1190.48
5th	0.00071	1408.45	0.0011	909.09	0.00108	925.93	0.0095	105.26	0.00064	1562.50
4th	0.00088	1136.36	0.00093	1075.27	0.00096	1041.67	0.00078	1282.05	0.00068	1470.59
3rd	0.00072	1388.89	0.00083	1204.82	0.00078	1282.05	0.00076	1315.79	0.00063	1587.30
2nd	0.00064	1562.50	0.00047	2127.66	0.00043	2325.58	0.00934	107.07	0.00062	1612.90
1st	-	-	-	-	-	-	-	-	-	-

	N-S Frames							
	Frame #5		Frame #6		Frame #7		Frame #8	
Level	Disp X	1/Disp	Disp X	1/Disp	Disp X	1/Disp	Disp X	1/Disp
Roof	-	-	0.00302	331.1258	-	-	0.03958	25.26529
Mez.	-	-	0.00376	265.9574	-	-	0.00465	215.0538
Pent.	-	-	0.00397	251.8892	0.00406	246.3054	-	-
11th	-	-	0.00311	321.5434	0.00323	309.5975	-	-
10th	-	-	0.00288	347.2222	0.00297	336.7003	-	-
9th	-	-	0.00283	353.3569	0.00292	342.4658	-	-
8th	-	-	0.00273	366.3004	0.00281	355.8719	-	-
7th	-	-	0.00267	374.5318	0.00275	363.6364	-	-
6th	-	-	0.00255	392.1569	0.00261	383.1418	-	-
5th	-	-	0.00223	448.4305	0.00222	450.4505	-	-
4th	-	-	0.00245	408.1633	0.00239	418.41	-	-
3rd	-	-	0.00247	404.8583	0.00238	420.1681	-	-
2nd	0.00737	135.6852	0.00219	456.621	0.00219	456.621	-	-
1st	-	-	-	-	-	-	-	-

Once the structural model was completed, it was possible to insert the loading. Wind analysis resulted in a base shear of 1525 kips in the East-West direction, and 506 kips in the North-South direction, while seismic analysis resulted in a 1001 kips base shear acting in both directions. These analyses as well as the vertical distributions of these loads were determined by hand and entered into the model (See previous wind and seismic load section). It is important to note that resultant wind loads act at the geometric center of the building while seismic loads act at the center of mass. These were determined by the model but verified by hand.

The lateral loads will also combine with gravity loads to further increase displacements and loads in the lateral framing, known as the P-delta effect. This justifies the used of the various multiple load combinations presented; however, it was determined through the computer modeling that the effect of these loads were minimal and greatly outweighed by the lateral loads.

Once entered into the model, the computer distributed the loads according to relative stiffness, giving displacements, story shears, and member forces. Not surprisingly, wind controlled in the East-West direction with the 1.2 D + 0.5 Lp ±1.6W load combination due to the large surface area of that elevation. In the North-South direction, seismic loads controlled with the 1.2 D± 1.0E load combination, which isn't surprising either since the seismic base shear was nearly double that of the wind base shear in this direction. The resulting story shears of the two controlling cases are presented in table 5; however, a full list of all 37 load cases analyzed by RAM is available upon request.

Table 5: Building Story Shears

Level	N-S (+E)	N-S (-E)	E-W (+W)	E-W (-W)
Roof	138.35	-138.47	114.77	-114.42
Mez.	182.6	-182.67	362.53	-325.41
Pent.	334.56	-334.55	544.6	-536.64
11th	470.31	-470.3	744.53	-745.79
10th	585.87	-585.86	-1141.39	846.13
9th	702.39	-702.38	1140.44	-1143.6
8th	796.88	-796.89	1337.44	-1333.25
7th	878.17	-878.17	1523.59	-1519.65
6th	938.75	-938.76	1704.53	-1701.29
5th	992.94	-992.97	1922.07	-1918.99
4th	1078.72	-1078.85	2117.78	-2110.29
3rd	1116.92	-1117.1	2300.82	-2299.13
2nd	1094.87	-1095	2463.78	-2471.79
1st	-648.81	650.59	112.76	104.75

Frame shears are also presented in table 6 in order to demonstrate the distribution according to relative stiffness as well as to provide a comparison to hand calculations. The hand calculations were made by distributing the floor shears to the various frames using a calculated relative stiffness factor. This is the compared to the RAM value for frame story shears. The differences between the hand-calculated frame story shears and RAM calculate frame story shears are similar for only a few of the frames, which is most likely due to the fact that the hand calculations are a simplification. An example of one frame is placed below, in order to demonstrate the comparison as well as show the RAM calculated values. A complete set of calculations for all of the frames is available in the appendix.

Table 6: Frame Shear Comparison								
Relative Stiffness Properties						Frame Shear Check		
	Frame #0							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N)$	H_s	$\frac{H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$	RAM Value
Roof	0.00	505.05	73.54	37141.41	0.256565	114.77	29.45	60.42
Mez.	0.00	444.44	75.51	33560.00	0.262375	362.53	95.12	138.71
Pent.	0.00	813.01	76.16	61918.70	0.133853	544.6	72.90	85.06
11th	0.00	854.70	76.09	65034.19	0.155041	744.53	115.43	56.21
10th	0.00	869.57	76.04	66121.74	0.154729	-1141.39	-176.61	310.69
9th	0.00	934.58	76.05	71074.77	0.161370	1140.44	184.03	190.47
8th	0.00	1000.00	75.93	75930.00	0.163852	1337.44	219.14	249.31
7th	0.00	1063.83	75.75	80585.11	0.161531	1523.59	246.11	304.53
6th	0.00	1282.05	75.46	96743.59	0.175581	1704.53	299.28	328.19
5th	0.00	1408.45	74.94	105549.30	0.210597	1922.07	404.78	536.17
4th	0.00	1136.36	73.99	84079.55	0.131078	2117.78	277.59	371.01
3rd	0.00	1388.89	72.22	100305.56	0.141186	2300.82	324.84	387.28
2nd	0.00	1562.50	71.76	112125.00	0.178369	2463.78	439.46	346.31
1st								

Torsion

Torsion occurs when there is an eccentricity between the center of rigidity and the center of mass (for seismic loads), or center of geometry (for wind loads). Torsional shear forces are dependent on the relative stiffness of a floors lateral framing, its eccentricity, and the story shear. If the eccentricities and loadings are high enough, torsional shear can have significant additive affects on lateral systems. Table 7 contains the calculations for finding eccentricity in this building. Wind controlled in the East-West (y) direction, so the center of geometry for this direction used, while the center of mass was used in the other direction, in which seismic controlled.

Table 7: Eccentricity Calculations								
	Center of Rigidity		Center of Mass/ Geom		eccentricity		% of building length	
	x	y	x	y	x	y	x	y
Roof	185.59	104.54	174.08	119.22	11.51	14.68	3%	22%
Mez.	186.34	106.51	195.77	123.44	9.43	16.93	3%	26%
Pent.	187.51	107.16	172	108.27	15.51	1.11	4%	1%
11th	186.87	107.09	171.86	108.31	15.01	1.22	4%	1%
10th	187.96	107.04	171.82	108.31	16.14	1.27	5%	1%
9th	184.33	107.05	171.87	108.31	12.46	1.26	4%	1%
8th	183.31	106.93	171.85	108.33	11.46	1.40	3%	1%
7th	182.06	106.75	171.84	108.34	10.22	1.59	3%	1%
6th	179.77	106.46	171.73	108.36	8.04	1.90	2%	2%
5th	176.97	105.94	173.98	109.21	2.99	3.27	1%	3%
4th	176	104.99	186.19	94.89	10.19	10.10	3%	6%
3rd	167.42	103.22	198.79	106.05	31.37	2.83	8%	2%
2nd	156.41	102.76	183.31	100.1	26.90	2.66	7%	2%
1st	186.31	95.14	186.31	95.14	0.00	0.00	0%	0%

The eccentricities for wind loads were not too large in this building, and were limited to the penthouse floors where the controlling combined story loading was minimal, resulting in minimal shears. An example of one of the East-West frames, seen in table 8, demonstrates the torsional shear equation as well some of the typical torsional shear forces seen in the building. A complete set of torsional calculations for all of the frames can be found in the appendix, which shows how the torsional shear was greatest on the lowest floors. This is expected since the building geometry greatly increased in size in one direction on these floors, creating a significant eccentricity.

Table 8: Torsional Shear								
Relative Stiffness Properties						Torsional Shear		
	Frame #0							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N^2)$	e	H_s	$\frac{e*H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$
Roof	0.00	505.05	73.54	37141.41	0.003489	14.68	138.35	7.09
Mez.	0.00	444.44	75.51	33560.00	0.003475	16.93	182.6	10.74
Pent.	0.00	813.01	76.16	61918.70	0.001758	1.11	334.56	0.65
11th	0.00	854.70	76.09	65034.19	0.002038	1.22	470.31	1.17
10th	0.00	869.57	76.04	66121.74	0.002035	1.27	585.87	1.51
9th	0.00	934.58	76.05	71074.77	0.002122	1.26	702.39	1.88
8th	0.00	1000.00	75.93	75930.00	0.002158	1.40	796.88	2.41
7th	0.00	1063.83	75.75	80585.11	0.002132	1.59	878.17	2.98
6th	0.00	1282.05	75.46	96743.59	0.002327	1.90	938.75	4.15
5th	0.00	1408.45	74.94	105549.30	0.002810	3.27	992.94	9.12
4th	0.00	1136.36	73.99	84079.55	0.001772	10.10	1078.72	19.30
3rd	0.00	1388.89	72.22	100305.56	0.001955	2.83	1116.92	6.18
2nd	0.00	1562.50	71.76	112125.00	0.002486	2.66	1094.87	7.24
1st								

The seismic eccentricities and controlling combined story loads were large enough to combine to create very significant torsional shear values, as seen in table 9. Once again, the larger base geometry created a greater eccentricity and larger shear values. This will have a significant effect on the sizing of the lateral system in the North-South direction.

Table9: Torsional Shear								
	Frame #6							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N^2)$	e	H_s	$\frac{e*H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$
Roof	0.00302	331.13	96.99	32114.54	0.010059	11.51	114.77	13.29
Mez.	0.00376	265.96	97.74	25993.59	0.008058	9.43	362.53	27.55
Pent.	0.00397	251.89	98.91	24913.32	0.006324	15.51	544.6	53.41
11th	0.00311	321.54	98.27	31596.75	0.006411	15.01	744.53	71.65
10th	0.00288	347.22	99.36	34498.58	0.006308	16.14	1141.39	116.21
9th	0.00283	353.36	95.73	33825.41	0.006607	12.46	1140.44	93.89
8th	0.00273	366.30	94.71	34690.81	0.006690	11.46	1337.44	102.54
7th	0.00267	374.53	93.46	35002.21	0.006804	10.22	1523.59	105.94
6th	0.00255	392.16	91.17	35751.33	0.007004	8.04	1704.53	95.98
5th	0.00223	448.43	88.37	39625.96	0.007214	2.99	1922.07	41.46
4th	0.00245	408.16	87.40	35671.80	0.007263	10.19	2117.78	156.73
3rd	0.00247	404.86	78.82	31909.27	0.008276	31.37	2300.82	597.34
2nd	0.00219	456.62	67.81	30961.60	0.007340	26.90	2463.78	486.44
1st	-	-						

Strength Check

Since the existing member sizes were used in the RAM model, a strength check was performed by comparing the frame member loads found through the RAM model with the original design. The basic frame shears have already been backed up with hand calculations, which should support this analysis. The following images demonstrate that the capacities of the braced frame's members are adequate. Discrepancies occur because the RAM model was created without all of the gravity members in place in order to simplify the lateral analysis. Also, the 1st floor was modeled as the ground level in order to obtain accurate story drift, but this reduced the amount of load reaching the braces on the lowest level.

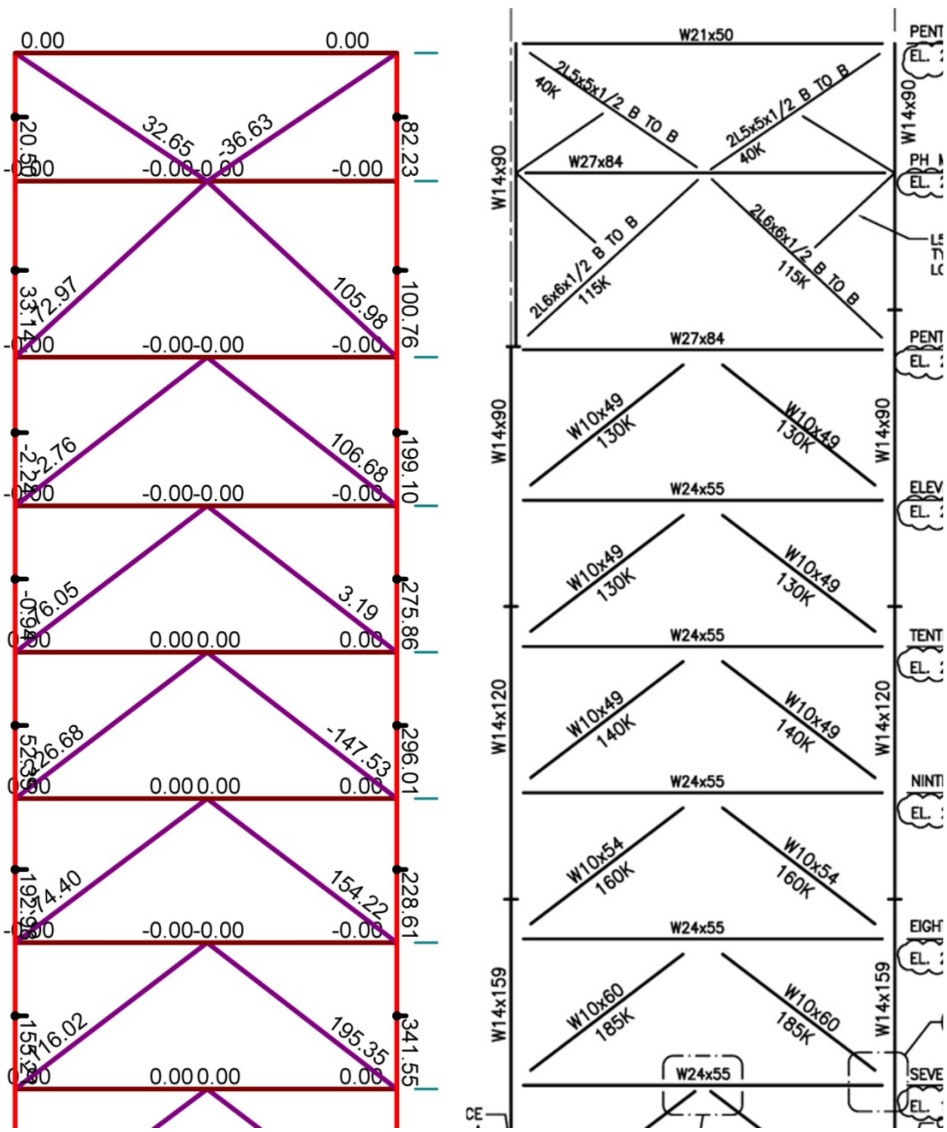

Figure 8: Strength check. Model left, original right.

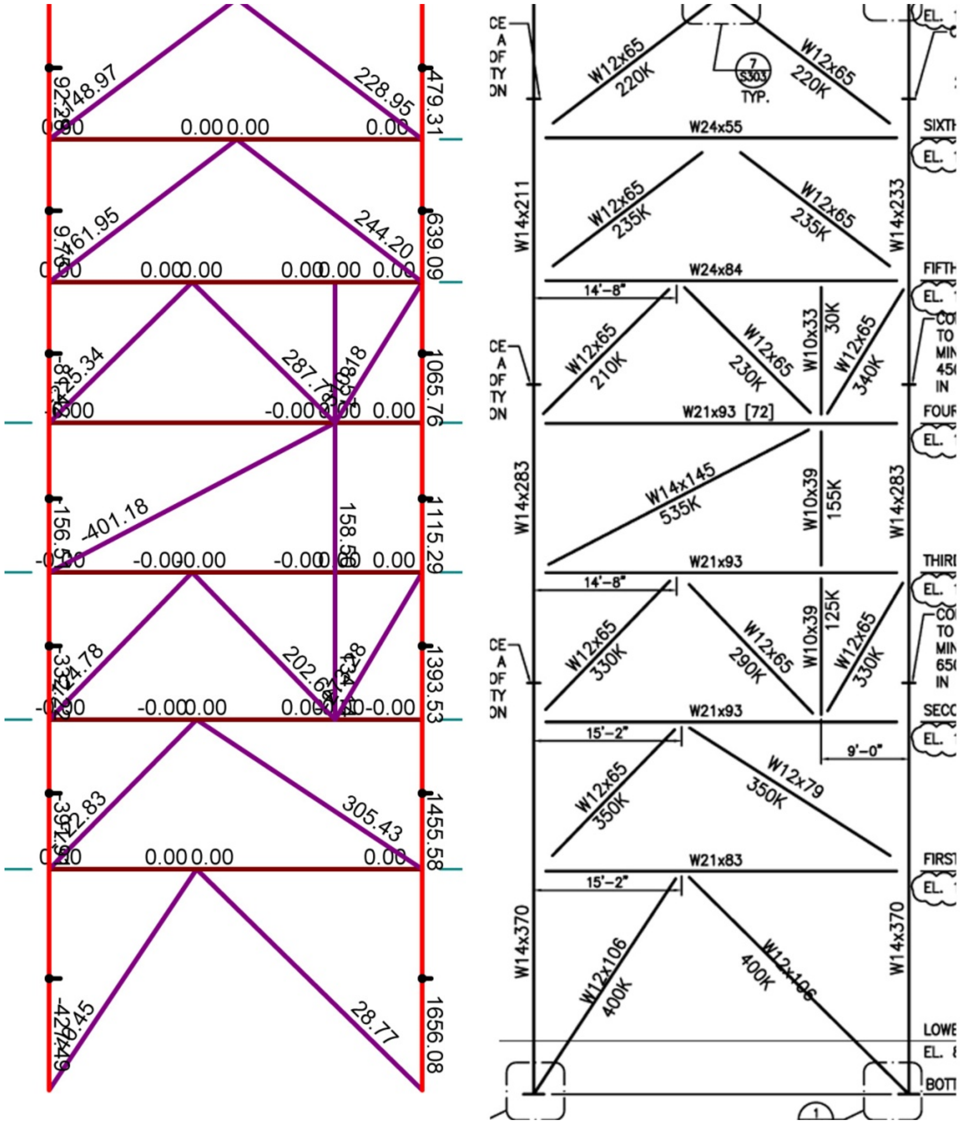

Figure 9:Strength check continued. Model left, original right.

Overturning Moments

Overturning moments were calculated in order to check the lateral frame's stability. Frame story shears were multiplied by their heights to get the overturning moment. Reactions were then found and compared to the bearing capacities listed in the structural notes, which is 60,000psf end bearing, and 2000psf tensile.

Table 10: Bearing Capacity			
	Story Shear	Height	Moment
Roof	60.42	194.46	11749.27
Mez.	78.29	181.50	14209.64
Pent.	-53.65	163.75	-8785.19
11th	-141.28	148.67	-21004.1
10th	-254.47	134.00	-34099
9th	501.15	119.33	59802.23
8th	58.84	104.67	6158.783
7th	55.23	90.00	4970.7
6th	23.65	75.33	1781.555
5th	207.99	60.67	12618.75
4th	-165.16	46.00	-7597.36
3rd	16.27	30.67	499.0009
2nd	-40.96	15.33	-627.917
1st	-351.23	0.00	0
Sum			39676.39
Bay width: 38.333'			
Reactions			1035.045
Caisson size: 96" Ø			8'
Bearing capacity: 60,000psf			3014.4 > 1035, ok
Caisson depth: 35'			
Allowable side resistance: 7000psf			3077.2 >1035, ok

Story Drifts

The final check for this report is an analysis of the story drift. The IBC limits overall, as well as story drift to H/400. That means an overall building drift less than six inches for this building. The maximum drift value was just a hair under the limit for the wind load case but the seismic loading resulted in twice the allowable drift. This is a bit high; however, the original design documents indicate that the building is not specifically detailed for seismic, which is at least a partial explanation. Otherwise, the wind load story drift is also acceptable.

Table 11: Story Drift

		E-W				N-S			
Level	Height	Disp	ΔDisp	H/400		Disp	ΔDisp	H/400	
Roof	194.46	5.99	1.01	0.03	ok	13.54	0.17	0.03	fail
Mez.	181.50	4.98	0.61	0.04	ok	13.37	0.25	0.04	fail
Pent.	163.75	4.37	0.44	0.04	ok	13.12	0.68	0.04	fail
11th	148.67	3.93	0.36	0.04	ok	12.44	0.84	0.04	fail
10th	134.00	3.57	0.14	0.04	ok	11.6	1.01	0.04	fail
9th	119.33	3.43	0.44	0.04	ok	10.59	1.19	0.04	fail
8th	104.67	2.99	0.46	0.04	ok	9.4	1.3	0.04	fail
7th	90.00	2.53	0.447	0.04	ok	8.1	1.38	0.04	fail
6th	75.33	2.083	0.963	0.04	ok	6.72	1.33	0.04	fail
5th	60.67	1.12	0.45	0.04	ok	5.39	1.4	0.04	fail
4th	46.00	0.67	0	0.04	ok	3.99	1.55	0.04	fail
3rd	30.67	0.67	0.37	0.04	ok	2.44	1.49	0.04	fail
2nd	15.33	0.3	0.3	0.04	ok	0.95	0.95	0.04	fail
1st	0.00	0	0	0.00	-	0	0	0.00	-
Max:	H/400	6" >5.99, ok				6" <13.54, fail			

Conclusions

The purpose of this technical report was to analyze the lateral system of the Temple University Multipurpose Health Science Center. This analysis included load determination, distribution, and calculation of torsional shears, with checks for strength, overturning moment, and story drift. Lateral load analysis yielded a wind base shear of 1525 kips in the East-West direction and 506 kips in the North-South direction, while seismic analysis resulted in a 1001 kips base shear acting in both directions. These loads were then manually distributed vertically, and then laterally distributed using a RAM Structural System model. Wind load combinations in the East-West direction and seismic load combinations in the North-South direction controlled the distribution. Relative stiffness was checked manually, since it is the guiding concept behind these distributions. After the relative stiffness of the frames was calculated, the story shear given by the RAM output could be manually checked. Significant discrepancies were most likely due to the simplified analysis method used to determine the relative stiffness of the frames.

An analysis of torsional shear was also included using center of mass and geometry, which was verified by hand. It was found that torsional shear was greatest for both controlling load conditions due to the asymmetrical base of the building, but the seismic torsional shear was especially high due to the size of the base shear in the North-South direction, which was much higher than the wind load combinations in the same direction.

Since member sizes were included in the computer model, the output member forces were compared to the capacities listed in the design documents, yielding satisfactory results. Additionally, an overturning moment of nearly 40,000 k-ft was calculated using story shear data and compared to bearing capacities derived from the design documents. Both tension capacity (3077 kips) and compressions bearing capacities (3014 kips) proved adequate to resist the overturning moment reactions of 1035 kips of a typical frame. Lastly, a story drift analysis yielded wind load drifts under the allowable 6”, but seismic loads did not meet restrictions with drifts around 13”. This is most likely due to a combination of analysis error and the fact that the building was not specifically designed for seismic loading.

Appendix: Wind Load Calculations

Wind : ASCE 7-05 | Method 2 : Analytical Procedure | 1

This method was used since height > 60'

① Basic windspeed (Fig 6-1)
$V = 90$ mph

Wind Directionality Factor (Table 6-4)
$K_d = 0.85$

② Importance Factor
Occupancy III Table 1-1

$I = 1.15$

③ Exposure Category
Urban : B

④ Topographic Factor
$K_{zt} = 1$

⑤ Gust Effect Factor
$T = 1.044s$ (from Table 12.8-2)
$n_1 = \frac{1}{1.044} = 0.96 < 1 \longrightarrow$ Flexible Structure

$$G_f = 0.925 \left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}} \right)$$

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2 \ln(3600 n_1)} + \frac{0.577}{\sqrt{2 \ln(3600 n_1)}} = 4.180$$

Resonant Response Factor

$$L_{\bar{z}} = \ell \left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 487.948$$

$\bar{z} = 0.6h = 0.6(195') = 117'$
$\ell = 320$ ft (Table 6-2)
$\bar{\epsilon} = 1/3.0$
$\bar{b} = 0.45$
$\bar{\alpha} = 1/4.0$

$$\bar{V}_{\bar{z}} = \bar{b} \left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V \left(\frac{88}{60}\right) = 0.45 \left(\frac{117'}{33}\right)^{1/4.0} (90)\left(\frac{88}{60}\right) = 81.509$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.96(487.948)}{81.509} = 5.747$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(5.747)}{(1 + 10.3(5.747))^{5/3}} = 0.046$$

2

R_h : $n = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.96)(195')/81.509 = 10.565$

$R_h = \frac{1}{n} - \frac{1}{2n^2}(1-e^{-2n})$

$= \frac{1}{10.565} - \frac{1}{2(10.565)^2}(1-e^{-2(10.565)}) = 0.090$

R_B : $n = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.96)(352)/81.509 = 19.071$

* $B \approx 352'$ for most of building

$R_B = \frac{1}{19.071} - \frac{1}{(2)(19.071)^2}(1-e^{-(2)(19.071)})$

$= 0.051$

R_L : $n = 15.4 n_1 L / \bar{V}_{\bar{z}} = 15.4(0.96)(85)/81.509 = 15.417$

* $L \approx 85'$ for most of building

$R_L = \frac{1}{15.417} - \frac{1}{2(15.417)^2}(1-e^{-2(15.417)}) = 0.063$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$

* Assume $\beta = 0.05$

$= \sqrt{\frac{1}{0.05}(0.046)(0.090)(0.051)(0.53+0.47(0.063))}$

$= 0.049$

Background Response

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{352+195}{487.948}\right)^{0.63}}}$

$= 0.772$

Intensity of Turbulence

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{117}\right)^{1/6} = 0.243$

$c = 0.30$ Table 6-2

$G_f = 0.925\left(\frac{1 + 1.7(0.243)\sqrt{(3.4)^2(0.772)^2 + (4.180)^2(0.049)^2}}{1 + 1.7(3.4)(0.243)}\right) = 0.803$

(6) Enclosure Factor : Enclosed

(7) Internal Pressure Coefficient Figure 6-5

$GC_{pi} = +0.18$
-0.18

3

(8) External Pressure Coefficient Fig 6-6

Windward wall $C_p = 0.8$ use q_z
Leeward wall $C_p = -0.5$ use q_h ($L/B = 0.241$)

Sidewall $C_p = -0.7$ use q_h

Roof $C_p = -0.18$ ($h/L = 2.294$)

(9) Velocity Pressure q_z or q_h

$$q_z = 0.00256\, k_z\, k_{zt}\, k_d\, V^2 I$$
$$= 0.00256\, k_z\, (1)(0.85)(90)^2(1.15)$$
$$= 20.269\, k_z$$

Table 6-3 Exp. B, Case 2

(See Spreadsheet Table 4)

(10) MWFRS for Flexible Buildings

Windward

$$p = q\, G_f\, C_p \pm q_i (G C_{pi})$$ $q_i = q_h = 20.269(1.17) = 23.71$
$$p = q(0.803)(0.8) \pm 23.71(0.18)$$
$$= 0.64q$$

Leeward

$$p = q_h\, G_f\, C_p \pm q_i (G C_{pi})$$
$$q_h = 20.269(1.17) = 23.715$$
$$= 23.715(0.803)(0.5) \pm 4.27$$
$$= 9.52$$

Internal Pressure Cancels

(11) Approximate Face Area E-W

$(352' \times 198' + 50' \times 45') = 70890$ sf

Table 6: Windward Pressure (psf)						
Height	kz	qz=20.269*kz	p=q*G_f*C_p	± qi*(GCpi)	=	p
0-15	0.57	11.55	7.42	0	=	7.42
20	0.62	12.57	8.07	0	=	8.07
25	0.66	13.38	8.59	0	=	8.59
30	0.7	14.19	9.11	0	=	9.11
40	0.76	15.40	9.90	0	=	9.90
50	0.81	16.42	10.55	0	=	10.55
60	0.85	17.23	11.07	0	=	11.07
70	0.89	18.04	11.59	0	=	11.59
80	0.93	18.85	12.11	0	=	12.11
90	0.96	19.46	12.50	0	=	12.50
100	0.99	20.07	12.89	0	=	12.89
120	1.04	21.08	13.54	0	=	13.54
140	1.09	22.09	14.19	0	=	14.19
160	1.13	22.90	14.71	0	=	14.71
180	1.17	23.71	15.23	0	=	15.23
200	1.2	24.32	15.62	0	=	15.62

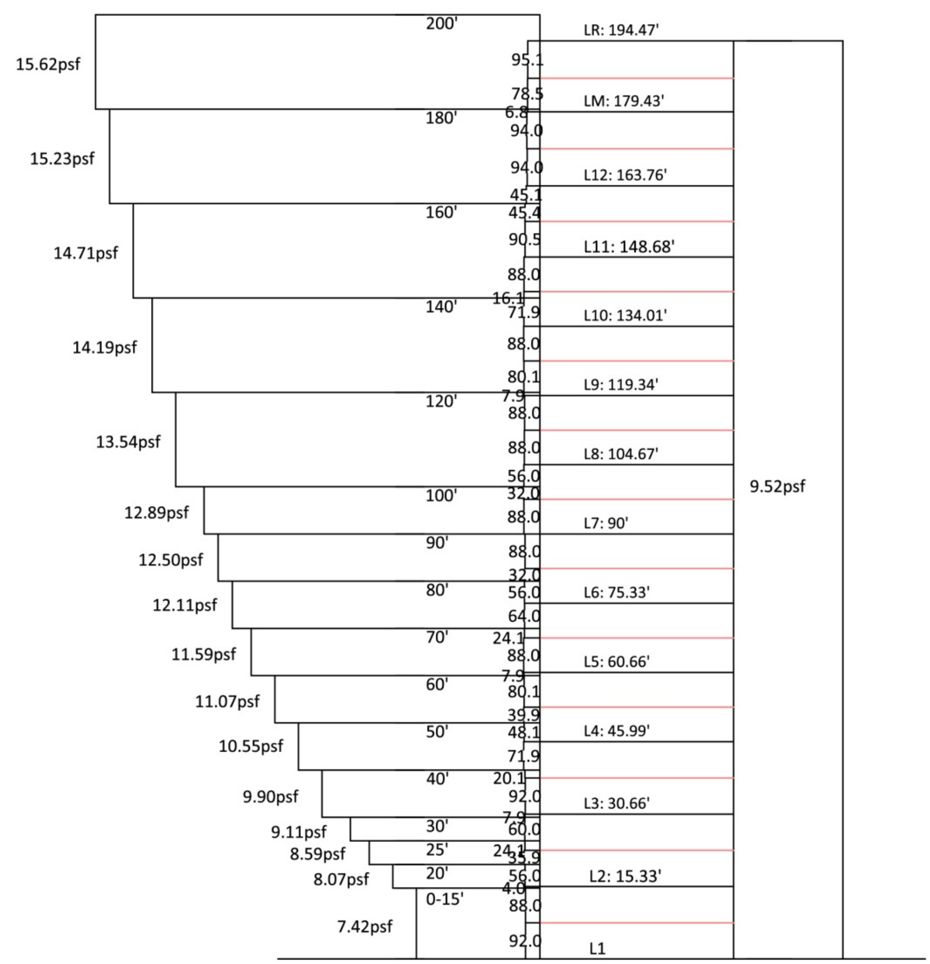

Table 7: Detailed Calculations - N-S								
				Windward	Leeward	Windward kips	Leeward kips	Total Kips
Floor	Height	Elevation	Bldtg Width	PLF vert W	PLF vert L	Bldg Load	Bldg Load	
1	15.33	0	160.1667	0.00	0.00	0.00	0.00	0.00
2	15.33	15.33	160.1667	120.50	145.89	19.30	23.37	42.67
3	15.33	30.66	160.1667	145.21	145.97	23.26	23.38	46.64
4	14.67	45.99	160.1667	158.84	142.80	25.44	22.87	48.31
5	14.67	60.66	109.8334	203.29	171.28	22.33	18.81	41.14
6	14.67	75.33	109.8334	177.70	139.71	19.52	15.34	34.86
7	14.67	90	109.8334	186.20	139.63	20.45	15.34	35.79
8	14.67	104.67	109.8334	196.87	139.63	21.62	15.34	36.96
9	14.67	119.34	109.8334	202.96	139.63	22.29	15.34	37.63
10	14.67	134.01	109.8334	208.86	139.63	22.94	15.34	38.28
11	15.08	148.68	109.8334	218.86	141.61	24.04	15.55	39.59
Penthouse	15.67	163.76	109.8334	232.26	146.37	25.51	16.08	41.59
Mezzanine	15.04	179.43	109.8334	230.18	142.24	25.28	15.62	40.90
Roof	0	194.47	109.8334	123.83	75.45	13.60	8.29	21.89
Total						285.58	220.66	506.24

Appendix: Seismic Load Calculations

Seismic 1

The USGS Earthquake Ground Motion Parameter Java Application at http://earthquake.usgs.gov/research/hazmaps/design was used to determine SDS and SD1.

A Site Class B was obtained by inputting these coordinates
Lat: 40.006164
Long: -75.151794

However, the geotechnical report classified the site as C so this was used for my analysis since the source is more accurate and the calculations are more conservative

The ASCE 7-05.12.8 Equivalent Lateral Force Procedure was used

$S_{DS} = 0.214$
$S_{D1} = 0.068$

12.8-2 Seismic Response Coeff.

$C_S = \frac{S_{DS}}{(R/I)}$

R=3.25 (Table 12.2-1): Drawing S520 states that structural was not specifically detailed for seismic. Also, a review of braced frame connections did not indicate moment connections. Values were picked for B4: Ordinary Steel Concentrically braced frames

$I = 1.25$ for Occupancy III using Tables 1-1, 11.5.1

$C_S = \frac{0.214}{(3.25/1.25)} = 0.084$

Limits

$T = C_t h_n^x = 0.02(195ft)^{0.75} = 1.044s$ (Table 12.8-2)

$T_L = 6s$ (Figure 22.15)

$C_S = \frac{S_{D1}}{T(\frac{R}{I})}$ for $T \leq T_L$

$= \frac{0.068}{1.044(\frac{3.25}{1.25})} = 0.025$

Seismic 2

Seismic Weight

Dead Load (77.1)(463,800 sf) = 35,754 k

$V = C_s W = 0.028(35,759) = 1,001\ k$

Seismic Distribution ASCE 12.8.3

$F_x = C_{vx} V$

$$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$$

$$k = 1 + \frac{(2-1)}{(2.5-0.5)}(1.044-0.5) = 1.272$$

Appendix: Frame Shear Checks

Note: Orange indicates wind loading in the East-West direction, while blue indicates seismic loading in the North-South direction.

Table 7: Eccentricity Calculations

	Center of Rigidity		Center of Mass/ Geom		eccentricity		% of building length	
	x	y	x	y	x	y	x	y
Roof	185.59	104.54	174.08	119.22	11.51	14.68	3%	22%
Mez.	186.34	106.51	195.77	123.44	9.43	16.93	3%	26%
Pent.	187.51	107.16	172	108.27	15.51	1.11	4%	1%
11th	186.87	107.09	171.86	108.31	15.01	1.22	4%	1%
10th	187.96	107.04	171.82	108.31	16.14	1.27	5%	1%
9th	184.33	107.05	171.87	108.31	12.46	1.26	4%	1%
8th	183.31	106.93	171.85	108.33	11.46	1.40	3%	1%
7th	182.06	106.75	171.84	108.34	10.22	1.59	3%	1%
6th	179.77	106.46	171.73	108.36	8.04	1.90	2%	2%
5th	176.97	105.94	173.98	109.21	2.99	3.27	1%	3%
4th	176	104.99	186.19	94.89	10.19	10.10	3%	6%
3rd	167.42	103.22	198.79	106.05	31.37	2.83	8%	2%
2nd	156.41	102.76	183.31	100.1	26.90	2.66	7%	2%
1st	186.31	95.14	186.31	95.14	0.00	0.00	0%	0%

Relative Stiffness Properties						Frame Shear Check		
	Frame #0							
Level	Disp Y	K_{SF} = 1/Disp	C_N	K_{SF}*C_N	(K_{SF}*C_N) / ∑(K_{SF}*C_N)	H_s	H_s(K_{SF}*C_N) / ∑(K_{SF}*C_N)	RAM Value
Roof	0.00	505.05	73.54	37141.41	0.256565	114.77	29.45	60.42
Mez.	0.00	444.44	75.51	33560.00	0.262375	362.53	95.12	138.71
Pent.	0.00	813.01	76.16	61918.70	0.133853	544.6	72.90	85.06
11th	0.00	854.70	76.09	65034.19	0.155041	744.53	115.43	56.21
10th	0.00	869.57	76.04	66121.74	0.154729	-1141.39	-176.61	310.69
9th	0.00	934.58	76.05	71074.77	0.161370	1140.44	184.03	190.47
8th	0.00	1000.00	75.93	75930.00	0.163852	1337.44	219.14	249.31
7th	0.00	1063.83	75.75	80585.11	0.161531	1523.59	246.11	304.53
6th	0.00	1282.05	75.46	96743.59	0.175581	1704.53	299.28	328.19
5th	0.00	1408.45	74.94	105549.30	0.210597	1922.07	404.78	536.17
4th	0.00	1136.36	73.99	84079.55	0.131078	2117.78	277.59	371.01
3rd	0.00	1388.89	72.22	100305.56	0.141186	2300.82	324.84	387.28
2nd	0.00	1562.50	71.76	112125.00	0.178369	2463.78	439.46	346.31
1st								
	Frame #1							
Level	Disp Y	K_{SF} = 1/Disp	C_N	K_{SF}*C_N	(K_{SF}*C_N) / ∑(K_{SF}*C_N)	H_s	H_s(K_{SF}*C_N) / ∑(K_{SF}*C_N)	RAM Value
Roof	0.05601	17.85	19.46	347.44	0.002400	114.77	0.28	0.85
Mez.	0.06413	15.59	17.49	272.73	0.002132	362.53	0.77	6.2
Pent.	0.00105	952.38	16.84	16038.10	0.034670	544.6	18.88	83.36

11th	0.00115	869.57	16.91	14704.35	0.035055	744.53	26.10	300.27
10th	0.00113	884.96	16.96	15008.85	0.035122	-1141.39	-40.09	357.94
9th	0.00111	900.90	16.95	15270.27	0.034670	1140.44	39.54	209.09
8th	0.00109	917.43	17.07	15660.55	0.033794	1337.44	45.20	262.47
7th	0.00108	925.93	17.25	15972.22	0.032016	1523.59	48.78	278.23
6th	0.00096	1041.67	17.54	18270.83	0.033160	1704.53	56.52	369.2
5th	0.0011	909.09	18.06	16418.18	0.032758	1922.07	62.96	298.2
4th	0.00093	1075.27	19.01	20440.86	0.031867	2117.78	67.49	479.58
3rd	0.00083	1204.82	20.78	25036.14	0.035240	2300.82	81.08	431.36
2nd	0.00047	2127.66	21.24	45191.49	0.071891	2463.78	177.12	520.3
1st	-	-						
	Frame #2							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N)$	H_s	$\frac{H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$	RAM Value
Roof	0.05558	17.99	50.46	907.88	0.006271	114.77	0.72	0.66
Mez.	0.0634	15.77	48.49	764.83	0.005979	362.53	2.17	5.99
Pent.	0.00109	917.43	47.84	43889.91	0.094879	544.6	51.67	140.93
11th	0.00119	840.34	47.91	40260.50	0.095981	744.53	71.46	290.34
10th	0.00116	862.07	47.96	41344.83	0.096750	-1141.39	-110.43	353.57
9th	0.00114	877.19	47.95	42061.40	0.095497	1140.44	108.91	220.79
8th	0.00111	900.90	48.07	43306.31	0.093452	1337.44	124.99	267.48
7th	0.00109	917.43	48.25	44266.06	0.088730	1523.59	135.19	278.32
6th	0.00105	952.38	48.54	46228.57	0.083901	1704.53	143.01	273.1
5th	0.00108	925.93	49.06	45425.93	0.090636	1922.07	174.21	318.3
4th	0.00096	1041.67	50.01	52093.75	0.081213	2117.78	171.99	337.13
3rd	0.00078	1282.05	51.78	66384.62	0.093440	2300.82	214.99	496.51
2nd	0.00043	2325.58	52.24	121488.37	0.193264	2463.78	476.16	754.5
1st	-	-						

Relative Stiffness Properties Cont.						Frame Shear Check Cont.		
	Frame #3							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N)$	H_s	$\frac{H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$	RAM Value
Roof	0.05518	18.12	143.46	2599.86	0.017959	114.77	2.06	0.17
Mez.	0.06869	14.56	141.49	2059.83	0.016104	362.53	5.84	5.98
Pent.	0.00107	934.58	140.84	131626.17	0.284544	544.6	154.96	214.43
11th	0.00116	862.07	140.91	121474.14	0.289593	744.53	215.61	151.97
10th	0.00114	877.19	140.96	123649.12	0.289347	-1141.39	-330.26	428.17
9th	0.00112	892.86	140.95	125848.21	0.285728	1140.44	325.86	298.31
8th	0.00109	917.43	141.07	129422.02	0.279284	1337.44	373.53	343.46
7th	0.00103	970.87	141.25	137135.92	0.274886	1523.59	418.81	380.63
6th	0.00096	1041.67	141.54	147437.50	0.267587	1704.53	456.11	461.23
5th	0.0095	105.26	142.06	14953.68	0.029836	1922.07	57.35	17.56
4th	0.00078	1282.05	143.01	183346.15	0.285833	2117.78	605.33	555.26
3rd	0.00076	1315.79	144.78	190500.00	0.268140	2300.82	616.94	531.26
2nd	0.00934	107.07	145.24	15550.32	0.024738	2463.78	60.95	22.45
1st	-	-						
	Frame #4					H_s	$H_s(K_{SF}*C_N)$	RAM Value

Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N)$		$\sum(K_{SF}*C_N)$	
Roof	0.00198	505.05	205.46	103767.68	0.716805	114.77	82.27	62
Mez.	0.00223	448.43	203.49	91251.12	0.713409	362.53	258.63	159.03
Pent.	0.00097	1030.93	202.84	209113.40	0.452053	544.6	246.19	29.58
11th	0.00114	877.19	202.91	177991.23	0.424330	744.53	315.93	19.73
10th	0.00112	892.86	202.96	181214.29	0.424053	-1141.39	-484.01	518.51
9th	0.00109	917.43	202.95	186192.66	0.422735	1140.44	482.10	166.21
8th	0.00102	980.39	203.07	199088.24	0.429618	1337.44	574.59	226.88
7th	0.00092	1086.96	203.25	220923.91	0.442837	1523.59	674.70	275.19
6th	0.00084	1190.48	203.54	242309.52	0.439771	1704.53	749.60	288.02
5th	0.00064	1562.50	204.06	318843.75	0.636172	1922.07	1222.77	731.09
4th	0.00068	1470.59	205.01	301485.29	0.470009	2117.78	995.38	378.18
3rd	0.00063	1587.30	206.78	328222.22	0.461993	2300.82	1062.96	458.26
2nd	0.00062	1612.90	207.24	334258.06	0.531739	2463.78	1310.09	771.96
1st	-	-						
	Totals							
Level				$\sum(K_{SF}*C_N)$				
Roof				144764.26				
Mez.				127908.51				
Pent.				462586.27				
11th				419464.41				
10th				427338.82				
9th				440447.31				
8th				463407.11				
7th				498883.22				
6th				550990.02				
5th				501190.84				
4th				641445.60				
3rd				710448.54				
2nd				628613.25				
1st								

Relative Stiffness Properties Cont.						Frame Shear Check Cont.		
	Frame #5							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N)$	H_s	$\frac{H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$	RAM Value
Roof	-	-			-			
Mez.	-	-			-			
Pent.	-	-			-			
11th	-	-			-			
10th	-	-			-			
9th	-	-			-			
8th	-	-			-			
7th	-	-			-			
6th	-	-			-			
5th	-	-			-			
4th	-	-			-			
3rd	-	-			-			
2nd	0.00737	135.69	131.14	17793.65	0.29	1094.87	313.1454	143.13
1st	-	-						
	Frame #6							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N)$	H_s	$\frac{H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$	RAM Value
Roof	0.00302	331.13	96.99	32114.54	0.975553	138.35	134.97	134.77
Mez.	0.00376	265.96	97.74	25993.59	0.787568	182.6	143.81	5.69
Pent.	0.00397	251.89	98.91	24913.32	0.625449	334.56	209.25	26.9
11th	0.00311	321.54	98.27	31596.75	0.630023	470.31	296.31	69.6
10th	0.00288	347.22	99.36	34498.58	0.626735	585.87	367.19	54.49
9th	0.00283	353.36	95.73	33825.41	0.632482	702.39	444.25	58.21
8th	0.00273	366.30	94.71	34690.81	0.633596	796.88	504.90	46.58
7th	0.00267	374.53	93.46	35002.21	0.635863	878.17	558.40	40.5
6th	0.00255	392.16	91.17	35751.33	0.638488	938.75	599.38	28.84
5th	0.00223	448.43	88.37	39625.96	0.637451	992.94	632.95	18.72
4th	0.00245	408.16	87.40	35671.80	0.634729	1078.72	684.69	21.13
3rd	0.00247	404.86	78.82	31909.27	0.652291	1116.92	728.56	55.08
2nd	0.00219	456.62	67.81	30961.60	0.497670	1094.87	544.88	69.22
1st	-	-						
	Frame #7							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N)$	H_s	$\frac{H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$	RAM Value
Roof	-	-			-			
Mez.	-	-			-			
Pent.	0.00406	246.31	60.57	14919.36	0.374551	334.56	125.31	166.91
11th	0.00323	309.60	59.93	18554.98	0.369977	470.31	174.00	68.64
10th	0.00297	336.70	61.02	20546.33	0.373265	585.87	218.68	60.66
9th	0.00292	342.47	57.39	19655.00	0.367518	702.39	258.14	57.73
8th	0.00281	355.87	56.37	20061.42	0.366404	796.88	291.98	48.99
7th	0.00275	363.64	55.12	20044.58	0.364137	878.17	319.77	38.53
6th	0.00261	383.14	52.83	20242.38	0.361512	938.75	339.37	34.69
5th	0.00222	450.45	50.03	22537.21	0.362549	992.94	359.99	35.24

4th	0.00239	418.41	49.06	20528.28	0.365271	1078.72	394.03	62.17
3rd	0.00238	420.17	40.48	17009.50	0.347709	1116.92	388.36	27.79
2nd	0.00219	456.62	29.47	13457.81	0.216318	1094.87	236.84	112.25
1st	-	-						
Relative Stiffness Properties Cont.						Frame Shear Check Cont.		
	Frame #8							
Level	Disp Y	K_{SF} = 1/Disp	C_N	K_{SF}*C_N	(K_{SF}*C_N) / ∑(K_{SF}*C_N)	H_s	H_s(K_{SF}*C_N) / ∑(K_{SF}*C_N)	RAM Value
Roof	0.03958	25.27	31.85	804.76	0.024447	138.35	3.38	2.96
Mez.	0.00465	215.05	32.60	7011.29	0.212432	182.6	38.79	41.18
Pent.	-	-			-			
11th	-	-			-			
10th	-	-			-			
9th	-	-			-			
8th	-	-			-			
7th	-	-			-			
6th	-	-			-			
5th	-	-			-			
4th	-	-			-			
3rd	-	-			-			
2nd	-	-			-			
1st	-	-			-			
	Totals							
Level				∑(K_{SF}*C_N)				
Roof				32919.30				
Mez.				33004.88				
Pent.				39832.68				
11th				50151.74				
10th				55044.91				
9th				53480.41				
8th				54752.23				
7th				55046.79				
6th				55993.71				
5th				62163.17				
4th				56200.08				
3rd				48918.77				
2nd				62213.06				
1st								

Appendix: Torsional Shear Calculations

Note: Orange indicates wind loading in the East-West direction, while blue indicates seismic loading in the North-South direction.

Table 7: Eccentricity Calculations								
	Center of Rigidity		Center of Mass/ Geom		eccentricity		% of building length	
	x	y	x	y	x	y	x	y
Roof	185.59	104.54	174.08	119.22	11.51	14.68	3%	22%
Mez.	186.34	106.51	195.77	123.44	9.43	16.93	3%	26%
Pent.	187.51	107.16	172	108.27	15.51	1.11	4%	1%
11th	186.87	107.09	171.86	108.31	15.01	1.22	4%	1%
10th	187.96	107.04	171.82	108.31	16.14	1.27	5%	1%
9th	184.33	107.05	171.87	108.31	12.46	1.26	4%	1%
8th	183.31	106.93	171.85	108.33	11.46	1.40	3%	1%
7th	182.06	106.75	171.84	108.34	10.22	1.59	3%	1%
6th	179.77	106.46	171.73	108.36	8.04	1.90	2%	2%
5th	176.97	105.94	173.98	109.21	2.99	3.27	1%	3%
4th	176	104.99	186.19	94.89	10.19	10.10	3%	6%
3rd	167.42	103.22	198.79	106.05	31.37	2.83	8%	2%
2nd	156.41	102.76	183.31	100.1	26.90	2.66	7%	2%
1st	186.31	95.14	186.31	95.14	0.00	0.00	0%	0%

Table 8: Torsional Shear								
Relative Stiffness Properties						Torsional Shear		
	Frame #0							
Level	Disp Y	K_{SF} = 1/Disp	C_N	K_{SF}*C_N	(K_{SF}*C_N) / ∑(K_{SF}*C_N^2)	e	H_s	e*H_s(K_{SF}*C_N) / ∑(K_{SF}*C_N)
Roof	0.00	505.05	73.54	37141.41	0.003489	14.68	138.35	7.09
Mez.	0.00	444.44	75.51	33560.00	0.003475	16.93	182.6	10.74
Pent.	0.00	813.01	76.16	61918.70	0.001758	1.11	334.56	0.65
11th	0.00	854.70	76.09	65034.19	0.002038	1.22	470.31	1.17
10th	0.00	869.57	76.04	66121.74	0.002035	1.27	585.87	1.51
9th	0.00	934.58	76.05	71074.77	0.002122	1.26	702.39	1.88
8th	0.00	1000.00	75.93	75930.00	0.002158	1.40	796.88	2.41
7th	0.00	1063.83	75.75	80585.11	0.002132	1.59	878.17	2.98
6th	0.00	1282.05	75.46	96743.59	0.002327	1.90	938.75	4.15
5th	0.00	1408.45	74.94	105549.30	0.002810	3.27	992.94	9.12
4th	0.00	1136.36	73.99	84079.55	0.001772	10.10	1078.72	19.30
3rd	0.00	1388.89	72.22	100305.56	0.001955	2.83	1116.92	6.18
2nd	0.00	1562.50	71.76	112125.00	0.002486	2.66	1094.87	7.24
1st								
	Frame #1							
Level	Disp Y	K_{SF} = 1/Disp	C_N	K_{SF}*C_N	(K_{SF}*C_N) / ∑(K_{SF}*C_N^2)	e	H_s	e*H_s(K_{SF}*C_N) / ∑(K_{SF}*C_N)
Roof	0.05601	17.85	19.46	347.44	0.000123	14.68	138.35	0.25
Mez.	0.06413	15.59	17.49	272.73	0.000122	16.93	182.6	0.38
Pent.	0.00105	952.38	16.84	16038.10	0.002059	1.11	334.56	0.76

11th	0.00115	869.57	16.91	14704.35	0.002073	1.22	470.31	1.19
10th	0.00113	884.96	16.96	15008.85	0.002071	1.27	585.87	1.54
9th	0.00111	900.90	16.95	15270.27	0.002045	1.26	702.39	1.81
8th	0.00109	917.43	17.07	15660.55	0.001980	1.40	796.88	2.21
7th	0.00108	925.93	17.25	15972.22	0.001856	1.59	878.17	2.59
6th	0.00096	1041.67	17.54	18270.83	0.001891	1.90	938.75	3.37
5th	0.0011	909.09	18.06	16418.18	0.001814	3.27	992.94	5.89
4th	0.00093	1075.27	19.01	20440.86	0.001676	10.10	1078.72	18.26
3rd	0.00083	1204.82	20.78	25036.14	0.001696	2.83	1116.92	5.36
2nd	0.00047	2127.66	21.24	45191.49	0.003385	2.66	1094.87	9.86
1st	-	-						

	Frame #2							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N^2)$	e	H_s	$e*H_s(K_{SF}*C_N)$ / $\sum(K_{SF}*C_N)$
Roof	0.05558	17.99	50.46	907.88	0.000124	14.68	138.35	0.25
Mez.	0.0634	15.77	48.49	764.83	0.000123	16.93	182.6	0.38
Pent.	0.00109	917.43	47.84	43889.91	0.001983	1.11	334.56	0.74
11th	0.00119	840.34	47.91	40260.50	0.002003	1.22	470.31	1.15
10th	0.00116	862.07	47.96	41344.83	0.002017	1.27	585.87	1.50
9th	0.00114	877.19	47.95	42061.40	0.001992	1.26	702.39	1.76
8th	0.00111	900.90	48.07	43306.31	0.001944	1.40	796.88	2.17
7th	0.00109	917.43	48.25	44266.06	0.001839	1.59	878.17	2.57
6th	0.00105	952.38	48.54	46228.57	0.001728	1.90	938.75	3.08
5th	0.00108	925.93	49.06	45425.93	0.001847	3.27	992.94	6.00
4th	0.00096	1041.67	50.01	52093.75	0.001624	10.10	1078.72	17.69
3rd	0.00078	1282.05	51.78	66384.62	0.001805	2.83	1116.92	5.70
2nd	0.00043	2325.58	52.24	121488.37	0.003700	2.66	1094.87	10.77
1st	-	-						

Relative Stiffness Properties Cont.						Torsional Shear Cont.		
	Frame #3							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N^2)$	e	H_s	$e*H_s(K_{SF}*C_N)$ / $\sum(K_{SF}*C_N)$
Roof	0.05518	18.12	143.46	2599.86	0.000125	14.68	138.35	0.25
Mez.	0.06869	14.56	141.49	2059.83	0.000114	16.93	182.6	0.35
Pent.	0.00107	934.58	140.84	131626.17	0.002020	1.11	334.56	0.75
11th	0.00116	862.07	140.91	121474.14	0.002055	1.22	470.31	1.18
10th	0.00114	877.19	140.96	123649.12	0.002053	1.27	585.87	1.53
9th	0.00112	892.86	140.95	125848.21	0.002027	1.26	702.39	1.79
8th	0.00109	917.43	141.07	129422.02	0.001980	1.40	796.88	2.21
7th	0.00103	970.87	141.25	137135.92	0.001946	1.59	878.17	2.72
6th	0.00096	1041.67	141.54	147437.50	0.001891	1.90	938.75	3.37
5th	0.0095	105.26	142.06	14953.68	0.000210	3.27	992.94	0.68
4th	0.00078	1282.05	143.01	183346.15	0.001999	10.10	1078.72	21.78
3rd	0.00076	1315.79	144.78	190500.00	0.001852	2.83	1116.92	5.85
2nd	0.00934	107.07	145.24	15550.32	0.000170	2.66	1094.87	0.50
1st	-	-						

	Frame #4					e	H_s	$e*H_s(K_{SF}*C_N)$

Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N^2)$			$\sum(K_{SF}*C_N)$
Roof	0.00198	505.05	205.46	103767.68	0.003489	14.68	138.35	7.09
Mez.	0.00223	448.43	203.49	91251.12	0.003506	16.93	182.6	10.84
Pent.	0.00097	1030.93	202.84	209113.40	0.002229	1.11	334.56	0.83
11th	0.00114	877.19	202.91	177991.23	0.002091	1.22	470.31	1.20
10th	0.00112	892.86	202.96	181214.29	0.002089	1.27	585.87	1.55
9th	0.00109	917.43	202.95	186192.66	0.002083	1.26	702.39	1.84
8th	0.00102	980.39	203.07	199088.24	0.002116	1.40	796.88	2.36
7th	0.00092	1086.96	203.25	220923.91	0.002179	1.59	878.17	3.04
6th	0.00084	1190.48	203.54	242309.52	0.002161	1.90	938.75	3.85
5th	0.00064	1562.50	204.06	318843.75	0.003118	3.27	992.94	10.12
4th	0.00068	1470.59	205.01	301485.29	0.002293	10.10	1078.72	24.98
3rd	0.00063	1587.30	206.78	328222.22	0.002234	2.83	1116.92	7.06
2nd	0.00062	1612.90	207.24	334258.06	0.002566	2.66	1094.87	7.47
1st	-	-						
				Totals				
Level				$\sum(K_{SF}*C_N)$				
Roof				144764.26				
Mez.				127908.51				
Pent.				462586.27				
11th				419464.41				
10th				427338.82				
9th				440447.31				
8th				463407.11				
7th				498883.22				
6th				550990.02				
5th				501190.84				
4th				641445.60				
3rd				710448.54				
2nd				628613.25				
1st								

Relative Stiffness Properties Cont.						Torsional Shear Cont.		
	Frame #5							
Level	Disp Y	K_{SF} = 1/Disp	C_N	K_{SF}*C_N	(K_{SF}*C_N) / ∑(K_{SF}*C_N^2)	e	H_s	$\frac{e*H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$
Roof	-	-			-			
Mez.	-	-			-			
Pent.	-	-			-			
11th	-	-			-			
10th	-	-			-			
9th	-	-			-			
8th	-	-			-			
7th	-	-			-			
6th	-	-			-			
5th	-	-			-			
4th	-	-			-			
3rd	-	-			-			
2nd	0.00737	135.69	131.14	17793.65	0.002181	26.90	2463.78	144.5457012
1st	-	-						
	Frame #6							
Level	Disp Y	K_{SF} = 1/Disp	C_N	K_{SF}*C_N	(K_{SF}*C_N) / ∑(K_{SF}*C_N^2)	e	H_s	$\frac{e*H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$
Roof	0.00302	331.13	96.99	32114.54	0.010059	11.51	114.77	13.29
Mez.	0.00376	265.96	97.74	25993.59	0.008058	9.43	362.53	27.55
Pent.	0.00397	251.89	98.91	24913.32	0.006324	15.51	544.6	53.41
11th	0.00311	321.54	98.27	31596.75	0.006411	15.01	744.53	71.65
10th	0.00288	347.22	99.36	34498.58	0.006308	16.14	1141.39	116.21
9th	0.00283	353.36	95.73	33825.41	0.006607	12.46	1140.44	93.89
8th	0.00273	366.30	94.71	34690.81	0.006690	11.46	1337.44	102.54
7th	0.00267	374.53	93.46	35002.21	0.006804	10.22	1523.59	105.94
6th	0.00255	392.16	91.17	35751.33	0.007004	8.04	1704.53	95.98
5th	0.00223	448.43	88.37	39625.96	0.007214	2.99	1922.07	41.46
4th	0.00245	408.16	87.40	35671.80	0.007263	10.19	2117.78	156.73
3rd	0.00247	404.86	78.82	31909.27	0.008276	31.37	2300.82	597.34
2nd	0.00219	456.62	67.81	30961.60	0.007340	26.90	2463.78	486.44
1st	-	-						
	Frame #7							
Level	Disp Y	K_{SF} = 1/Disp	C_N	K_{SF}*C_N	(K_{SF}*C_N) / ∑(K_{SF}*C_N^2)	e	H_s	$\frac{e*H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$
Roof	-	-			-			
Mez.	-	-			-			
Pent.	0.00406	246.31	60.57	14919.36	0.006184	15.51	544.6	52.23
11th	0.00323	309.60	59.93	18554.98	0.006173	15.01	744.53	68.99
10th	0.00297	336.70	61.02	20546.33	0.006117	16.14	1141.39	112.68
9th	0.00292	342.47	57.39	19655.00	0.006404	12.46	1140.44	90.99
8th	0.00281	355.87	56.37	20061.42	0.006500	11.46	1337.44	99.62
7th	0.00275	363.64	55.12	20044.58	0.006606	10.22	1523.59	102.86
6th	0.00261	383.14	52.83	20242.38	0.006843	8.04	1704.53	93.77
5th	0.00222	450.45	50.03	22537.21	0.007246	2.99	1922.07	41.64
4th	0.00239	418.41	49.06	20528.28	0.007445	10.19	2117.78	160.66

3rd	0.00238	420.17	40.48	17009.50	0.008589	31.37	2300.82	619.93
2nd	0.00219	456.62	29.47	13457.81	0.007340	26.90	2463.78	486.44
1st	-	-						
Relative Stiffness Properties Cont.						Torsional Shear Cont.		
	Frame #8							
Level	Disp Y	K_{SF} = 1/Disp	C_N	$K_{SF}*C_N$	$(K_{SF}*C_N) / \sum(K_{SF}*C_N^2)$	e	H_s	$\frac{e*H_s(K_{SF}*C_N)}{\sum(K_{SF}*C_N)}$
Roof	0.03958	25.27	31.85	804.76	0.000767	11.51	114.77	1.01
Mez.	0.00465	215.05	32.60	7011.29	0.006516	9.43	362.53	22.28
Pent.	-	-			-			
11th	-	-			-			
10th	-	-			-			
9th	-	-			-			
8th	-	-			-			
7th	-	-			-			
6th	-	-			-			
5th	-	-			-			
4th	-	-			-			
3rd	-	-			-			
2nd	-	-			-			
1st	-	-			-			
	Totals							
Level				$\sum(K_{SF}*C_N)$				
Roof				32919.30				
Mez.				33004.88				
Pent.				39832.68				
11th				50151.74				
10th				55044.91				
9th				53480.41				
8th				54752.23				
7th				55046.79				
6th				55993.71				
5th				62163.17				
4th				56200.08				
3rd				48918.77				
2nd				62213.06				