St. Vincent Mercy Medical Center Heart Pavilion

Toledo, Ohio

Technical Report III

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EXECUTIVE SUMMARY

St. Vincent Mercy Medical Center Heart Pavilion is a four story hospital that provides diagnostics, surgery, and patient care. It was constructed for St. Vincent's Mercy Medical Center Campus, established in 1855, in downtown Toledo, Ohio.

The facility is approximately 144,000 square feet and reaches a height of 57'-5" above grade with a typical floor to floor height of approximately 14 feet. A typical interior bay is 30 feet by 35 feet and is comprised of composite steel with a concrete slab on deck. The lateral system utilizes steel moment frames due to limited floor space. Drilled caissons and spread footings make up the foundation system. The ground floor is a reinforced slab on grade with grade beams between caissons to transfer wall load into the foundation.

In this third technical report, a thorough lateral analysis for St. Vincent Mercy Medical Center Heart Pavilion is studied through detailed hand calculations and modeling via RAM Structural System and SAP 2000. Lateral forces determined in the preliminary analysis prepared for Technical Report I were verified using RAM Structural System. Upon comparison, it was concluded that the preliminary calculations provided satisfactory approximations of story loads. In order to determine how lateral loads were distributed throughout the lateral system, the relative stiffness of each moment frame was calculated. Once the relative stiffness of each frame was determined, torsion forces were calculated in order to determine the controlling load case for the structural design of this building.

Due to the unique shape of this building, wind loading induced greater torsion effects than seismic loading. However, since the soil is classified as site class E, the seismic base shear, when coupled with the torsion effects for seismic, ultimately controls the design of the building.

Spot checks for lateral members were also prepared to better understand why member sizes were chosen by the engineer of record. It was determined that the girders and columns within the lateral system are more than adequate for strength requirements. The girders meet deflection criteria by approximately 14% while columns meet drift limitations by approximately 37%. Further investigation of these member sizes will be conducted in efforts to further optimize the structural system. Overall, the lateral system of the Heart Pavilion meets strength requirements as well as serviceability requirements.

INTRODUCTION: ST. VINCENT MERCY MEDICAL CENTER HEART PAVILION

St. Vincent's Heart Pavilion is one of the seven hospitals that comprise Mercy Health Partners. As Toledo's first and only facility for the treatment of vascular disease, St. Vincent's Heart Pavilion has become a staple within the community. St. Vincent's Mercy Medical Center Campus is now able to take a leadership role in providing education to its students as well as saving lives through the treatment of vascular disease.

Modernization is emphasized through the façade of St. Vincent Mercy Medical Center Heart Pavilion. As one approaches the building from the North, a beautiful curtain wall composed of curved aluminum and spandrel glass is seen, thus adding great verticality to the building. As the eye gazes along the façade, stone bands and brick veneer promote horizontal progression to an attractive vertical component of stairs wrapped in stone veneer and spandrel glass. The eye is then led to the pedestrian bridge, connecting the Heart Pavilion to a parking garage, which shows off its structure through exposed chevron bracing.

The structure of the Heart Pavilion is comprised of a composite steel floor system that utilizes steel moment frames to resist lateral forces. Drilled caissons and spread footings make up the foundation system. The ground floor is a reinforced slab on grade with grade beams between caissons to transfer wall load into the foundation.

The purpose of Technical Report III is to gain a better understanding of how wind and seismic forces are distributed throughout the lateral system and to determine which lateral force controls the design of the structure. Upon completion of this report, conclusions will be drawn on the validity of member sizes chosen for the lateral system.

EXISTING STRUCTURAL DESCRIPTION

Floor System

St. Vincent Mercy Medical Center Heart Pavilion's typical floor system is made up of composite steel framing and normal weight concrete, creating a total floor thickness of 6½". Composite action is created by the use of 2" 20 gauge steel deck with 5½" long, ¾" diameter shear studs evenly spaced over the length of each beam. Even though a composite system is used, the girders are actually non-composite. In order to avoid coping of the infill beams, the girders are placed 2" higher than the beams on a typical floor and 1½" higher on the roof (see figure to the right). This system saved money and fabrication time which resulted in faster steel erection. Strength requirements are met by approximately 4.2% while deflection criteria are met by approximately 26%. Please reference Technical Report I for detailed calculations checking member validity and deflection criteria.

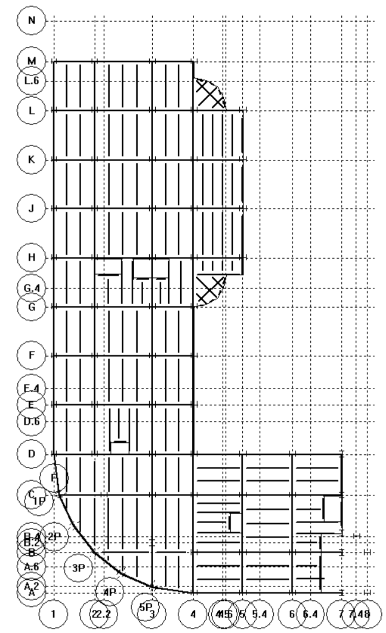


Figure 1: *Typical Floor Layout*

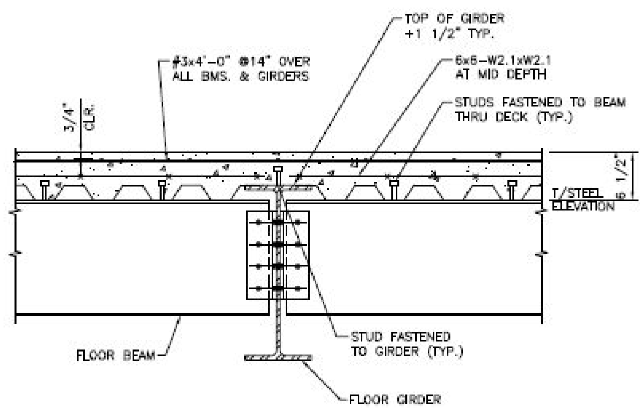


Figure 2: *Detail of Existing Composite Steel Floor System*

Columns

The columns used in St. Vincent Mercy Medical Center Heart Pavilion range from W10x119's to W12x210's, depending on their location within the building. While these sizes may seem large based purely on gravity, each column must resist induced moment since all columns are part of a moment frame. These impacts are investigated in the "Member Verification" section of this report. Pipe columns are used to support the roof for the main entrance lobby and the emergency vestibule canopy. All of the main building columns are spliced at the 2nd-3rd floor. Base plates range in thickness from 1" to 2 ¼" depending on which columns they are supporting. Each base plate utilizes a standard 4 bolt connection using either ¾" A325 or 1 ¼" A325 bolts.

Lateral System

At the time of design, braced frames were thought to be architecturally incompatible with this floor plan. As a result, steel moment frames were used for the lateral load resisting system. Please reference figure 3, indicating the lateral system in red. Please reference Appendix A for a larger view of all floor plans.

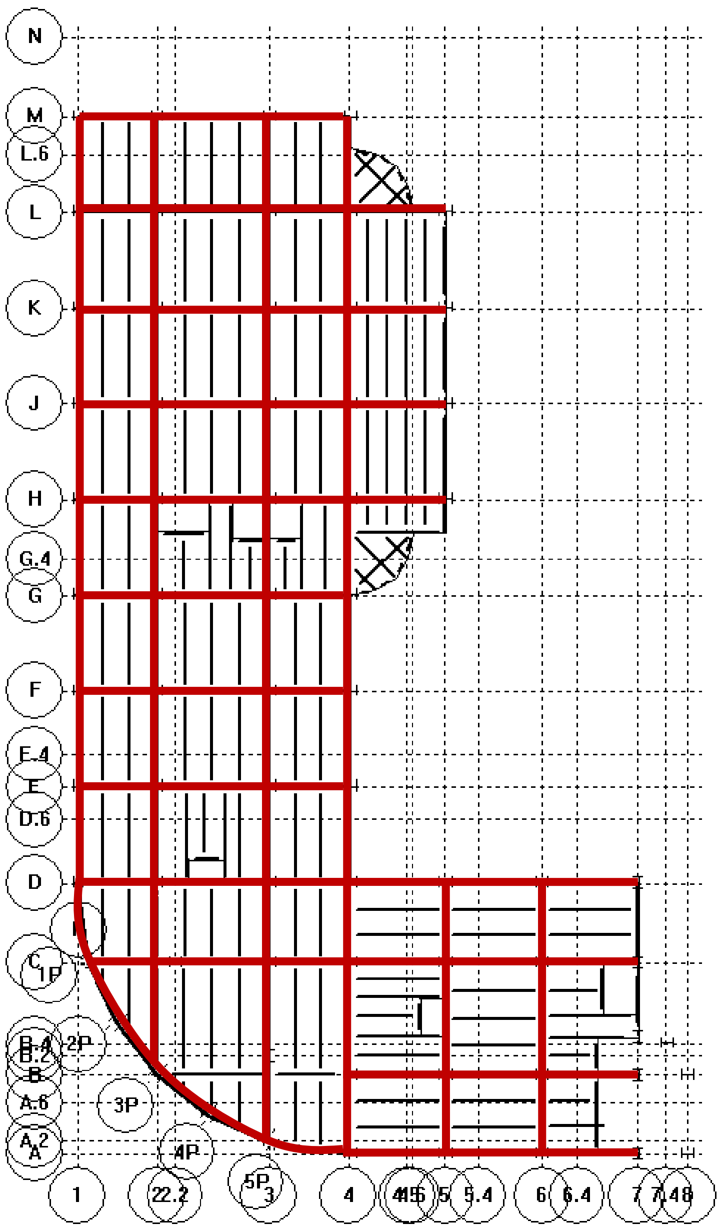


Figure 3: *Typical Floor Plan Indicating Lateral System*

The moment frames are connected in two different fashions as seen in figures 4 and 5 below. The beam to column web moment connection is comprised of flange plates that are fillet welded to the column web and flange. The beam flanges are full-penetration welded to these plates. The beam to column flange moment connection utilizes double angles connecting the beam to the column flange, where the beam flange is then full penetration welded to the column flange.

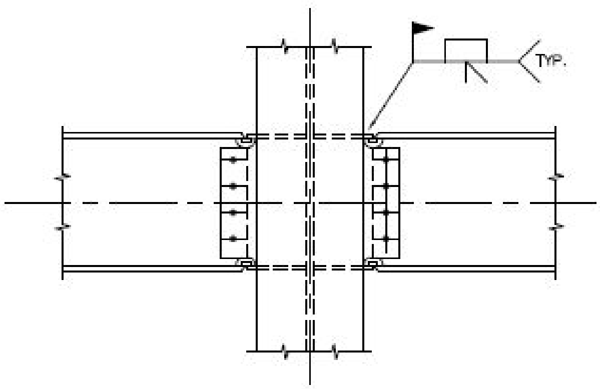


Figure 4: *Beam to Column Web Connection*

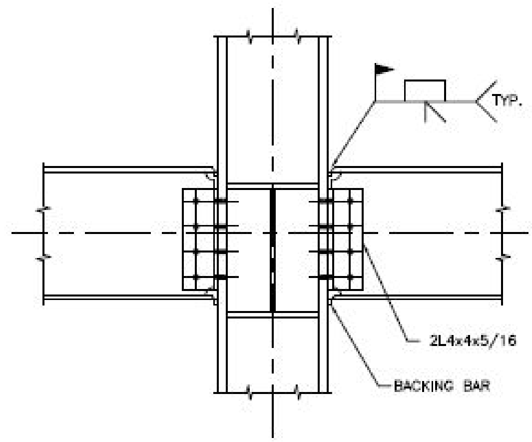


Figure 5: *Beam to Column Flange Connection*

Foundation System

The foundation system is made up of 80 drilled caissons and 6 spread footings that support the entrance lobby. The caisson caps are a uniform size of 4'x4'x3' thick. Between caissons are grade beams, varying in depth from 2' to 4' depending on the location, which transfer façade and wall load to the foundation system. The ground (main) floor rests on a 6" concrete slab reinforced with W/4x4-W4.0x4.0 welded wire fabric. Please reference the Member Verification section for detailed calculations checking overturning moment on the foundation system.

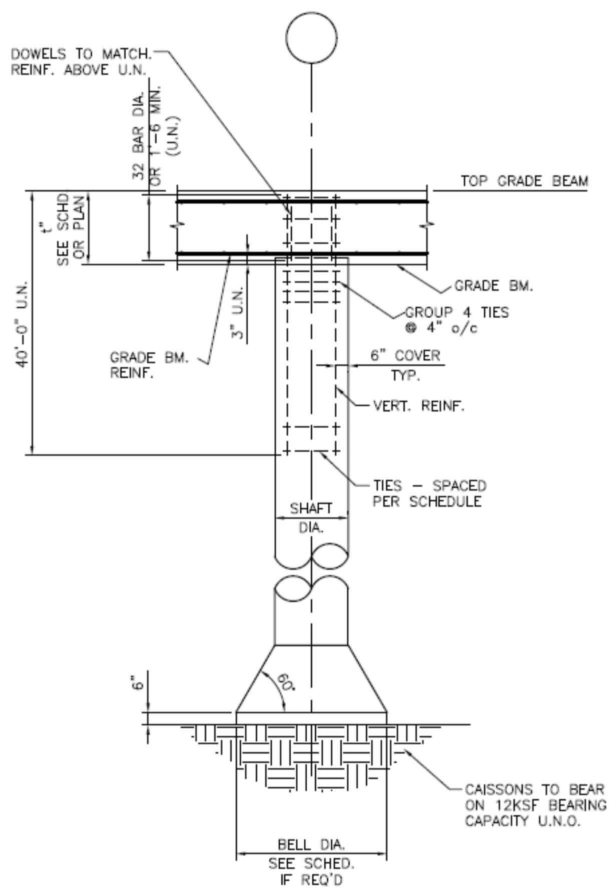


Figure 6: *Caisson Detail at Interior Grade Beam*

CODE REFERENCES AND LOAD COMBINATIONS

Various references were used by the engineer of record in order to carry out the structural design of St. Vincent Mercy Medical Center Heart Pavilion:

- The 2002 International Building Code as amended by the State of Ohio
- The Building Code Requirements for Structural Concrete (ACI 318-02), American Concrete Institute
- Specification for the Design, Fabrication, and Erection of Structural Steel for Buildings —Load and Resistance Factor Design, Third Edition, American Institute of Steel Construction
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-02), American Society of Civil Engineers

Lateral serviceability requirements for St. Vincent Mercy Medical Center Heart Pavilion used by the engineer of record are as follows:

Δ_{WIND} = H/500 Allowable Building Drift

Δ_{WIND} = H/400 Allowable Story Drift

$\Delta_{SEISMIC}$ = 0.015h_{sx} Allowable Story Drift

The following load cases were considered within the analysis of this technical report per IBC 2006, § 1605:

1.4 (Dead)

1.2 (Dead) + 1.6 (Live) + 0.5 (Roof Live)

1.2 (Dead) + 1.6 (Roof Live) + 1.0 (Live or 0.8 Wind)

1.2 (Dead) + 1.6 (Wind) + 1.0 (Live) + 0.5 (Roof Live)

1.2 (Dead) + 1.0 (Seismic) + 1.0 (Live)

0.9 (Dead) + 1.6 (Wind)

0.9 (Dead) + 1.0 (Seismic)

Load combinations including wind and seismic loading were applied in various directions within the computer analysis. Snow loading was not included within this analysis. A detailed list of all load combinations applied within the computer model is available upon request. After completion of torsion analysis, it was ultimately concluded that seismic loading controls the design of this structure.

MATERIALS

Multiple materials were used for the construction of St. Vincent Mercy Medical Center Heart Pavilion. The details of these materials are listed as follows:

Concrete

Foundations	f'_c = 3000 psi
Walls	f'_c = 3000 psi
Slabs	f'_c = 3500 psi
Grade Beams	f'_c = 4000 psi

Reinforcing Steel

Reinforcing Bar	A.S.T.M. A-615 GRADE 60
Tie Wire	A.S.T.M. A-82
Welded Wire Fabric	A.S.T.M. A-185

Structural Steel

Wide Flange	A.S.T.M. A992
Angle, Plate, Channel	A.S.T.M. A36
Connection Bolts	A.S.T.M. A325
Anchor Bolts	A.S.T.M. A307 OR A36
Square/Rectangle (HSS)	A.S.T.M. A500, GRADE B
Round (HSS)	A.S.T.M. A500, GRADE B

Metal Deck and Shear Studs

Composite Floor	2" 20. GA.
Roof Deck	1 ½" 22 GA.
Shear Studs	¾" x 5 ½"

LATERAL FORCE RESISTING SYSTEM ANALYSIS

A three dimensional structural model was generated in RAM Structural System to verify hand calculations associated with story forces and base shears due to lateral loads. In addition, the center of rigidity was also verified using this model.

Isometric View of Structural Model

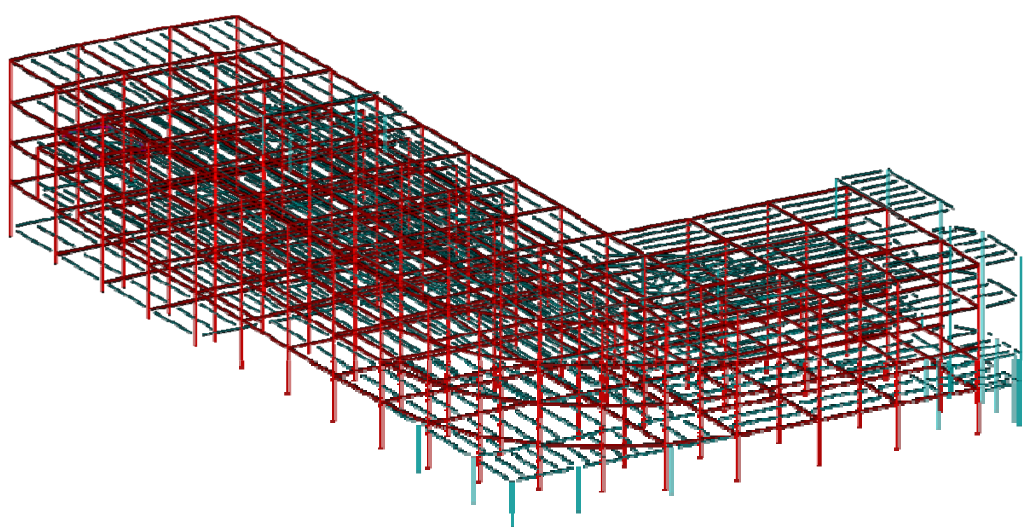


Figure 7: *Structural Model Including Gravity Elements*

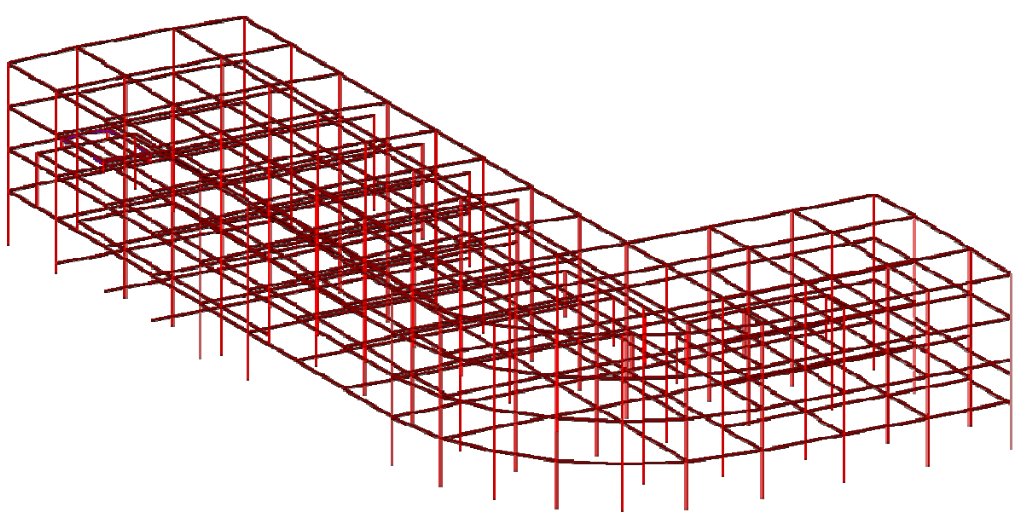


Figure 8: *Structural Model Displaying Only Lateral Elements*

Applicable Loads

Loading conditions are a very important consideration for the design of any structure. The dead load conditions assumed by the engineer of record at the time of design and live load conditions obtained from ASCE 7-02 are provided for reference in figure 9 below. The dead and live load values listed in this figure are the values used in the Member Verification section of this report.

Applicable Loads			
Dead Loads		**Live Loads**	
Concrete	150 PCF	1st Floor Corridors	100 PSF
Steel	490 PCF	Lobbies	100 PSF
Partitions	20 PSF	Loading Dock	100 PSF
M.E.P.	10 PSF	Penthouse Floor	100 PSF
Windows & Framing	10 PSF	Corridors above 1st Floor	80 PSF
Finishes & Misc.	5 PSF	Patient Rooms	60 PSF
Roof	20 PSF	Operating Rooms	60 PSF
		Bridge Floor	60 PSF
		Roof	20 PSF

Figure 9: *Applicable Loads*

Wind Criteria

Wind loads were analyzed using the analytical procedure of ASCE 7-05 §6.5 for Technical Report I and are analyzed using RAM Structural System for Technical Report III. The assumptions listed below were used to determine gust effect factors, wind pressures, and story shears for both reports. Please refer to Appendix B for detailed calculations regarding wind analysis.

Basic Wind Speed V…………………………………. 90 mph

Exposure Category…………………………………... B

Importance Factor…………………………………… 1.15

Internal Pressure Coefficient……………………… +/- 0.18

Directionality Factor………………………………… 0.85

Topography Factor……………………………………1.0

Wind Hand Calculations vs. RAM Output

The following figures were provided for a comparison of values obtained from the preliminary analysis in Technical Report I and those obtained from the three-dimensional RAM model.

	Wind Design (E-W Direction)					
	Story Loads (k)			Story Shears (k)		
Story	Hand Calculations	RAM Output	% Difference	Hand Calculations	RAM Output	% Difference
Roof	28	38.85	27.9	0	29.56	-
3	53	43.96	20.6	28	64.23	56.4
2	50	41.91	19.3	81	92.03	12.0
1	48	52.98	9.4	131	126.58	3.5
Total Base Shear (k)	179	177	1.1	179	179.56	0.3
Overturning Moment (ft-k)	6,043	9,026	33.1			

Figure 10: *Story Loads for E-W Direction*

The value obtained for base shear in the E-W direction matches very closely with the output from RAM Structural System. The loads on specific floors vary slightly due to simplified assumptions made within the calculations in the preliminary analysis. First, the pedestrian bridge connecting the second floor with the adjacent parking garage on the south side of the building was neglected for its contribution to wind loads. Second, the curtain wall on the corner of the building was taken to be a rectangular shape as opposed to a curve as seen in figure 11 to the right. In addition, wind effects on the roof and canopy entrance were neglected. Please reference Appendix B for detailed calculations for the loads obtained in the preliminary analysis that were used for this technical report.

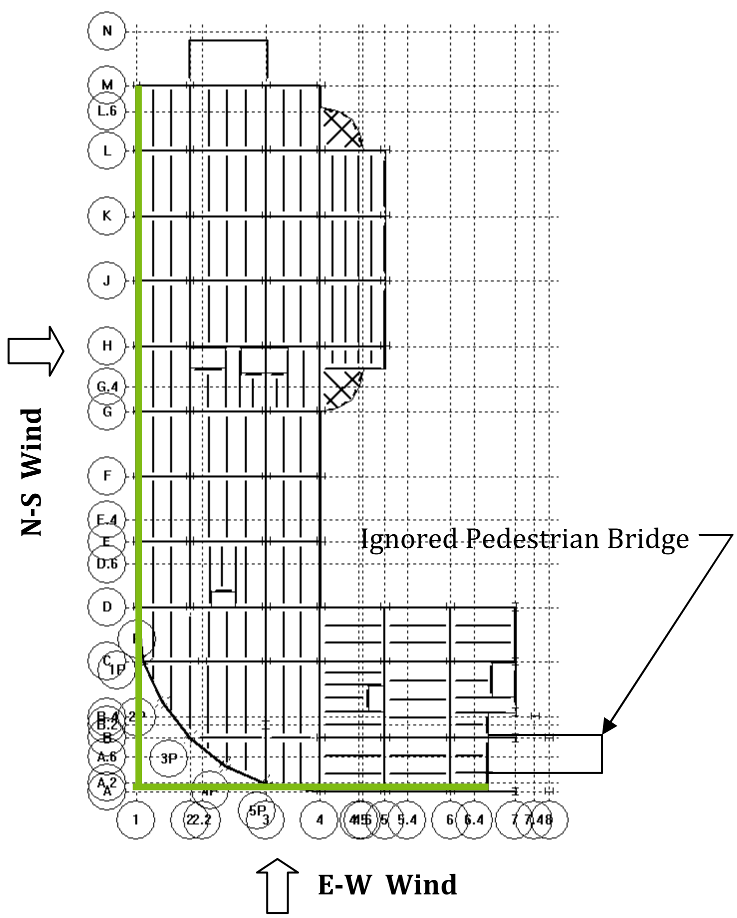
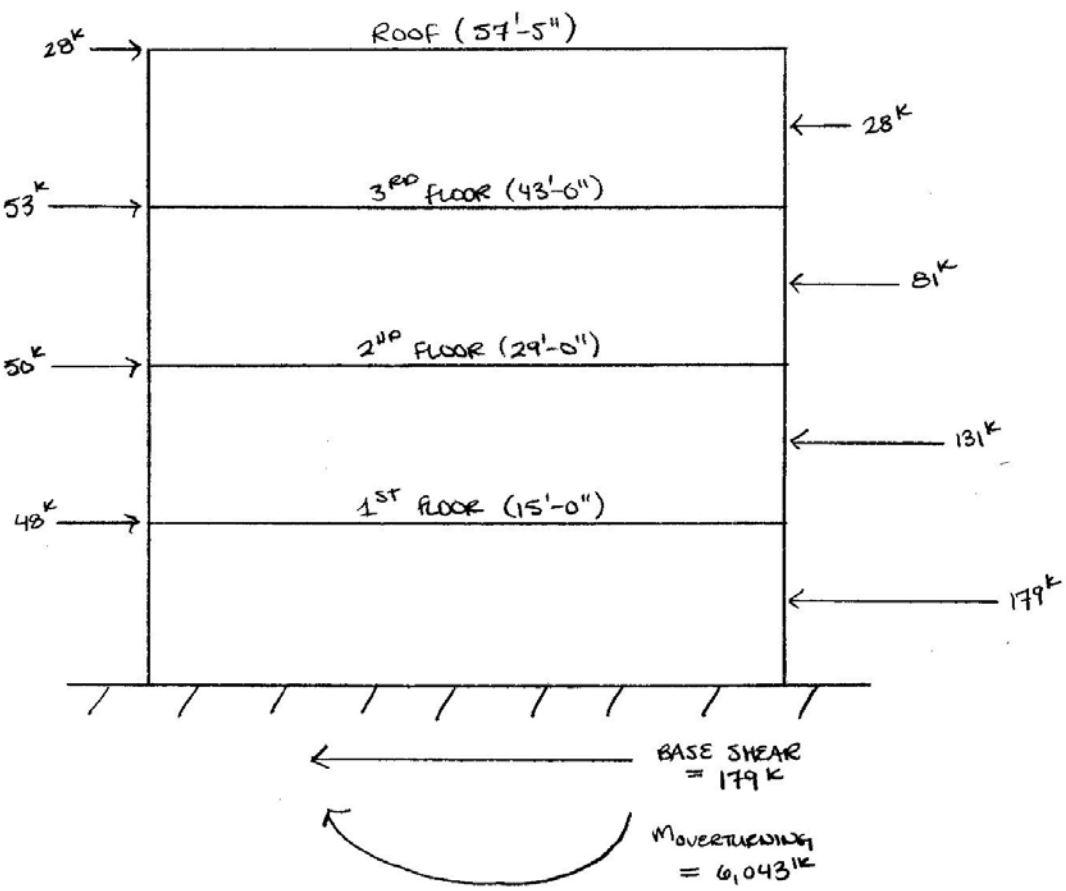


Figure 11: *Typical Floor Plan*

WIND STORY LOAD DIAGRAM

EAST-WEST DIRECTION



SAMPLE CALCULATION — LOAD ON 3RD FLOOR

$$\text{STORY LOAD} = \frac{14.31\,psf - (-7.54\,psf) + 13.42\,psf - (-7.54\,psf)}{2}\left[\frac{14.4' + 14'}{2}\right]\left[\frac{175}{1000}\right]$$

$$= 53^K$$

PLEASE REFERENCE APPENDIX B FOR WIND ANALYSIS CALCULATIONS

Wind Hand Calculations vs. RAM Output

The following figures were provided for a comparison of values obtained from the preliminary analysis in Technical Report I and those obtained from the three-dimensional RAM model.

	Wind Design (N-S Direction)					
	Story Loads (k)			Story Shears (k)		
Story	Hand Calculations	RAM Output	% Difference	Hand Calculations	RAM Output	% Difference
Roof	57	59.3	3.9	0	44.83	-
3	111	80.89	37.2	57	107.27	46.9
2	105	83.36	26.0	168	164.77	2.0
1	102	97.72	4.4	273	230.26	18.6
Total Base Shear (k)	375	321	16.8	375	327.98	14.3
Overturning Moment (ft-k)	12,618	15,418	18.2			

Figure 12: *Story Forces for Wind Design in N-S Direction*

The base shear and story loads in the N-S direction are slightly off from the values obtained from RAM Structural System. This variance is due to simplified assumptions made within the calculations in the preliminary analysis. First, the protruding loading dock located on the main floor of the building was neglected for its contribution to wind loads. Second, the curtain wall on the corner of the building was taken to be a rectangular shape as opposed to a curve as seen in figure 13 to the right. In addition, wind effects on the roof and canopy entrance were neglected. Please reference Appendix B for detailed calculations for the loads obtained in the preliminary analysis that were used for this technical report.

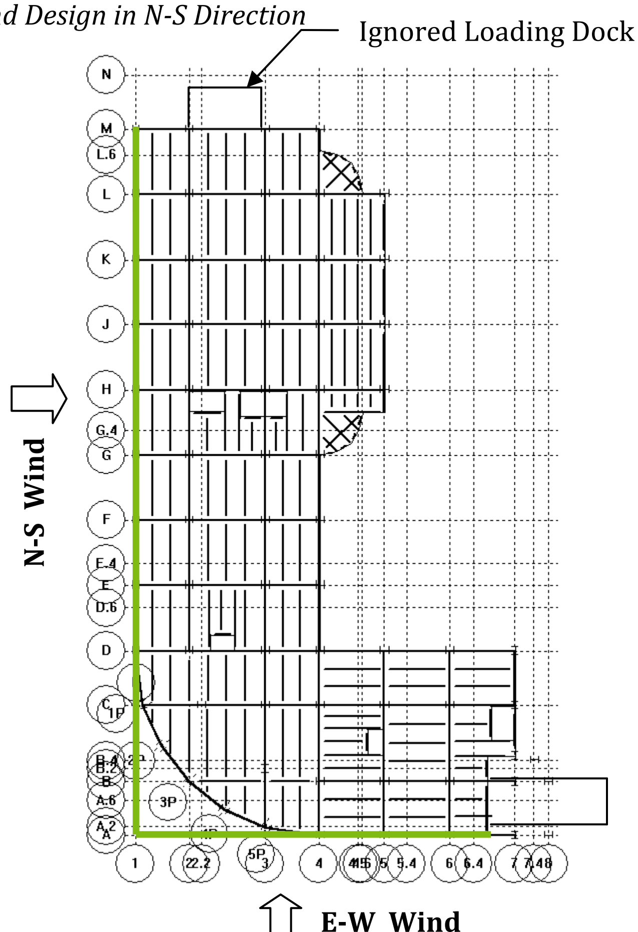
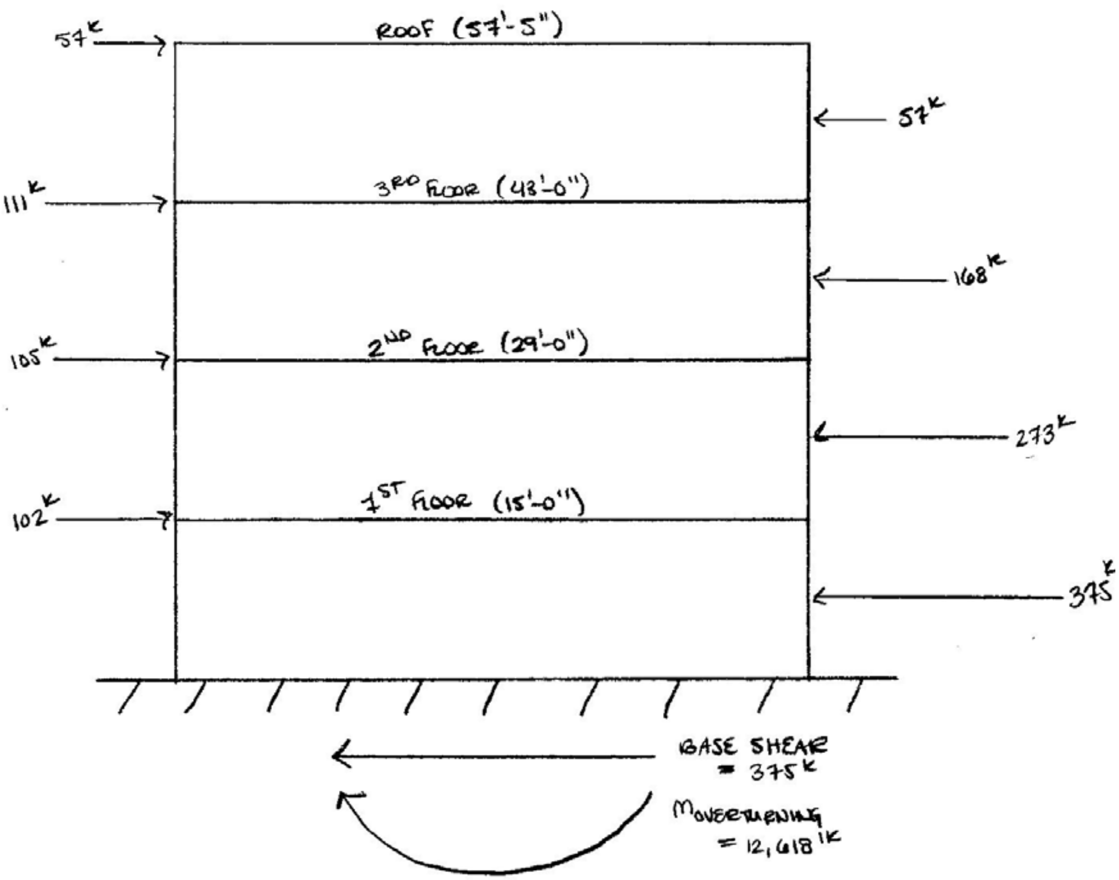


Figure 13: *Typical Floor Plan*

WIND STORY LOAD DIAGRAM

NORTH - SOUTH DIRECTION



SAMPLE CALCULATION — LOAD ON 3RD FLOOR

$$\text{STORY LOAD} = \frac{13.89psf - (-9.83psf) + 13.03psf - (-9.83psf)}{2}\left[\frac{14.4' + 14'}{2}\right]\left[\frac{335}{1000}\right]$$

$$= 111^k$$

PLEASE REFERENCE APPENDIX B FOR WIND ANALYSIS CALCULATIONS

Seismic Criteria

Seismic loads were analyzed using chapters 11 and 12 of ASCE 7-05. Please refer to Appendix C for detailed calculations used to obtain building weight as well as base shear and overturning moment distribution for each floor as seen in Figure 9 on the following page.

Occupancy Category……………………………….. IV

Importance Factor…………………………………. 1.5

Spectral Response Accelerations

S_s………………………………………………….. 0.170

S_1…………………………………………………... 0.056

Site Class…………………………………………….. E

Site Class Factors

F_a…………………………………………………... 2.5

F_v…………………………………………………... 3.5

S_{ms}……………………………………………………… 0.425

S_{m1}……………………………………………………… 0.196

S_{ds}……………………………………………………… 0.283

S_{d1}……………………………………………………… 0.131

Seismic Design Category……………………………… B

Response Modification Factor…………………….. 3.0

Seismic Period Coefficient (C_t)…………………...0.028

Seismic Period Coefficient (C_s)…………………...0.092

Seismic Hand Calculations vs. RAM Output

The following figures were provided for a comparison of values obtained from the preliminary analysis in Technical Report I and those obtained from the three-dimensional RAM model.

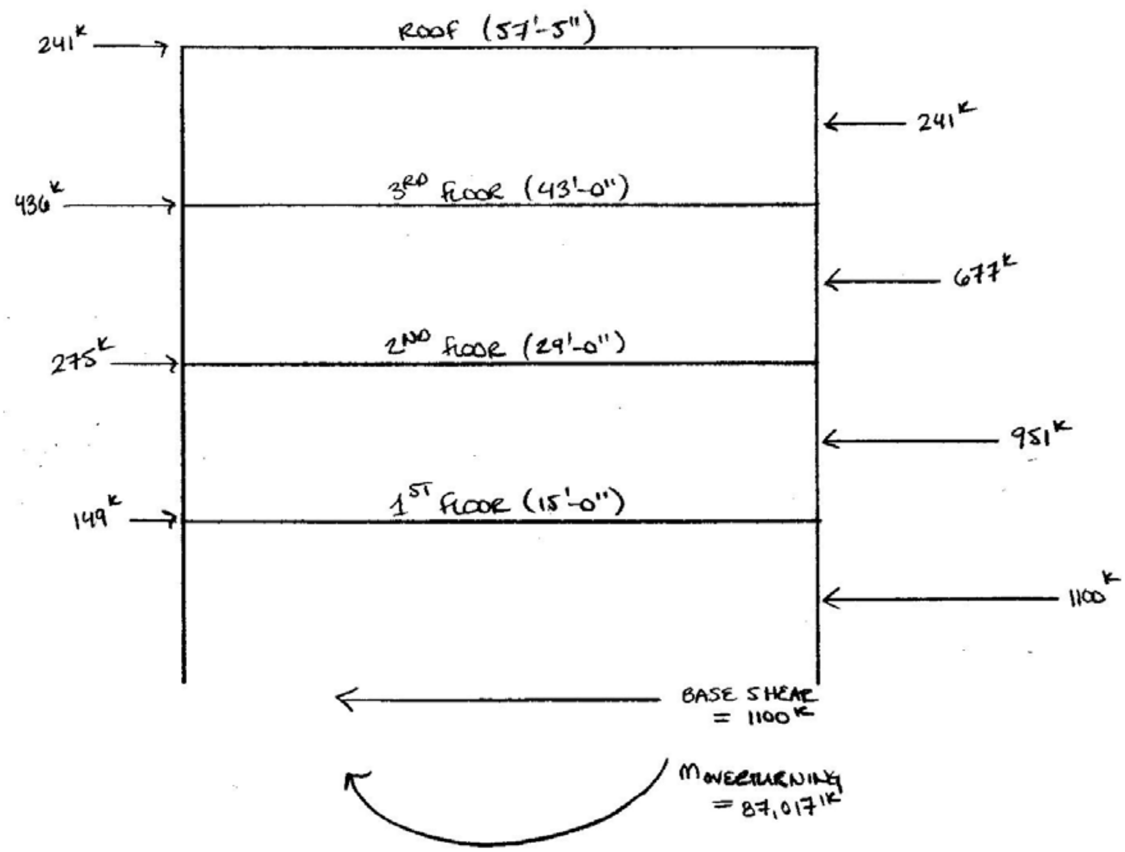
	Seismic Design					
	Story Loads (k)			Story Shears (k)		
Story	Hand Calculations	RAM Output	% Difference	Hand Calculations	RAM Output	% Difference
Roof	241	231.85	3.9	241	235.41	2.4
3	436	395.18	10.3	677	647.49	4.6
2	275	259.65	5.9	951	916.40	3.8
1	149	157.31	5.3	1100	1112.48	1.1
Total Base Shear (k)	1100	1044	5.4	1100	1112.48	1.1
Overturning Moment (ft-k)	87,017	84,617	2.8			

Figure 14: *Story Forces for Seismic Design*

The base shear value for this building seems extremely high at first glance, however, the nature of the soil within the site had a significant impact on the determination of this value. Based on field and laboratory test data within the geotechnical report for the site, it was determined that more than 10 feet of soil located 12 to 40 feet below existing grade has an un-drained shear strength of less than 500 psf. As a result, the site is characterized by the Ohio Building Code as Seismic Site Class E, “Soft Soil Profile”. This means that the soil is very weak and cannot take great shear force. If the soil was classified as Seismic Site Class B, the base shear would be reduced by approximately 60%. Without considering torsion effects, this reduction leads to a wind-controlled design.

Due to the fact that the soil is soft in nature, seismic forces control the design of this building even when torsion effects are considered, as seen in the “Torsion Effects” section later in this report.

SEISMIC STORY LOAD DIAGRAM



DISTRIBUTION FACTORS

Lateral forces are distributed based on frame relative stiffness. The composite floor system was treated as a rigid diaphragm, thus distributing lateral loads to each moment frame within the building based on their relative stiffness. Figures 15 and 16 below show how each frame was analyzed to determine the relative stiffness. The intersection of column lines A and 1 are taken as x=0.00 and y=0.00, respectively.

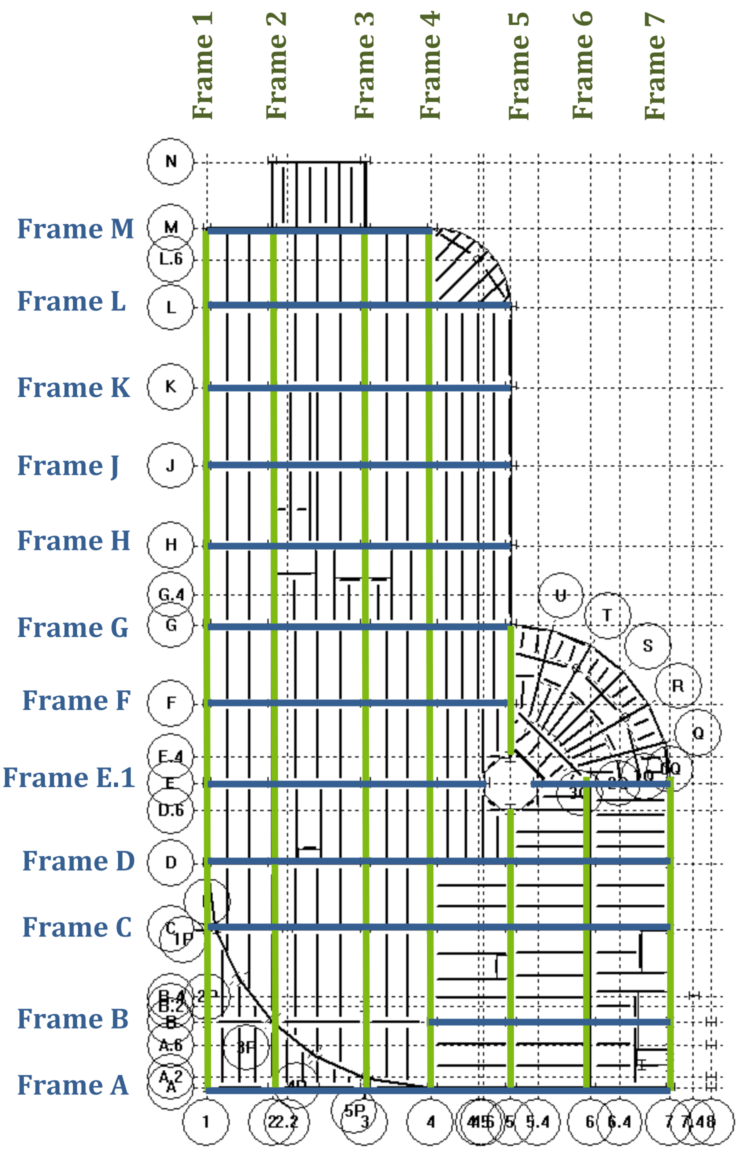


Figure 15: *Moment Frames on 1st and 2nd Floor*

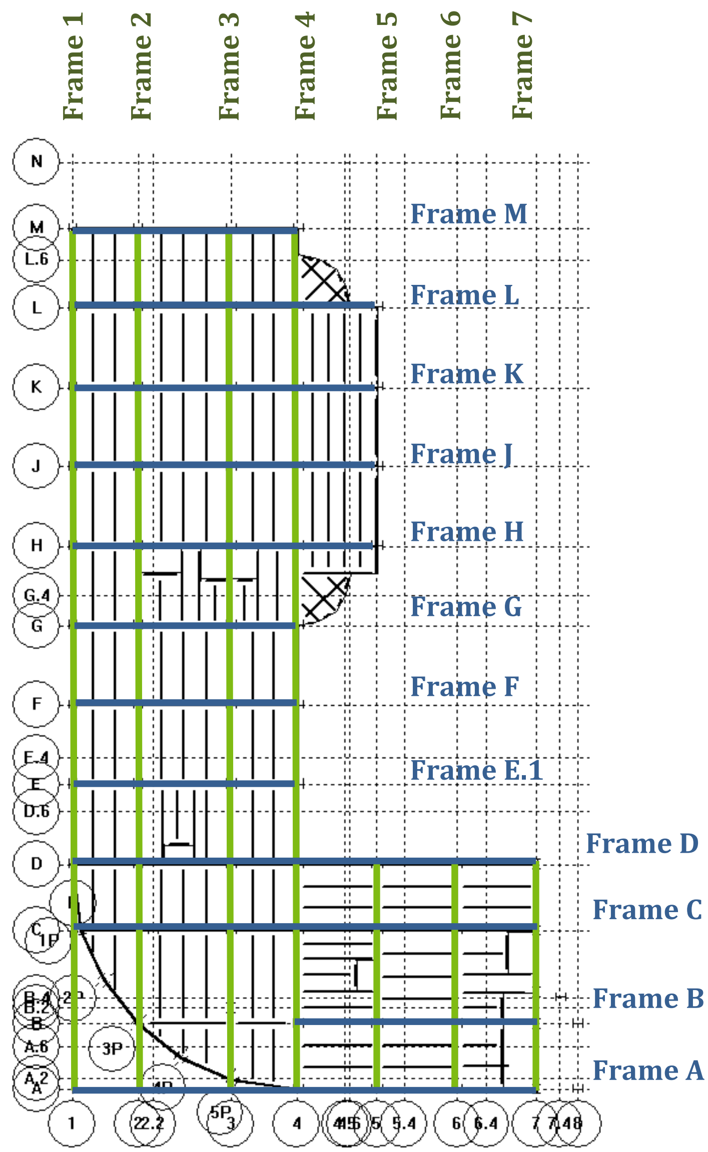


Figure 16: *Moment Frames on 3rd Floor and Roof*

Frame Modeling with SAP 2000

Relative stiffness of each frame was determined using SAP 2000. A one kip load was applied, the deflection was measured, and the inverse was taken, thus producing the relative stiffness of that frame. Most frames had the same number of bays for the entire height of the building. However, since the main and first floors are larger than the second and third floors, some frames have two different stiffness, depending on which floor is being analyzed. An example of this case is seen in frame F in figure 17 and 18. For these cases, a one kip load was applied at the top of the frame to measure stiffness for the 3rd floor and roof and the procedure was repeated with a one kip load applied to the 2nd floor to measure stiffness for the 1st and 2nd floor. For this reason, a particular frame may have differing stiffness and distribution factors for different floors as seen in figure 19 on the following page.

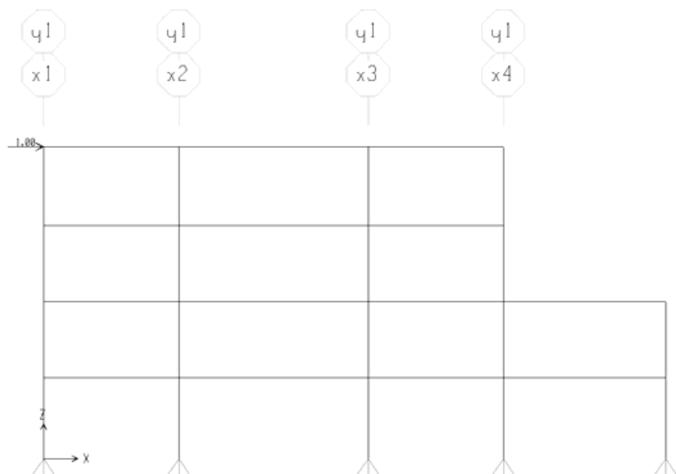


Figure 17: *Frame F*

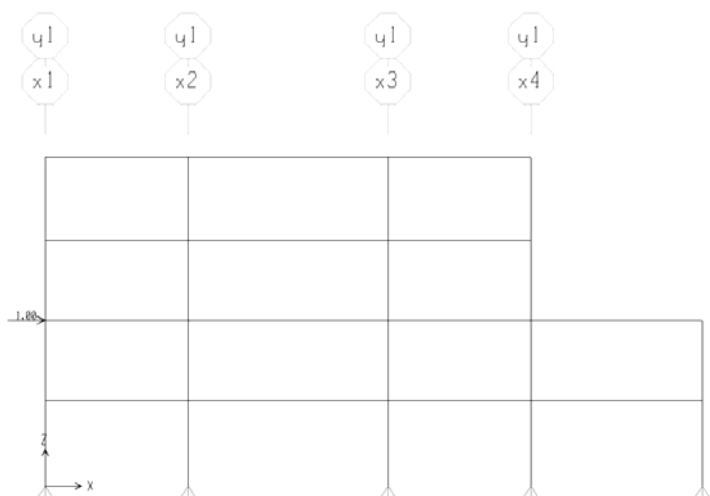


Figure 18: *Frame F*

Relative Stiffness and Distribution Factors

Relative stiffness was computed using SAP 2000 for each frame using the concept that stiffness is load divided by deflection. This procedure was carried out for each moment frame within the building and figure 19 below was provided based upon this data.

Frame	Stiffness (k/in)				Distribution Factors			
	Roof	3rd Floor	2nd Floor	1st Floor	Roof	3rd Floor	2nd Floor	1st Floor
A	26.67	26.67	26.67	26.67	0.070	0.070	0.070	0.070
B	39.68	39.68	39.68	39.68	0.104	0.104	0.104	0.104
C	39.68	39.68	39.68	39.68	0.104	0.104	0.104	0.104
D	39.68	39.68	39.68	39.68	0.104	0.104	0.104	0.104
E.1	42.74	42.74	42.74	42.74	0.112	0.112	0.112	0.112
E.2	18.05	18.05	18.05	18.05	0.047	0.047	0.047	0.047
F	24.69	24.69	43.48	43.48	0.065	0.065	0.096	0.096
G	24.69	24.69	43.48	43.48	0.069	0.069	0.096	0.096
H	25.84	33.90	33.90	33.90	0.068	0.075	0.075	0.075
J	25.84	33.90	33.90	33.90	0.068	0.075	0.075	0.075
K	25.84	33.90	33.90	33.90	0.068	0.075	0.075	0.075
L	25.84	33.90	33.90	33.90	0.068	0.075	0.075	0.075
M	21.74	21.74	21.74	21.74	0.057	0.057	0.057	0.057
					1.0	1.0	1.0	1.0
1	53.48	53.48	53.48	53.48	0.138	0.138	0.138	0.138
2	64.52	64.52	64.52	64.52	0.167	0.167	0.167	0.167
3	64.52	64.52	64.52	64.52	0.167	0.167	0.167	0.167
4	64.52	64.52	64.52	64.52	0.167	0.167	0.167	0.167
5.1	23.58	23.58	23.58	23.58	0.061	0.061	0.061	0.061
5.2	46.08	46.08	46.08	46.08	0.119	0.119	0.119	0.119
6	24.04	24.04	45.66	45.66	0.062	0.062	0.112	0.112
7	45.66	45.66	45.66	45.66	0.118	0.118	0.118	0.118
					1.0	1.0	1.0	1.0

Figure 19: *Frame Stiffness and Distribution Factors*

TORSION EFFECTS

When the resultant shear force of lateral loads acts at an eccentricity, the resultant force will try to twist the building around its center of rigidity. This concept is known as torsion. Depending on the building footprint, torsion effects can have a significant impact on the controlling load case used for structural design. The irregular shape of St. Vincent Mercy Medical Center Heart Pavilion creates significant torsion effects on certain frames due to where the center of rigidity is located. Please refer to figures 20 and 21 and below, where the center of mass is shown in red and the center of rigidity is shown in blue.

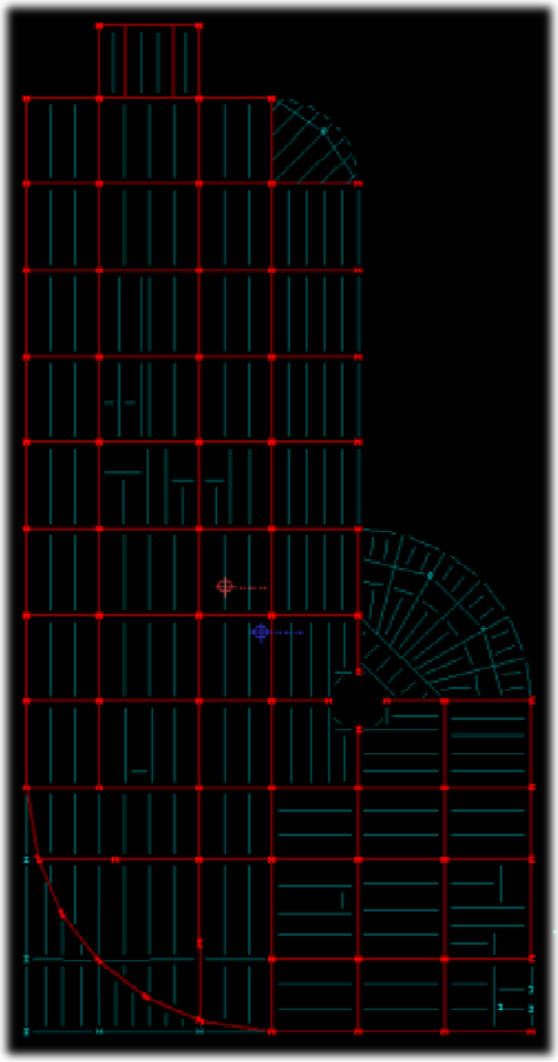


Figure 20: *Center of Mass and Rigidity for 1st and 2nd Floor*

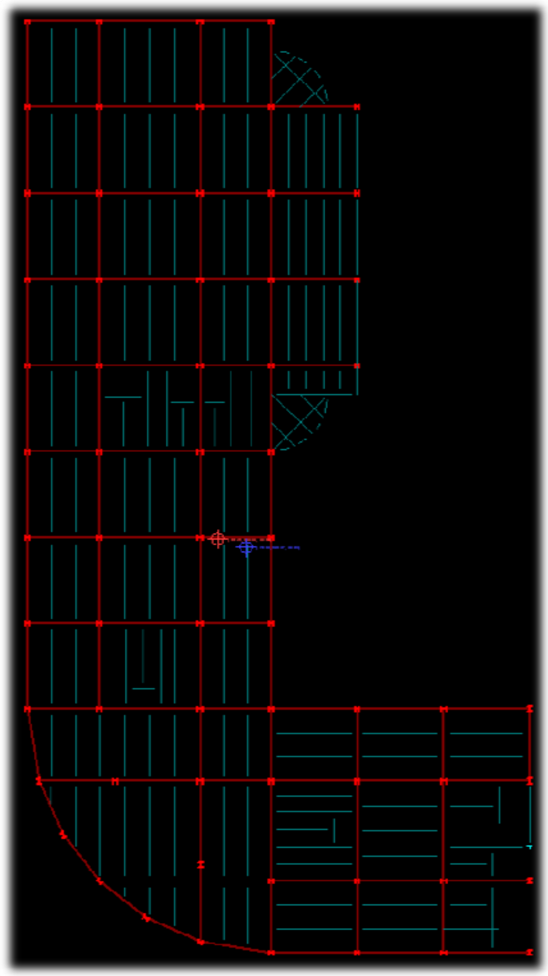


Figure 21: *Center of Mass and Rigidity for 3rd Floor and Roof*

Center of Rigidity and Center of Mass

Center of rigidity was calculated by hand for all floors by multiplying frame stiffness by the distance the frame is from the origin and dividing by the sum of all stiffness times the distance from the origin. Figure 22 was provided based upon this data. The equations used are as follows:

$$K_{ix}*d_{ix}/\Sigma K_{ix} \text{ (for frames 1-7)}$$

$$K_{iy}*d_{iy}/\Sigma K_{iy} \text{ (for frames A-M)}$$

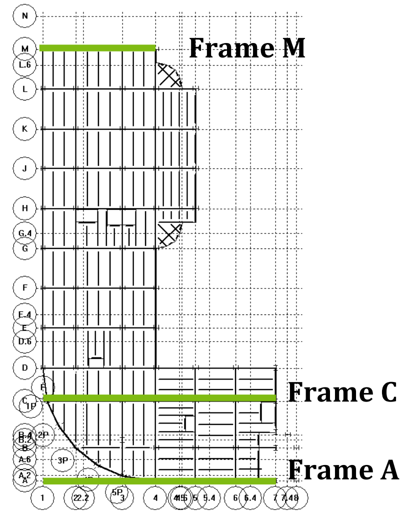
The intersection of column lines A and 1 are taken as x=0.00 and y=0.00, respectively.

	Hand Calculations		**Ram Output**							
	Center of Rigidity (ft)		Center of Rigidity (ft)		% Diff. $_x$	% Diff. $_y$	Center of Mass (ft)		Eccentricities (ft)	
Floor	x_R	y_R	x_R	y_R			x_M	y_M	e_x	e_y
Roof	78.82	149.77	70.28	144.78	12.2	3.4	74.25	135.82	4.57	13.95
3	78.82	149.77	72.74	142.15	8.4	5.4	63.32	142.85	15.5	6.92
2	82.33	159.03	76.10	141.37	8.2	12.5	66.25	144.44	16.08	14.59
1	82.33	159.03	81.43	139.20	1.1	14.2	69.11	154.69	13.22	4.34

Figure 22: *Center of Mass and Center of Rigidity*

The RAM output for the center of rigidity is very close to what was calculated by hand. The variation is a result of ignoring the entrance canopies, pedestrian bridge, and openings in the floors for the hand calculations. After verifying the center of rigidity values obtained from RAM Structural System by hand, it was concluded that the center of mass values obtained from RAM would be satisfactory to use within this report for torsion calculations.

Torsion Effects from Wind Loading



Torsion due to wind loading is caused by the eccentricity measured from the geometric center of pressure to the center of rigidity. Frame A was chosen in the North-South direction for torsion analysis because of the assumption made that the curved curtain wall was actually rectangular. It was desired to see what effects torsion would have on this frame. Frame C was also chosen for torsion analysis because this frame has the greatest distribution factor in this direction; therefore it will receive larger forces, which may be significant when doing spot checks. Frame M was also analyzed because this is the farthest frame from the center of rigidity; therefore it should receive the most torsion force as seen in figure below. The total forces for these frames are calculated by adding the direct force and the torsion force. These forces are then multiplied by a factor of 1.6 because this is the LRFD load factor for wind.

	Story	Force N-S (X Dir.) (K)	Force E-W (Y Dir.) (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
				Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame A	Roof	57	28	3.99	0.17	6.6
	3	111	53	7.77	0.32	12.9
	2	105	50	7.35	0.12	11.9
	1	102	48	7.14	0.11	11.6

Figure 23: *Total Force on Frame A Including Torsion Effects*

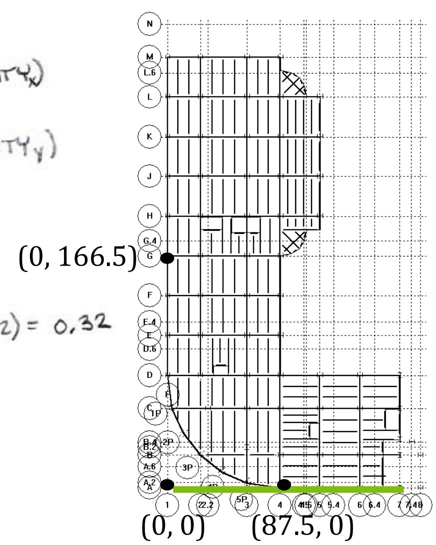
SAMPLE CALCULATION OF TORSIONAL FORCES — 3RD FLOOR FRAME A

$$\text{TORSIONAL MOMENT}_x = \text{STORY FORCE}\,(\text{CENTER OF WALL} - \text{CENTER OF RIGIDITY}_x)$$
$$= 111\,(166.5 - 78.82) = 9732^{\text{'K}}$$

$$\text{TORSIONAL MOMENT}_y = \text{STORY FORCE}\,(\text{CENTER OF WALL} - \text{CENTER OF RIGIDITY}_y)$$
$$= 111\,|87.5 - 149.77| = 3300^{\text{'K}}$$

$$\text{TORSIONAL FORCE} = \frac{K_i x_i}{I_p} M_y + \frac{K_i y_i}{I_p} M_x$$

$$= \frac{26.67\,|87.5 - \overset{COR_x}{78.82}|}{(\underbrace{12.28\times10^6}_{I_x} + \underbrace{3.56\times10^6}_{I_y})}(3300) + \frac{26.67\,(166.5 - \overset{COR_y}{149.77})}{(\underbrace{12.28\times10^6}_{I_x} + \underbrace{3.56\times10^6}_{I_y})}(9732) = 0.32$$

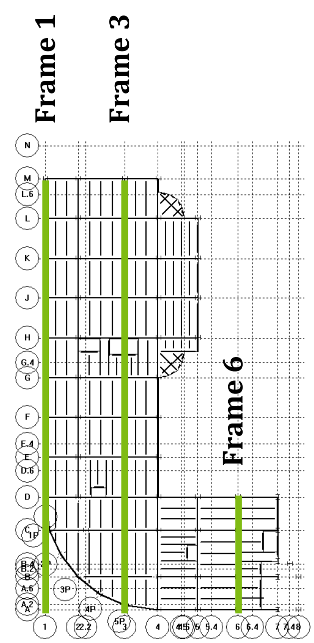


	Story	Force N-S (X Dir.) (K)	Force E-W (Y Dir.) (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
				Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame C	Roof	57	28	5.94	1.06	11.2
	3	111	53	11.56	0.89	19.9
	2	105	50	10.94	1.67	20.2
	1	102	48	10.62	1.62	19.6

Figure 24: *Total Force on Frame C Including Torsion Effects*

	Story	Force N-S (X Dir.) (K)	Force E-W (Y Dir.) (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
				Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame M	Roof	57	28	3.25	1.34	7.4
	3	111	53	6.33	2.61	14.3
	2	105	50	5.99	1.87	12.6
	1	102	48	5.82	1.81	12.2

Figure 25: *Total Force on Frame M Including Torsion Effects*



In the East-West Direction, Frame 1, 3 and 6 were chosen for torsion analysis. Since the curtain wall along frame 1 was assumed to be rectangular instead of a curve, it was desired to see what effects torsion would have upon this frame. Frame 3 was also chosen for torsion analysis because this frame has the greatest distribution factor in this direction; therefore it will receive larger forces, which may be significant when doing spot checks. Frame 6 was also analyzed because this is the farthest frame from the center of rigidity; therefore it should receive the most torsion force as seen in figure below. It was chosen over frame 7 because frame 6 continues up through all four floors of the building. The total forces for these frames are calculated by adding the direct force and the torsion force. These forces are then multiplied by a factor of 1.6 because these loads are from wind.

	Story	Force N-S (X Dir.) (K)	Force E-W (Y Dir.) (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
				Fiy=(Kiy/ΣKiy)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 1	Roof	57	28	3.88	0.75	7.39
	3	111	53	7.34	1.43	14.02
	2	105	50	6.92	0.99	12.65
	1	102	48	6.64	0.95	12.15

Figure 26: *Total Force on Frame 1 Including Torsion Effects*

	Story	Force N-S (X Dir.) (K)	Force E-W (Y Dir.) (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
				Fiy=(Kiy/ΣKiy)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 3	Roof	57	28	4.68	0.47	8.24
	3	111	53	8.85	0.92	15.62
	2	105	50	8.35	0.48	14.13
	1	102	48	8.01	0.46	13.57

Figure 27: *Total Force on Frame 3 Including Torsion Effects*

	Story	Force N-S (X Dir.) (K)	Force E-W (Y Dir.) (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
				Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 6	Roof	57	28	1.74	0.84	4.1
	3	111	53	3.30	1.51	7.7
	2	105	50	5.60	2.54	13.0
	1	102	48	5.37	2.46	12.5

Figure 28: *Total Force on Frame 6 Including Torsion Effects*

Torsion Effects from Seismic Loading

Torsion due to seismic loading is caused by the eccentricity measured from center of mass to center of rigidity. For purpose of determining the controlling load case for the structural design of St. Vincent Mercy Medical Center Heart Pavilion, the same frames for both directions were chosen for torsion analysis due to seismic loading. The total forces for these frames are calculated by adding the direct force and the torsion force. These forces are then multiplied by a factor of 1.0 because this is the LRFD load factor for seismic.

	Story	Force (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame A	Roof	241	16.87	0.33	17.2
	3	436	30.52	1.75	32.3
	2	275	19.25	0.76	20.0
	1	149	10.43	0.32	10.7

Figure 29: *Total Force on Frame A Including Torsion Effects*

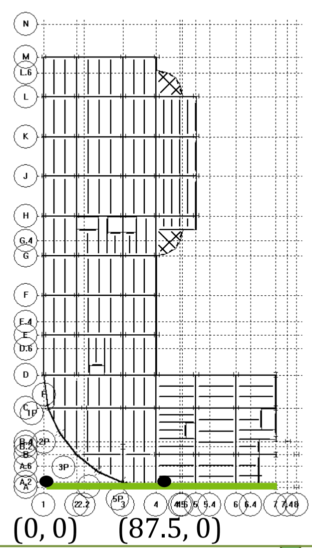
SAMPLE CALCULATION OF TORSIONAL FORCE — 3^{RD} FLOOR FRAME A

$$\text{TORSIONAL MOMENT}_x = \text{STORY FORCE}\,(\text{CENTER OF MASS}_x - \text{CENTER OF RIGIDITY}_x)$$
$$= 436\,|63.32 - 78.82| = 6759^{\,1k}$$

$$\text{TORSIONAL MOMENT}_y = \text{STORY FORCE}\,(\text{CENTER OF MASS}_y - \text{CENTER OF RIGIDITY}_y)$$
$$= 436\,|142.85 - 149.77| = 3017^{\,1k}$$

$$\text{TORSIONAL FORCE} = \frac{K_i x_i}{I_p} m_y + \frac{K_i y_i}{I_p} m_x$$

$$= \frac{26.67(87.5 - \underbrace{78.82}_{COR_x})}{(\underbrace{12.28\times10^6}_{I_x} + \underbrace{3.56\times10^6}_{I_y})}(3017) + \frac{26.67\,|0 - \underbrace{149.77}_{COR_y}|}{(\underbrace{12.28\times10^6}_{I_x} + \underbrace{3.56\times10^6}_{I_y})}(6759) = 1.75^{k}$$



	Story	Force (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame C	Roof	241	25.10	0.30	25.4
	3	436	45.41	1.45	46.9
	2	275	28.64	0.65	29.3
	1	149	15.52	0.28	15.8

Figure 30: *Total Force on Frame C Including Torsion Effects*

	Story	Force (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame M	Roof	241	13.75	0.44	14.2
	3	436	24.88	1.85	26.7
	2	275	15.69	0.73	16.4
	1	149	8.50	0.31	8.8

Figure 31: *Total Force on Frame M Including Torsion Effects*

	Story	Force (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
			Fix=(Ki/ΣKi)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 1	Roof	241	33.35	0.96	34.3
	3	436	60.34	1.18	61.5
	2	275	38.06	0.40	38.5
	1	149	20.62	0.19	20.8

Figure 32: *Total Force on Frame 1 Including Torsion Effects*

	Story	Force (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 3	Roof	241	40.24	0.33	40.6
	3	436	72.80	0.69	73.5
	2	275	45.92	0.19	46.1
	1	149	24.88	0.12	25.0

Figure 33: *Total Force on Frame 3 Including Torsion Effects*

	Story	Force (K)	Direct Force (k)	Torsion Force (k)	Total Factored Force on Each Story (K)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame 6	Roof	241	14.99	0.49	15.5
	3	436	27.12	2.04	29.2
	2	275	30.78	1.54	32.3
	1	149	16.67	0.66	17.3

Figure 34: *Total Force on Frame 6 Including Torsion Effects*

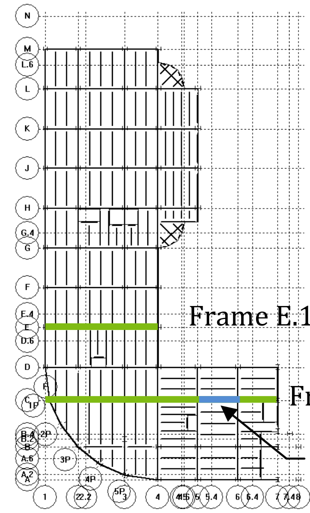
Controlling Load Case

After taking torsion effects from lateral loads into account as well as the load combination factors for both wind and seismic, it was concluded that seismic loading controls the structural design of St. Vincent Mercy Medical Center Heart Pavilion. This was expected as the base shear for seismic loads was approximately 1100 kips as opposed to a base shear of 375 kips for wind in the North-South direction. However, a torsion analysis was necessary because greater torsion forces were generated by wind loading. This result was expected because the eccentricity for torsion from wind is measured from the center of pressure to the center of rigidity whereas the eccentricity for torsion from seismic is measured from the center of mass to the center of rigidity. Based upon this conclusion, the controlling LRFD load combination for this structure is 1.2 (Dead) + 1.0 (Seismic) + 1.0 (Live) since the structural design is ultimately controlled by seismic loading.

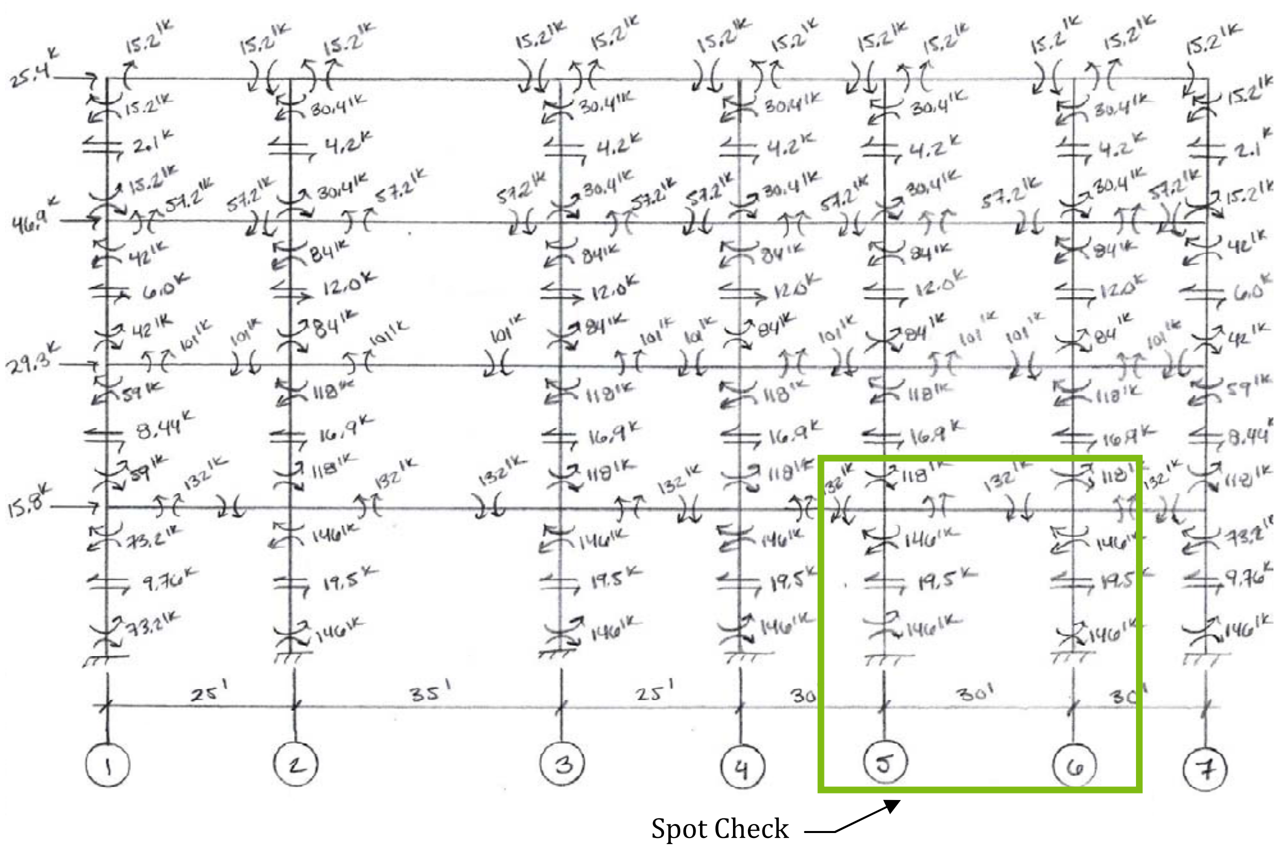
MEMBER VERIFICATION

In order to check the validity of member sizes, a portal analysis was done with the loads obtained from the torsion analysis. Moments due to dead and live loads were then added to the moments found from the portal analysis.

Portal Analysis on Frame C



Frame C was chosen for a portal analysis in the North-South direction even though Frame E.1 has a higher distribution factor because the girders along Frame C will have a more critical negative moment. Since no beams frame into the girder pointed out on the plan, the distributed load from dead and live loads will be evenly distributed across this member. As a result, negative moment is created at the ends of the girder. As seen in the floor plan on the right, infill beams frame into all girders along Frame E.1, creating no negative moment on the ends and a maximum positive moment in the center of the girder. This is not the most critical case of moment that the girders will take while considering lateral forces as well as gravity.



Verification of Girder from Frame C

SPOT CHECKS ON FRAME C

- CHECK FRAME C EVEN THOUGH FRAME E.1 HAS HIGHER DISTRIBUTION FACTOR B/C GIRDERS IN FRAME C WILL BE MORE CRITICAL. GIRDERS IN FRAME C ARE MORE CRITICAL B/C THEY TAKE NEGATIVE MOMENT DUE TO DISTRIBUTED DL + LL AS OPPOSED TO GIRDERS IN FRAME E.1. GIRDERS IN FRAME E.1 HAVE INFILL BMS FRAMING INTO THEM, CREATING POSITIVE MOMENT WITHIN THESE GIRDERS.

GIRDER CHECK : CHECK GIRDER BTW. COL LINES 5 + 6 SINCE IT WILL TAKE NEGATIVE MOMENT

FROM PORTAL ANALYSIS : $M_{LL} = 132^{1K}$ ON W24×84 GIRDER

FIND MOMENT DUE TO DL+LL —

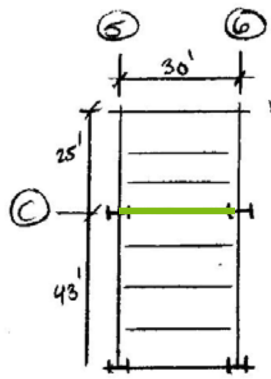
$w_{LL} = 60\text{ psf}$

$w_{DL} = 110\text{ psf}$ [CALCULATED FLOOR WEIGHT IN SEISMIC SECTION, PLEASE REFER TO APPENDIX C. TAKE 110 psf FOR ALL FLOORS]

GIRDER FRAMES INTO W12×170 + W12×120

$$l_n = 30' - \left(\frac{14.0 + 13.1}{2\,(12)}\right) = 28.9'$$

← $\frac{1}{2}$ AVG. DEPTH OF COLS



BY MOMENT COEFFICIENT METHOD (ACI 8.3.3)

NEG. MOMENT @ INT. FACE OF SUPPORT =

$$M^- = \frac{w_u l_n^2}{11} = \frac{1.83\text{ klf}\,(28.9)^2}{11} = 139^{1K}$$

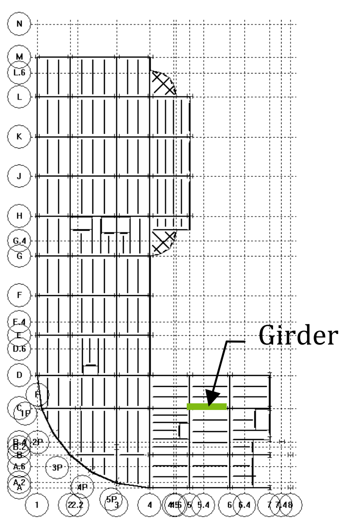
$1.2\,(DL) + 1.0\,(SEISMIC) + 1.0\,(LL)$

= CONTROLLING LOAD CASE SINCE SEISMIC DESIGN CONTROLS

$w_u = 1.2(110\text{ psf}) + 1.0\,(60) = 192\text{ psf}$

$$\text{TRIB WIDTH} = \frac{(43/4 + 25/3)}{2} = 9.54'$$

$w_u = 192\text{ psf}\,(9.54') = 1832\text{ plf} = 1.83\text{ klf}$



$M_{TOTAL} = 132^{1K} + 139^{1K} = 271^{1K}$

FROM TABLE 3-1 : $\phi M_n = 840^{1K}$

$\phi M_n = 840^{1K} > M_u = 271^{1K}$ ✓OK

SINCE THIS MEMBER IS MORE THAN ADEQUATE FOR STRENGTH, LET'S CHECK SERVICEABILITY

$$\Delta_{TOTAL\ LOAD} = \frac{5wl^4}{384EI} = \frac{5(1.83)(30)^4(1728)}{384\,(29000)\,(2370)} = 0.49''$$

$$l/240 = (30\times12)/240 = 1.5'' > 0.49'' \quad ✓OK$$

Verification of Column from Frame C

FRAME C COLUMN CHECK

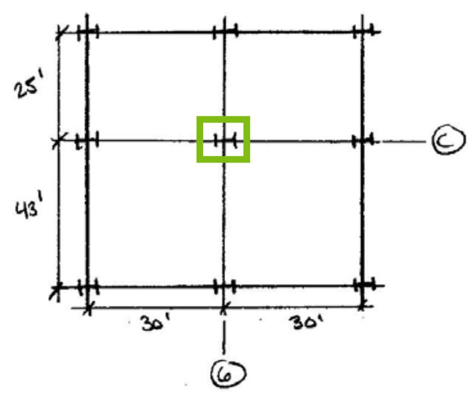
CHECK COL C-6 W 12×120

FROM PORTAL ANALYSIS: $M_u = 146^{K}$

FIND P_u FROM DL + LL ON FLOOR —

$A_T = 30'(34')(3 \text{ FLOORS ABOVE}) = 3060 ft^2$

$A_I = 4(3060 ft^2) = 12240 ft^2$



$L_R = 60psf\left[0.25 + \frac{15}{\sqrt{12240}}\right] = 23.1 psf$

$0.40(60psf) = 24psf$

$23.1 psf < 24 psf \quad \therefore$ USE 24 psf

$P_{LL} = 12240 ft^2 (24psf) = 294^{K}$

$P_{DL} = 110psf(4 \text{ FLOORS})(34 \times 30) = 449^{K}$

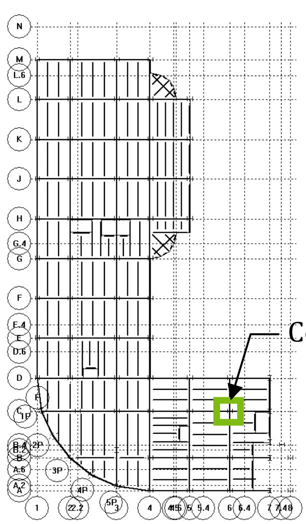
$P_u = 1.2(449) + 1.0(294) = 834^{K}$

TABLE 6-1: KL = 15' (STORY HT.)

FOR W12×120 $p = 0.802\times10^{-3}/k$ $b_x = 1.32\times10^{-3}/k$

$pP_u + b_x M_u = 0.802\times10^{-3}(834) + 1.32\times10^{-3}(146) = 0.862$

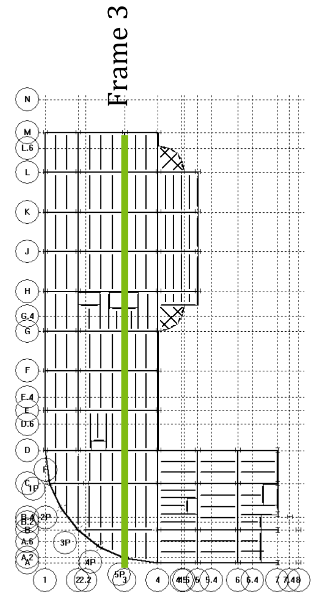
$0.862 < 1.0$ ✓OK



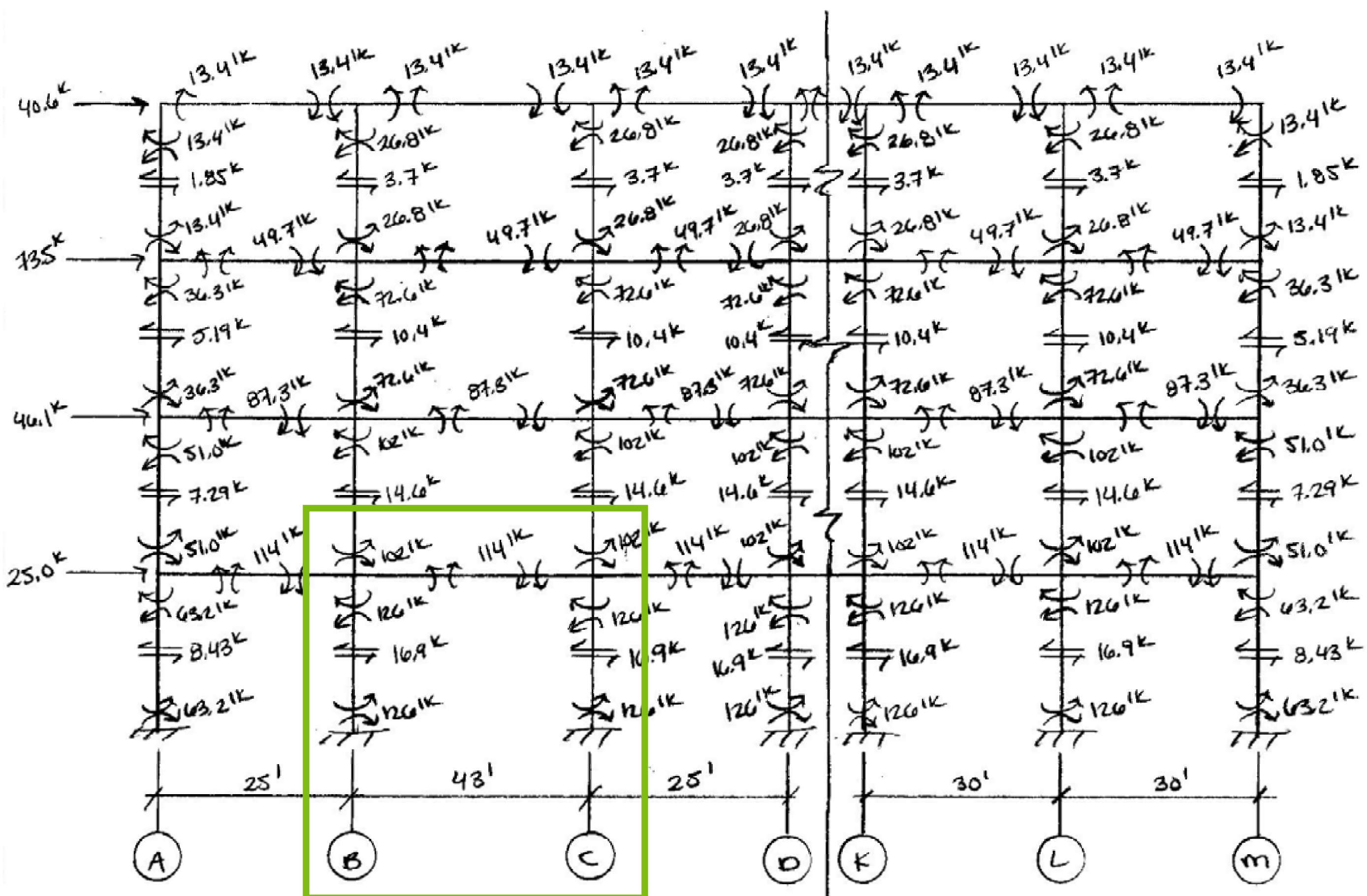
Col C-6

Portal Analysis on Frame 3

Frame 3 was chosen for a portal analysis in the East-West direction because it has the highest distribution factor for this direction and all the girders will have induced negative moment since there are no infill beams framing into these girders. If there were beams framing into the girders, there would be a positive moment generated in the middle of the girder and no negative moment at the ends. This situation is not the most critical loading condition for girders included in moment frames because lateral loads create moments in the ends of the girders.



The portal method for frame 3 was carried out for every bay within the frame; however some repetitious bays were left out of the drawing below for simplicity.



Spot Check

Verification of Girder from Frame 3

<u>SPOT CHECKS ON FRAME 3</u>

– CHECK FRAME 3 BECAUSE THIS FRAME HAS HIGHEST DISTRIBUTION FACTOR FOR THIS DIRECTION. –

<u>GIRDER CHECK</u>: CHECK GIRDER BTW. COL. LINES B + C SINCE IT IS THE LONGEST SPAN

FROM PORTAL ANALYSIS: $M_u = 114^{1k}$ ON W24X84 GIRDER

FIND MOMENT DUE TO DL + LL –

$w_{LL} = 60 psf$

$w_{DL} = 110 psf$ [CALCULATED FLOOR WEIGHT IN SEISMIC SECTION, PLEASE REFER TO APPENDIX C. TAKE 110 psf FOR ALL FLOORS]

GIRDER FRAMES INTO (2) W12X170 COLS

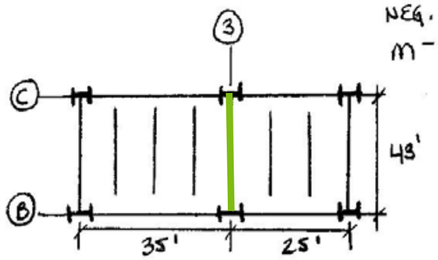
$\ell_n = 43' - (0.96''/12) = 42.9'$

t_{WEB} OF W24X84

BY MOMENT COEFFICIENT METHOD (ACI 8.3.3)

NEG. MOMENT @ INT. FACE OF SUPPORT =

$$M^- = \frac{w_u \ell_n^2}{11} = \frac{1.64 klf (42.9')^2}{11} = 274^{1k}$$



$w_u = 1.2(110 psf) + 1.0(60 psf) = 192 psf$

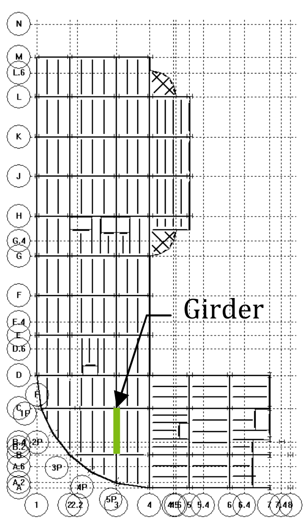
$\text{TRIB WIDTH} = \dfrac{(35/4 + 25/3)}{2} = 8.54'$

$w_u = 192 psf (8.54') = 1640 plf = 1.64 klf$

$M_{TOTAL} = 274^{1k} + 114^{1k} = 388^{1k}$

FROM TABLE 3-1: $\phi M_n = 840^{1k}$

$\phi M_n = 840^{1k} > M_u = 388^{1k}$ ✓ OK



SINCE THE MEMBER IS MORE THAN ADEQUATE FOR STRENGTH, LET'S CHECK SERVICEABILITY

$$\Delta_{TOTAL\ LOAD} = \frac{5 w \ell^4}{384 EI} = \frac{5(1.64 klf)(43')^4 (1728)}{384(29000)(2370)} = 1.84''$$

$$\ell/240 = (43 \times 12)/240 = 2.15''$$

$1.84'' < 2.15''$

THIS W24X84 GIRDER WAS CHOSEN BASED ON SERVICEABILITY

Verification of Column from Frame 3

FRAME 3 COLUMN CHECK

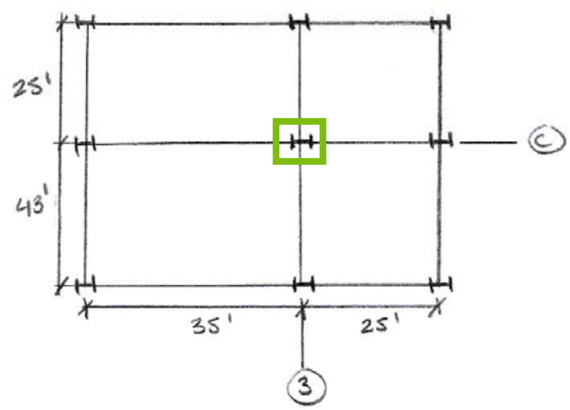
CHECK COL C-3 W12×170

FROM PORTAL ANALYSIS: $M_{LL} = 126^{1K}$

FIND P_u FROM DL + LL ON FLOOR —

$A_T = 30'(34')(3 \text{ FLOORS ABOVE}) = 3060 \text{ ft}^2$

$A_I = 4(3060 \text{ ft}^2) = 12240 \text{ ft}^2$



$L_R = 60 \text{ psf}\left[0.25 + \frac{15}{\sqrt{12240}}\right] = 23.1 \text{ psf}$

$0.40(60 \text{ psf}) = 24 \text{ psf}$

$23.1 \text{ psf} < 24 \text{ psf}$ ∴ USE 24 psf

$P_{LL} = 12240 \text{ ft}^2 (24 \text{ psf}) = 294^K$

$P_{DL} = 110 \text{ psf } (4 \text{ FLOORS})(34 \times 30) = 449^K$

$P_u = 1.2(449) + 1.0(294) = 834^K$

TABLE 6-1; $KL = 15'$ (STORY HT.)

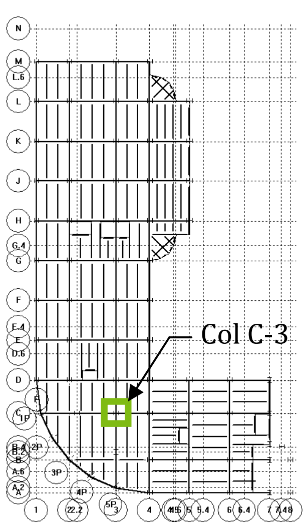
FOR W12×170 $p = 0.559 \times 10^{-3}/K$ $b_x = 0.881 \times 10^{-3}/K$

$pP_u + b_x M_u = 0.559 \times 10^{-3}(834) + 0.881 \times 10^{-3}(126) = 0.577$

$0.577 < 1.0$ ✓ OK

COLUMNS ARE MORE THAN ADEQUATE FOR STRENGTH

THEY MAY BE SIZED FOR DRIFT.



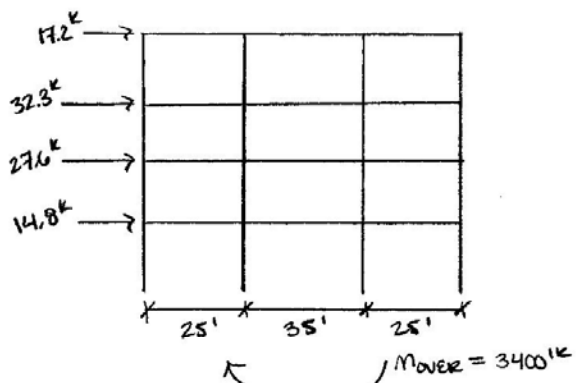
Overturning Moment on Foundation System

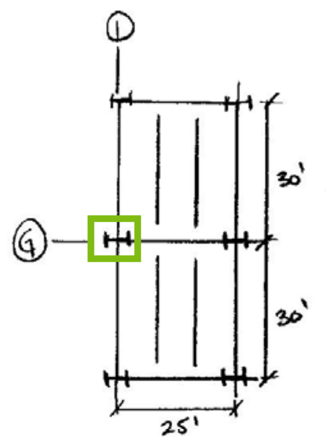
	Story	Force (K)	Direct Force (k)	Torsion Force (k)	Total Force on Each Story (K)
			Fix=(Kix/ΣKix)F	Fix=((Ki*xi)/Ip)M	F=DF+TF
Frame G	Roof	241	16.70	0.50	17.2
	3	436	30.22	2.10	32.3
	2	275	26.52	1.07	27.6
	1	149	14.37	0.45	14.8

Figure 36: *Total Force on Frame G due to Seismic Loading*

OVERTURNING ON FOUNDATIONS

CHOOSE FRAME G TO ANALYZE OVERTURNING AS IT WILL BE THE MOST CRITICAL DUE TO BEING ALONG THE SHORTEST WIDTH OF THE BUILDING.





$M_{OVER} = 17.2(57.4) + 32.3(43) + 27.6(29) + 14.8(15) = 3400'^k$

$\text{UPLIFT FORCE} = \dfrac{M_{OVER}}{\text{FRAME LENGTH}} = \dfrac{3400'^k}{85'} = 40^k$ UPLIFT

FIND IF DL ACTING ON COL G-1 WILL BE GREATER THAN UPLIFT

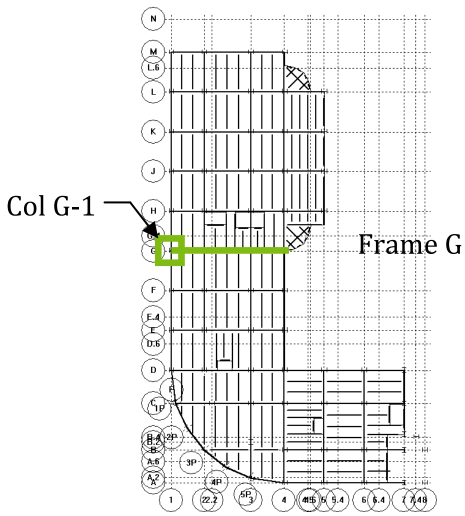
$w_{DL} = 110psf$

TRIB AREA OF COL G1 $= (30 \times \frac{25}{2})$(3 FLOORS ABOVE) $= 1125 ft^2$

$P_{DL} = 110psf\,(1125 ft^2) = 124^k$

$P_{DL} = 124^k > P_{UPLIFT} = 40^k$

∴ FOUNDATION WILL NOT OVERTURN



SERVICEABILITY REQUIREMENTS

Drift is an important serviceability requirement that can cause several problems within a building if the limitations are not met. Wind drift is not addressed in the code, however, standard engineering practice has employed a limitation of H/400 for many years. Seismic drift is addressed in ASCE 7-05 and is limited based on the occupancy category of the building. St. Vincent Mercy Medical Center is classified as occupancy category IV and normally would be limited to an allowable story drift of 0.010 h_{sx}. However, since the facility is only 4 stories, the allowable story drift is limited to 0.015h_{sx} per ASCE 7-05 Table 12.12-1. Story drift values for wind loading and story drift ratios for seismic loading were determined by RAM Frame. As seen in Figures 36 and 37 below, drift limitations are met for both wind and seismic loading.

Wind Drift									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) ΔWIND=H/400			Total Drift (in)	Allowable Total Drift (in) ΔWIND=H/400		
Roof	57.4	0.298	<	0.432	OK	0.848	<	1.722	OK
3	43	0.257	<	0.420	OK	0.550	<	1.290	OK
2	29	0.192	<	0.420	OK	0.293	<	0.870	OK
1	15	0.101	<	0.450	OK	0.101	<	0.450	OK

Figure 36: *Actual Drift due to Wind Loads vs. Industry Standard*

Seismic Drift					
Story	Story Height (ft)	Actual Drift Ratio		Allowable Drift Ratio	
Roof	57.4	0.0021	<	0.0075	OK
3	43	0.0036	<	0.0075	OK
2	29	0.0046	<	0.0075	OK
1	15	0.0047	<	0.0075	OK

Figure 37: *Actual Drift due to Seismic Loads vs. Code Limitations*

DETERMINE ALLOWABLE STORY DRIFT RATIO

ASCE 7-05 TABLE 12.2-1

C_d = DEFLECTION AMPLIFICATION FACTOR = 3 (FOR STEEL MOMENT FRAMES)

I = IMPORTANCE FACTOR = 1.5 (USED BY EOR)

$\delta_x = 0.015\,h_{sx}$ (TABLE 12.12-1)

$\delta_x = \dfrac{C_d\,\delta_{xe}}{I}$ (EQUATION 12.8-15)

$$\text{DRIFT RATIO}_{\text{ALLOW}} = \frac{\delta}{h_{sx}} = \frac{0.015\,(I)}{C_d} = \frac{0.015\,(1.5)}{3} = 0.0075$$

CONCLUSION

A better understanding of how lateral forces are distributed throughout a structure was gained upon completion of Technical Report III. When lateral forces are applied to the structure, the load follows the stiffness of the structure. Therefore, moment frames with a greater stiffness receive larger story loads. These story loads are generated as a result of lateral loads traveling through the floor diaphragm to the lateral resisting elements.

In efforts to determine which force controls the structural design, torsion analysis was investigated. Torsion due to wind loading is caused by the eccentricity measured from the center of the wall to the center of rigidity. This is because the center of pressure from the wind blowing will act at the center of the wall. Torsion due to seismic loading is caused by the eccentricity measured from the center of mass to the center of rigidity. Since the center of the wall in each direction and the center of rigidity are much farther apart than the center of mass and the center of rigidity, torsion forces from wind loading are greater. This was expected considering the shape of this building.

The total forces acting on each moment frame due to wind and seismic loading were compared in order to determine which lateral load controls the design of the structure. It was concluded that even though a greater torsion effect is seen from wind loading, seismic loading still controls the design of the structure. The soil is classified as seismic site class E and is very soft in nature. As a result, the seismic base shear for this structure was considerably affected. The torsion effects from wind acting on the structure are not great enough to overcome story loads generated by seismic loading.

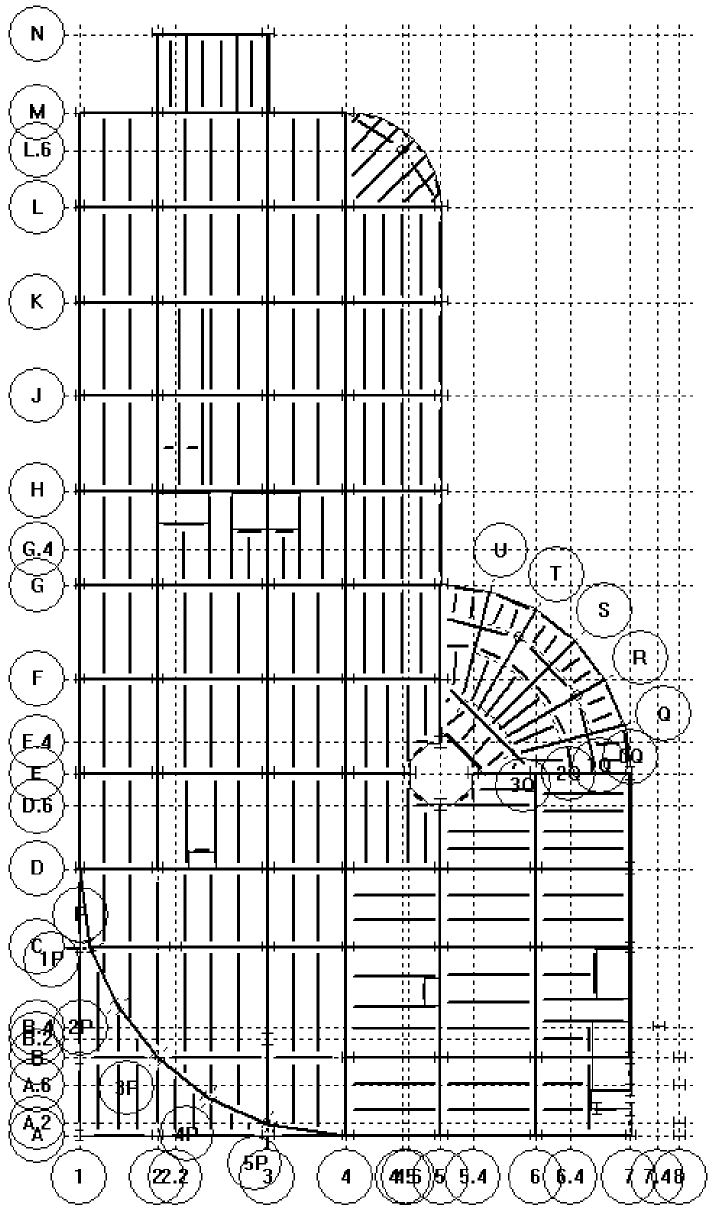
Overall, the lateral system of the Heart Pavilion meets strength requirements as well as serviceability requirements. It was determined that the girders and columns within the lateral system are more than adequate for strength requirements. Upon completion of serviceability checks due to dead and live loads, it was found that the girders meet deflection criteria by approximately 14%. Upon completion of checking lateral drift limitations, it was found that the actual seismic drift ratio on the first floor is 0.0047 compared to an allowable seismic drift ratio of 0.0075. It was concluded that column sizes meet allowable drift limitations by approximately 37%. Further investigation of these member sizes will be conducted in efforts to further optimize the structural system.

All design values used and procedures carried out were done in accordance with applicable codes. Please refer to the appendices for further review of detailed notes, figures, or tables regarding this matter. Questions should be directed to Kristen M. Lechner via email: kml5016@psu.edu.

APPENDIX A: BUILDING LAYOUT

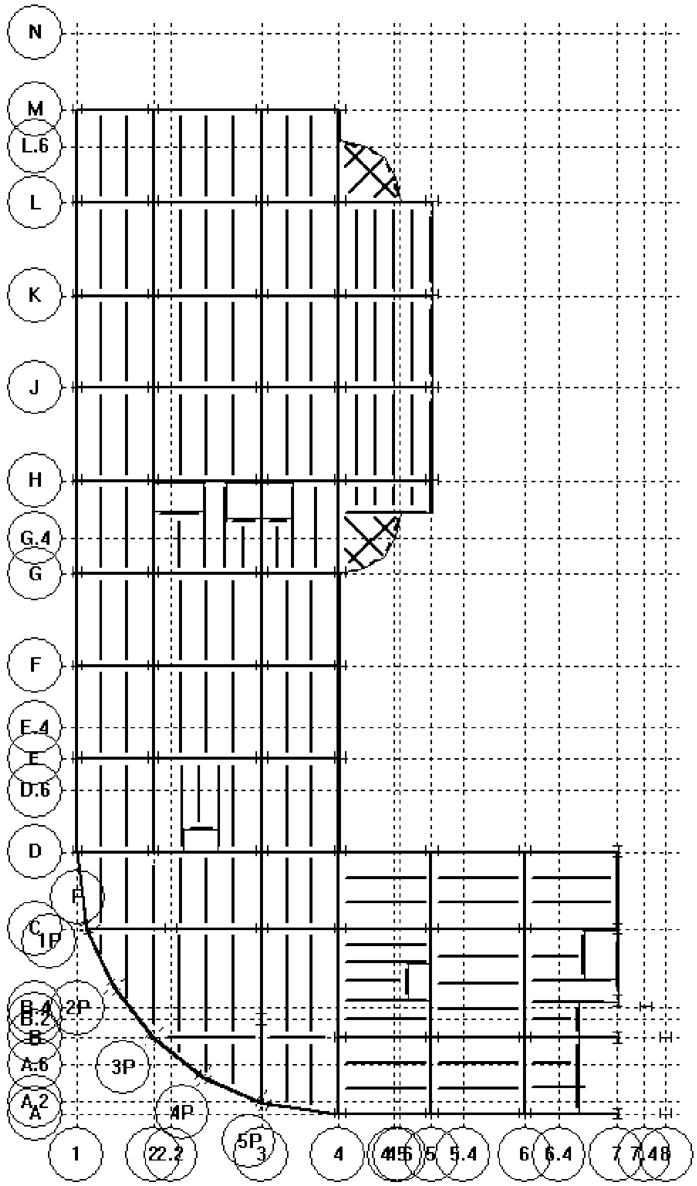
Photos courtesy of Ruby + Associates

Existing Floor Layout



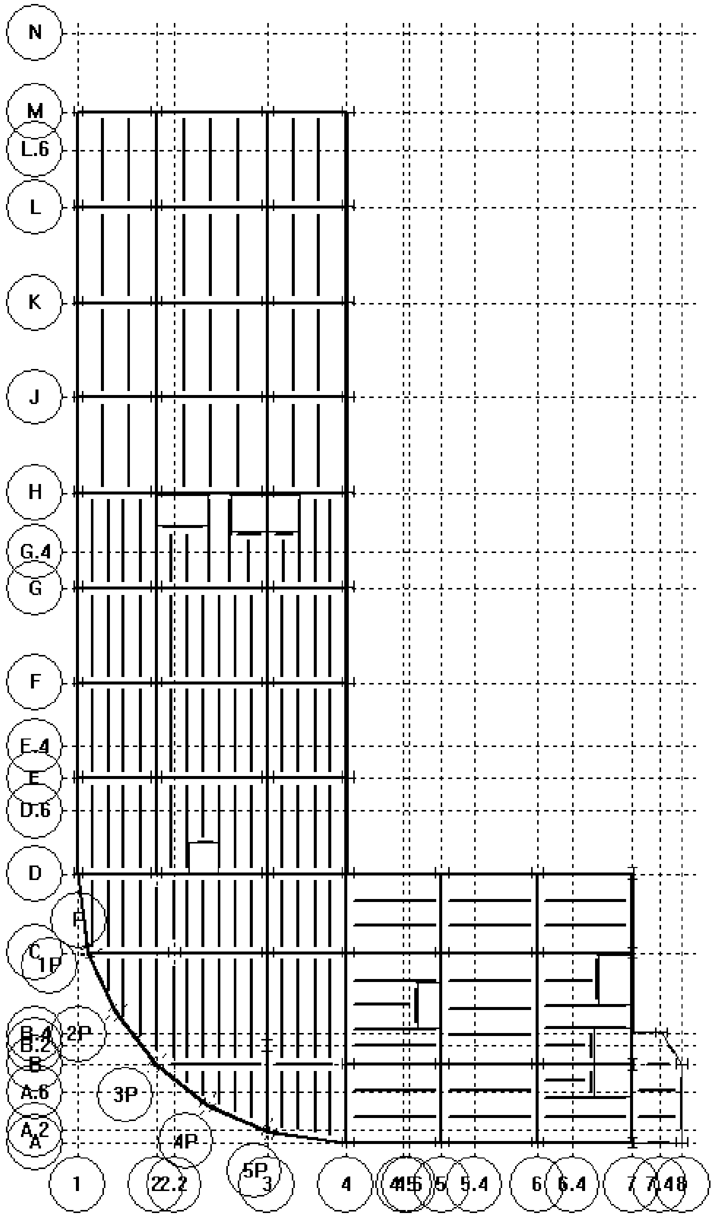
First Floor Plan N

Existing Floor Layout



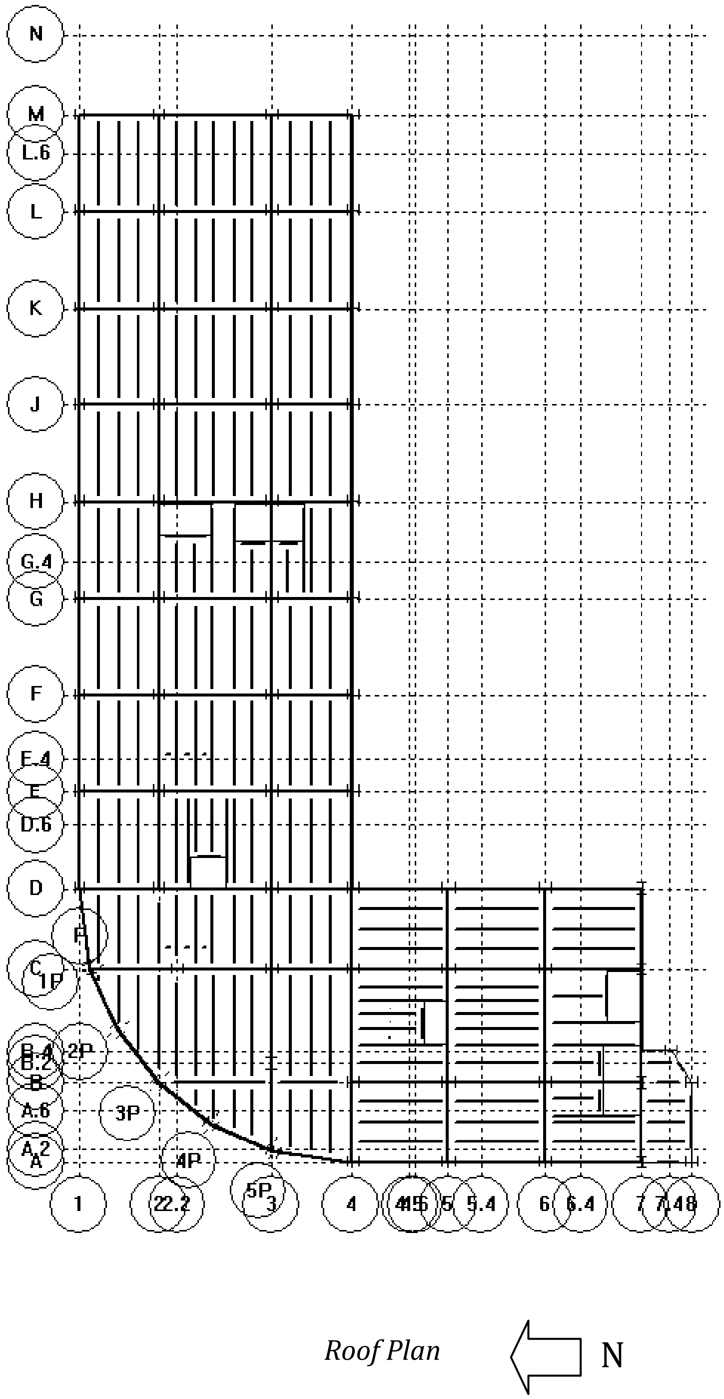
Second Floor Plan N

Existing Floor Layout



Third Floor Plan

Existing Floor Layout



Roof Plan

-End of Section-

APPENDIX B: WIND ANALYSIS

Photo courtesy of www.wbdg.org

Main Wind Force Resisting System

Building Information	
Number of Floors	4
Building Height (ft)	57.4
N-S Building Length (ft)	335
E-W Building Length (ft)	175
L/B in N-S Direction	1.91
L/B in E-W Direction	0.52

Building Location Factors	
Basic Wind Speed (V) mph	90
Exposure Category	B
Importance Factor (I)	1.15
Wind Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0

Variables to Obtain Gust Factor		
Variable	Wind Direction	
	N-S	E-W
n_1 (Hz)	0.869	0.869
Stiffness	Flexible	Flexible
B	335	175
L	175	335
h	57.4	57.4
g_q	3.4	3.4
g_v	3.4	3.4
g_r	4.16	4.16
z_{BAR}	34	34
ε_{BAR}	0.333	0.333
L_{BAR}	320	320
b_{BAR}	0.45	0.45
α_{BAR}	0.25	0.25
Iz_{BAR}	0.298	0.298
Lz_{BAR}	325	325
Q	0.765	0.814
Vz_{BAR}	60.0	60.0
N_1	4.7	4.7
n_h	3.82	3.82
n_B	22.32	11.66
n_L	39.03	74.71
R_h	0.227	0.227
R_B	0.044	0.082
R_L	0.025	0.013
R_n	0.0528	0.0528
R	0.0755	0.1028
G_f	**0.791**	**0.822**

Main Wind Force Resisting System

Floor Height (ft)	Level	Total Height (ft)	K_z	q_z	Wind Pressures (psf)					
					N-S Windward	N-S Leeward	N-S Side Wall	E-W Windward	E-W Leeward	E-W Side Wall
14.40	Roof	57.40	0.84	17.09	13.89	-9.83	-12.54	14.31	-7.54	-12.91
14.00	3	43.00	0.78	15.74	13.03	-9.83	-12.54	13.42	-7.54	-12.91
14.00	2	29.00	0.69	14.06	11.97	-9.83	-12.54	12.32	-7.54	-12.91
15.00	1	15.00	0.57	11.65	10.44	-9.83	-12.54	10.73	-7.54	-12.91

Distribution of Windward and Leeward Pressures

Level	Wind Design					
	Load (k)		Shear (k)		Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	57	28	0	0	3284	1580
3	111	53	57	28	4764	2287
2	105	50	168	81	3037	1450
1	102	48	273	131	1533	726
Total	375	179	375	179	12618	6043

Total Base Shear from Windward and Leeward Pressures

WIND DESIGN

USING ANALYTICAL PROCEDURE —

$V = 90$ MPH (FIGURE 6-1)

$K_d = 0.85$ (TABLE 6-4)

$I = 1.15$ (TABLE 6-1)

EXPOSURE B (REFERENCE § 6.5.6)

$K_{zt} = 1.0$ (FIGURE 6-4)

VELOCITY PRESSURE EXPOSURE COEFFICIENT, K_h (TABLE 6-3)

BUILDING HEIGHT = 57'5"

HEIGHT	K_h
50'	0.81
57.4'	0.84
60'	0.85

VELOCITY PRESSURE (q_p)

$$q_p = 0.00256 K_h K_{zt} K_d V^2 I$$

$$q_p = 0.00256(0.84)(1.0)(0.85)(90)^2(1.15) = 17.0 \text{ psf}$$

COMBINED NET PRESSURE COEFFICIENT (GC_{pn}) (REFERENCE § 6.5.12.2.4)

$GC_{pn} = 1.5$ (WINDWARD)

$GC_{pn} = -1.0$ (LEEWARD)

COMBINED NET DESIGN PRESSURE ON PARAPET, p_p

$$p_p = q_p GC_{pn}$$

$p_p = 17.0(1.5) = 25.5$ psf (WINDWARD)

$p_p = 17.0(-1.0) = -17.0$ psf (LEEWARD)

FORCES ON PARAPETS

HT. OF PARAPET = 4'-1½"

$F = 25.5 \text{ psf}(4.125') = 105.2 \text{ plf}$ (WINDWARD)

$F = +17.0 \text{ psf}(4.125') = 70.1 \text{ plf}$ (LEEWARD)

APPROXIMATE FUNDAMENTAL FREQUENCY, η_1 (REFERENCE C6.5.8 IN COMMENTARY OF ASCE 7-05)

FOR STEEL MOMENT RESISTING FRAMES,

$$\eta_1 = \frac{22.1}{H^{0.8}} = \frac{22.1}{(57.4)^{0.8}} = 0.869 \text{ Hz} < 1.0$$

∴ FLEXIBLE STRUCTURE

Hand Calculations from Preliminary Analysis

OBTAIN GUST EFFECT FACTOR —

$$g_a = g_v = 3.4$$

$$g_r = \sqrt{2\ln(3600\eta_1)} + \frac{0.577}{\sqrt{2\ln(3600\eta_1)}}$$

$$g_r = \sqrt{2\ln(3600(0.869))} + \frac{0.577}{\sqrt{2\ln(3600(0.869))}} = 4.156$$

$$\bar{z} = 0.6h = 0.6(57.4') = 34.4' \quad ; \quad z_{min} = 30'$$

$$34.4' > 30' \quad \therefore \text{OK}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{34.4}\right)^{1/6} = 0.298$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{34.4}{33}\right)^{1/3} = 324.5$$

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

$Q = 0.765$ FOR N-S $(B = 335')$

$Q = 0.814$ FOR E-W $(B = 175')$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{34.4}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = 60.0$$

$$N_1 = \frac{\eta_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.869(324.5)}{60.0} = 4.70$$

$$R_n = \frac{7.47N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(4.70)}{(1+10.3(4.7))^{5/3}} = 0.053$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{3.82} - \frac{1}{2(3.82)^2}(1-e^{-2(3.82)}) = 0.228$$

$$\eta = 4.6\eta_1 h/\bar{V}_{\bar{z}} = 4.6(0.869)(57.4)/60.0 = 3.82$$

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

$$\eta = 4.6\eta_1 B/\bar{V}_{\bar{z}} = 4.6(0.869)(335)/60 = 22.3 \quad [\text{N-S DIR.}]$$

$$= 4.6(0.869)(175)/60 = 11.7 \quad [\text{E-W DIR.}]$$

$$R_B = \frac{1}{22.3} - \frac{1}{2(22.3)^2}(1-e^{-2(22.3)}) = 0.044 \quad [\text{N-S DIR.}]$$

$$R_B = \frac{1}{11.7} - \frac{1}{2(11.7)^2}(1-e^{-2(11.7)}) = 0.082 \quad [\text{E-W DIR.}]$$

Hand Calculations from Preliminary Analysis

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

$$\eta = 15.4 n_1 L/\bar{V}_{\bar{z}} = 15.4(0.869)(175)/60 = 39.0 \quad [N\text{-}S]$$
$$= 15.4(0.869)(335)/60 = 74.7 \quad [E\text{-}W]$$

$$R_L = \frac{1}{39} - \frac{1}{2(39)^2}\left(1-e^{-2(39)}\right) = 0.025 \quad [N\text{-}S]$$

$$R_L = \frac{1}{74.7} - \frac{1}{2(74.7)}\left(1-e^{-2(74.7)}\right) = 0.013 \quad [E\text{-}W]$$

$$R = \sqrt{\frac{1}{\beta}(R_n R_h R_B)(0.53+0.47R_L)} \quad \text{WHERE } \beta = 0.05$$

$$R = \sqrt{\frac{1}{0.05}\left[(0.053)(0.228)(0.044)(0.53+0.47(0.025))\right]} = 0.076 \quad [N\text{-}S]$$

$$R = \sqrt{\frac{1}{0.05}\left[(0.053)(0.228)(0.082)(0.53+0.47(0.013))\right]} = 0.103 \quad [E\text{-}W]$$

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

$$G_f = 0.925\left(\frac{1+1.7(0.298)\sqrt{3.4^2(0.765)^2+4.156^2(0.076)^2}}{1+1.7(3.4)(0.298)}\right) = 0.791 \quad [N\text{-}S]$$

$$G_f = 0.925\left(\frac{1+1.7(0.298)\sqrt{3.4^2(0.814)^2+4.156^2(0.103)^2}}{1+1.7(3.4)(0.298)}\right) = 0.822 \quad [E\text{-}W]$$

VELOCITY PRESSURE, q_z

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$
$$q_z = 0.00256(0.84)(1.0)(0.85)(90)^2(1.15) = 17.0 \text{ psf}$$

FOR WIND IN E-W DIRECTION (REFERENCE FIGURE 6-6)

WINDWARD WALL – $C_p = 0.8$

LEEWARD WALL – $L/B = 335/175 = 1.914$

$C_p = -0.318$ (BY INTERPOLATION)

SIDE WALL – $C_p = -0.7$

Hand Calculations from Preliminary Analysis

FOR WIND IN N-S DIRECTION (REFERENCE FIGURE 6-6)

WINDWARD WALL — $C_p = 0.8$

LEEWARD WALL — $L/B = 175/335 = 0.522 \quad \therefore C_p = -0.5$

SIDE WALL — $C_p = -0.7$

NOT INCLUDING UPLIFT ON ROOF SINCE ROOF FRAMING MADE UP OF W-SHAPES

$q_i = q_h = q_z$ FOR TOP OF BLDG $= 17.0\,psf$

INTERNAL PRESSURE COEFFICIENT

$GC_{pi} = \pm 0.18$

DESIGN WIND PRESSURES — $p_z + p_h$ (EQ. 6-17)

WINDWARD WALLS:

$$p_z = q_z G C_p - q_h (GC_{pi})$$
$$p_z = (0.791)(0.8) q_z \pm 17.0(0.18) = (0.633 q_z \pm 3.06)\,psf \quad [N\text{-}S]$$
$$p_z = (0.822)(0.8) q_z \pm 17.0(0.18) = (0.658 q_z \pm 3.06)\,psf \quad [E\text{-}W]$$

LEEWARD WALLS + SIDE WALLS:

$$p_z = q_h G C_p - q_h (GC_{pi})$$
$$p_z = (17.0)(0.791) C_p \pm 17.0(0.18) = (13.4 C_p \pm 3.06)\,psf \quad [N\text{-}S]$$
$$p_z = (17.0)(0.822) C_p \pm 17.0(0.18) = (14.0 C_p \pm 3.06)\,psf \quad [E\text{-}W]$$

Wind Hand Calculations vs. RAM Output

East-West Direction

Story	Wind E-W Story Loads (k)		
	Hand Calculations	RAM Output	% Difference
Roof	28	38.85	27.9
3	53	43.96	20.6
2	50	41.91	19.3
1	48	51.98	7.7
Total Base Shear (k)	179	177	1.3
Overturning Moment (ft-k)	12,618	10,588	19.2

APPLIED STORY FORCES

Type: Wind_IBC03_1_Y

Level	Ht	Fx	Fy
	ft	kips	kips
H. PENT-H	74.04	0.00	0.00
H. PENT-L	73.37	0.00	0.00
ROOF	59.92	0.00	38.85
3RD	45.50	0.00	43.96
2ND	31.50	0.00	35.58
BRIDGE PLAT	26.50	0.00	6.33
COOLING TOWER	21.00	---	---
STUBS	20.04	---	---
1ST	17.50	0.00	34.54
LOW 1ST	16.50	0.00	3.06
MAIN	2.50	0.00	14.38
H COL BASES	1.67	---	---
		0.00	176.71

Wind Hand Calculations vs. RAM Output

North-South Direction

Story	Wind N-S Story Loads (k)		
	Hand Calculations	RAM Output	% Difference
Roof	57	59.30	3.9
3	111	80.89	37.2
2	105	83.36	26.0
1	102	97.72	4.4
Total Base Shear (k)	375	321	16.8
Overturning Moment (ft-k)	12,618		

APPLIED STORY FORCES

Type: Wind_IBC03_1_X

Level	Ht ft	Fx kips	Fy kips
H. PENT-H	74.04	0.00	0.00
H. PENT-L	73.37	0.00	0.00
ROOF	59.92	59.30	0.00
3RD	45.50	80.89	0.00
2ND	31.50	74.83	0.00
BRIDGE PLAT	26.50	8.53	0.00
COOLING TOWER	21.00	---	---
STUBS	20.04	---	---
1ST	17.50	73.31	0.00
LOW 1ST	16.50	4.95	0.00
MAIN	2.50	19.46	0.00
H COL BASES	1.67	---	---
		321.27	0.00

-End of Section-

APPENDIX B: SEISMIC ANALYSIS

Photo courtesy of
www.science.howstuffworks.com

Seismic Force Resisting System

Occupancy Category	IV
Importance Factor (I)	1.5
S_s	0.170
S_1	0.056
Site Class	E
Total Building Height (ft)	57.4
T_a	0.715
T_L	12
Frequency (Hz)	1.40
Structural Behavior	Rigid Diaphragm
Total Weight (k)	12043

S_{ms}	0.425
S_{m1}	0.196
S_{ds}	0.283
S_{d1}	0.131
SDC	B
R	3.0
C_s	0.091
k	1.11
Base Shear (k)	1100

Base Shear and Overturning Moment Distribution

Story	h_x (ft)	Story Weight (k)	$h_x^k W_x$	C_{vx}	$F_x = C_{vx}V$	V_x (k)	M_x (ft-k)
Roof	57.4	1132	100432	0.219	241	241	13817
3	43	2824	181955	0.396	436	677	29103
2	29	2751	114571	0.250	275	951	27591
1	15	3100	62203	0.135	149	1100	16507
Main	0	2236	0	0.000	0	1100	0
Total	57.4	12043	459162	1.000	**1100**		**87017**
Base Shear =	1100	k					

Seismic Force Resisting System: Floor Weights

Floor 2					
	Approx. Area =		25,120	SF	
	Floor to Floor Ht. =		14	ft	
Walls:			**Superimposed:**		
Perimeter =	755	ft	Partitions =	20	psf
Height =	14	ft	MEP =	10	psf
Unit Wt. =	20	psf	Finishes =	5	psf
Weight=	**211**	**k**	**Weight=**	**879**	**k**
Slab:					
		Thk. =	4.5	in	
		Unit Wt. =	150	pcf	
		Weight=	**1413**	**k**	
Columns:					
Shape	Quantity	Weight (lb/ft)	Column Height (ft)	Total Weight (k)	
W10x112	7	112	14	10.98	
W12x40	3	40	14	1.68	
W12x96	1	96	14	1.34	
W12x120	5	120	14	8.40	
W12x136	3	136	14	5.71	
W12x152	3	152	14	6.38	
W12x170	18	170	14	42.84	
W12x210	6	210	14	17.64	
			Weight =	**95**	**k**
Beams:					
Shape	Quantity	Weight (lb/ft)	Beam Length (ft)	Total Weight (k)	
W12x22	3	22	25	1.65	
W14x22	2	22	19.5	0.86	
W16x26	12	26	25	7.80	
W16x26	44	26	30	34.32	
W18x40	2	40	25	2.00	
W18x40	1	40	30	1.20	
W24x55	14	55	25	19.25	
W24x55	5	55	30	8.25	
W24x68	6	68	25	10.20	
W24x68	26	68	30	53.04	
W24x84	3	84	25	6.30	
W24x84	3	84	30	7.56	
			Weight =	**152**	**k**
	2nd Floor Weight =	**2751**	**k OR**	**110**	**psf**

This table is provided to show the method used for determining floor weights. All tables are available upon request.

SEISMIC DESIGN

OCCUPANCY CATEGORY : III

IMPORTANCE FACTOR : 1.5 ⟵ VALUE USED BY DESIGN ENGINEER

SITE CLASS : E

S_S : 0.170
S_1 : 0.056 } USING APPLET W/ LONGITUDINAL + LATITUDE COORDINATES OF SITE

R : 3.0

h_n : 57.4 ft

T_L : 12 [FIG. 22-15 ASCE 7-05]

C_t : 0.028
x : 0.8 } FOR STEEL MOMENT FRAMES - TABLE 12.8-2

USING TABLE 11.4-1 (ASCE 7-05)

$S_{MS} = F_a S_S = (2.5)(0.170) = 0.425$

USING TABLE 11.4-2 (ASCE 7-05)

$S_{M1} = F_v S_1 = (3.5)(0.056) = 0.196$

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.425) = 0.283$
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.196) = 0.131$ } USING TABLE 11.6-1 + 11.6-2 (ASCE 7-05) SDC = B

RESPONSE MODIFICATION FACTOR = R = 3.0

$T_a = C_t h_n^x = 0.028\,(57.4)^{0.8} = 0.715$

$T_S = S_{D1}/S_{DS} = 0.131/0.283 = 0.463$

$0.8T_S = 0.8(0.463) = 0.370$

$0.8T_S < T_a$ ∴ TABLE 11.6-1,2 GIVES VALUES FOR C_t + x ✓

$T_L = 12$ [FIG 22-15 ASCE 7-05]

$$C_S = \begin{cases} \dfrac{S_{DS}}{(R/I)} = \dfrac{0.283}{(3/1.5)} = 0.1415 \\[2ex] \dfrac{S_{D1}}{(T\cdot R/I)} = \dfrac{0.131}{(0.715(3/1.5))} = 0.0916 \quad \geq 0.01 \\[2ex] \dfrac{S_{D1}\cdot T_L}{(T^2\cdot R/I)} = \dfrac{0.131\,(12)}{(0.715^2(3/1.5))} = 1.537 \end{cases} \quad \text{MIN}$$

$C_S = 0.0916 \approx 0.092$

$f = 1/T = 1/0.715 = 1.40 > 1.0$ ∴ RIGID DIAPHRAGM

Hand Calculations from Preliminary Analysis

SEE EXCEL SPREADSHEET FOR FLOOR WEIGHTS

MAIN FLOOR: 47,410 SF 47.2 psf

1ST FLOOR: 25,120 SF 123 psf

2ND FLOOR: 25,120 SF 110 psf

3RD FLOOR: 25,120 SF 112 psf

ROOF: 25,120 SF 45 psf

W_T = TOTAL BLDG. WT.

$$W_T = 47{,}410(47.2) + 25{,}120(123) + 25{,}120(110) + 25{,}120(112) + 25{,}120(45)$$

$$W_T = 12{,}043{,}000 \text{ lbs} = 12{,}043^k$$

$$V = C_s W_T$$

$$V = 0.092(12043^k) = 1100^k$$

* NOTE: BASE SHEAR VALUE IS HIGH DUE TO BEING IN SITE CLASS E AND USING AN IMPORTANCE FACTOR OF 1.5

Seismic Hand Calculations vs. RAM Output

East-West Direction

Story	Seismic E-W Story Loads (k)		
	Hand Calculations	RAM Output	% Difference
Roof	241	231.85	3.9
3	436	395.18	10.3
2	275	259.65	5.9
1	149	157.31	5.3
Total Base Shear (k)	1100	1044	5.4
Overturning Moment (ft-k)	87,017		

APPLIED STORY FORCES

Type: EQ_IBC03_Y_+E_F

Level	Ht ft	Fx kips	Fy kips
H. PENT-H	74.04	0.00	0.00
H. PENT-L	73.37	0.00	0.00
ROOF	59.92	0.00	231.85
3RD	45.50	0.00	395.18
2ND	31.50	0.00	259.65
BRIDGE PLAT	26.50	0.00	0.00
COOLING TOWER	21.00	---	---
STUBS	20.04	---	---

Loads and Applied Forces

RAM Frame v11.2
DataBase: St. V

Level	Ht ft	Fx kips	Fy kips
1ST	17.50	0.00	152.66
LOW 1ST	16.50	0.00	0.00
MAIN	2.50	0.00	4.65
H COL BASES	1.67	---	---
		0.00	1044.00

Seismic Hand Calculations vs. RAM Output

North-South Direction

Story	Seismic N-S Story Loads (k)		
	Hand Calculations	RAM Output	% Difference
Roof	241	231.85	3.9
3	436	395.18	10.3
2	275	259.65	5.9
1	149	157.31	5.3
Total Base Shear (k)	1100	1044	5.4
Overturning Moment (ft-k)	87,017		

APPLIED STORY FORCES

Type: EQ_IBC03_X_+E_F

Level	Ht ft	Fx kips	Fy kips
H. PENT-H	74.04	0.00	0.00
H. PENT-L	73.37	0.00	0.00
ROOF	59.92	231.85	0.00
3RD	45.50	395.18	0.00
2ND	31.50	259.65	0.00
BRIDGE PLAT	26.50	0.00	0.00
COOLING TOWER	21.00	---	---
STUBS	20.04	---	---

Loads and Applied Forces

RAM Frame v11.2
DataBase: St. V

Level	Ht ft	Fx kips	Fy kips
1ST	17.50	152.66	0.00
LOW 1ST	16.50	0.00	0.00
MAIN	2.50	4.65	0.00
H COL BASES	1.67	---	---
		$\boxed{1044.00}$	0.00

-End of Section-