Kaleida Health – Global Heart and Vascular Institute
University at Buffalo – CTRC/Incubator

Buffalo, New York

Technical Report #3

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November 29, 2010

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Technical Report #3

Executive Summary

The following document is the third technical report of senior thesis and includes information regarding the structural lateral system of the Kaleida Health and University at Buffalo, Global Heart and Vascular Institute. This project will be referred to throughout this report simply as GHVI. This report includes a structural system overview, a summary of building loads, and an in-depth lateral system analysis.

GHVI is a ten story medical facility in the city of Buffalo, NY. The building is square in shape with a length and width of 221 feet, and a height of 185 feet. The foundation is made of grade beams and steel helical piles that are driven 82 to 87 feet deep. Floor construction entails composite metal deck resting on steel superstructure. A standard bay size of 31'-6" by 31'-6" is used throughout the building, utilizing W14 columns of varying weight to make up the gravity system. The lateral system is comprised of braced frames which are located near the perimeter of the building.

In order to analyze the lateral system of GHVI a three dimensional computer model was created using ETABS. The steel columns and braces of each frame were modeled and assigned material and frame section properties. Columns were pinned at the base and beams and braces were released to prevent them from taking out-of-plane bending. A rigid diaphragm was drawn with additional area masses to model each floor level.

The ETABS model was used to determine the relative stiffness of each frame. A 100 kip load was applied to the top of each individual frame, and the lateral displacement was measured. From these values the stiffness and relative stiffness of the frames was calculated. With the relative stiffness it was then possible to distribute the lateral load to the building. After confirming the location of the centers of rigidity, mass, and pressure, it was determined that both direct and torsional shear must be considered for this building.

Seven basic load combinations were taken from ASCE 7-10 and input into the ETABS model to determine the controlling load case for this building. Although unexpected, seismic loading was shown to control in both the North-South and East-West directions. With this controlling load combination determined, it was then possible to examine drift, consider overturning moment and foundation impact, and perform spot checks of different lateral members. In the end all story and total drifts were found to be acceptable, the foundation is sufficient, and the inspected lateral members were adequate.

Introduction

GHVI is a state-of-the-art medical facility and a fundamental component in a joint undertaking between Kaleida Health Systems and the University at Buffalo School of Medicine. The building spans ten levels and includes exam rooms, classrooms, offices, a café, a wellness center and library, and a research facility. It is intended to bring patients, surgeons, and researchers together to collaborate in an unprecedented way.

Key themes considered throughout the design were collaboration, flexibility, and comfort. Kaleida Health Systems sought a structure that would link clinical and research work and combine all vascular disciplines. A spirit of collaboration was the driving force behind bringing both Kaleida and the University at Buffalo together in a single structure. Keeping this in mind, the design team developed the facility with a “collaborative core” which enables interaction among those working within the facility. This collaborative learning environment brings together research, ideas, and solutions and results in better patient care.

A universal grid design increases the flexibility of space and achieves measurable advantage in initial capital cost, speed to market, operating economy, and future adaptability. The universal grid is comprised of three 10’-6” building modules and forms a 31’-6” x 31’-6” structural grid capable of integrating the building’s diverse functions. When combined with an 18’ floor-to-floor height, the flexible grid creates an open plan capable of adapting to present and future healthcare needs. The building will be able to incorporate unknown, but rapidly changing technological developments within the industry, also giving it longevity through its adaptability.

With comfort in mind, a separate “hotel” level was designed on the second floor and separated from the procedural floors. Functionally, the “hotel” is comprised of private patient rooms and a small lounge area. Other family lounges are also provided and the perimeter of the building is shaped to bring in as much natural daylight as possible. The vision of GHVI is to create an atmosphere that is more than a simple hospital, but instead a facility for world-class treatment and state-of-the-art technology.

Structural System Overview

Foundation

Based on the recommendations of the October 2008 Geotechnical Report by Empire Geo-Services, Inc., the foundation of GHVI consists of grade beams and pile caps placed on top of steel helical piles.

The helical piles are HP12x74 sections with an allowable axial capacity of 342 kips (171 tons) which are driven to absolute refusal on limestone bedrock 82 to 87 feet below the sub-basement finish level. Grade beams and pile caps have a concrete strength of 4000 psi, and it should be noted that the width of the grade beams equals that of the pile caps at the foundations of the braced frames. The grade beams provide resistance to lateral column base movement, and the pile caps link the steel helical piles and the structural steel columns of the superstructure.

Spanning the grade beams is the sub-basement floor, a 5" slab-on-grade. Due to the slope of the site, part of this sub-basement is below grade, and therefore a one foot thick foundation wall slopes along the west elevation of the sub-basement.

Floor System

The floors of GHVI consist of 3" composite metal deck with a total slab thickness ranging from 4" to 7½". The metal deck is 18-gage galvanized steel sheets resting on various different beam and girder sizes. These sizes change throughout the structure because of the various functions of the spaces. The bay sizes through the building are mostly 31'-6" by 31'-6", with beams spaced at 10'-6". As was discussed in the introduction, this universal grid design increases the future flexibility of the space. A slight variation in the floor can be seen on Levels 6-8. On these levels, part of the floor structure is left open to provide for the collaborative atrium that was designed to bring the various disciplines together.

Gravity System

Steel columns are used throughout the building to transmit the gravity load to the foundation. All of the columns in the building are W14s, but they range in weight from 68 lb/ft to 370 lb/ft, and they are typically spliced every 36 feet. These columns provide an 18' floor-to-floor height, which also contributes to the universal grid and future flexibility of the space.

Lateral System

The lateral system of GHVI utilizes braced frames located near the perimeter of the building, all of which are HSS sections. A braced frame system is ideal in steel buildings because of its low cost compared to moment connection frames. There are moment connections in some parts of this structure, but they are used to support the small amount of slab overhang that is cantilevered. These moment connections may actually add some stiffness to the lateral system, but they cannot be included in the lateral system design. Figure A depicts the location of the braced frames on the outer part of the structure, and an elevation of each frame is shown on the following two pages.

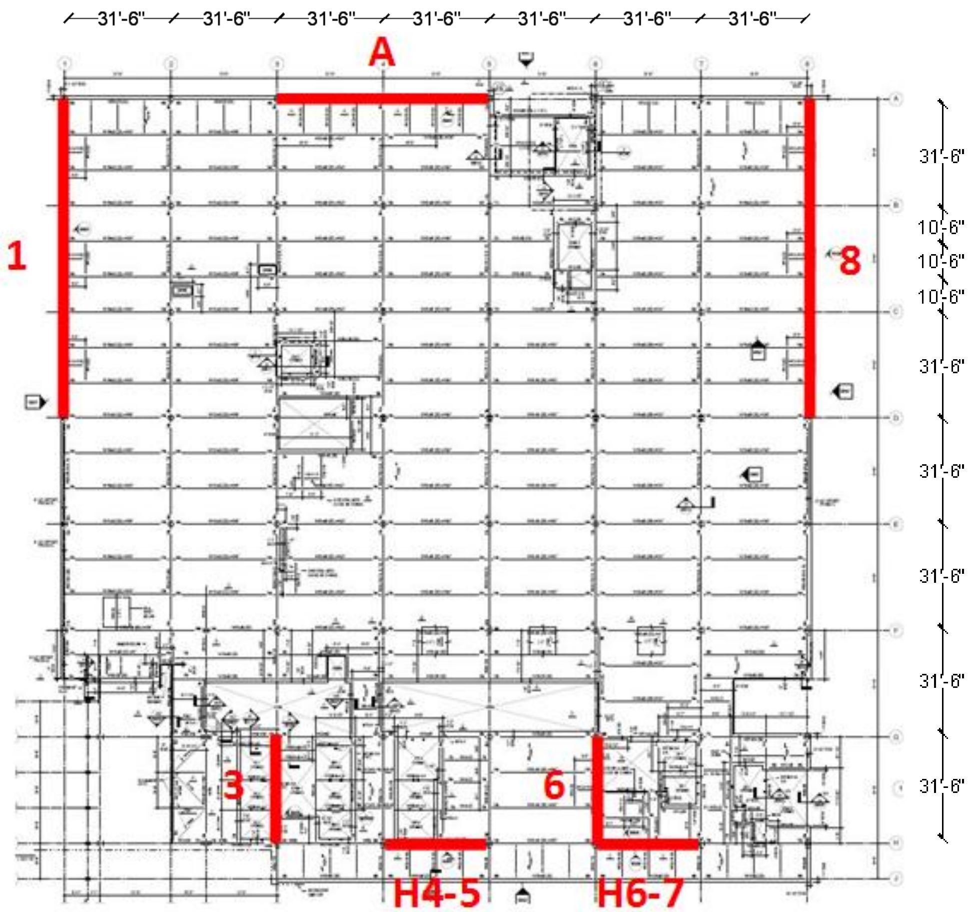
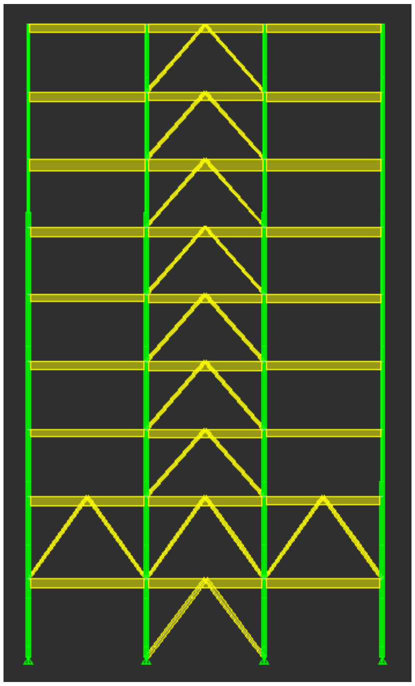
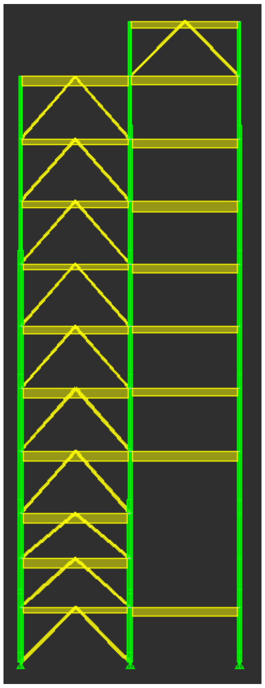


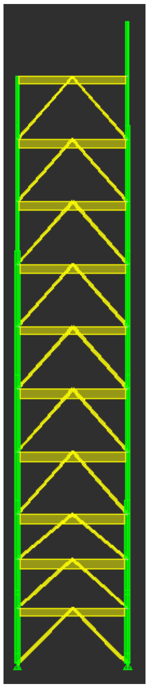
Figure A – Level Two Framing Plan with Braced Frames Highlighted (Cannon Design)



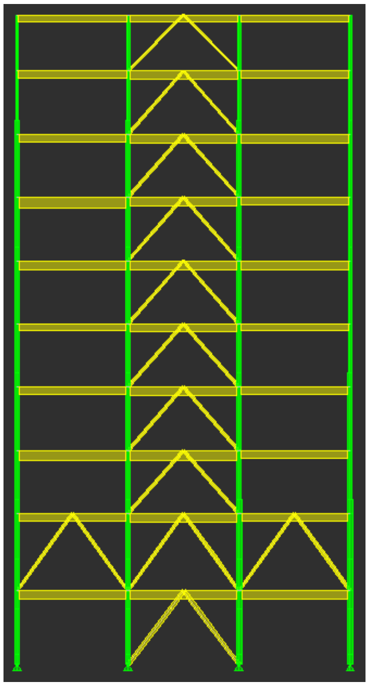
Frame 1



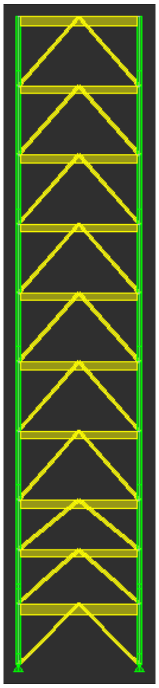
Frame 3



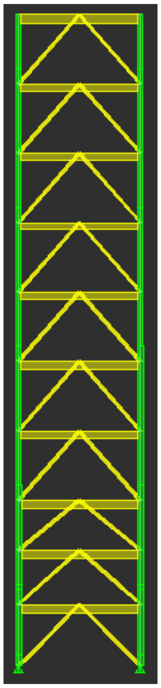
Frame 6



Frame 8



Frame H4-5



Frame H6-7

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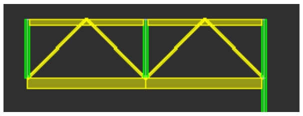
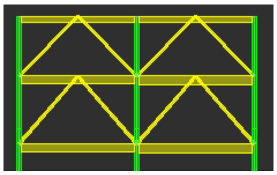
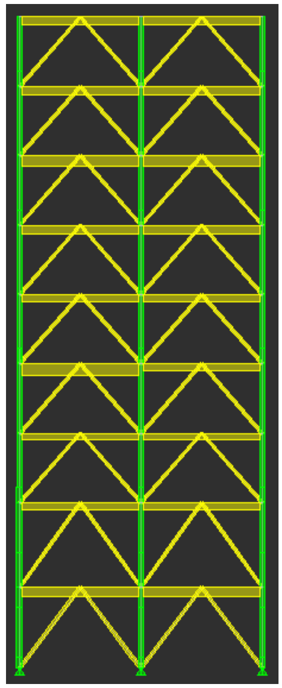


Frame G



Frame C



Frame A

Codes and References

Original Design Codes

- Model Building Code:
  Building Code of New York State 2007

- Design Codes:
  "Load and Resistance Factor Design Specification for Structural Steel Buildings," AISC

  "Code of Standard Practice for Steel Buildings and Bridges", AISC

  "Manual of Steel Construction - Load and Resistance Factor Design," AISC

  ACI 318-05, Building Code Requirements for Structural Concrete

  American Society of Civil Engineers, ASCE/SEI 7-02,
  Minimum Design Loads for Buildings and Other Structures

Thesis Design Codes

- National Model Building Code:
  2009 International Building Code

- Design Codes:
  Steel Construction Manual 13th edition, AISC

  ACI 318-05, Building Code Requirements for Structural Concrete

  PCI Design Handbook, 6th Edition

  RSMeans Building Construction Cost Data 2011 Book

  American Society of Civil Engineers, ASCE/SEI 7-10,
  Minimum Design Loads for Buildings and Other Structures

- Deflection Criteria:
  Allowable Building Drift (Wind) = H/400

  Allowable Story Drift (Seismic) = $0.010h_{sx}$

Materials

Structure Steel:

Type	Standard	Grade
Wide Flange Shapes, WT's	ASTM A-992	
Channels & Angles	ASTM A-36	
Pipe	ASTM A-53	Grade B
Hollow Structural Sections (Rectangular & Round)	ASTM A-500	Grade B
Base Plates	ASTM A-572	Grade 42
All Other Steel Members	ASTM A-36	

Concrete:

Type	f'c (psi)	Unit Weight (pcf)
Pile Caps	4000	150
Grade Beams	4000	150
All Other Concrete	4000	150
Slabs-On-Grade	3000	150
Foundation Walls	4000	150

Reinforcing:

Type	Standard	Grade
Typical Bars	ASTM A-615	60
Welded Bars	ASTM A-706	60
Welded Wire Fabric	ASTM A-185	
Steel Fibers	ASTM A-820	Type 1
Bars Noted To Be Field Bent	ASTM A-615	40

Connectors:

Type	Standard
High Strength Bolts, Nuts, & Washers	ASTM A-325 or A-490 (min. 3/4 Diameter)
Anchor Rods	ASTM F1554
Welding Electrode	E70XX
Steel Deck Welding Electrode	E60XX min.

Building Loads

Design Floor Dead Loads

The dead loads shown below are a combination of information obtained from Cannon Design and values determined from ASCE 7-10.

Typical Floor

Steel Deck and 7.5" Slab	75.0 psf
Steel Beams	12.0 psf
Total	87.0 psf

Typical Roof

3' Steel Deck	4.5 psf
Adhered Membrane	2.0 psf
4" Rigid Insulation	6.0 psf
1/2" Protection Board	2.0 psf
Total	14.5 psf

Electrical and Mechanical Areas

Steel Deck and 7.5" Slab	75.0 psf
Steel Beams	12.0 psf
Concrete Pad	25.0 psf
Total	112.0 psf

Vivarium (Level 7)

Steel Deck and 7.5" Slab	75.0 psf
Membrane and 6" LTWT Topping	65.0 psf
Steel Beams	12.0 psf
Masonry Partitions	73.0 psf
Total	225.0 psf

Superimposed Dead Load

MEP	15.0 psf
Ceiling	5.0 psf
Leveling Concrete for Deflection	5.0 psf
Total	25.0 psf

Exterior Curtain Wall – 15.0 psf

Partitions – 10.0 psf

Floor Live Loads

The live loads shown below are a combination of information obtained from Cannon Design and values determined from ASCE 7-10.

Occupancy or Use	Design (psf)	ASCE 7-10 (psf)
Vivarium	80	60
Hotel (Patient) Floor	125	40
Procedure and Lab Floors	125	60
Mechanical Floors	150	--
Mechanical Floors with Catwalks below	175	--
Electrical Floors	200	--
Mechanical Mezzanine (Low)	40	40
Storage	--	20
Lobby	--	100
Stairs	--	100
Corrridors	--	100
Roof	--	20

It should be noted that there is a large difference between the live loads used by Cannon Design and the live loads referenced from ASCE 7-10. This difference can most likely be attributed to the fact that the building was designed to adapt to the ever changing needs of the healthcare industry. By over-designing the floors, it can be assured that they can be used for a variety of functions in the future without the need for redesign and renovation. Where there is a discrepancy the design load was used.

Wind Loads

The wind loads for GHVI were analyzed in Technical Report 1 using Chapters 26 and 27 of ASCE 7-10, and revisions were made for this report due to corrections in story height and internal pressure. Wind loads for the Main Wind-Force Resisting System were determined using the directional procedure for buildings of all heights. Based on an occupancy category of IV, a basic wind speed of 120 mph was used to find the windward and leeward pressures. By code, flexible buildings can be affected by wind gusts and have the potential for resonance response. Because this building is considered flexible, a gust-effect factor also had to be determined. Detailed calculations including the initial parameters, an effective length check, gust-effect factor calculations, wind pressure coefficients, and the calculated wind pressures can be found in Appendix B.

Wind Story Forces							
		Load (kips)		Shear (kips)		Moment (ft-kips)	
Level	**Height (ft)**	**N-S**	**E-W**	**N-S**	**E-W**	**N-S**	**E-W**
Roof	185	57.3	80.4	0.0	0.0	10599.02	14872.28
9	169	146.3	169.4	57.3	80.4	24730.99	28634.67
8	151	176.4	176.4	203.6	249.8	26634.72	26634.72
7	133	172.8	172.8	380.0	426.2	22985.1	22985.1
6	115	168.8	168.8	552.8	599.0	19415.03	19415.03
5	97	164.5	164.5	721.7	767.9	15956.65	15956.65
4	79	159.7	159.7	886.2	932.4	12616.63	12616.63
3	61	153.7	153.7	1045.9	1092.1	9375.787	9375.787
2	43	126.2	126.2	1199.6	1245.8	5427.234	5427.234
1	30	85.0	85.0	1325.8	1372.0	2550.022	2550.022
Mechanical	21	51.9	51.9	1410.8	1457.0	1089.248	1089.248
Basement	16	72.8	72.8	1462.7	1508.9	1164.818	1164.818
	Total	1535.5	1581.7	1535.5	1581.7	152545.3	160722.2

Table 1 – Wind loads, shears, and moments calculated for each story

From Table 1 it can be seen that there is a base shear of 1535.5 kips in the North-South direction and 1581.7 kips in the East-West direction. This is expected, due to the fact that the area of wind projection decreases slightly at the roof level in the North-South direction. As was discussed in Technical Report 1, these base shears are larger than the values determined by the design engineer using ASCE 7-02. It is probable that this large difference can be attributed to a difference in the two codes. In ASCE 7-02, the basic wind speed for Buffalo, NY is 90 mph, whereas in ASCE 7-10, the basic wind speed is 120 mph.

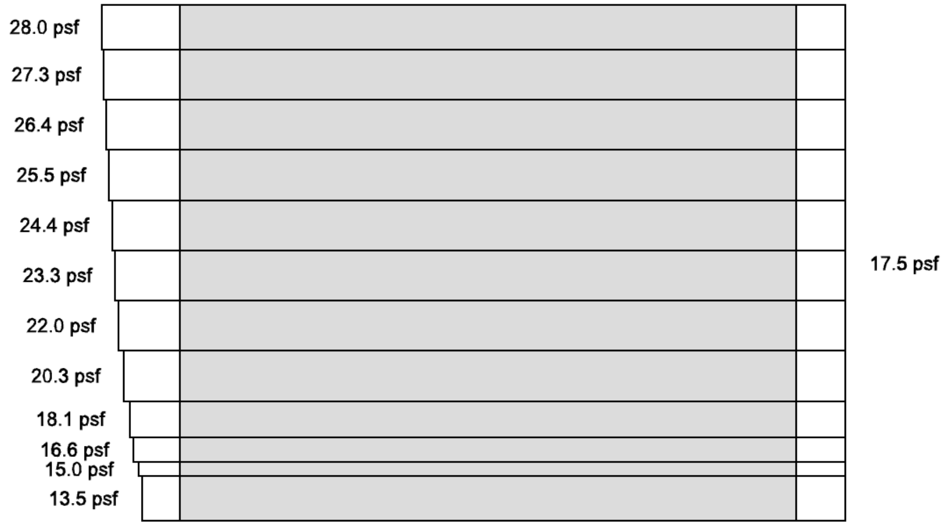


Figure B – Wind pressure diagram for East-West direction

Figure B shows the wind pressure diagram for the East-West direction. The windward loads are on the left, and the leeward loads are on the right. Figure C shows the wind force diagram and the base shear the building experiences.

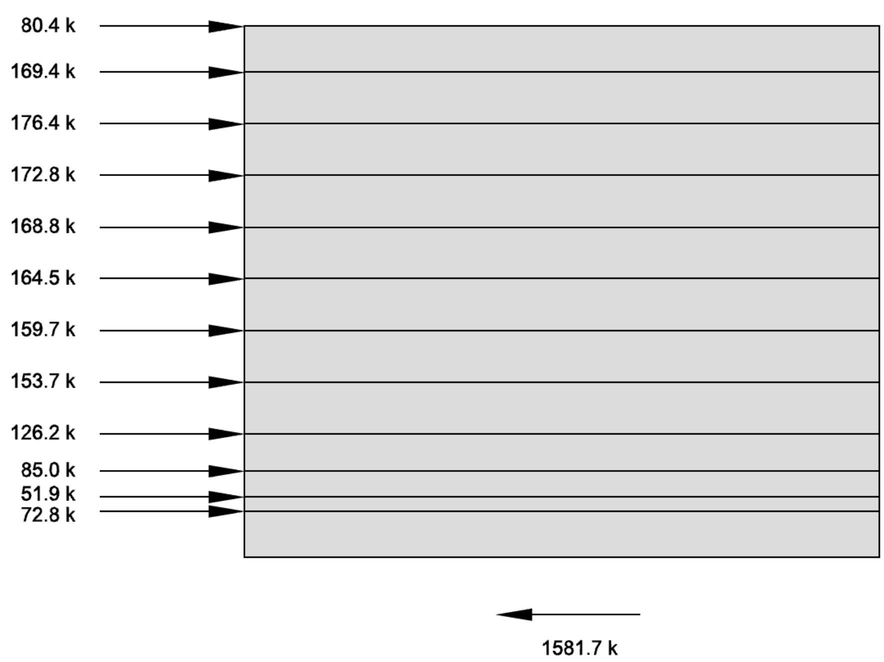


Figure C – Wind force diagram for East-West direction

Seismic Loads

Seismic analysis for GHVI was done in Technical Report 1, with reference to Chapters 11 and 12 in ASCE 7-10. Minor changes were made for this report, however the building was still assumed to be square for simplicity. The first step in this analysis was the estimated summation of the entire building weight above grade, which included the beams, columns, composite slab, exterior walls, superimposed dead load, and partitions of each level. An Excel spreadsheet was set up to go through the building floor-by-floor and estimate as precisely as possible the building weight. The estimated building weight was found to be 52636 kips. The Equivalent Lateral Force Procedure was then used to determine the base shear and this base shear was then distributed to the diaphragm of each level as seen in Table 2. A more detailed set of calculations can be found in Appendix C.

Level	h_i (ft)	h (ft)	w (k)	$w*h^k$	C_{VX}	f_i (k)	V_i (k)	M_i (ft-k)
Roof	16	185	1056	4457134	0.049	64	64	11929
9	18	169	4089	14929393	0.164	216	280	36501
8	18	151	6354	19373949	0.213	280	561	42322
7	18	133	6437	16021249	0.176	232	793	30826
6	18	115	6395	12614520	0.139	182	975	20987
5	18	97	6167	9266678	0.102	134	1109	13004
4	18	79	6202	6711654	0.074	97	1206	7671
3	18	61	6433	4604533	0.051	67	1273	4063
2	13	43	6067	2482343	0.027	36	1309	1544
1	9	30	958	220336	0.002	3	1312	96
Mechanical	5	21	1652	214933	0.002	3	1315	65
Basement	16	16	826	69572	0.001	1	1316	16
		Σ =	52635.7	90966294	1.000	1316		169025

Table 2 – Seismic Design Loads

Table 2 shows a total base shear of 1316 kips, and an overturning moment of 169025 foot-kips. The design engineers calculated a base shear of 1030 kips using ASCE 7-02. The difference in these two numbers can be attributed to the fact that the design engineers used a smaller total building weight, but more importantly because the response modification factor for steel ordinary concentrically braced frames was 5 in ASCE 7-02, and is now 3.25 in ASCE 7-10. A lower R value results in a more conservative number, hence a higher base shear.

Snow Loads

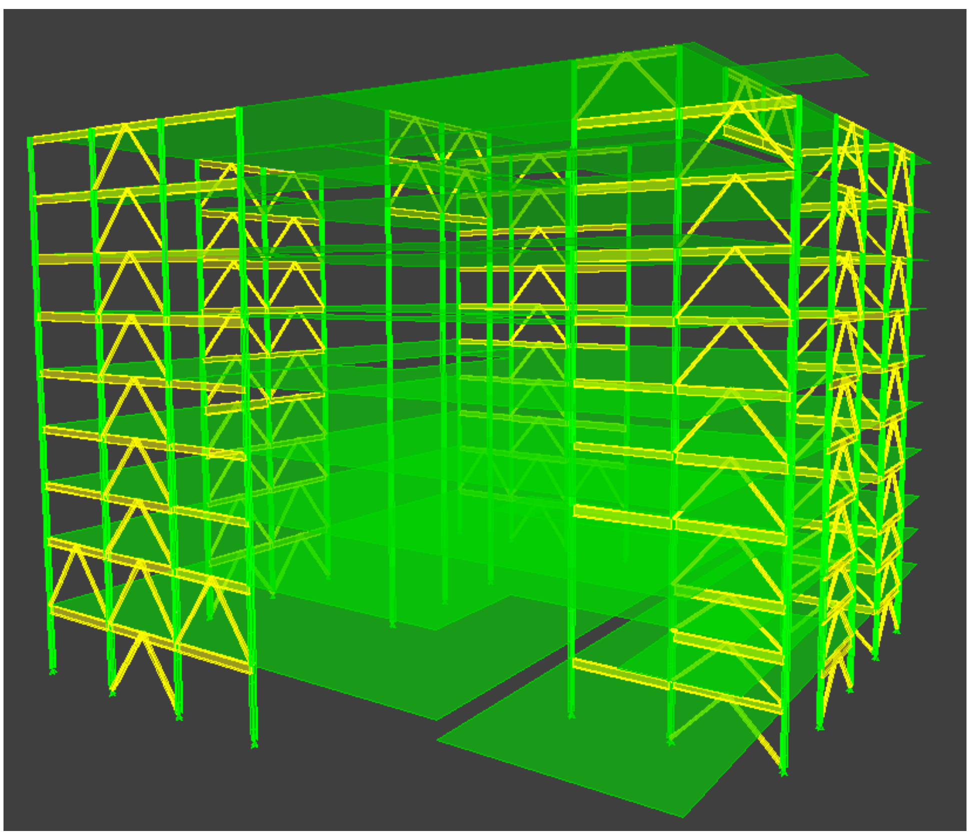
Snow loading for GHVI was calculated based on Chapter 7 in ASCE 7-10. A ground snow load of 50 psf was determined from a site-specific case study provided by Cannon Design. The exposure factor, thermal factor, and importance factor were then obtained from the code and used to calculate the flat roof snow load of 42 psf, which matched the value obtained by the design engineers. Because part of the roof is lower than the rest of the building, drift calculations were performed to find the maximum snow loading in these areas. The detailed calculations for snow loading can be found in Appendix D.

Computer Model

In order to analyze the lateral system of GHVI a computer model was created using ETABS. This model was used for both 2D and 3D analysis. The structure as a whole was examined with a 3D model to determine how it would react to various load types and combinations. Also, each frame was studied in two dimensions to determine the relative stiffness.

The steel columns and braces of the building were modeled by assigning material and frame section properties as per the structural plans. Columns were pinned at the base after careful consideration and consultation with the design structural engineer. Major and minor moments (M33 & M22) were released at the start and end of all beams. Braces were released of major and minor moment (M33 & M22) at the bottom, and major moment, minor moment, and torsion (M33, M22, & T) at the top to reduce out of plane bending. Finally, rigid diaphragms were drawn at each level and assigned with additional area masses to account for the weight of each floor.

Hand calculations were conducted to verify the accuracy of the computer model, and it proved extremely helpful in visualizing how the structure actually works. The image below shows the 3D model that was used in analysis.



Lateral System Analysis

Relative Stiffness of Lateral Elements

The relative stiffness of each frame was calculated for both the North-South and East-West directions, and is shown in the tables below. Finding the relative stiffness of each frame provides a reasonable method of distributing the lateral load throughout the building. It was done by placing a 100 kip load at the top of each individual frame, and then measuring the lateral displacement in inches. The formula for stiffness is:

$$k_i = \frac{P}{\delta}$$

where k_i is the stiffness, P is the force, or 100 kips, and δ is the lateral displacement. After the stiffness for each frame was found, they were summed, and used to find the relative stiffness with the equation:

$$Relative\ Stiffness = \frac{k_i}{\sum k_i}$$

Refer to Appendix E for a hand calculated lateral load distribution.

East-West Direction Relative Stiffness				
Frame	Load (k)	Displacement (in)	Stiffness (k/in)	Relative Stiffness
A	100	0.7201	138.8656	0.5998
C	100	1701.1382	0.0588	0.0003
G	100	1.20E+14	8.3682E-13	3.61419E-15
H 4-5	100	2.1855	45.7565	0.1976
H 6-7	100	2.1342	46.8564	0.2024
		Σ =	231.5373	1.0000

Table 3 – East-West Relative Stiffness

North-South Direction Relative Stiffness				
Frame	Load (k)	Displacement (in)	Stiffness (k/in)	Relative Stiffness
1	100	2.6558	37.6541	0.2166
3	100	2.0998	47.6232	0.2739
6	100	2.0732	48.2352	0.2774
8	100	2.4785	40.3468	0.2321
		Σ =	173.8593	1.0000

Table 4 – North-South Relative Stiffness

Center of Rigidity

The center of rigidity was calculated for each floor using the relative stiffness of each frame and the following equations:

$$X_r = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \qquad Y_r = \frac{\sum k_{ix} y_i}{\sum k_{ix}}$$

Table 5 displays the hand calculations for the center of rigidity at each floor, which were then compared to the values from ETABS, as shown in Table 6. Because the hand calculations are similar for most of the floors, it can be concluded that the ETABS values can be used with confidence.

Hand Calculated Center of Rigidity		
Level	**X_r**	**Y_r**
Roof	135.24	157.50
9	112.12	132.29
8	112.12	132.29
7	112.12	132.28
6	112.12	132.28
5	112.12	132.28
4	112.12	132.28
3	112.12	132.28
2	112.12	132.28
1	110.55	0.00
Mechanical	114.06	220.50
Basement	110.55	0.00

Table 5 – Hand Calculated COR

ETABS Center of Rigidity		
Level	**X_r**	**Y_r**
Roof	108.05	146.68
9	107.10	139.05
8	108.03	135.33
7	108.80	133.43
6	108.59	130.10
5	108.85	126.27
4	109.11	122.96
3	109.20	117.87
2	109.37	117.05
1	111.31	30.26
Mechanical	109.63	184.23
Basement	109.22	9.79

Table 6 – ETABS COR

Center of Mass and Center of Pressure

Both the center of mass and the center of pressure values were obtained for each floor from the ETABS model, and are shown in the tables below. These are important because earthquake forces act at the center of rigidity, and wind forces act at the center of pressure. If the center of rigidity or the center of pressure differs from the center of mass, a torsional force is induced from the eccentricity. As it can be seen from the Tables 6, 7, and 8, the centers of rigidity and pressure are in fact different from the center of mass, and thus torsion must be considered. Refer to Appendix E for detailed torsional shear hand calculations.

Center of Mass		
Level	**X_r**	**Y_r**
Roof	147.046	103.721
9	101.661	113.185
8	116.754	107.896
7	116.537	107.944
6	116.824	107.832
5	111.083	109.001
4	110.392	110.359
3	110.513	110.219
2	114.372	113.557
1	145.971	50.005
Mechanical	101.492	145.657
Basement	110.543	30.694

Table 7 – Center of Mass

Center of Pressure		
Level	**X_r**	**Y_r**
Roof	141.75	110.25
9	110.25	110.25
8	110.25	110.25
7	110.25	110.25
6	110.25	110.25
5	110.25	110.25
4	110.25	110.25
3	110.25	110.25
2	110.25	110.25
1	126	78.75
Mechanical	110.25	141.75
Basement	110.25	31.5

Table 8 – Center of Pressure

Load Combinations

There are the seven basic load combinations prescribed by ASCE 7-10 section 2.3.2 that were considered for this building:

1) $1.4D$
2) $1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R)$
3) $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.5W)$
4) $1.2D + 1.0W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5) $1.2D + 1.0E + L + 0.2S$ ← Controlling Load Combination
6) $0.9D + 1.0W$
7) $0.9D + 1.0E$

In all, 13 different load cases were input into ETABS for analysis. Snow load was previously calculated and is larger than the roof live load and the rain load. Therefore, the snow load controlled in any combination that included these three load types. Also, in the combinations with wind or earthquake, both an East-West (X) direction and a North-South (Y) direction were considered.

After checking the deflection of a point at the roof level, it can be concluded that combinations five and seven control the design of this building. These cases include the X and Y earthquake forces, which implies that seismic loading controls design. Combination five was used as the controlling case because it would have a greater impact on the gravity system as well. Refer to Table 9 below for the deflections under each load combination.

Level	Diaphragm	Load	UX	UY
Roof	D1	1	-0.0925	-0.0157
Roof	D1	2	-0.0792	-0.0134
Roof	D1	3	-0.0792	-0.0134
Roof	D1	4	1.4121	0.2193
Roof	D1	5	0.1146	1.6404
Roof	D1	6	2.9035	0.452
Roof	D1	7	0.3085	3.2943
Roof	D1	8	0.4316	4.0899
Roof	D1	9	3.2690	0.5334
Roof	D1	10	2.9233	0.4554
Roof	D1	11	0.3283	3.2977
Roof	D1	12	3.2888	0.5368
Roof	D1	13	0.4514	4.0932

Table 9 – Roof Level Deflections under Load Combinations

Drift Analysis

Story drift and total drift were determined for the controlling seismic loading and wind loading. Checking seismic drift is necessary from a strength standpoint, in order to prevent building damage or failure. Wind drift is a serviceability issue, and addressing it is necessary to prevent sway that would cause discomfort to building occupants, as well as damage to curtain walls and other façade components.

For seismic loading, drift values were obtained from the ETABS model and were then compared to the allowable story drift and total drift of $0.010h_{sx}$. The wind load drifts were also acquired from ETABS, but they were evaluated against the limit of H/400.

As it can be seen from the following tables, all story drift and total drift values were well within the allowable limits. It was expected that drift would be acceptable when compared with code limits, but the large differences between code and modeled values may indicate a problem with the computer model. Further examination may be needed to assure that these values are in fact correct and no mistakes have been made.

Controlling Seismic Drift: East-West							
Level	Height (ft)	Story Drift (in)	Allowable Story Drift (in)		Total Drift (in)	Allowable Total Drift (in)	
Roof	185	0.000818	0.16	Acceptable	0.020288	1.85	Acceptable
9	169	0.001823	0.18	Acceptable	0.019470	1.69	Acceptable
8	151	0.002348	0.18	Acceptable	0.017647	1.51	Acceptable
7	133	0.002453	0.18	Acceptable	0.015299	1.33	Acceptable
6	115	0.002322	0.18	Acceptable	0.012846	1.15	Acceptable
5	97	0.002099	0.18	Acceptable	0.010524	0.97	Acceptable
4	79	0.001916	0.18	Acceptable	0.008425	0.79	Acceptable
3	61	0.001647	0.18	Acceptable	0.006509	0.61	Acceptable
2	43	0.001578	0.13	Acceptable	0.004862	0.43	Acceptable
1	30	0.001293	0.09	Acceptable	0.003284	0.3	Acceptable
Mechanical	21	0.001115	0.05	Acceptable	0.001991	0.21	Acceptable
Basement	16	0.000876	0.16	Acceptable	0.000876	0.16	Acceptable

Table 10 – East-West Direction Controlling Seismic Drift

Controlling Seismic Drift: North-South							
Level	**Height (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)**		**Total Drift (in)**	**Allowable Total Drift (in)**	
Roof	185	0.002216	0.16	Acceptable	0.024016	1.85	Acceptable
9	169	0.002552	0.18	Acceptable	0.021800	1.69	Acceptable
8	151	0.002757	0.18	Acceptable	0.019248	1.51	Acceptable
7	133	0.002643	0.18	Acceptable	0.016491	1.33	Acceptable
6	115	0.002576	0.18	Acceptable	0.013848	1.15	Acceptable
5	97	0.00239	0.18	Acceptable	0.011272	0.97	Acceptable
4	79	0.002092	0.18	Acceptable	0.008882	0.79	Acceptable
3	61	0.001735	0.18	Acceptable	0.006790	0.61	Acceptable
2	43	0.001077	0.13	Acceptable	0.005055	0.43	Acceptable
1	30	0.000928	0.09	Acceptable	0.003978	0.3	Acceptable
Mechanical	21	0.001992	0.05	Acceptable	0.003050	0.21	Acceptable
Basement	16	0.001058	0.16	Acceptable	0.001058	0.16	Acceptable

Table 11 – North-South Direction Controlling Seismic Drift

Wind Drift: East-West							
Level	**Height (ft)**	**Story Drift (in)**	**Allowable Story Drift (in)**		**Total Drift (in)**	**Allowable Total Drift (in)**	
Roof	185	0.000744	0.48	Acceptable	0.018664	5.55	Acceptable
9	169	0.001531	0.54	Acceptable	0.017920	5.07	Acceptable
8	151	0.001908	0.54	Acceptable	0.016389	4.53	Acceptable
7	133	0.001981	0.54	Acceptable	0.014481	3.99	Acceptable
6	115	0.001904	0.54	Acceptable	0.012500	3.45	Acceptable
5	97	0.001776	0.54	Acceptable	0.010596	2.91	Acceptable
4	79	0.001688	0.54	Acceptable	0.008820	2.37	Acceptable
3	61	0.001532	0.54	Acceptable	0.007132	1.83	Acceptable
2	43	0.001591	0.39	Acceptable	0.005600	1.29	Acceptable
1	30	0.001726	0.27	Acceptable	0.004009	0.90	Acceptable
Mechanical	21	0.001286	0.15	Acceptable	0.002283	0.63	Acceptable
Basement	16	0.000997	0.48	Acceptable	0.000997	0.48	Acceptable

Table 12 – East-West Direction Wind Drift

Wind Drift: North-South							
Level	Height (ft)	Story Drift (in)	Allowable Story Drift (in)		Total Drift (in)	Allowable Total Drift (in)	
Roof	185	0.001652	0.48	Acceptable	0.019438	5.55	Acceptable
9	169	0.001932	0.54	Acceptable	0.017786	5.07	Acceptable
8	151	0.002047	0.54	Acceptable	0.015854	4.53	Acceptable
7	133	0.001973	0.54	Acceptable	0.013807	3.99	Acceptable
6	115	0.001956	0.54	Acceptable	0.011834	3.45	Acceptable
5	97	0.001875	0.54	Acceptable	0.009878	2.91	Acceptable
4	79	0.001713	0.54	Acceptable	0.008003	2.37	Acceptable
3	61	0.001505	0.54	Acceptable	0.006290	1.83	Acceptable
2	43	0.000976	0.39	Acceptable	0.004785	1.29	Acceptable
1	30	0.000892	0.27	Acceptable	0.003809	0.90	Acceptable
Mechanical	21	0.001811	0.15	Acceptable	0.002917	0.63	Acceptable
Basement	16	0.001106	0.48	Acceptable	0.001106	0.48	Acceptable

Table 13 – North-South Direction Wind Drift

Overturning and Impact on Foundation

Overturning moments are a result of wind and seismic loading, and cause the building to try and 'topple over'. This 'toppling' produces uplift in the foundation, and the foundation must be able to resist this uplift. The foundation of GHVI consists of steel helical piles with an allowable axial capacity of 342 kips. These piles are driven to refusal at about a depth of 82 to 87 feet.

In order to check the foundation of this building against uplift the controlling load combination was placed on the ETABS model in both the East-West direction and the North-South direction. From the model the reactions at the base of the structure were found, and negative reactions were deemed significant. A negative reaction on the base means that there is a positive uplift force on the foundation. The location of each uplift occurrence was determined, and the foundation plan was referenced to determine the type of pile cap and the number of piles at this region. The axial load was calculated for this part of the foundation, and it was then compared to the uplift force. The foundation was found to be adequate for each location of uplift. Refer to Tables 14 and 15 for the uplift locations, forces, and corresponding axial capacities.

Level	Point	Load	FZ	Pile Cap	Axial Capacity (k)
Base	207	8	205	-	-
Base	208	8	1331	-	-
Base	209	8	-1130	PC7	2394
Base	210	8	-104	PC4	1368
Base	250	8	86	-	-
Base	254	8	1149	-	-
Base	255	8	-1004	PC7	2394
Base	264	8	1312	-	-
Base	265	8	-867	PC7A	2394
Base	271	8	-153	PC7	2394
Base	277	8	-29	PC7	2394
Base	279	8	183	-	-
Base	285	8	49	-	-
Base	286	8	64	-	-
Base	287	8	140	-	-
Base	316	8	46	-	-
Base	317	8	51	-	-
Base	318	8	53	-	-
Base	336	8	189	-	-
Base	344	8	-777	PC5	1710
Base	562	8	944	-	-
Base	563	8	-114	PC4	1368

Table 14 – North-South Base Reactions and Corresponding Pile Axial Capacity

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Level	Point	Load	FZ	Pile Cap	Axial Capacity (k)
Base	207	9	63	-	-
Base	208	9	327	-	-
Base	209	9	-117	PC7	2394
Base	210	9	29	-	-
Base	250	9	56	-	-
Base	254	9	93	-	-
Base	255	9	81	-	-
Base	264	9	-100	PC7	2394
Base	265	9	-989	PC7A	2394
Base	271	9	1380	-	-
Base	277	9	-1105	PC7	2394
Base	279	9	1259	-	-
Base	285	9	-1219	PC7	2394
Base	286	9	64	-	-
Base	287	9	1409	-	-
Base	316	9	-62	PC6	2052
Base	317	9	51	-	-
Base	318	9	161	-	-
Base	336	9	29	-	-
Base	344	9	143	-	-
Base	562	9	19	-	-
Base	563	9	52	-	-

Table 15 – East-West Base Reactions and Corresponding Pile Axial Capacity

Lateral Member Spot Checks

Lateral member spot checks were performed on three braces and two columns at different levels of Braced Frame A, as shown in Figure D. A brace was analyzed at the top, middle, and base of the structure, and a column was investigated at the middle and base of the structure. The loads for each member were obtained from the ETABS model considering the controlling load combination. All five members seem to be slightly oversized, and this may be a result of drift requirements. Detailed hand calculations of these spot checks can be found in Appendix F.

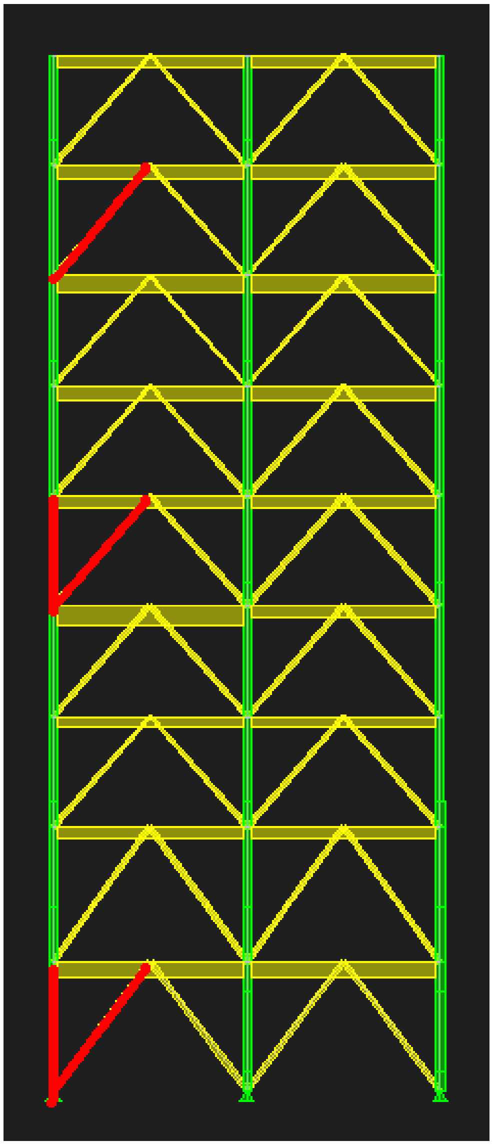

Figure D – Frame A

Conclusion

This third technical report has investigated the existing lateral system of the Kaleida Health and University at Buffalo Global Heart and Vascular Institute with respect to strength and serviceability requirements.

An ETABS model was constructed and used to study the building in both two and three dimensions. The steel columns and braces of each frame were modeled and assigned material and frame section properties. Columns were pinned at the base and beams and braces were released to prevent them from taking out-of-plane bending. A rigid diaphragm was drawn with additional area masses to model each floor level. Hand calculations were also conducted to verify the accuracy of the computer model.

Seven basic load combinations were taken from ASCE 7-10 and considered to determine the controlling load case for this building. Due to the fact that combinations with wind or earthquake include both a North-South (Y) and an East-West (X) direction, 13 load cases were actually input into the ETABS model. Although unexpected, seismic loading was shown to control in both the North-South and East-West directions.

The drift analysis included a strength check of the controlling seismic load combination, and a serviceability check of the wind forces acting on the building. Seismic drift values were obtained from the ETABS model and were checked against the allowable story drift and total drift of $0.010h_{sx}$. The wind load drifts were also acquired from ETABS, but they were evaluated against the limit of H/400. All story drift and total drift values were within the allowable limits, and although this was expected, the large differences between code and modeled values may indicate a problem with the computer model. Further investigation into the computer model and drift values may be required in a future report.

Overturning moments were considered with regard to the foundations. The controlling seismic load combination was applied to the ETABS model and uplift forces were found at the base of some of the frames. After checking all critical points at the base of the structure, the foundation was found to be adequately designed for uplift.

Five lateral spot checks were performed on columns and braces in Frame A. Because Frame A resists lateral load in the East-West direction, the controlling East-West seismic load combination was used to determine the loads. A brace was analyzed at the top, middle, and base of the structure, and a column was investigated at the middle and base of the structure. Hand calculations were performed and all five of the members were found to be sufficient.

Appendix

Appendix A: Typical Floor Plans and Elevations

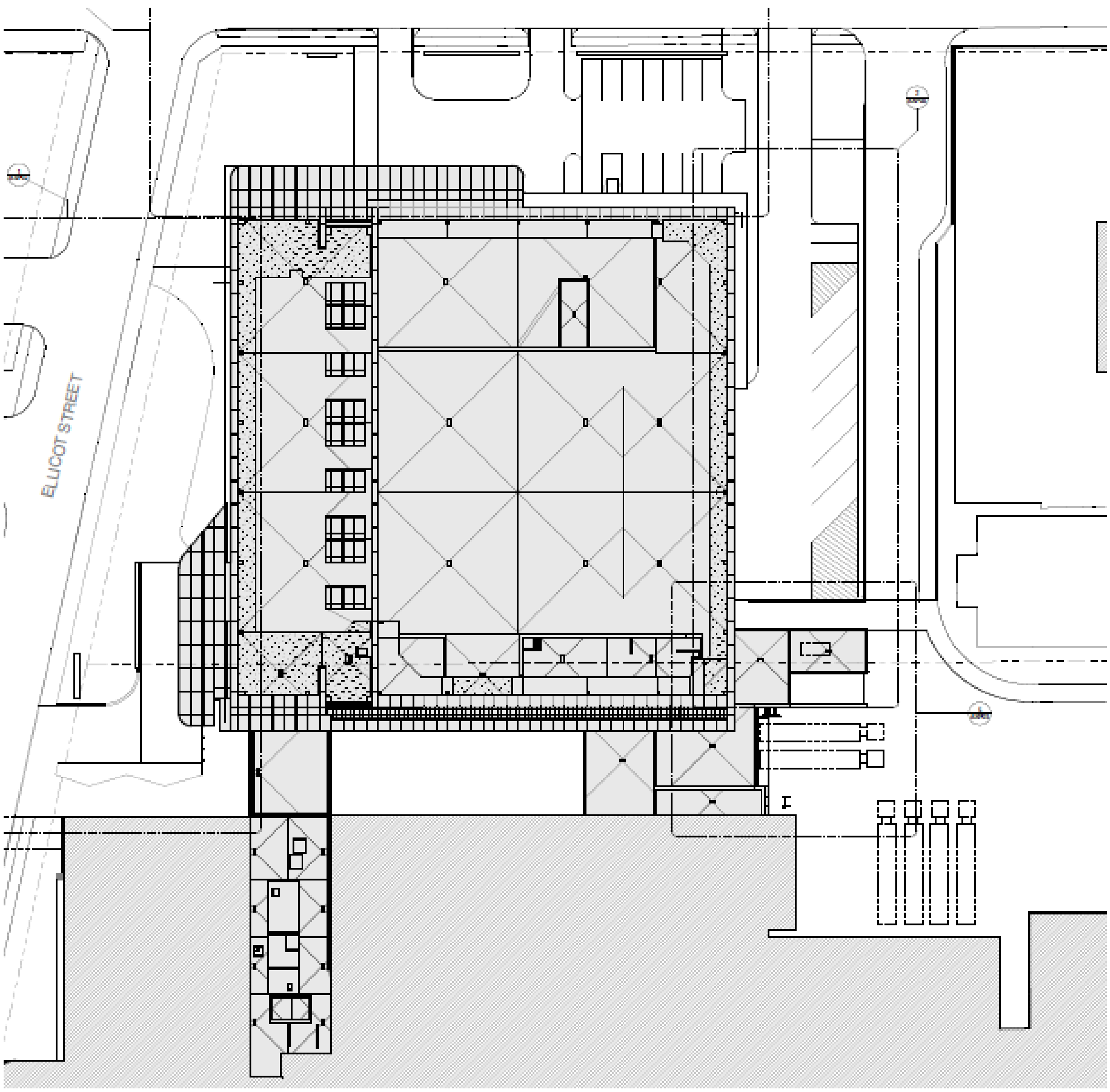

Figure E – Site Plan (Cannon Design)

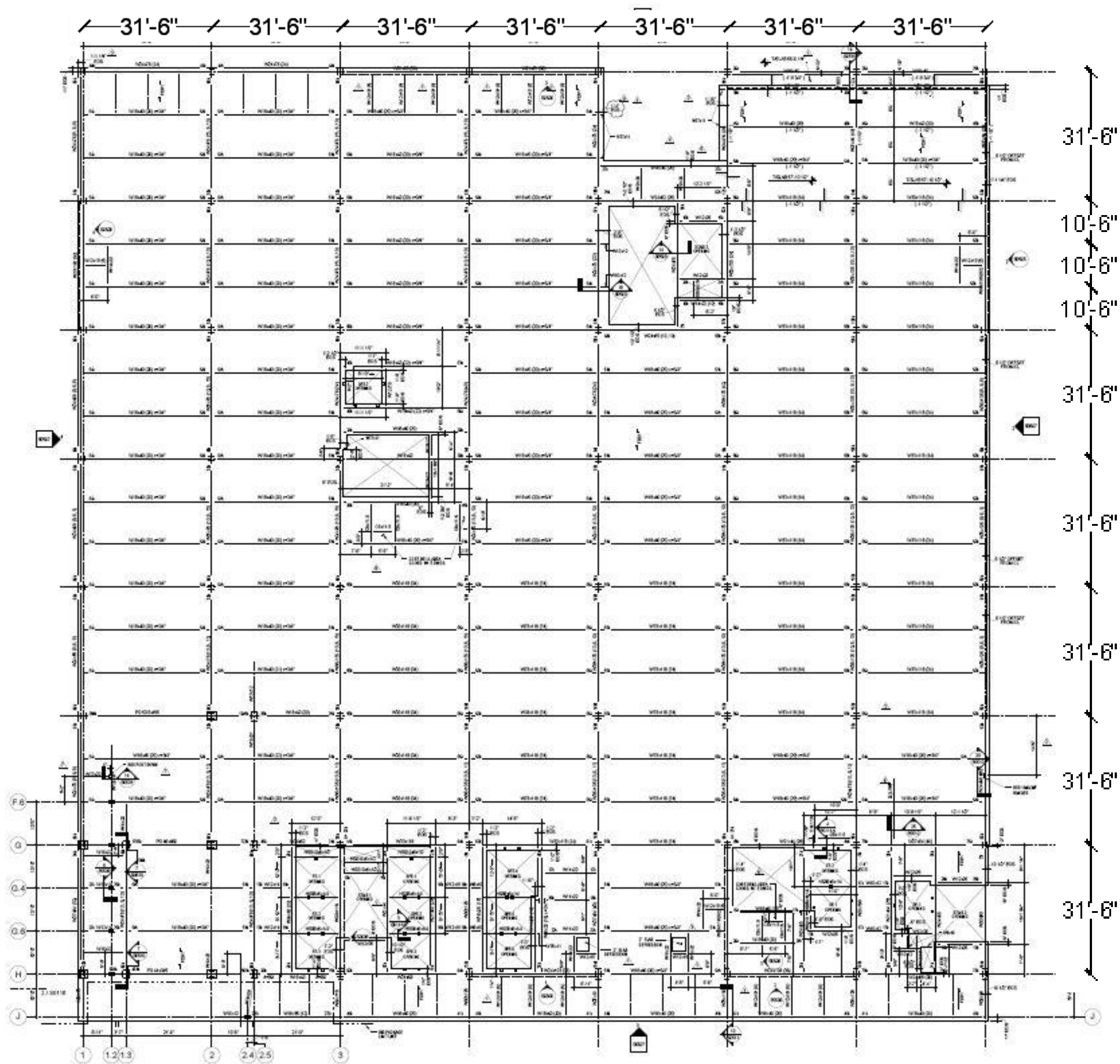

Figure F – Typical floor framing plan (Cannon Design)

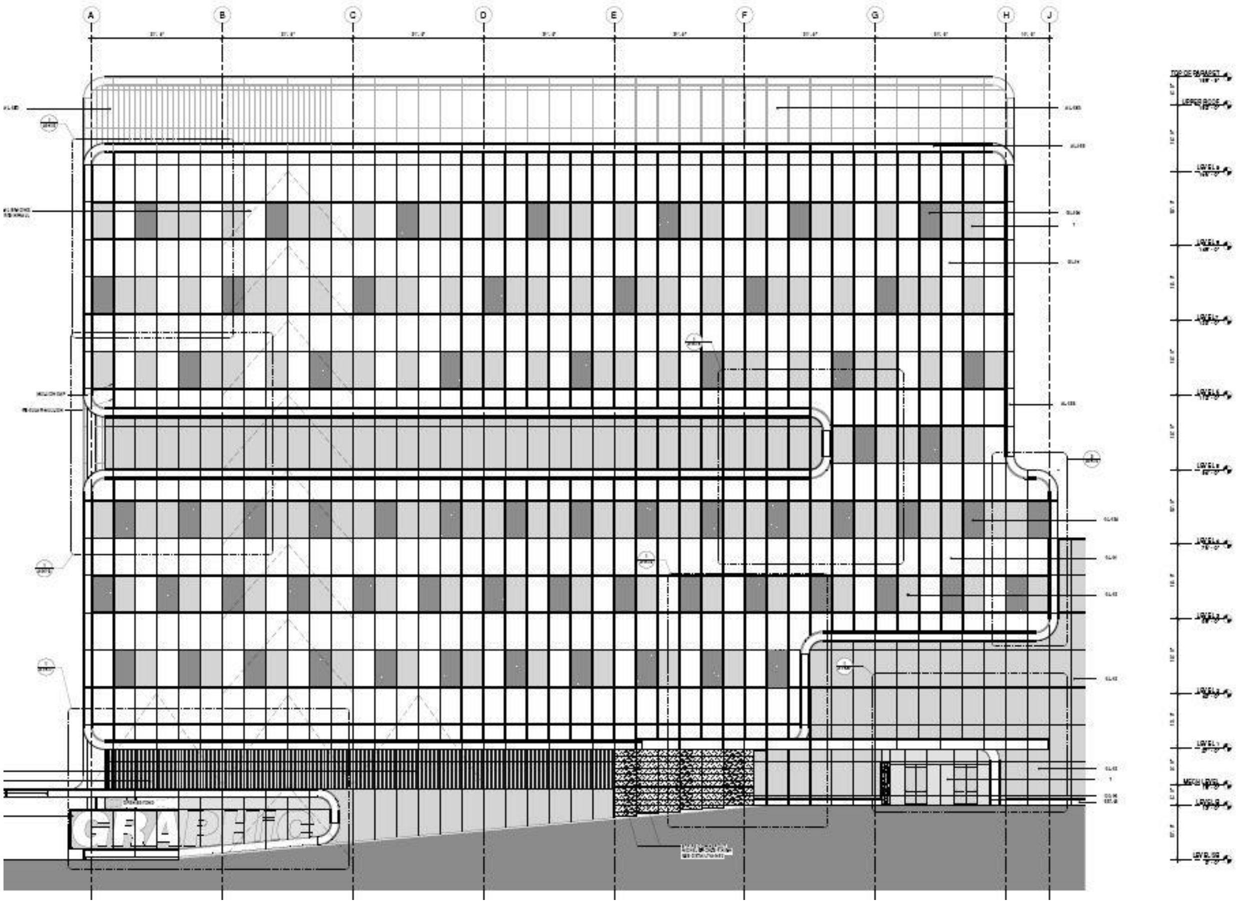

Figure G – West Elevation (Cannon Design)

Appendix B: Wind Analysis

The following table contains the initial parameters used in the wind analysis as determined from ASCE 7-10:

V	120
K_d	0.85
Exposure	B
K_{zt}	1
GC_{pi}	0.18

Table 16 - Parameters

The following table contains the effective length calculations completed to assure that the natural frequency could be approximated:

N-S Direction				E-S Direction			
Level	**h_i**	**l_i**	**h_il_i**	**Level**	**h_i**	**l_i**	**h_il_i**
Sub basement	13	221	2873	Sub basement	13	174	2262
Basement	18	221	3978	Basement	18	221	3978
Mechanical	27	221	5967	Mechanical	27	221	5967
1	40	221	8840	1	40	221	8840
2	58	221	12818	2	58	221	12818
3	76	221	16796	3	76	221	16796
4	94	221	20774	4	94	221	20774
5	112	221	24752	5	112	221	24752
6	130	221	28730	6	130	221	28730
7	148	221	32708	7	148	221	32708
8	166	221	36686	8	166	221	36686
9	189	158	29862	9	189	221	41769
Σ =	1071		224784	**Σ =**	1071		236080
L_{eff} =	209.9			**L_{eff} =**	220.4		

Table 17 – Effective Length Check Calculations

The following table contains the calculations to determine the gust-effect factor:

Gust Effect Calculation		
	N-S	**E-W**
B	221	221
L	221	221
h	189	189
n_a	0.3968	0.3968
	FLEXIBLE	FLEXIBLE
I_z	0.244	0.244
c	0.30	0.30
z	113.4	113.4
g_Q	3.4	3.4
g_v	3.4	3.4
g_R	3.96	3.96
R	0.575	0.575
R_n	0.0956	0.0956
N_1	1.777	1.777
L_z	482.89	482.89
V_z	107.83	107.83
R_h	0.2638	0.2638
n	3.20	3.20
R_B	0.2316	0.2316
n	3.74	3.74
R_L	0.0767	0.0767
n	12.52	12.52
Q	0.799	0.799
β	0.01	0.01
G_f	0.95	0.95

Table 18 – Gust Effect Calculations

The following table contains the wind pressure coefficients:

Wind Pressure Coefficients			
Surface	**L/B**	**Cp**	**Use With**
Windward	All	0.8	q_z
Leeward	1	-0.5	q_h
Side	All	-0.7	q_h

Table 19 – Wind Pressure Coefficients

The following tables contains the wind pressure in pounds per square feet for both the windward and leeward directions:

	Level	Height (ft)	K_z	q_z	Wind Pressure	
					N-S	E-W
Windward	Top of Parapet	189	1.18	37.1	90.4	90.4
	Upper Roof	184	1.18	36.8	34.6	34.6
	9	166	1.14	35.8	33.8	33.8
	8	148	1.11	34.7	33.0	33.0
	7	130	1.07	33.4	32.0	32.0
	6	112	1.02	32.0	30.9	30.9
	5	94	0.97	30.5	29.8	29.8
	4	76	0.91	28.6	28.4	28.4
	3	58	0.84	26.4	26.7	26.7
	2	40	0.76	23.8	24.7	24.7
	1	27	0.68	21.2	22.7	22.7
	Mechanical	18	0.60	18.8	20.9	20.9
	Basement	13	0.57	17.9	20.2	20.2

Table 20 – Windward Wind Pressures

	Level	q_h	Wind Pressure	
			N-S	E-W
Leeward	Top of Parapet	37.1	-61.4	-61.4
	Remaining	37.1	-24.3	-24.3

Table 21 – Leeward Wind Pressures

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USE ASCE 7-10 — MWFRS (DIRECTIONAL PROCEDURE)

27.2.1

- BASIC WIND SPEED (26.5)
  OCCUPANCY CATEGORY IV (TABLE 1.5-1)
  ↳ USE FIGURE 26.5-1B → $V = 120$ mph

- WIND DIRECTIONALITY FACTOR (26.6)
  $K_d = 0.85$ (TABLE 26.6-1)

- EXPOSURE CATEGORY (26.7)
  B

- TOPOGRAPHIC FACTOR (26.8)
  $K_{zt} = 1.0$

- GUST-EFFECT FACTOR (26.9)
  RIGID?
  26.9.2.1 APPROXIMATE NATURAL FREQUENCY LIMITATIONS
  1) BUILDING HEIGHT = 189' < 300' ✓ OK
  2) BUILDING HEIGHT = $189' \overset{?}{<} 4L_{eff}$
  CHECK N-S DIRECTION:

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = 209.9 \qquad 4(209.9) = 839.6 > 189 \ \checkmark \text{ OK}$$

CHECK E-W DIRECTION

$$L_{eff} = 220.4 \qquad 4(220.4) = 881.6 > 189 \ \checkmark \text{ OK}$$

∴ CAN APPROXIMATE

26.9.3
STRUCTURAL STEEL BUILDING WITH BRACED-FRAME (26.9-4)

$$n_a = \frac{75}{h} = \frac{75}{189} = 0.3968 < 1.0 \text{ Hz} \rightarrow \text{FLEXIBLE}$$

26.9.5 FLEXIBLE BUILDING

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right] = 0.95 \text{ (FOR BOTH N-S/E-W)}$$

$B = L$

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$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{113.4}\right)^{1/6} = 0.244$$

$c = 0.30$

$$\bar{z} = \max\begin{vmatrix} 0.6h = 0.6(189) = 113.4 \\ z_{min} = 30 \end{vmatrix}$$

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} = 3.96$$

$n_1 = n_a = 0.3968$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.575$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(1.777)}{[1+10.3(1.777)]^{5/3}} = 0.0956$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.3968(482.89)}{107.83} = 1.777$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{113.4}{33}\right)^{1/3} = 482.89$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{113.4}{33}\right)^{1/4}\left(\frac{88}{60}\right)(120) = 107.83$$

R_h: $$\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.3968)(189)}{107.83} = 3.20$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = 0.2638$$

R_B: $$\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.3968)(221)}{107.83} = 3.74 \quad R_B = 0.2317$$

R_L: $$\eta = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.3968)(221)}{107.83} = 12.52 \quad R_L = 0.0769$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.799$$

ASSUME $\beta = 0.01 \rightarrow$ 1% DAMPING FOR STEEL STRUCTURE

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WILLIAM McDEVITT | TECH REPORT #3 | WIND ANALYSIS (3)

- ENCLOSURE CLASSIFICATION (26.10)

ENCLOSED

- INTERNAL PRESSURE COEFFICIENT

ENCLOSED BUILDING = ±0.18

- WALL PRESSURE COEFFICIENTS, C_p

SURFACE	L/B	C_p	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	1	-0.5	q_h
SIDE	ALL	-0.7	q_h

- 27.4.2 ENCLOSED FLEXIBLE BUILDING

$$p = q G_f C_p - q_i (GC_{pi})$$

- 27.4.5 PARAPETS

$$P_p = q_p (GC_{pn})$$

GC_{pn} = + 1.5 FOR WINDWARD PARAPET
− 1.0 FOR LEEWARD PARAPET

$$P_p = 37.1(1.5) = \underbrace{55.65}_{\text{WINDWARD}}\ \text{psf}$$

$$P_p = 37.1(-1.0) = \underbrace{-37.1}_{\text{LEEWARD}}\ \text{psf}$$

- DESIGN PRESSURES (N-S + E-W WILL BE EQUAL BECAUSE B = L)

WINDWARD: $p = q_z G_f C_p - q_h (GC_{pi})$

$$p = q_z(0.95)(0.8) - 37.1(\pm 0.18) = 0.76 q_z + 6.14 \text{ psf}$$

ADD P_p = 55.65 psf TO PARAPET

LEEWARD: $p = q_h G_f C_p - q_h (GC_{pi})$

$$p = 37.1(0.95)(-0.5) - 37.1(\pm 0.18) = -24.30 \text{ psf}$$

ADD P_p = −37.1 psf TO PARAPET

Appendix C: Seismic Analysis

The following table contains an example summation of the weight of a floor for use in seismic analysis:

Level 6					
Steel-Beams	**Type**	**Number**	**Length(ft)**	**Weight (lb/ft)**	**Weight (lb)**
	W27x94	154	31.5	94	455994
	W30x108	56	31.5	108	190512
				Total Beams	646506.0
Steel-Columns	**Type**	**Number**	**Length(ft)**	**Weight (lb/ft)**	**Weight (lb)**
	W14x68	5	18	68	6120
	W14x74	3	18	74	3996
	W14x90	31	18	90	50220
	W14x109	15	18	109	29430
	W14x120	10	18	120	21600
				Total Columns	111366.0
Deck	**Type**	**Weight (psf)**	**Area (ft2)**	**Weight (lb)**	
	3" (7.5")	75	48841	3663075.0	
			Total Deck	3663075.0	
			Total L6 Weight (lb)	4420947.0	

Table 22 – Example Weight Summation

The following table contains the summation of the total building weight above grade:

Level	**Weight (k)**
Roof	1056
9	4089
8	6354
7	6437
6	6395
5	6167
4	6202
3	6433
2	6067
1	958
Base/Mech	2478
Total	52636

Table 23 – Total Weight

(1)

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ASCE/SEI 7-10

11.4.2 SITE CLASS

D – AS PER GEOTECHNICAL REPORT

11.4.3 SPECTRAL RESPONSE ACCELERATION

$S_s = 0.277$ BUFFALO, NY 14203

$S_1 = 0.058$

F_a: D: $S_s < 0.25$: 1.6; 0.277: $\boxed{1.58}$; $S_s = 0.5$: 1.4 $F_a = 1.58$

F_v: D: $S_1 \leq 0.1$: 2.4 $F_v = 2.4$

$S_{MS} = F_a S_s = 1.58(0.277) = 0.438$

$S_{M1} = F_v S_1 = 2.4(0.058) = 0.139$

11.4.4 DESIGN SPECTRAL RESPONSE ACCELERATION

OK FOR ELFP

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.438) = 0.292 \Rightarrow$ SEISMIC DESIGN CATEGORY C ✓

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.139) = 0.093$ $(0.167 \leq S_{DS} < 0.33 + \text{IV})$

ADJACENT STRUCTURE WITHIN 10'

12.8 EQUIVALENT LATERAL FORCE PROCEDURE

12.8.1 SEISMIC BASE SHEAR

$V = C_s W$

$W = 52636\text{K}$ (TABULATED IN EXCEL)

12.8.1.1 SEISMIC RESPONSE COEFFICIENT

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

$R = 3.25$ (TABLE 12.2-1, STEEL ORDINARY CONCENTRICALLY BRACED FRAMES – NL)

$I_e = 1.50$ (TABLE 1.5-2, RISK IV)

$$C_s = \frac{0.292}{\left(\frac{3.25}{1.50}\right)}$$

$C_s = 0.135 \Rightarrow$ SHALL NOT EXCEED: $C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ FOR $T \leq T_L$

OR

$C_s = \frac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)}$ FOR $T > T_L$

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T = FUNDAMENTAL PERIOD – 12.8.2
T_L = LONG-PERIOD TRANSITION PERIOD(S) – 11.4.5 $T_L = 6$ (FIGURE 22-12)

12.8.2 PERIOD DETERMINATION

$$T = \min \begin{cases} C_u T_a \\ T_b \text{ - CAN BE DETERMINED LATER FROM COMPUTER MODEL} \end{cases}$$

$C_u = 1.7$ (TABLE 12.8-1, $S_{D1} = 0.093 \leq 0.1$)

$T_a = C_t h_n^{x}$ $\quad C_t = 0.02 \quad x = 0.75$ (TABLE 12.8-2, ALL OTHER STRUCTURAL SYSTEMS)

$T_a = 0.02(184)^{0.75}$ $\quad h_n = 184$ ft

$T_a = 0.999$

$T = C_u T_a = 1.7(0.999) = 1.698$ s ← SAME FOR BOTH N-S/E-W DIR.

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \dfrac{0.292}{\left(\frac{3.25}{1.50}\right)} = 0.135 \\[2ex] \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{0.093}{1.698\left(\frac{3.25}{1.50}\right)} = 0.025 \\[2ex] \dfrac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)} = \dfrac{0.093(6)}{(1.698)^2\left(\frac{3.25}{1.50}\right)} = 0.089 \end{cases}$$

$V = C_s W = 0.025(52636) = 1316$ k

12.8.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx} V$

$w_x \neq w_i$ ✓
$h_x \neq h_i$ ✓

$$C_{vx} = \frac{w_x h_x^{k}}{\sum_{i=1}^{n} w_i h_i^{k}}$$

T	0.5	1.698	2.5
k	1	$\boxed{1.599}$	2

$k = 1.599$

SEE EXCEL SPREADSHEET FOR DETERMINATION OF VERTICAL FORCES

Appendix D: Snow Loading

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7.2 GROUND SNOW LOADS

P_g = CS FROM FIGURE 7-1

AS PER CASE STUDY, $P_g = 50$ psf

7.3 FLAT ROOF SNOW LOADS, P_f

$P_f = 0.7\, C_e C_t I_s P_g$

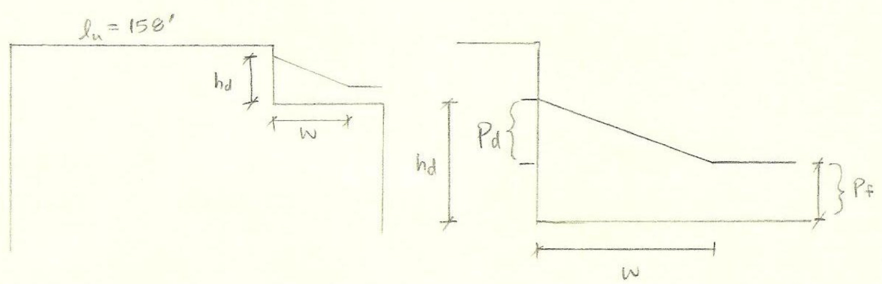
$P_f = 0.7(1.0)(1.0)(1.2)(50) = 42$ psf

7.3.1 $C_e = 1.0$ (EXPOSURE B, PARTIALLY EXPOSED - PARAPET)

7.3.2 $C_t = 1.0$

7.3.3 $I_s = 1.2$ (RISK CATEGORY IV)

7.7 DRIFT



$h_d = 0.43 \sqrt[3]{l_u}\, \sqrt[4]{P_g + 10} - 1.5$

$h_d = 0.43(158)^{1/3}(50+10)^{1/4} - 1.5 = 4.97$ ft

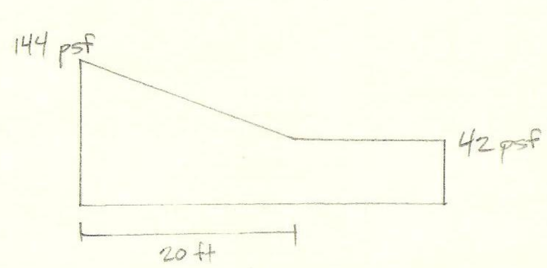
$w = 4h_d = 4(4.97)$

$w = 19.9$ ft

$\gamma = \begin{cases} 0.13 P_g + 14 = 0.13(50) + 14 = 20.5 \text{ pcf} \\ \text{min } 30 \text{ pcf} \end{cases}$

$P_d = h_d \gamma = 4.97(20.5) = 101.9$ psf

$P_{total} = 42 + 101.9 = 143.9$ psf



Appendix E: Lateral Load Distribution

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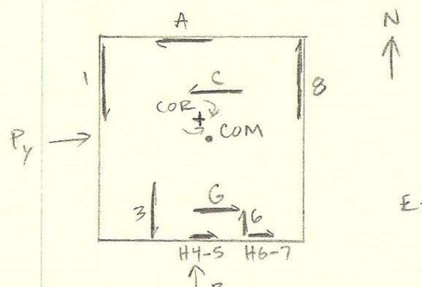


EARTHQUAKE CONTROLS SO USE $P_y = 1316k$

	FRAME	STIFFNESS (K)
E-W	A	138.9
	C	0.0588
	G	8.37×10^{-13}
	H4-5	45.8
	H6-7	46.8
N-S	1	37.7
	3	47.6
	6	48.2
	8	40.3

ASSUME:
COM = 117 ft, 108 ft
COR = 109 ft, 130 ft
e = 8 , 22

NORTH-SOUTH

DIRECT SHEAR:

$$F_{D1} = \frac{37.7}{37.7+47.6+48.2+40.3}(1316) = 285\ k \downarrow$$

$$F_{D3} = \frac{47.6}{37.7+47.6+48.2+40.3}(1316) = 360\ k \downarrow$$

$$F_{D6} = \frac{48.2}{173.8}(1316) = 365\ k \downarrow$$

$$F_{D8} = \frac{40.3}{173.8}(1316) = 305\ k \downarrow$$

TORSIONAL SHEAR:

$$\theta_t = \frac{1316(8)}{37.7(109)^2+47.6(46)^2+48.2(48.5)^2+40.3(111.5)^2+138.9(90.5)^2+0.0588(27.5)^2+8.37\times10^{-13}(98.5)^2+45.8(130)^2+46.8(130)^2}$$

$\theta_t = 2.72\times10^{-3}$ RADIANS

$F_{it} = k_i d_i \theta_t$

$F_{T1} = 37.7(109)(2.72\times10^{-3}) = 11.2\ k \downarrow$ $\qquad F_{TA} = 34.2\ k \leftarrow$

$F_{T3} = 47.6(46)(2.72\times10^{-3}) = 6.0\ k \downarrow$ $\qquad F_{TC} = 0.004\ k \leftarrow$

$F_{T6} = 48.2(48.5)(2.72\times10^{-3}) = 6.4\ k \uparrow$ $\qquad F_{TG} = 2.24\times10^{-13}\ k \rightarrow$

$F_{T8} = 40.3(111.5)(2.72\times10^{-3}) = 12.2\ k \uparrow$ $\qquad F_{TH4-5} = 16.2\ k \rightarrow$

$F_{TH6-7} = 16.5\ k \rightarrow$

WILLIAM McDEVITT | TECH REPORT #3 | LATERAL LOAD DIST. 2

TOTAL SHEAR

$F_1 = 296.2\ k \downarrow$
$F_3 = 366\ k \downarrow$
$F_6 = 358.6\ k \downarrow$
$F_8 = 292.8\ k \downarrow$

$F_A = 34.2\ k \leftarrow$
$F_C = 0.004\ k \leftarrow$
$F_G = 2.24 \times 10^{-13}\ k \rightarrow$
$F_{H4-5} = 16.2\ k \rightarrow$
$F_{H6-7} = 16.5\ k \rightarrow$

EAST - WEST

DIRECT SHEAR:

$$F_{DA} = \frac{138.9}{231.6}(1316) = 789.3\ k \leftarrow$$

$$F_{DC} = \frac{0.0588}{231.6}(1316) = 0.33\ k \leftarrow$$

$$F_{DG} = \frac{8.37 \times 10^{-13}}{231.6}(1316) = 4.76 \times 10^{-12}\ k \leftarrow$$

$$F_{DH4-5} = \frac{45.8}{231.6}(1316) = 260.2\ k \leftarrow$$

$$F_{DH6-7} = \frac{46.8}{231.6}(1316) = 265.9\ k \leftarrow$$

TORSIONAL SHEAR

$$\theta_t = \frac{1316(22)}{3865643.6} = 7.49 \times 10^{-3}\ \text{RADIANS}$$

$F_{TA} = 94.2\ k \rightarrow$
$F_{TC} = 0.012\ k \rightarrow$
$F_{TG} = 6.18 \times 10^{-13}\ k \leftarrow$
$F_{TH4-5} = 44.6\ k \leftarrow$
$F_{TH6-7} = 46.6\ k \leftarrow$

$F_{T1} = 30.8\ k \downarrow$
$F_{T3} = 16.4\ k \uparrow$
$F_{T6} = 17.5\ k \uparrow$
$F_{T8} = 33.7\ k \downarrow$

TOTAL SHEAR

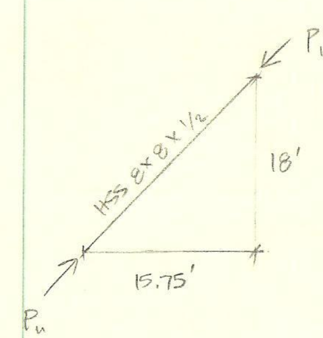
$F_A = 695.1\ k \leftarrow$
$F_C = 0.318\ k \leftarrow$
$F_G = 5.38 \times 10^{-12}\ k \leftarrow$
$F_{H4-5} = 304.8\ k \leftarrow$
$F_{H6-7} = 312.5\ k \leftarrow$

$F_1 = 30.8\ k \downarrow$
$F_3 = 16.4\ k \uparrow$
$F_6 = 17.5\ k \uparrow$
$F_8 = 33.7\ k \downarrow$

Appendix F: Lateral Member Spot Checks

WILLIAM McDEVITT | TECH REPORT #3 | SPOT CHECKS (1)

BRACE ON FRAME A AT LEVEL 7



$1.2D + 1.0E_x + L + 0.2S$ (COMBO 9)

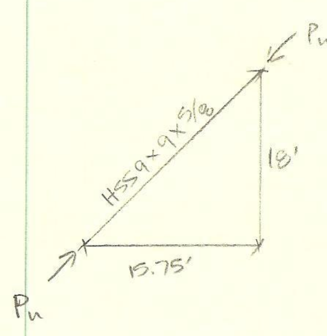
HSS8x8x½, $F_y = 46$ ksi

$L_b = \sqrt{18^2 + 15.75^2} = 24'$

FROM TABLE 4-4 (p4-52)

$\phi P_n = 305K > 67K$ ✓ ∴ OK

BRACE ON FRAME A AT LEVEL 4



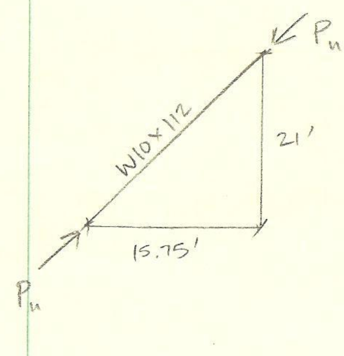
HSS9x9x⅝, $F_y = 46$ ksi

$L_b = 24'$

FROM TABLE 4-4 (p4-51)

$\phi P_n = 477K > 137K$ ✓ ∴ OK

BRACE ON FRAME A AT BASE LEVEL



W10x112, $F_y = 50$ ksi

$L_b = \sqrt{21^2 + 15.75^2} = 26'$

FROM TABLE 4-1 (p4-19)

$\phi P_n = 549K > 179K$ ✓ ∴ OK

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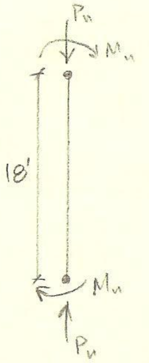
WILLIAM MCDEVITT | TECH REPORT #3 | SPOT CHECKS (2)

COLUMN ON FRAME A AT LEVEL 4



$P_u = 287$ k
$M_u = 495$ 'k

W14x145

FROM TABLE 6-1 (pg 6-68)

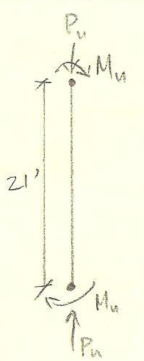
$L = 18'$ $\quad r_x/r_y = 1.59$

$KL = 18(1.59) = 28.62'$

	$p \times 10^{-3}$	$b \times 10^{-3}$
28	0.877	1.02
28.62	0.899	1.03
30	0.947	1.04

$287(0.899 \times 10^{-3}) + 495(1.03 \times 10^{-3}) = 0.77 < 1.0 \checkmark \therefore$ OK

COLUMN ON FRAME A AT LEVEL 1



$P_u = 984$ k
$M_u = 1424$ 'k

W14x370

FROM TABLE 6-1 (pg 6-64)

$L = 21'$ $\quad r_x/r_y = 1.66$

$KL = 21(1.66) = 34.86$

	$p \times 10^{-3}$	$b \times 10^{-3}$
34	0.397	0.343
34.86	0.412	0.344
36	0.431	0.345

$984(0.412 \times 10^{-3}) + 1424(0.344 \times 10^{-3}) = 0.90 < 1.0 \checkmark \therefore$ OK