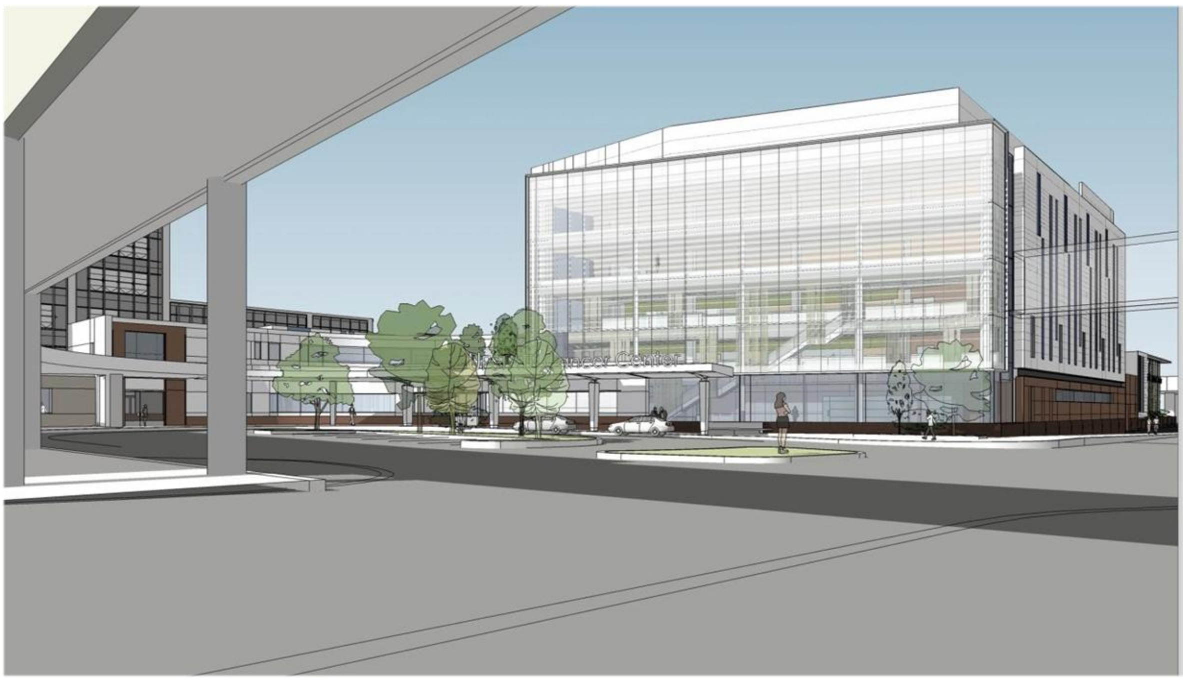
SUNY

Upstate Cancer Center

Syracuse, New York



Technical Report 1

Michael Kostick | Structural Option

Existing Conditions

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Executive Summary

The intent of this report is to gain a thorough understanding of the current existing structure of the SUNY Upstate Cancer Center located in Syracuse, New York. In order to successfully achieve this task, the structural concepts of the current design will be examined; design values for gravity and lateral loads will be calculated; and typical structural framing components will be checked for suitability under gravity loads. All drawings and specifications as well as rendered images have been provided by EwingCole.

A brief overview of each structural system present in the building has been provided in efforts to help understand how each of the systems operates independently and cohesively of each other within the overall structural system. Design codes and standards used for analysis purposes are discussed and related to original documents pooled for the initial design of building. Materials types and properties used in the original design remained unchanged for analysis purposed carried out through this report.

Applicable building loads generalized as either gravity or lateral loads were determined for the given structure by use of applicable codes, such as the 2009 International Building Code, ASCE 7-10, and the AISC Manual for Steel Construction 14th Edition. Gravity loads for the Upstate Cancer Center consisted of snow load, dead load, and live load. Calculations provided a max snow load, considering drifting effects, of 143 psf. Dead loads were established while finding the overall building weight, and mainly were composed of structural members, material weights, and wall and floor assemblies. These values are tabulated later in the report. Live load values were gathered from the appropriate code literature and compared for similarities and differences to the original design live loads.

Lateral loads consisted of wind and seismic loads and were calculated in accordance with the respective chapters of ASCE 7-10. In order to produce a wind analysis by hand, a simplification of the building's geometries had to be used. Wind analysis was carried out for each direction of loading, North-South and East-West. The resulting wind base shears were 319.2 kips and 288.42 kips, and the resulting wind overturning moments were 11826 ft-k and 10911 ft-k for the North-South and East-West directions, respectively. The large difference in design pressures and analysis pressures have been attributed to the use of differing design codes. Seismic load analysis resulted in the conclusion that seismic loading will drive the design of the lateral system for the building. Seismic base shears and overturning moments were calculated individually for three separate portions of the building. These divisions were determined based on the locations of building expansion joints. In summary the highest seismic base shear was more than double the wind base shear and the seismic overturning moment was nearly three times that of the wind overturning moment. This was the reasoning behind the conclusion that seismic loads will control the design of the later system.

Finally, structural elements of a typical bay were checked for adequate strength as well as serviceability issues including total and live load deflections, wet concrete deflections, and unshored strength of composite framing members. Items that were checked included composite metal floor deck, composite wide flange beams and girders, and a typical gravity column. In summary, all elements checked, met or exceeded loading and deflection requirements.

Introduction

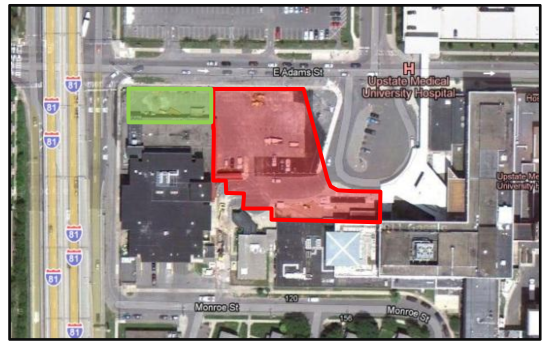


Figure 1 Aerial map locating the building site. (Courtesy of Google Maps)

The State University of New York's Upstate Medical University, located in Syracuse, New York will serve as the home to the new Upstate Cancer Center. Taking the place of an existing parking lot to the northwest of the Upstate Medical University Hospital, the new center will not only serve as the region's premiere outpatient adult and pediatric cancer center, but also link the university's Regional Oncology Center (ROC), Gamma Knife Center, and the Upstate Medical University Hospital. (See Figure 1)

Upon its completion, the five-story building will rise 72 feet to the roof level, 90 feet to the top of the rooftop parapets, and encompass 90,000 square feet. Floor one will house administration services, the radiology department, as well as intra operative suites. The second floor will be reserved for medical oncology while the third floor will be devoted entirely for pediatric oncology. Floors four and five will consist of shell space intended for future outfit and expansion. A two-story central plant containing electrical transformers and a full mechanical space serves as linkage between the cancer center and the existing ROC. (See Figure 1 – highlighted green)

The building is primarily clad in a soothing white insulated metal paneling with cold form metal stud back up. This metal paneling is rather haphazardly disrupted by varying widths and heights of vertical bands of glazing. These bands consist of both vision and spandrel glazing, which is used to transition floor levels, hiding mechanical space and the structural floor. The exterior façade culminates at the three-story, northeast facing entrance atrium. Featuring a custom frit pattern, the northeast facing façade is enclosed by a full height, glazed curtain wall which provides solar shading as well as an aesthetically pleasing view. (See Figure 2 below)

Figure 2 Exterior rendering of northeast entry façade. (Courtesy of EwingCole)

Upstate is committed to providing a comforting environment for their patients, providing amenities such as a meditation room, a boutique for gifts and apparel, and a four-season roof top healing garden. These gardens not only serve as a refreshing oasis, but also help to reduce the cooling costs for the Upstate Cancer Center, adding to their goal of achieving USGBC LEED Silver certification. Preliminary Construction on the 74 million dollar center began in March of 2011 and is expected to be completed by September of 2013.

Structural Systems

Structural Key Plan

In an attempt to better understand the building geometries, a key plan overview of the site has been created. Main divisions of the building were divided and designated based on the location of expansion joints. Included in this reference diagram are basic dimensions, story counts, roof elevations, and primary building function or name. These building names will apply to data, calculations, and descriptions later in this report.

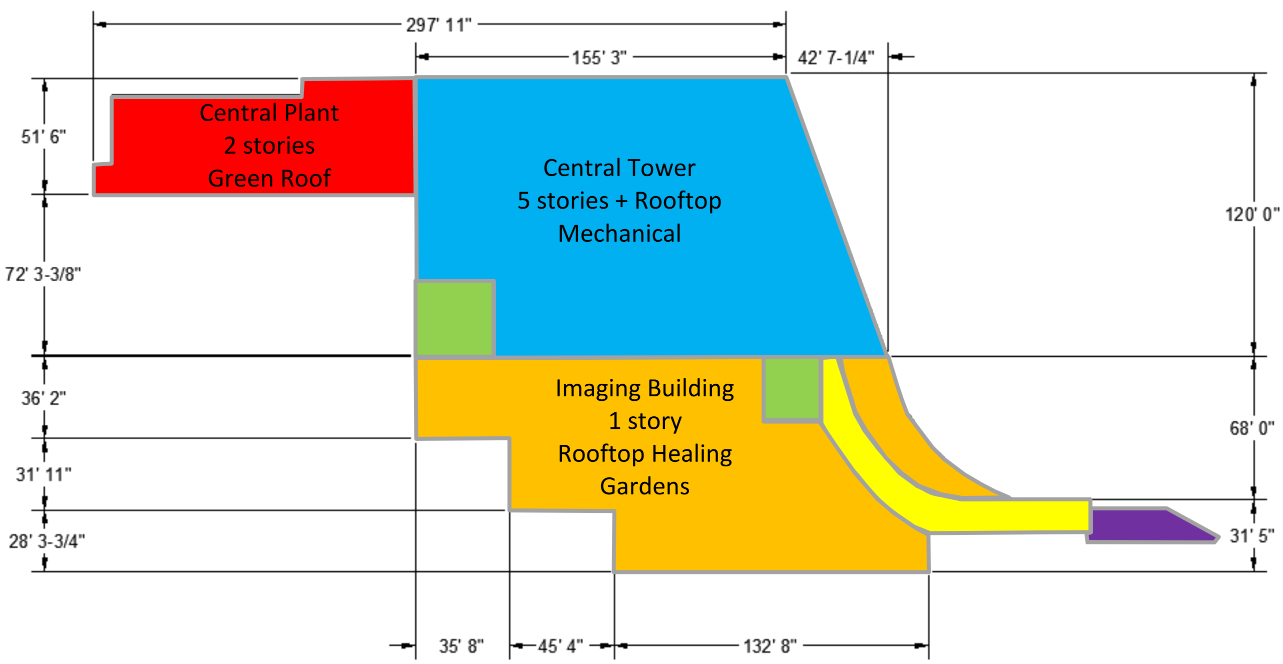


Figure 3 Building key plan showing main building divisions, dimensions, and description. Diagram key given below.

Diagram Key / Roof Elevations

- Central Tower – 72'-0"
- Central Plant – 30'-0"
- Public Access Corridor – 30'-0"
- Imaging Building – 16'-0"
- Elevator Core Shafts – 86' 6"
- Covered Entry Walkway

Foundation

Atlantic Testing Laboratories (ATL), at the request of Upstate Medical University, conducted a subsurface and geotechnical evaluation of the project site. Testing purposes were to determine the subsurface soil and ground water conditions at the site, and assess their engineering significance. Several boring tests, locations specified by architect/engineer EwingCole, were performed by ATL, to a minimum depth of 12 feet throughout the site. Subsurface soil composition beneath the initial layers of top soil and asphalt, mainly consisted of silty, gravelly, sand; silty clay and clayey silt, organic silt; debris (brick and ash); and weathered gypsum. Weathered bedrock was discovered at depths ranging from 12 to 28 feet at different boring locations. Beneath the weathered rock, lies bedrock that consists of shale, gypsum, and dolostone deposits.

ATL's discoveries resulted in their recommendation of using a structural slab supported by a deep foundation system consisting of drilled piers (caissons) bearing on dolostone bedrock. The allowable rock bearing capacity of the specified bedrock was assessed at 40 kips per square foot (40 ksf). ATL recommends a minimum pier diameter of 30 inches drilled a minimum of 24 inches into the bedrock.

Following these recommendations, EwingCole designed a foundation consisting of cast-in-place grade beams (4000 psi minimum compressive strength) resting on drilled caissons (5000 psi minimum compressive strength) with a poured slab on grade (4000 psi minimum compressive strength). All reinforcing was specified as ASTM A615 Grade 60. Grade beams range in depth from 16 to 66 inches and in width from 18 to 116 inches. Typical longitudinal bars are number eights to number tens with use of number three or number four stirrups. The slab on grade is most commonly a depth of six inches with some areas up to twelve inches thick, reinforced with number four to number six longitudinal bars. A typical grade beam section is shown below. (Figure 5)

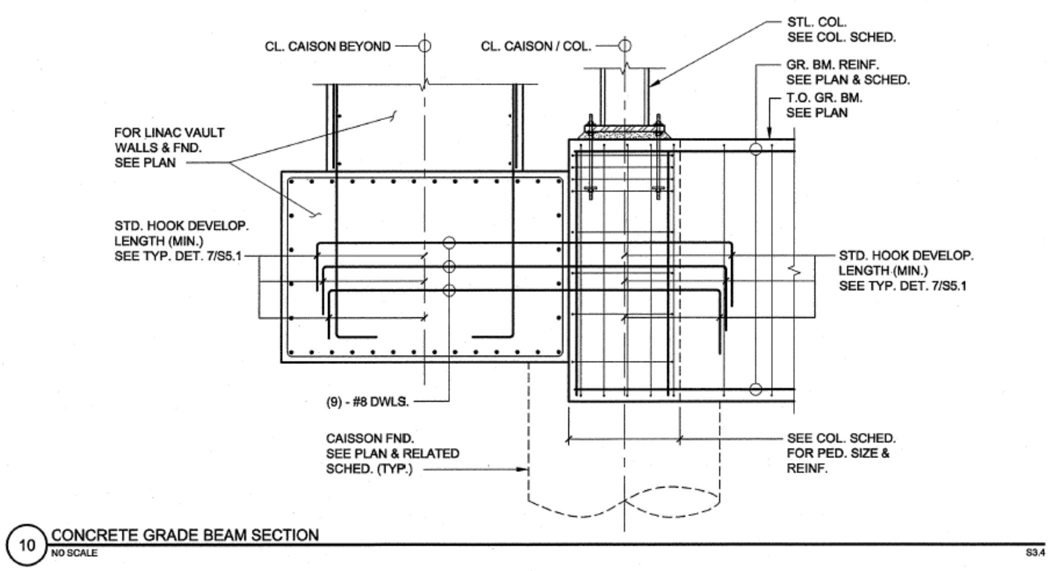


Figure 5 Typical grade beam section from sheet S3.4

Framing System

The superstructure of the Upstate Cancer Center is composed of structural ASTM A992 GR 50 wide flange steel shapes. Columns are almost exclusively sized as W12's with a few exceptions, W14's, and spliced at a height of 36 feet, mid-way through floor three. This provides a typical floor to floor height of 14 feet with a ground floor height of 16 feet. Column weights vary from 24 lb/ft to 210 lb/ft.

A typical bay size throughout the building measures 30'-0" by 30'-0" with infill beams spaced evenly at a distance of 10'-0" on center, spanning 30'-0" from girder to girder. Beams and Girders were designed compositely with the floor system through use of ¾" by 5 inch long shear studs welded on the center line of the members. In addition to this, infill beams were generally designed with a ¾" camber to compensate for excessive deflection. On a typical floor, beams range in size from W12x14's to W16x31's with the most common size being a W16x26. Girders range in size from W18x35's to W30x90's with the most common size being a W24x68 on a typical floor. Figure 6 shows a typical floor framing plan for floors two through four.

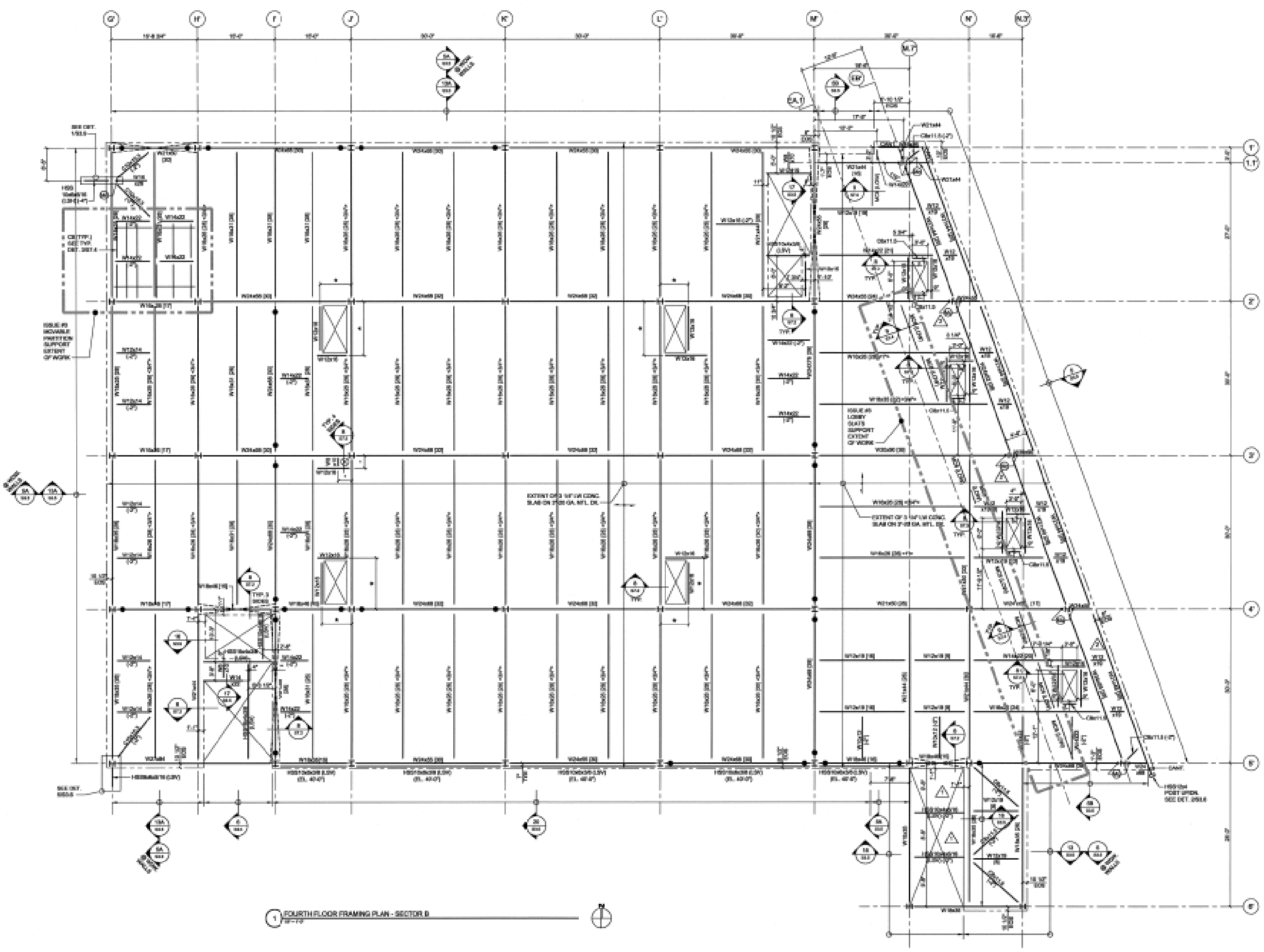


Figure 6 Typical framing layout (Central Tower) Floors two – four

Floor System

All elevated floors of the cancer center utilize a composite flooring system working integrally with the structural framing members discussed in the previous section. A typical floor assembly is comprised of 3 inch 20 gage galvanized steel deck with 3 ¼ inch lightweight concrete topping (110 pcf, 3000 psi minimum compressive strength), a total thickness of 6 ¼ inches. The deck is reinforced with ASTM A185 6x6 welded wire fabric (WWF). On the fifth floor, a 60'-0" by 30'-0", two bay, section of floor reserved for a future MRI or PET-CV unit, uses a larger topping thickness of 5 ¼ inches. The floor assembly for this particular area results as 3 inch 20 gage galvanized steel deck with 5 ¼ inch lightweight concrete topping, a total thickness of 8 ¼ inches, and ASTM A185 6x6 welded wire fabric.

All decking is specified as a minimum of two span continuous. The typical span length is approximately 10'-0" spanning perpendicular to the infill beams, typically W16x26's. In the two story central plant, housing the center's mechanical equipment, typical deck spans decrease to approximately 6'-0" to 7'-0". The decrease of span length allows the floor system to support a larger superimposed load, i.e. mechanical and electrical equipment.

Roof System

The Upstate Cancer Center uses three separate roofing assemblies; metal roof deck; concrete roof deck; and a green roof. The metal roof deck is the most commonly used assembly of the three and consists of a 60 mil EPDM membrane, 5/8 inch cover board, 4 inch minimum rigid insulation, and a gypsum thermal barrier. This composition is used in combination with a 3 inch 18 gage galvanized metal roof deck atop the five story central tower, and with a 1 ½ inch 18 gage galvanized metal roof deck atop the second floor public access corridor spanning from the Upstate Cancer Center to the Upstate Medical University Hospital. In place of the metal deck and gypsum thermal barrier, the concrete roof deck assembly employs a poured concrete deck with a minimum of 2 inches of concrete topping. This assembly is used in one location, the lower level roof supporting auxiliary mechanical equipment.

Green roofing systems have been incorporated into the design of the Upstate Cancer Center for both aesthetic and energy saving purposes. The typical green roof assembly consists of native plants grown in approximately 12 inches of top soil. Beneath the soil surface is a composition of a drainage boards, rigid insulation, a root barrier, as well as roofing membrane. All of this is supported by a composite 3 inch 20 gage galvanized steel deck with 3 ¼ inch lightweight concrete topping, a thickness of 6 ¼ inches, reinforced with ASTM A185 6x6 welded wire fabric. The green roof assemblies are located atop the two story central plant as well as the single story imaging building.

Lateral System

Lateral forces acting on the building are mainly opposed by a series of ordinary steel braced frames running in the East-West and North-South directions inside the central tower. These braced frames generally run the full height of the building, from ground level to the roof. Frames are located, surrounding the elevator cores, along the exterior wall of the building, and along interior framing lines. (See Figure 7 for frame locations, highlighted in blue)

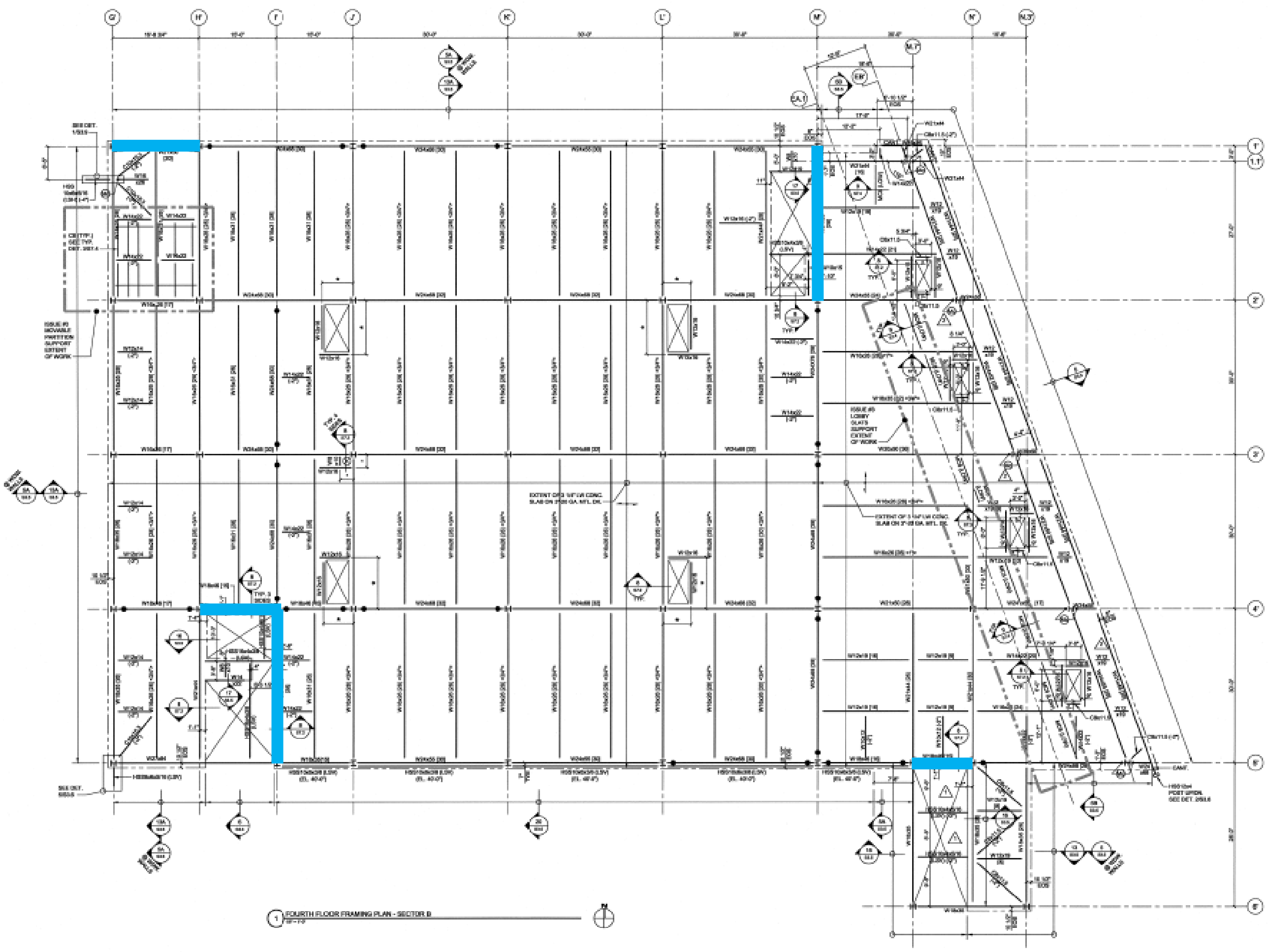


Figure 7 Location of braced frames in the central tower.

All columns used in the braced frames are W12's ranging in size from a W12x106 to a W12x210. The diagonal members used for the frames are generally W10's with W8's being used at the upper levels. Sizes of these members range from W8x31 to W10x88. The bolted connections for the frames were not detailed for seismic resistance and therefore a response modification factor of 3.0 was used for calculation purposes. Figure 8 below displays an elevation of the braced frame located long grid line I' between lines 4' and 5'.

Braced frames are used in conjunction with moment frames in the central plant. Braced frames run in the East-West direction along the exterior walls of the building, while moment frames run in the North-South direction along interior framing lines. The moment frames allow for more accessible floor space to be utilized for the movement of mechanical equipment. The brace frame composition for the central plant is similar to that described previously. The typical moment frame uses a bolted moment connection with most welding prefabricated in the shop.

Similar braced frames are used as the main lateral resisting system within the imaging building. Figure 9 displays the location of braced (blue) and moment (red) frames in the central plant as well as the imaging building.

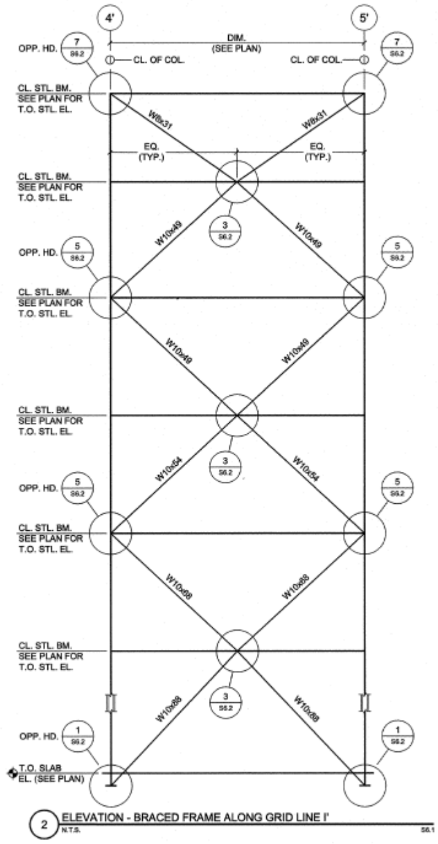
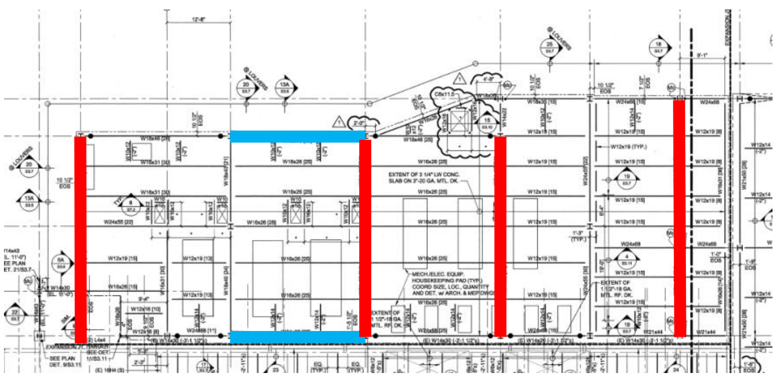


Figure 8 Braced frame elevation along grid line I' between lines 4' & 5'



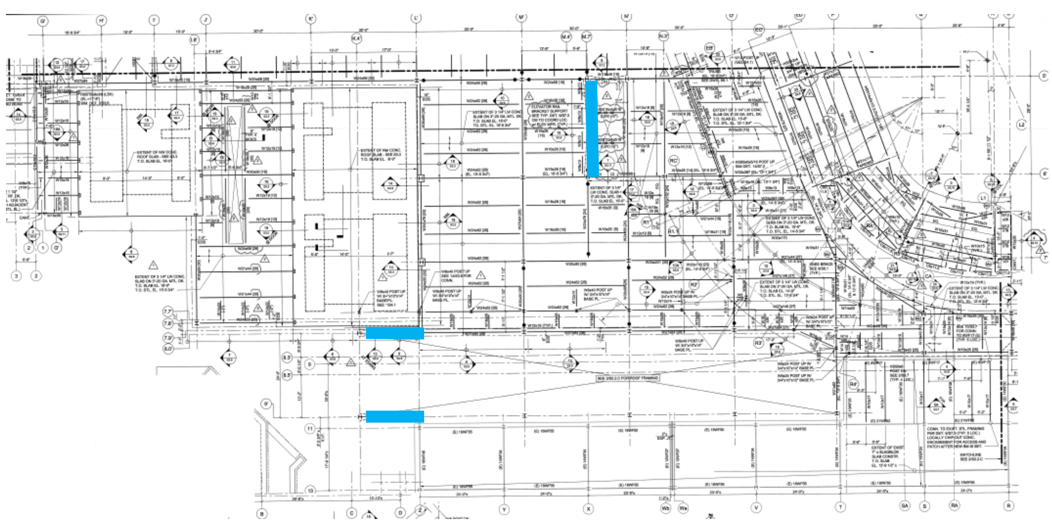


Figure 9 Floor plans showing braced (blue) and moment (red) frames locations in the central plant (above) and imaging building (right).

Design Codes and Standards

Referencing sheet G.2.1, the following codes were applicable in the design of the Upstate Cancer Center:

- 2007 Building Code of New York State (Based on IBC 2003)
    - IBC 2003 - International Building Code, 2003 Edition
    - ASCE 7-02 – Minimum Design Loads for Buildings and Other Structures, 2002 Edition
- 1997 Life Safety Code (NFPA 101)
- Sprinkler Code – NFPA 13-02
- National Electrical Code, 2005 Edition
- 2007 Plumbing Code of New York State (Based on the 2003 IPC)
- 2007 Fire Code of New York State (Based on the 2003 IFC)
- 2007 Energy Conservation Construction Code of New York State
- 2007 Mechanical Code of New York State (Based on the 2003 IMC)
- 2007 Fuel Gas Code of New York State (Based on the 2003 IFGC)
- Accessibility – ICC/ANSI A117.1-03
- 1997 AIA Guidelines for Design & Construction of Healthcare Facilities
- Health Care – NFPA 99-1996
- Fire Alarm Code – NFPA 72-02 (Amended)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

Calculations and analyses included within this report have been carried out with use of the following codes and standards:

- IBC 2009 – International Building Code, 2009 Edition
- ASCE 7-10 – Minimum Design Loads for Building and Other Structures, 2010 Edition
- AISC Manual of Steel Construction, 14th Edition, Load Resistance Factor Design (LRFD)

*NOTE: References made to 2007 Building Code of New York State for special case items.

Materials

Materials		
Structural Steel		
Item	**Grade**	**Strength, fy (ksi)**
Wide Flange Structural Shapes	A992 GR 50	50
Base Plates / Moment Plates / Spice Plates	ASTM 572 GR 50	50
Hollow Structural Steel	ASTM A 500 GR B	46
Angles / Channels / Other Plates	A36	36
Concrete		
Item	**Weight (pcf)**	**Strength, f'c (psi)**
Piers / Caissons	Normal Weight (145)	5000
Slab on Grade (SOG)	Normal Weight (145)	4000
Walls / Beams / Equipment Pads / Sidewalks	Normal Weight (145)	4000
Lower Mechanical Roof Slab Deck	Normal Weight (145)	3500
Typical Slab Deck	Light Weight (110)	3000
Masonry		
Item	**Grade**	**Strength (psi)**
Concrete Masonry Unit (CMU)	ASTM C 90	1900
Type S Mortar	ASTM C 270	1800
Fine Grout	--	3000
Cold Formed Metal Framing		
Item	**Grade**	**Strength (ksi)**
6" Cold Form Metal Framing	ASTM 653	50

Table 1 Compilation of building materials used in the design and construction of the Upstate Cancer Center.

Building Loads

The following sections convey the various loads that were tabulated for the Upstate Cancer Center and used to spot check selected member sizes and design. Loads considered acting on the structure were dead, live, snow, wind, and seismic. Values were verified against provided data for accuracy where given.

Dead Load

Dead load was calculated for the building accounting for loading that was considered permanent over the life of the building. Items that were included in the dead load determination consisted of framing members (beams and girders); columns; floor assemblies (metal deck, concrete slab, etc.); exterior wall assemblies (façade weights); mechanical, electrical, and plumbing (MEP) equipment; ceiling and floor finishings; and any permanent equipment that was specified. Values for weights of common building materials were either gathered from literature or assumed based on engineering judgment. In cases of uncertainty, values were always calculated conservatively.

Because the building is separated into three separate pieces, loads were tabulated individually for each piece. Discrepancies between listed weights are most likely due to different assumptions of superimposed dead loads. The table below (Table 2) lists typical values for various components of the structural system. It should be noted that MEP equipment, ceiling and floor finishings are considered in one category, superimposed dead load. Also, any weights particular to a specific floor, such as air handling units or medical equipment, are not included.

Dead Loads	
Description	**Load**
Beams / Girders	6.5 psf
Columns	2.25 psf
Floor Systems:	
1-1/2" Metal Roof Deck	13.74 psf
3" Metal Roof Deck	14.56 psf
3" Composite Deck w/ 3-1/4" LW Topping	46 psf
3" Composite Deck w/ 5-1/4" LW Topping	64 psf
Green Roof	154.5 psf
Facades:	
Curtain Wall Glazing	15 psf
Insulated Metal Paneling	21 psf
Brick Veneer	45 psf
Super Imposed Dead Load:	
Central Tower / Imaging Building	25 psf
Central Plant	60 psf

Table 2 Break down of typical dead loads. Note: Central Plant Superimposed Dead Load considers the weight of unaccounted mechanical equipment.

In order to determine the weight of individual floors and subsequently the total weight of the building, individual assembly weights were taken by their appropriate area and summed.

Live Load

Design live loads were specified on sheet SG.1 in accordance with the 2007 New York State Building Code. The loads given were not descriptive of their classification, but simply were listed as "Typical Floor Live Load," etc. To produce accurate and comparable loads, assumptions were made with engineering judgment regarding usage of spaces as well as future changes. Because floors four and five are left unoccupied for future expansion, they will be designed to the highest live load found on the remaining three floors to compensate for the uncertainty of occupancy. Live load values were obtained from the International Building Code, 2009 edition, using Table 1607.1, and cross-referenced with ASCE 7-10 using Table 4-1. Table 3 below summarizes the comparison of live load values chosen for design versus the live load values used for analyses in this report.

Live Loads			
Occupancy Type	Design Live Load (psf) N. Y. State Building Code (2007)	Analysis Live Load (psf) IBC 2009 / ASCE 7-10	Comments
Public Space / Typical Floor	100	100	Use of higher load to account for undesigned core floors four and five
Corridors	100	100	
Mechanical Building Spaces	250	250	Heavy manufacturing areas used for comparison
Typical Roof	45	20	Snow Load may control over roof live load
Rooftop Gardens	100	100	
Rooftop Mechanical Locations	150	125	Light manufacturing areas used for comparison

Table 3 Live load comparison between initial design and loads used in analyses in this report

Snow Load

Snow Load was calculated for the Upstate Cancer Center using ASCE 7-10 Section 7.3, flat roof snow loads. Upon viewing the ground snow load map provided in ASCE 7-10 (Figure 7-1), it was determined that Syracuse, New York requires a case study ground snow load. Figure 1608.2 of the 2007 Building Code of New York State was referenced, leading to a ground snow load of 50 psf. The appropriate factors were used in calculating a flat roof snow load of 42 psf. This load agrees with the flat roof snow load value provided on the structural drawings. A summary of snow load calculation values can be found in Table 4.

Flat Roof Snow Load Calculation	
Factor	Value
Ground Snow Load, p_g	50 psf
Exposure Factor, C_e	1.0
Temperature Factor, C_t	1.0
Importance Factor, I_s	1.2
Flat Roof Snow Load, p_f	42 psf

Table 4 Compilation of snow load calculation factors

Because the Upstate Cancer Center has varying roof heights, there is potential for snow accumulation in these regions causing a larger than expected load. Ten roof locations were chosen to figure out the worst case, maximum snow drift load. Full detailed drift calculations can be view in Appendix A. The max drift snow load of 143 psf is in compliance with the structural engineer’s note for max snow drift load of 150 psf. Below is a diagram, detailing the geometry of the max snow drift occurring between the lower roof of the central plant and the west façade of the central tower.

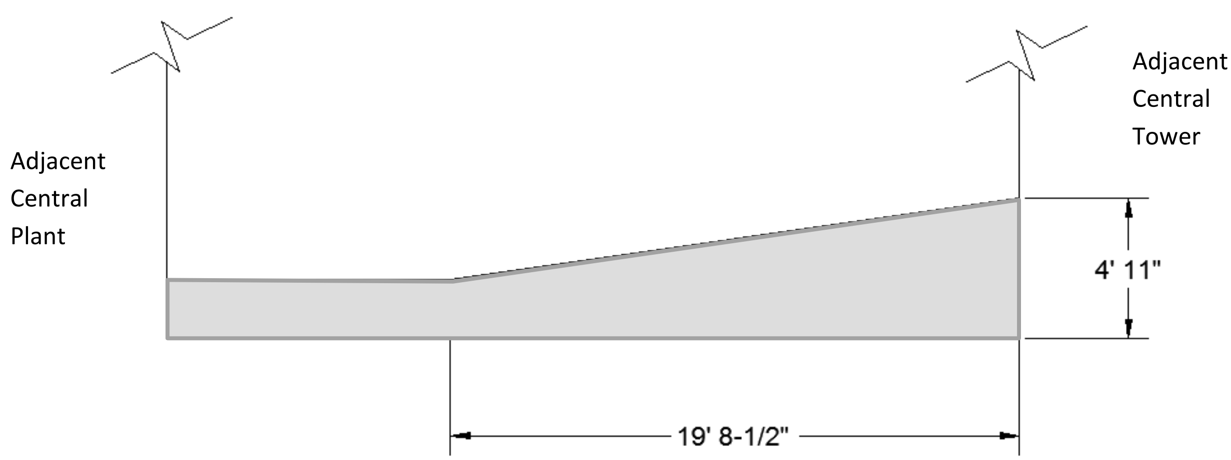


Figure 10 Snow drift geometry of max load 143 psf between Central Tower and the lower roof of the central plant

Wind Load

Wind loads were calculated for the cancer center using the Main Wind Force Resisting System (MWFRS) directional procedure for buildings of all heights specified by ASCE 7-10 Chapter 27. Because the building consists of varying roof heights, assumptions were made to simplify the geometry. The vertical geometries were broken down into two pieces, a large base consisting of two stories with a mean roof height of 30'-0", and an upper portion with a square footage approximately one third of the larger base and a mean roof height equal to 72'-0". A Google SketchUp model, provided in Figure 11 below represents the original and simplified building geometries.

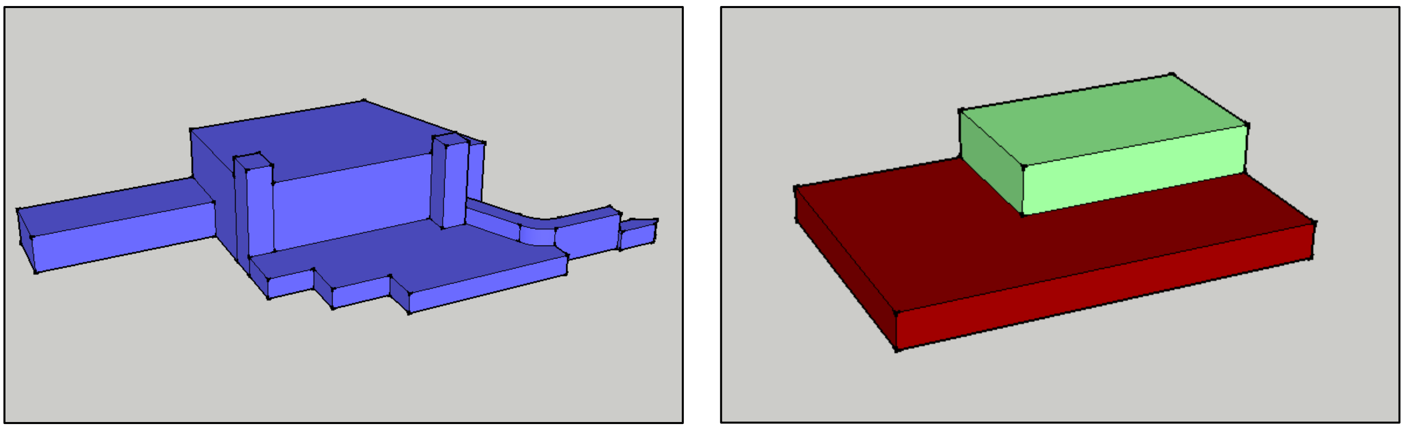


Figure 11 Google SketchUp models representing original building geometries (above left) and simplified geometry used for wind analysis (above right)

Gust effect factor calculations were carried out separately for each portion of the building. Using section 26.9.3, the building's lower bound frequency was estimated to be 1.042 Hertz. Since this value is less than 1.0 Hertz, the building can be classified as rigid by definition stated in Section 26.2. This classification was confirmed by inverting the building's period determined in the seismic analysis. The gust factors for the East-West and North-South directions of the upper portion of the building were determined by Equation 26.9-7. Since the lower portion of the building's mean roof height was less than 60'-0", it is classified as a Low-Rise Building by definition stated in Section 26.2 and permitted to be considered rigid by Section 26.9.2. Thus, the gust effect factor for the lower portion of the building was taken to be 0.85 by Section 26.9.4. Detailed calculations used to determine gust factors and other preliminary wind calculations can be found in Appendix B.

The cancer center experiences full wind pressure acting upon its exterior cladding, shown in Tables 5 and 6 and Figures 12 and 13. This lateral force is then transferred to the metal stud back-up wall, anchored to the floor slabs. From the floor slabs, load is carried to the vertical frames of the building and eventually to the foundation. Following this path, wind pressures were resolved into lateral forces acting at each story level. Visual representation of this data can be found in Tables 7 and 8 and Figures 14 and 15.

Atop the five story central tower are eighteen foot tall parapet/screen walls that surround the rooftop mechanical equipment. Wind loads for these walls were calculated in accordance with Section

27.4.5 and are tabulated in Tables 7 and 8. In addition, wind loads for roof top mechanical equipment, such as air handling units and cooling towers, have been calculated for the Upstate Cancer Center by Chapter 29. To simplify the amount of calculations, the worst case scenario was assumed for all rooftop equipment.

Wind Pressures (E-W Direction)						
Location	Level	Distance (ft)	K_z	q_z	q_h	Wind Pressure (psf)
Windward Walls	Ground	0.0	0.57	17.86	28.20	17.22
	Two	16.0	0.59	18.49	28.20	17.65
	Three	30.0	0.70	21.93	28.20	19.99
	Four	44.0	0.78	24.44	28.20	21.30
	Five	58.0	0.85	26.63	28.20	22.76
	Roof	72.0	0.90	28.20	28.20	23.80
	Parapet	90.0	0.96	30.08	-	45.12
Leeward	1-3	0.0 - 30.0	0.70	21.93	28.20	-12.35
	4-Roof	44.0 - 72.0	0.90	28.20	28.20	-13.74
	Parapet	90.0	0.96	30.08	-	-30.08
Side Walls	1-3	0.0 - 30.0	0.90	28.20	28.20	-21.86
	4-Roof	44.0 - 72.0	0.90	28.20	28.20	-21.46
Upper Roof (h=72' 0")	-	0' - 36'	0.90	28.20	28.20	-26.14
	-	36' - 72'	0.90	28.20	28.20	-26.14
	-	72' - 144'	0.90	28.20	28.20	-16.78
	-	>144'	0.90	28.20	28.20	-12.10
Lower Roof (h=30' 0")	-	0' - 15'	0.70	21.93	21.93	-20.73
	-	15' - 30'	0.70	21.93	21.93	-20.73
	-	30' - 60'	0.70	21.93	21.93	-13.27
	-	> 60'	0.70	21.93	21.93	-9.54

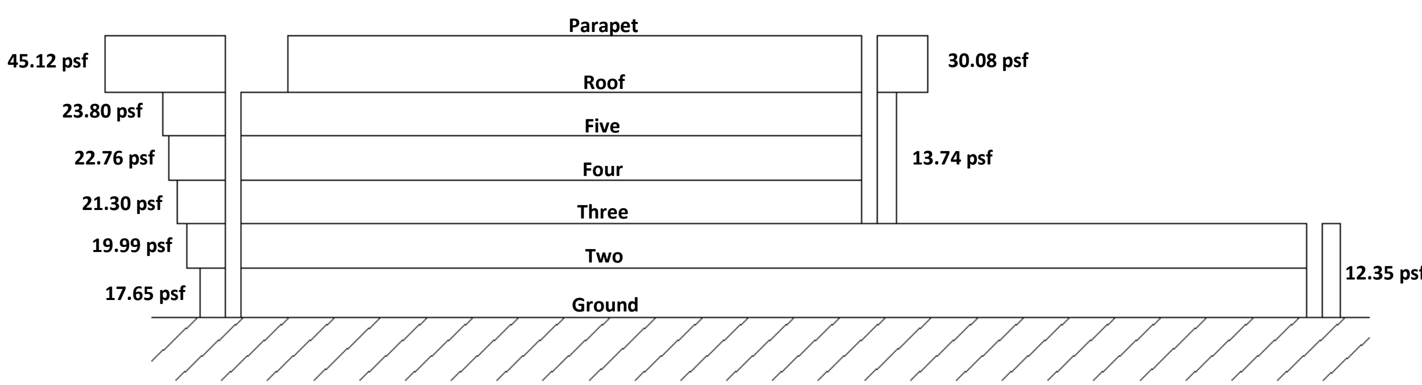


Table 5 / Figure 12 Table and Diagram of wind pressures in the East-West direction
NOTE: Roof uplift pressures displayed on the Story Force Diagram (Figure 14)

Wind Pressures (N-S Direction)						
Location	**Level**	**Distance (ft)**	**K_z**	**q_z**	**q_h**	**Wind Pressure (psf)**
Windward Walls	Ground	0.0	0.57	17.86	28.20	17.22
	Two	16.0	0.59	18.49	28.20	17.65
	Three	30.0	0.70	21.93	28.20	19.99
	Four	44.0	0.78	24.44	28.20	20.91
	Five	58.0	0.85	26.63	28.20	22.34
	Roof	72.0	0.90	28.20	28.20	23.35
	Parapet	90.0	0.96	30.08	-	45.12
Leeward	1-3	0.0 - 30.0	0.90	28.20	28.20	-17.06
	4-Roof	44.0 - 72.0	0.90	28.20	28.20	-16.50
	Parapet	90.0	0.96	30.08	-	-30.08
Side Walls	1-3	0.0 - 30.0	0.90	28.20	28.20	-21.86
	4-Roof	44.0 - 72.0	0.90	28.20	28.20	-21.07
Upper Roof (h=72' 0")	-	0' - 36'	0.90	28.20	28.20	-27.46
	-	36' - 72'	0.90	28.20	28.20	-24.72
	-	72' - 144'	0.90	28.20	28.20	-17.41
	-	>144'	0.90	28.20	28.20	-13.76
Lower Roof (h=30' 0")	-	0' - 15'	0.70	21.93	21.93	-20.73
	-	15' - 30'	0.70	21.93	21.93	-20.73
	-	30' - 60'	0.70	21.93	21.93	-13.27
	-	> 60'	0.70	21.93	21.93	-9.54

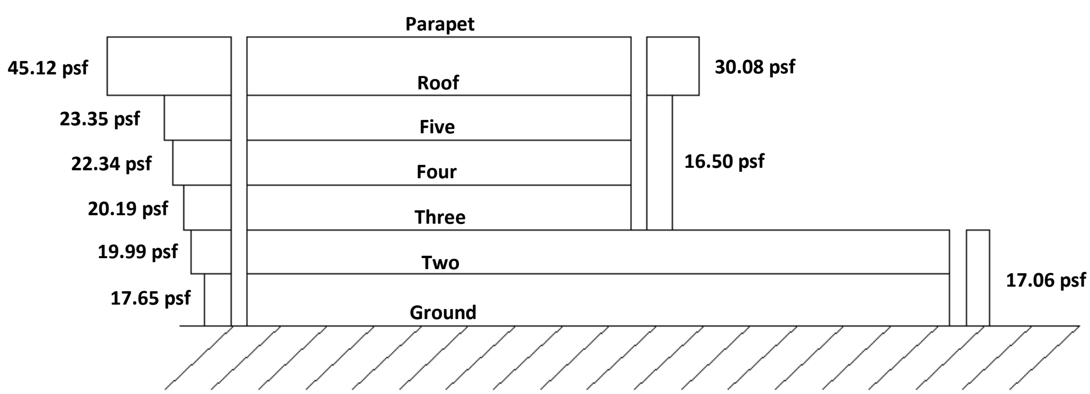


Table 6 / Figure 13 Table and Diagram of wind pressures in the North-South direction
NOTE: Roof uplift pressures displayed on the Story Force Diagram (Figure 15)

Wind Forces (E-W Direction)						
Floor Level	Elevation (ft)	Façade Area (ft^2)	Total Pressure (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.0	960.0	29.6	28.39	288.42	0.00
Second	16.0	1800.0	30.0	53.99	260.04	863.85
Third	30.0	1680.0	32.3	54.33	206.05	1629.87
Fourth	44.0	1680.0	35.0	58.87	151.72	2590.27
Fifth	58.0	1680.0	36.5	61.32	92.85	3556.36
Roof	72.0	840.0	37.5	31.53	31.53	2270.32
			Total Base Shear =	288.42		
					Total Overturning Moment =	10910.67
Parapet	90.0	2160.0	75.2	162.44	-	-
Mech. Equip.	90.0	-	-	6.50	-	-

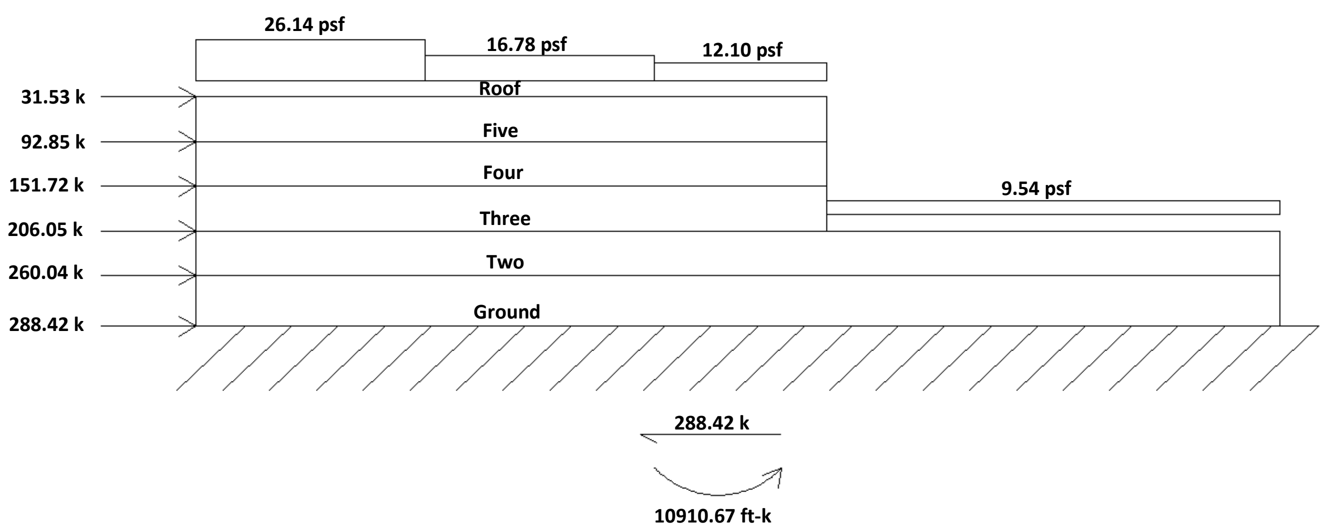


Table 7 / Figure 14 Table and diagram of wind forces in the East-West direction

Roof

Wind Forces (N-S Direction)						
Floor Level	Elevation (ft)	Façade Area (ft^2)	Total Pressure (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.0	960.0	34.3	32.91	319.20	0.00
Second	16.0	1800.0	34.7	62.48	286.29	999.62
Third	30.0	1680.0	37.1	62.25	223.81	1867.47
Fourth	44.0	1680.0	37.4	62.85	161.56	2765.45
Fifth	58.0	1680.0	38.8	65.24	98.71	3783.86
Roof	72.0	840.0	39.8	33.47	33.47	2410.00
			Total Base Shear =	319.20		
					Total Overturning Moment =	11826.41
Parapet	90.0	2160.0	75.2	162.44	-	-
Mech. Equip.	90.0	-	-	22.50	-	-

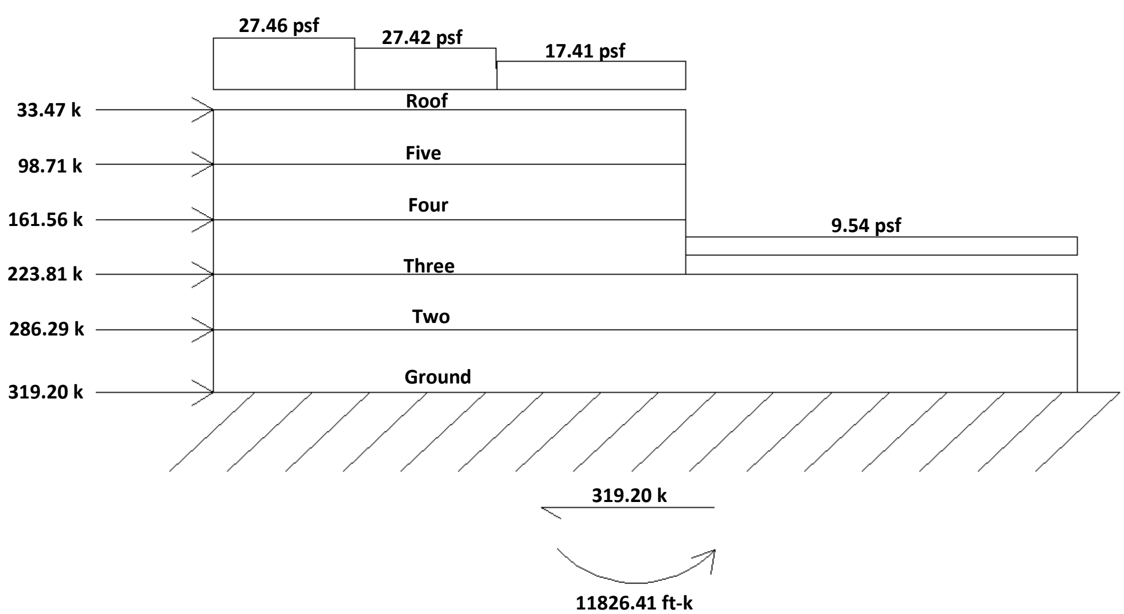


Table 8 / Figure 15 Table and diagram of wind forces in the North-South direction

In summary, the wind analysis produced base shears of 288.42 kips and 319.20 kips in the East-West and North-South directions respectively. The difference in base shears is due largely in part to the fact that the North and South facades have a larger surface area normal to the wind pressure, creating larger story forces with relatively the same external pressure.

Calculated wind pressures differed by as much as 10 pounds per square foot above the designed wind load pressures stated on Sheet SG.1. This error is mainly attributed to differences in design codes. While all the parameters agreed with what was provided in the structural drawings, the base wind speed used in the design was specified as 90 mph (ASCE 7-02) while the analysis value used was 120 mph (ASCE 7-10). A sample calculation conducted using the 90 mph wind speed as opposed to 120 mph resulted in an error of approximately 8 percent. The resulting error is assumed to be rooted in the use of simplified geometries to calculate wind pressure and coefficients.

Seismic Load

Although Syracuse, New York is not necessarily known as "earthquake prone," seismic design loads were computed to determine the controlling lateral load used for the design of the lateral system of the Upstate Cancer Center. Seismic Loads were produced following the Equivalent Lateral Force Analysis procedure outlined in Chapter 12 of ASCE 7-10. Because of the location of expansion joints, the overall building was separated into three separate buildings; the Central Tower, the Central Plant, and the Imaging Building. Each portion of the building was assumed to respond to loading independently of each other, therefore seismic analysis was conducted for each piece. This assumption is justified by the listing of separate base shear values on structural Sheet SG.1 for the Central Tower and Central Plant.

Atlantic Testing Laboratories, the geotechnical firm responsible for providing sub-surface investigation of the site, concluded that the condition of the sub grade materials resulted in categorizing the site as Site Class D, defined by ASCE 7-10. Spectral response acceleration parameters for the short and one second periods were obtained from the USGS Seismic DesignMaps application, using site latitude of 43.04 degrees and longitude of 76.14 degrees. Resulting calculations classified the site as Seismic Design Category C.

In order to determine the appropriate base shears, each building's weight need to be established. This was done through use of an excel spread sheet. Only the weights of floors elevated above the ground level were considered in the calculations of total building weight. For the Central Tower, the total building weight was approximately 9115 kips. As previously mentioned, connections used on for the lateral system of the building were not detailed for seismic resistance as defined by AISC 341, therefore a seismic response modification factor of 3.0 was used for analysis purposes. A natural period of 0.494 seconds, natural frequency of 2.025 hertz, was determined confirming that the building is a rigid structure.

Seismic forces are mass related forces that originate from the distortion of the ground and the inertial resistance of the building. Most of the cancer center's building mass is focused in the floor slabs and the structural framing of beams and girders. These floors transfer the generated seismic loads to the structural frame of the building which subsequently transfers the force to the foundation through means of the braced frames. Seismic forces were calculated for each floor using Equation 12.8-11, Vertical Distribution of Forces, and are represented in tables 9-11 and figures 16-18. Because the structural system and response modification factor are the same for either direction, only one set of calculations needed to be performed. Preliminary seismic calculations can be found in Appendix C.

Seismic Forces -Central Tower (V_b = 697.3 kips, T=.494s, k=1.0)								
Story Level (i)	**Floor Height (h_i) ft**	**Story Height (h) ft**	**Floor Weight (w) kips**	**$w*h^k$**	**C_{VX}**	**Story Forces (f_i) kips**	**Story Shear (V_i) kips**	**Overturning Moment (k-ft)**
Roof	14	72	1480	106560	0.2735	190.7	190.7	13732.01
Fifth	14	58	1936	112288	0.2882	201.0	391.7	11656.52
Fourth	14	44	1889	83116	0.2133	148.8	540.5	6545.53
Third	14	30	1905	57150	0.1467	102.3	642.7	3068.63
Second	16	16	1905	30480	0.0782	54.6	697.3	872.86
Totals			9115	389594		697.3		35875.54

Table 9 Seismic forces for the Central Tower. (Both directions)

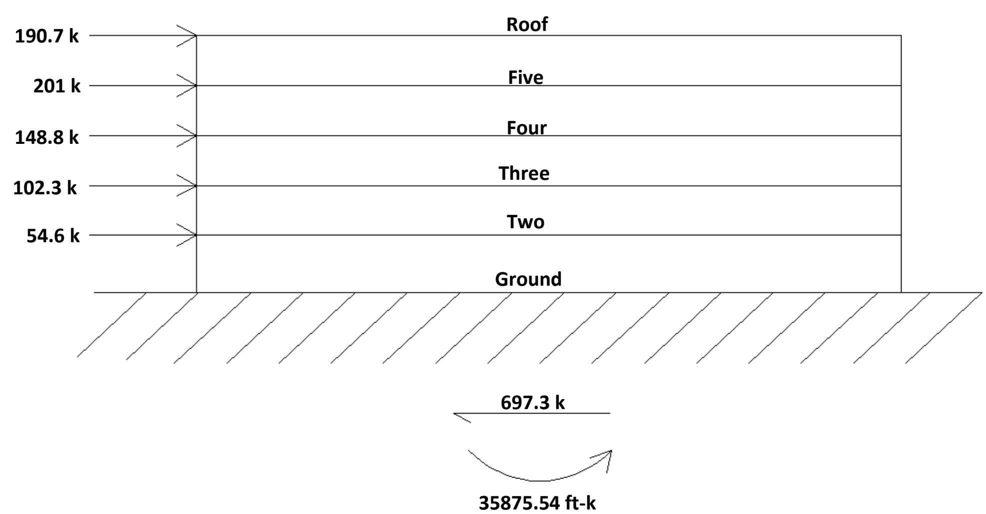


Figure 16 Diagram of Seismic forces for the Central Tower. (Both directions)

Seismic Forces - Central Plant (V_b = 212.8 kips, T=0.256s, k=1.0)								
Story Level (i)	Floor Height (h_i) ft	Story Height (h) ft	Floor Weight (w) kips	$w*h^k$	C_{VX}	Story Forces (f_i) kips	Story Shear (V_i) kips	Overturning Moment ft-k
Roof	14	30	1661.4	49842	0.7355	156.5	156.5	4695.72
Second	16	16	1120	17920	0.2645	56.3	212.8	900.42
Totals			2781.4	67762		212.8		5596.14

Table 10 Seismic forces for the Central Plant. (Both directions)

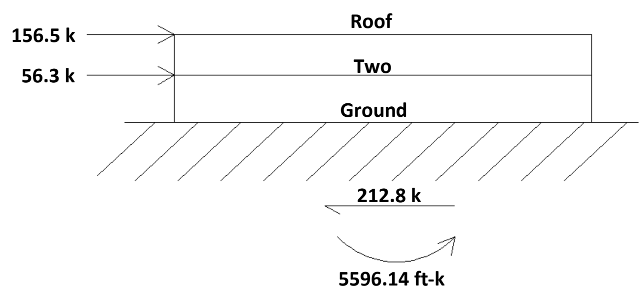


Figure 17 Diagram of Seismic forces for the Central Plant. (Both directions)

Seismic Forces - Imaging Building (V_b= 218 kips, T = 0.16s, k=1.0)								
Story Level (i)	Floor Height (h_i) ft	Story Height (h) ft	Floor Weight (w) kips	$w*h^k$	C_{VX}	Story Forces (f_i) kips	Story Shear (V_i) kips	Overturning Moment (ft-k)
Roof	16	16	2850	45600	1.0000	218.0	218.0	3488
Totals			2850	45600		218.0		3488

Table 11 Seismic forces for the Imaging Building. (Both directions)

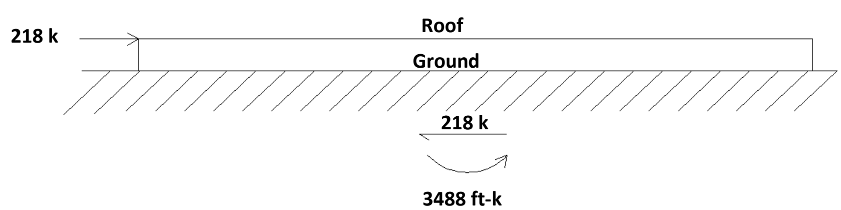


Figure 18 Diagram of Seismic forces for the Imaging Building. (Both directions)

The resulting base shear calculated through analysis for the Central Tower, 697.3 kips, was within one percent of the design base shear stated on Sheet SG.1. The base shear for the Central Plant, 212.8 kips, was accurate within fourteen percent of the design value. Error in this calculation most likely stemmed from the unknown quantity and mass of various pieces of equipment within the building. The base shear value for the Imaging Building was determined to be 218 kips. There was no value for comparison purpose provided on the drawings for this portion of the building.

Gravity Load Spot Checks

In order to assess the proper member and decking sizes used in the design of the Upstate Cancer Center, spot checks were conducted on a typical bay on floor level two defined by column lines, K' to L' and 3' to 4'. Spot checks consisted of decking, a typical beam, a typical girder, and a column. Figure 19 shows the typical bay analyzed for gravity load spot checks.

Figure 19 Diagram of typical bay chosen for gravity analysis (highlighted in green)

Decking

The most common decking system utilized throughout the cancer center, and the second floor is a composite deck consisting of a 3 inch 20 gage galvanized steel deck with 3 ¼" lightweight concrete topping. It was also noted in the initial code study that all floor decks provided will obtain a two-hour fire rating. Using the 2008 Vulcraft Steel Roof and Deck catalogue, a 3VLI20 composite deck with 3 ¼" lightweight concrete was the most suitable choice for the requirements. The max unshored construction span of 13'-3" is more than the typical 10'-0" span found in the chosen bay, and the allowable superimposed load is well above what is required by the building. In addition to this, the

3VLI20 assembly with 3 ¼" lightweight concrete topping will provide a two-hour fire rating with unprotected deck according to Underwriters Laboratories Inc. Full hand calculations can be found in Appendix D.

Beam & Girder

A typical W16x26 infill beam and W24x68 girder were check for proper strength, serviceability deflection, and construction deflections. The beam was composite design using 28 ¾" x 5" long headed shear studs to develop full strength with the concrete deck above. The number of shear studs used, 28, was more than required by analysis, but this is most likely to ensure that member will receive full strength of the concrete deck in compression. The W16x26 has enough moment capacity to carry the required loading without using shoring during construction. A ¾" camber at the center of the beam was provided to prevent excessive deflection of the member. The camber may also have been provided to counteract absolute deflection values, accounting not only for beam deflection solely, but in combination with the deflection of the composite girder. With the camber accounted for, the W16x26 was adequate for all serviceability criteria, as well as adequate for all strength requirements. Full hand calculations can be found in Appendix D.

W24x68 composite girders with 32, ¾" x 5" long headed shear studs carry the infill beam loads to the columns of the building. Loadings determined in the previous beam calculation were converted to point loads that were used to determine the adequacy of the girder. Once again, the number of shear studs provided exceeded the amount required by analysis, most likely for strength development purposes. The girders seemed more overdesigned in terms of strength than the beams were. This may have been done to compensate for inadequacies in beam design as well as redundancy. The composite girder was checked for serviceability issues, such as live load deflection, total load deflection, unshored strength, and wet concrete deflection. The W24x68 met all requirements. Full hand calculations can be found in Appendix D.

Column K' 2'

To finish the load path begun with the decking describe in the previous sections, column K'2' was checked for strength adequacy. Column K'2' was chosen because it is a typical interior column that is not part of braced frame. The column strength was calculated at floor two. At floor two, the member size is a W12x96 with a maximum axial load of 1020 kips at an unbraced length of 14'-0", which is far more than the required 762 kip load that was calculated. The unbraced length was chosen as 14'-0" because it is the typical floor to floor height in the cancer center and columns were assumed to be pinned at every floor level for analysis purposes. Since the column is not spliced until midway through the third floor, it makes sense that the column is overdesigned at the second floor. This will allow for the same column size to carry a larger load on the ground floor.

Conclusion

This investigative analysis has helped establish a better understanding of each individual structural system used, and how each system combined works as one structure. Although considered a single building from a nontechnical standpoint, structurally, the Upstate Cancer Center is really three separate buildings and must be treated so in structural design and analysis.

The majority of the effort put forth into this assignment was the determination of gravity and lateral loads on the building. With the aid of ASCE 7-10 as well as the provided structural and architectural drawings, superimposed loads could be determined more practically. Determination of building dead loads were conducted by establishing standard weights of common material, components, and assemblies, while live loads values were gathered from codes and standards such as the International Building Code as well as ASCE 7-10. These loadings would be the basis for several other calculations in this analysis and therefore needed to be resolved effectively but accurately.

Snow loading was calculated taking into consideration drifting effect and snow accumulation against areas of transitioning roof or building heights. These loads are necessary since they may be used in place of roof live load under certain loading conditions.

Typical framing members, such as infill beams, girders, and column were check for adequate strength as well as serviceability issues to reason if the correct sizing was used in the design of the cancer center. Along with the typical framing members, composite floor decking assemblies were also checked for strength and serviceability requirements. In future reports these structural components will also be checked for their adequacy in supporting lateral loading as well as gravity loading.

Lateral loads found on the building consisted of both seismic and wind loading. Wind loads were found not to control the design of the lateral system of the Upstate Cancer Center. It should be noted that the margin of error between the design wind values and those tabulated through this analysis is primarily caused by use of differing codes. Seismic base shears and overturning moments were at least twice as much as the calculated wind shear and nearly three times as much as wind overturning moment. This provides as evidence as to the fact that seismic loading will drive the design of the lateral system for the building. Seismic base shear values found through analysis were on target with the provided design values.

Appendix A: Snow Calculations

SUNY UPSTATE CANCER CENTER
AE THESIS | SNOW LOAD CALCULATION | MICHAEL KOSTICK | 1

LOCATION: SYRACUSE, NEW YORK
BUILDING TYPE: HEALTH CARE
ROOF TYPE: FLAT

ASCE 7-10 (SECTION 7.3) – FLAT ROOF SNOW LOADS

$$P_f = .7 C_e C_t I_s P_g \qquad (7.3-1)$$

$C_e = 1.0$ – EXPOSURE B; PARTIALLY EXPOSED DUE TO MECHANICAL EQUIPMENT AND SURROUNDING PARAPETS. (TABLE 7-2)

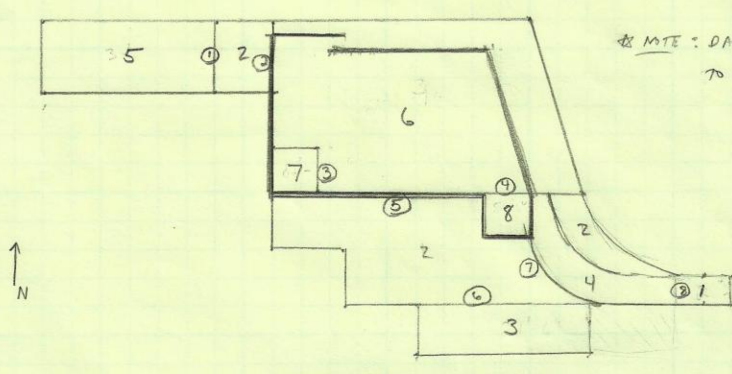
$C_t = 1.0$ (TABLE 7-3)

$I_s = 1.2$ (TABLE 1.5-2) – CATEGORY IV BUILDING

$P_g =$ C.S. – CASE-SPECIFIC (FIGURE 7-1)
= 50 psf (FROM 2007 BUILDING CODE OF NEW YORK STATE → FIGURE 1608.2)

$$P_f = .7(1.0)(1.0)(1.2)(50) = \underline{\underline{42\ psf}}$$

SNOW LOAD DUE TO DRIFTING



* NOTE: DARKENED LINES CORRESPOND TO SCREEN WALL (ELEVATION = 90'-0")

SIMPLIFIED ROOF PLAN

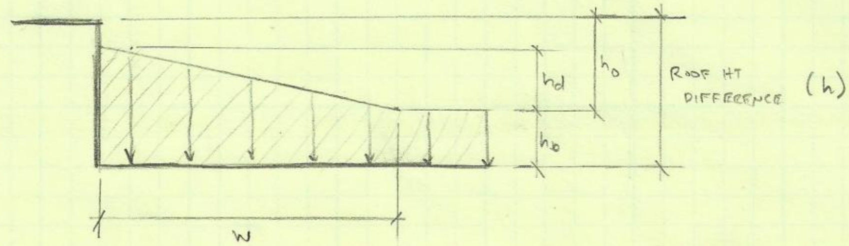
ROOF HEIGHTS
1 → 12'-9½"
2 → 17'-4"
3 → 21'-6"
4 → 28'-6"
5 → 30'
6 → 72'-¼"
7 → 82'-11½"
8 → 86'-6"

POTENTIAL DRIFT LOCATIONS
1. 5 → 2
2. 6 → 2
3. 7 → 6
4. 8 → 6
5. 6 → 2
6. 3 → 2
7. 2 → 4
8. 1 → 4

PARAPET LOCATIONS
(6 → 90'-0")

SUNY UPSTATE CANCER CENTER | SNOW LOADS | MICHAEL KOSTICK | 2
AE THESIS

SNOW ACCUMULATION DIAGRAM (FIG 7-8)



SNOW DENSITY (γ)

$$\gamma = .13p_g + 14 \leq 30\,pcf \quad (EQ\ 7.7\text{-}1)$$
$$= .13(50) + 14$$
$$\gamma = 20.5\ pcf \leq 30\ pcf \quad \text{OK} \checkmark$$

BASE SNOW ACCUMULATION HEIGHT (ft)

$h_b = P_s/\gamma$ WHERE $P_s = C_s P_f$ $\quad C_s = 1.0$ (FIGURE 7-2a) SLOPE = 0°

$= (1.0)(42.0)$

$P_s = 42.0\ psf$

$$h_b = 42.0/20.5 = 2.05\ ft$$

CALCULATION FOR LOCATION [ROOF 5 TO ROOF 2]

CHECK IF $\frac{h_c}{h_b} < .2$ $\quad h = 30' - 17.333' = 12.667'$

$h_c = h - h_b$

$= 12.667' - 2.05' = 10.62'$

$\frac{10.62'}{2.05} = 5.18 > .2 \quad \therefore$ CALCULATE DRIFT.

WINWARD DIR.

$h_d = .75(.43\sqrt[3]{l_u}\sqrt[4]{p_g+10} - 1.5)$

$l_u = 22'\text{-}10'' > 20' \quad \therefore \quad l_u = 22'\text{-}10''$

$h_d = .75(.43\sqrt[3]{22.833}\sqrt[4]{50+10} - 1.5) = 1.42\ ft$

NOTE: SEE SPREAD SHEET FOR ADDITIONAL SNOW LOAD CALCULATIONS

LEEWARD DIR

$l_u = 110'\text{-}4'' > 20' \quad \therefore \quad l_u = 110'\text{-}4''$

$h_d = .43\sqrt[3]{110.333}\sqrt[4]{50+10} - 1.5 = 4.24\,ft$

- LEEWARD CONTROLS $\rightarrow$ $h_d = 4.24\,ft < h_c$ OK ✓

$\therefore \quad w_d = 4h_d = 4(4.24) = 16.96\ ft$

FIND DRIFT SNOW LOAD (pd)

$p_d = h_d \gamma$

$= 4.24(20.5) = 86.92$ psf $= 87$ psf DRIFT SNOW LOAD

$+\ 42$ psf GROUND SNOW LOAD

129 psf

Drift Heights and Lengths

Adjacent Roofs

					Windward		Leeward			
Location		h	h_c	h_c/h_b	l_u	h_d	l_u	h_d	h_d (ft)	w_d (ft)
1	5 to 2	12.667	10.617	5.179024	22.833	1.421	110.333	4.240	4.240	16.96
2	6 to 2	54.688	52.638	25.67707	22.833	1.421	155.25	4.932	4.932	19.73
3	7 to 6	10.938	8.888	4.33561	155.25	3.699	31.729	2.289	3.699	14.80
4	8 to 6	14.479	12.429	6.062927	120	3.302	28	2.134	3.302	13.21
5	6 to 2	54.688	52.638	25.67707	68	2.539	120	4.403	4.403	17.61
6	3 to 2	4.167	2.117	1.032683	68	2.539	31.396	2.275	2.117	8.47
7	2 to 4	11.267	9.217	4.496098	213.667	4.241	20	1.749	4.241	16.96
8	1 to 4	15.708	13.658	6.662439	48.883	2.157	71	3.456	3.456	13.82

Screen Walls

Location		h	h_c	h_c/h_b	l_u	h_d	h_d (ft)	w_d (ft)
	6 to P (E-W)	17.979	15.929	7.770	177.25	3.917	3.917	15.668
	6 to P (N-S)	17.979	15.929	7.770	120	3.302	3.302	13.209
	7 to P	7.042	4.992	2.435	31.729	1.717	1.717	6.868
	8 to P	3.5	1.45	0.707	28	1.601	1.45	3.973

Total Max Drift Load

Adjacent Roofs

ϒ = 20.5 (Snow Density)

Location		h_d (ft)	p_d (psf)	w_d (ft)	p_g (psf)	Total Max Drift Load (psf)
1	5 to 2	4.24	87	17.0	42	129
2	6 to 2	4.93	101	19.7	42	143
3	7 to 6	3.70	76	14.8	42	118
4	8 to 6	3.30	68	13.2	42	110
5	6 to 2	4.40	90	17.6	42	132
6	3 to 2	2.12	43	8.5	42	85
7	2 to 4	4.24	87	17.0	42	129
8	1 to 4	3.46	71	13.8	42	113

Screen Walls

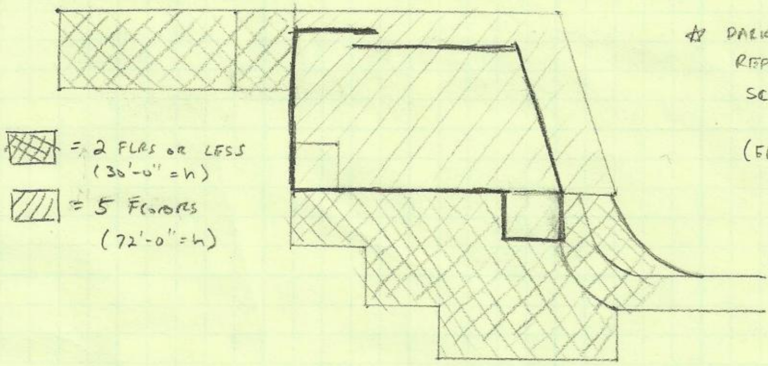
Location		h_d (ft)	p_d (psf)	w_d (ft)	p_g (psf)	Total Max Drift Load (psf)
	6 to P (E-W)	3.92	80	15.7	42	122
	6 to P (N-S)	3.30	68	13.2	42	110
	7 to P	1.72	35	6.9	42	77
	8 to P	1.45	30	4.0	42	72

Appendix B: Wind Calculations

SUNY UPSTATE CANCER CENTER | WIND ANALYSIS | MICHAEL KOSTICK | 1

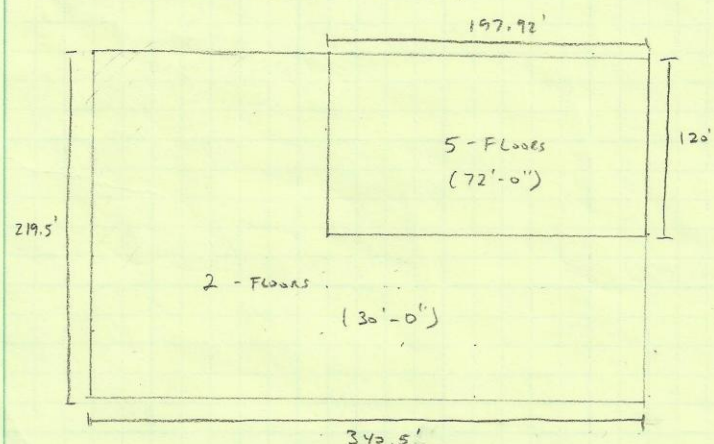
BUILDING SIMPLIFICATION

- BECAUSE OF VARYING ROOF HEIGHTS, WIND ANALYSIS IS BROKEN DOWN INTO TWO BUILDING GEOMETRIES.



☆ DARKENED LINES REPRESENT BUILDING SCREEN WALLS / PARAPETS (ELEV. TO TOP = 90'-0")

RESULTING SIMPLIFIED GEOMETRY



☆ PARAPETS ANALYZED SEPARATELY.

WIND DIMENSIONS PER FLOOR.

GROUND – 0'-0"
2ND – 16'-0"
3RD – 30'-0" } DIM (340.5' × 219.5')
4TH – 44'-0"
5TH – 58'-0"
ROOF – 72'-0" } DIM (197.92' × 120')
PARAPET – 90'-0" } CALCULATED SEPARATELY

SUNY UPSTATE CANCER CENTER | WIND LOADS MWFRS – ASCE 7-10 | MICHAEL KOSTICK | 2
AE THESIS

LOCATION: SYRACUSE, NEW YORK
BUILDING TYPE: HEALTHCARE
TOPOGRAPHY: HOMOGENOUS
TERRAIN: URBAN

☆ NOTE: WIND DESIGN W/ $h = 72'-0''$
↳ ROOF PARAPETS DESIGNED SEPARATELY (+ 90'-0")

RISK CATEGORY: IV (ASCE 7-10: TABLE 1.5-1)

BASIC WIND SPEED: $V = 120$ mph (FIGURE 26.5-1B)

DIRECTIONALITY FACTOR: $K_d = .85$ (TABLE 26.6-1)

EXPOSURE CATEGORY: EXPOSURE B (SPECIFIED IN STRUCTURAL NOTES
↳ JUSTIFIED BY ASCE 7-10 26.7.3)

TOPOGRAPHICAL FACTOR: $K_{zt} = 1.0$ (SECTION 26.8.1 – 26.8.2)

GUST FACTOR:

- DETERMINE IF FLEXIBLE OR RIGID (26.9.2)
  ↳ MEAN ROOF HEIGHT = 72'-0" > 60' ∴ NOT LOW RISE BLDG

- CHECK PROVISIONS FOR 26.9.3 (26.9.2.1)
  ↳ BLDG HEIGHT = 90' < 300' OK ✓

  ↳ $L_{EFF} = \dfrac{\Sigma h_i L_i}{\Sigma h_i}$

- E-W DIRECTION

$$L_{EFF} = \frac{(16')(297.92') + (30)(297.92) + (44+58+72)(155.25')}{16+30+44+58+72}$$

$= 185.08'$

$90' < 4(185.08')$

$90 < 740.32'$ OK ✓

- N-S DIRECTION

$$L_{EFF} = \frac{(16)(219.4) + (30+44+58+72)(120)}{16+30+44+58+72}$$

$= 127.23'$

$90 < 4(127.23')$

$90 < 508.92'$ OK ✓

∴ CAN APPROXIMATE LOWER BOUND FREQUENCY BY $n_a = 75/h$ (26.9-4)

GUST FACTOR DETERMINATION CONTINUED

· O/C IN COMPLIANCE W/ (26.9.2.1) → APPROXIMATE L.B. FREQUENCY PER SECTION (26.9.3)

↳ STRUCTURAL STEEL AND CONCRETE BUILDINGS W/ OTHER LATERAL-FORCE-RESISTING SYSTEMS:

$n_a = 75/h$ h = MEAN ROOF HEIGHT = 72'-0"

$= 75/72 = 1.042 \text{ Hz} > 1.0$

AS PER DEFINITION → BUILDING IS RIGID.

GUST FACTOR CALCULATION AS PER SECTION (26.9.4)

$$G_f = .925\left[\frac{1+1.7 g_Q I_{\bar z} Q}{1+1.7 g_v I_{\bar z}}\right]$$

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

$\bar z = .6(72)$ WHERE $h = 72'$-0"

$= 43.2 > 30 = z_{min}$ OK

$c = .30$ (TABLE 26.9-1)

$$I_{\bar z} = .30\left(\frac{33}{72}\right)^{1/6} = .287$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_{\bar z}}\right)^{.63}}}$$

B = HORIZ DIMENSION ⊥ TO WIND (ft)
$= 120'$

$h = 72'$

$L_{\bar z} = \ell\left(\frac{\bar z}{33}\right)^{\bar\epsilon}$ WHERE: $\ell = 320'$, $\bar\epsilon = 1/3.0$ (TABLE 26.9-1)

$= 320\left(\frac{43.2}{33}\right)^{1/3.0}$

$L_{\bar z} = 350.06$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{120+72}{350.06}\right)^{.63}}} = .836$$

$g_Q = 3.4$, $g_v = 3.4$

$$G_F = .925\left[\frac{1+1.7(3.4)(.287)(.836)}{1+1.7(3.4)(.287)}\right] = .83$$

FOR E-W DIRECTION CENTRAL TOWER

SUNY UPSTATE CANCER CENTER | WIND CALCULATIONS | MICHAEL KOSTICK | 4

GUST FACTOR FOR N-S DIRECTION, CENTRAL TOWER

$I_z = .287$ ☆ $h = 72'$ $B = 198'$ $L = 120'$

$$Q = \sqrt{\frac{1}{1 + .63\left(\frac{198+72}{350.06}\right)^{.63}}} = .807$$

$g_Q = 3.4$, $g_v = 3.4$

$$G_f = .925\left[\frac{1 + 1.7(3.4)(.287)(.807)}{1 + 1.7(3.4)(.287)}\right] = .81$$ FOR N-S DIRECTION CENTRAL TOWER

$\boxed{\text{LOWER PORTION OF BUILDING}}$

$h = 30'-0'' < 60'$ ∴ LOW RISE BUILDING → RIGID BUILDING

DIMENSIONS: $340.5' \times 219.5'$

☆ ALL VARIABLES ARE THE SAME → $G_f = .85$ (RIGID STRUCTURE)
$GC_{pi} = \pm .18$

PRESSURE COEFFICIENTS FOR LOWER PORTION OF BUILDING

WINDWARD $= .8$ (E-W + N-S)
SIDEWALL $= -.7$ (E-W + N-S)
LEEWARD $=$ $L/B = 340/219.5' = 1.55$ → ∴ $C_p = -.39$ [E-W DIRECTION]

$L/B = 219.5/340 = .65$ → ∴ $C_p = -.5$ [N-S DIRECTION]

ROOF:
$C_p = -.9$ $(0 - h/2)$
$C_p = -.9$ $(h/2 - h)$
$C_p = -.5$ $(h - 2h)$
$C_p = -.3$ $(> 2h)$

SAME FOR N-S + E-W DIRECTIONS

SUNY UPSTATE CANCER CENTER | WIND CALCULATIONS | MICHAEL KOSTICK | 5
AE THESIS

ENCLOSURE CLASSIFICATION: ENCLOSED (AS PER 26.2 DEFINITION)

INTERNAL PRESSURE COEFFICIENTS (GC_{pi})

$GC_{pi} = \pm .18$ (TABLE 26.11-1)

VELOCITY PRESSURE EXPOSURE COEFFICIENT (K_z or K_H)

FROM (TABLE 27.3-1) $K_z = 2.01\,(z/z_g)^{2/\alpha}$ FOR $15' \le z \le z_g$

EXP B

$\alpha = 7$ $z_g = 1200'$ $K_z = 2.01\,(15/z_g)^{2/\alpha}$ FOR $z < 15'$

EXAMPLE CALC FOR FOURTH LEVEL (44'-0")

$$K_z = 2.01\left(\frac{44}{1200}\right)^{2/7} = .78$$

DETERMINE VELOCITY PRESSURE (q_z / q_h)

$$q_z = .00256\,K_z\,K_{zt}\,K_d\,V^2 \quad \text{(EQ 27.3-1)}$$

$$q_0 = .00256(.78)(1.0)(.85)(120)^2$$

$$q_0 = 24.44\ \text{lb/ft}^2$$

DETERMINE EXTERNAL PRESSURE COEFFICIENT (C_p) (C_N)

WALL PRESSURE COEFFICIENTS, C_p (FIGURE 27.4-1)

WINWARD → $C_p = .8$ [USE w/ q_z]

SIDEWALL → $C_p = -.7$ [USE w/ q_h]

LEEWARD → $L/B = \frac{197.92}{120} = 1.65$

$C_p = -.37$ [USE w/ q_h] E-W DIR.

$L/B = 120/197.92 = .606$

$C_p = -.5$ [USE w/ q_h] N-S DIR

ROOF PRESSURE COEFFICIENTS (FIGURE 27.4-1)

N-S $h/L = 72/120 = .6$

E-W $h/L = 72/198 = .364$

HORIZ DIST FROM WINWARD	
0 - h/2 =	0' - 36'
h/2 - h =	36' - 72'
h - 2h =	72' - 144'
> 2h =	> 144'

$\theta^\circ = 0$ (FLAT ROOF)

$h/L = .364$ [EAST - WEST DIRECTION]

ROOF COEFFICIENTS

0'-36' → $C_p = -.9$
36'-72' $C_p = -.9$
72'-144' $C_p = -.5$
>144' $C_p = -.3$

$h/L = .6$ [NORTH - SOUTH DIRECTION]

ROOF COEFFICIENTS

0'-36' → $C_p = -.98$
36'-72' → $C_p = -.86$
72'-144' → $C_p = -.54$
>144' → $C_p = -.38$

THROUGH INTERPOLATION

CALCULATE WIND PRESSURE

↳ ENCLOSED AND PARTIALLY ENCLOSED RIGID BUILDINGS

FOR HEIGHT 44'-0" WINDWARD WALL E-W DIR

$$p = q\,G\,C_p - q_i(GC_{pi}) \qquad \text{(EQ 27.4-1)}$$

$$p = (24.44)(.831)(.8) - (28.2)(\pm.18) = 16.23 + 5.076$$

$$P_4 = 21.3 \text{ psf}$$

WIND PRESSURE DETERMINATION ON ROOFTOP PARAPETS

DESIGN PARAPETS IN ACCORDANCE W/ (SEC. 27.4.5)

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

q_p: VELOCITY PRESSURE @ TOP OF PARAPET

$GC_{pn} = +1.5$ (WINWARD PARAPET)
$= -1.0$ (LEEWARD PARAPET)

$$q_p = .00256(.96)(1.0)(.85)(120)^2$$
$$q_p = 30.1 \text{ psf}$$

WINDWARD DIRECTION

$$P_P = 30.1(1.5)$$

$$P_P = 45.15 \text{ psf}$$

LEEWARD DIRECTION

$$P_P = 30.1(1.0)$$

$$P_P = 30.1 \text{ psf}$$

CALCULATION OF WIND LOAD ON ROOFTOP EQUIPMENT (AHU'S) (CHAPTER 29) MWFRS

* ALTHOUGH SURROUNDED BY PARAPETS → CANT ASSUME ANY REDUCTION (SEC 29.1.4)

- BASIC WIND SPEED = 120 mph (FROM BEFORE) (V)
- WIND DIRECTIONALITY FACTOR = .85 (FROM PREVIOUS) (K_d)
- EXPOSURE CATEGORY = B (PREVIOUS)
- TOPOGRAPHIC FACTOR = 1.0 (PREVIOUS) (K_{zt})
- ENCLOSURE CALCULATIONS = ENCLOSED (PREVIOUS)
- VELOCITY PRESSURE COEFFICIENT K_z = .96 (TABLE 29.3-1 - z = 90')
- VELOCITY PRESSURE (q_z)

$$q_z = .00256\, K_z K_{zt} K_d V^2 \quad (\text{Eq } 29.3\text{-}1)$$

$$q_{90} = .00256(.96)(1.0)(.85)(120)^2 = 30.1 \text{ psf}$$

DESIGN BY SECTION 29.5

- $F = q_z G C_f A_f$ (Eq 29.5-1) WHERE:
  - q_z = 30.1 psf
  - G = .81 (N-S DIR), .83 (E-W DIR)
  - C_f = 1.3 (BOTH DIRECTIONS) (FIGURE 29.5-1)
  - A_f = 710 ft² (N-S), 200 ft² (E-W)

- $F = 30.1(.83)(1.3)(200)$

  $F = \underline{\underline{6496 \text{ lbs}}}$ (E-W DIR)

$F = 30.1(.81)(1.3)(710)$

$F = \underline{\underline{22504 \text{ lbs}}}$ (N-S DIR)

Wind Factor Criteria		
Risk Category	IV	ASCE 7-10: Table 1.5-1
Basic Wind Speed	120 mph	ASCE 7-10: Figure 26.5-1B
Directionality Factor (K_d)	0.85	ASCE 7-10: Table 26.6-1
Exposure Category	B	ASCE 7-10: Sect. 26.7.3
Topographical Factor (K_{zt})	1	ASCE 7-10: Sect. 26.8.1-26.8.2
Internal Pressure Coefficient (GC_{pi})	0.18	ASCE 7-10: Table 26.11-11

Gust Effect Factor (G_f) (ASCE 7-10: Sect. 26.9.4)		
Variable	N-S Wind	E-W Wind
B (ft)	198	120
L (ft)	120	198
h (ft)	72	72
n_a	1.042	1.042
z_{mean}	43.2	43.2
c	0.3	0.3
I_z	0.287	0.287
L_z	350.06	350.06
Q	0.807	0.836
g_Q	3.4	3.4
g_V	3.4	3.4
G_f	0.81	0.83
* **Note**: Calculated G_f only applies for upper portion of building (Floors 4-Roof). Lower structure mean roof height =30'-0" < 60'-0", and therefore can be considered rigid. (G_f = 0.85)		

Parapet (Screen Wall) Pressure (P_p) (ASCE 7-10: Section 27.4.5)		
Parameter	Windward	Leeward
Velocity Pressure, q_p	30.1 psf	30.1 psf
Pressure Coefficient, GC_{pi}	1.5	-1.0
Wind Pressure, p_p	45.15 psf	30.1 psf

External Pressure Coefficients (C_p)		
Description	N - S Wind	E-W Wind
Lower Building:		
L/B	0.65	1.55
Windward Walls	0.8	0.8
Leeward Walls	-0.5	-0.39
Side Walls	-0.7	-0.7
h/L	0.137	0.088
Roof - 0 to h/2	-0.9	-0.9
Roof - h/2 to h	-0.9	-0.9
Roof - h to 2h	-0.5	-0.5
Roof - >2h	-0.3	-0.3
Upper Building:		
L/B	0.606	1.65
Windward Walls	0.8	0.8
Leeward Walls	-0.5	-0.37
Side Walls	-0.7	-0.7
h/L	0.6	0.364
Roof - 0 to h/2	-0.98	-0.9
Roof - h/2 to h	-0.86	-0.9
Roof - h to 2h	-0.54	-0.5
Roof - >2h	-0.38	-0.3

Appendix C: Seismic Calculations

SUNY UPSTATE CANCER CENTER | SEISMIC CALCULATIONS | MICHAEL KOSTICK | 1
AE THESIS

LOCATION: SYRACUSE, NEW YORK ☆ CALCULATIONS ACCORDING TO ASCE 7-10 ☆
OCCUPANCY CATEGORY – IV – HEALTHCARE
SITE CLASS : D (SPECIFIED IN GEOTECHNICAL REPORT)
: STIFF SOIL

LAT: 43.04°
LON: -76.14°

SPECTRAL RESPONSE ACCELERATION PARAMETERS (MCE_R) FOR SHORT (S_{MS}) + 1 SEC (S_{M1}) PERIODS.

- $S_{MS} = F_a S_s$ (Eq 11.4-1)
- $S_{M1} = F_v S_1$ (Eq 11.4-2)

WHERE: • S_s = .143 [FROM USGS DESIGN TOOLS]
↳ BASE ON 0.2s SPECTRAL RESPONSE ACC.
• S_1 = .062 [FROM USGS DESIGN TOOLS]
↳ BASED ON 1s SPECTRAL RESP. ACC.

• F_a = 1.6 [TABLE 11.4-1]
↳ SITE CLASS D
• F_v = 2.4 [TABLE 11.4-2]
↳ SITE CLASS D

• $S_{MS} = (1.6)(.143) = .2288$
$S_{M1} = (2.4)(.062) = .1488$

DESIGN SPECTRAL ACCELERATION PARAMETERS

- $S_{DS} = \frac{2}{3} S_{MS}$ (Eq 11.4-3)
- $S_{D1} = \frac{2}{3} S_{M1}$ (Eq 11.4-4)

$S_{DS} = \frac{2}{3}(.229) = .153$ (SHORT PERIOD)
$S_{D1} = \frac{2}{3}(.149) = .099$ (1-SECOND PERIOD)

IMPORTANCE FACTOR

I_e = 1.50 [TABLE 1.5-2 RISK CATEGORY IV – HEALTHCARE]

SEISMIC DESIGN CATEGORY (SDC)

$S_{DS} = .153 \rightarrow .153 < .167 \rightarrow$ SDC = A [TABLE 11.6-1]

$S_{D1} = .099 \rightarrow .067 < .099 < .133 \rightarrow$ SDC = C [TABLE 11.6-2]

USE SDC = C – MORE SEVERE CASE

ANALYSIS PROCEDURE SELECTION

PER TABLE 12.6-1 → USE EQUIVALENT LATERAL FORCE ANALYSIS (12.8)
(SECTION 12.8)

SUNY UPSTATE CANCER CENTER | SEISMIC CALCULATIONS | MICHAEL KOSTICK | 2

EFFECTIVE SEISMIC WEIGHT – CENTRAL TOWER ONLY (SECT. 12.7.2)
W = 9115 kips (FROM SPREADSHEET)

SEISMIC BASE SHEAR [SEC. 12.8.1]

- $V = C_s W$ (Eq 12.8-1) WHERE C_s: SEISMIC RESPONSE COEFFICIENT
  W: EFFECTIVE SEISMIC WEIGHT.

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

WHERE $S_{DS} = .153$
$I_e = 1.5$ [11.5.1]
$R = 3$ [TABLE 12.2-1]
LATERAL SYSTEM NOT SPECIFICALLY DETAILED FOR SEISMIC RESISTANCE
* NOTED ON PLANS

$$C_s = \frac{.153}{\left(\frac{3}{1.5}\right)}$$

$$C_s = .0765$$

* CHECK C_s MAX
$T_L = 6s$ (FIG 22-12)

$$C_{s,max} = \begin{cases} \dfrac{S_{D1}}{T(R/I_e)} & \text{For } T < T_L \\ \dfrac{T_L S_{D1}}{T^2(R/I_e)} & \text{For } T > T_L \end{cases}$$

WHERE $T = T_a = C_t h_n^x$

$h_n = 72'-0''$
$C_t = .02$ $x = .75$
[TABLE 12.8-2 – "OTHER" STRUCTURAL SYSTEMS]

$T_a = C_t h_n^x$ [Eq 12.8-7]
$= .02(72)^{.75} = .494s$

$$C_{s\,MAX} = \frac{.099}{.494\left(\frac{3}{1.5}\right)} = .1002$$

$C_{s\,MIN} = .044 S_{DS} I_e \geq .01$ [Eq 12.8-5]
$= .044(.153)(1.5)$
$= .0101$

$V = (.0765)(9115) = 697.3^k$

VERTICAL DISTRIBUTION OF FORCES

$F_x = C_{vx} V$ [Eq 12.8-11] WHERE $C_{vx} = \dfrac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$ [Eq 12.8-12]

- $k = 1$ b/c $.494s < .5s$

* CALCULATIONS OF STORY FORCES FOR THIS AND OTHER BLDGS COMPLETE IN SPREADSHEET.

Initial Seismic Design Criteria		
Parameter	Value	Source
Site Class	D	Geotechnical Report
Short Spectral Response Acceleration (S_S)	0.143	USGS DesignMaps
1-sec. Spectral Response Acceleration (S_1)	0.062	USGS DesignMaps
Site Coefficient (F_a)	1.6	ASCE 7-10:Table 11.4-1
Site Coefficient (F_V)	2.4	ASCE 7-10:Table 11.4-2
Importance Factor (I_e)	1.50	ASCE 7-10: Table 1.5-2
Response Modification Factor (R)	3.0	Structural Notes
Long-Period Transition Period (T_L)	6 s	ASCE 7-10: Fig. 22-12

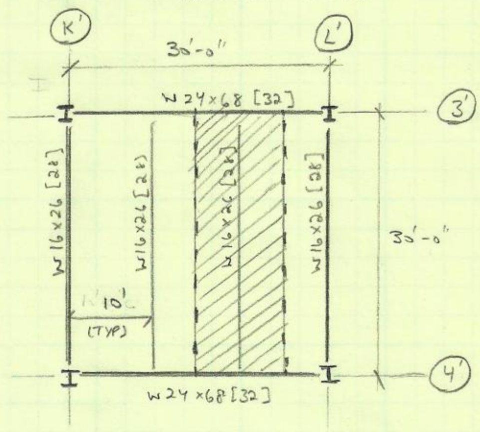
Seismic Design Parameters	
Parameter	Value
Modified Short Spectral Response Acceleration (S_{MS})	0.2288
Modified 1-sec. Spectral Response Acceleration (S_{M1})	0.1488
Design Short Spectral Response Accelerations (S_{DS})	0.153
Design 1-sec. Spectral Response Accelerations (S_{D1})	0.099
Seismic Design Category (S.D.C.)	C
Seismic Response Coefficient (C_S)	0.0765

Appendix D: Gravity Load Spot Checks

SUNY UPSTATE CANCER CENTER — AE THESIS | DECKING SPOT CHECKS | MICHAEL KOSTICK | 1

SPOT CHECKS PERFORMED ON TYPICAL BAY OF TYPICAL FLOOR

2ND FLOOR - CENTRAL TOWER



(TYP. BAY)
FLR 3

★ DECKING PRODUCTS AND VALUES TAKE S.D.I. VULCRAFT STEEL ROOF AND FLOOR DECK (2008)

LOADS ON DECK:
LL = 100 psf
SI_{DL} = 25 psf

NOTES:
- DECKING MUST SPAN MIN OF 2 SPANS
- 2HR FIRE RATING REQUIRED (UNPROTECTED DECK)

FLOOR ASSEMBLY
- 3-1/4" LW CONC ON 3"-20GA MTL DECK (COMPOSITE DECK)

TOTAL THICKNESS = 6.25"

TOTAL LOAD = 100 + 25 = 125 psf
↳ USE 3VLI20 (t=3.25") → TOTAL THICK = 6.25" [L.W. CONC.]
↳ CHECK 3-SPAN CONDITION [MAX SDI CONSTRUCTION SPAN = 13'-3"]

10' < 13'-3" OK ✓

CHECK LOADING → 10'-0" CLEAR SPAN:

149 psf > 125 psf OK ✓

- FIRE RATING
  ↳ 2HR, UNPROTECTED DECK → 3VLI
  ↳ 3 1/4" L.W. TOPPING REQUIRED OK ✓

- THE DECKING 3VLI20 w/ t=3.25 L.W. CONC. TOPPING (110 PCF) WILL SUPPORT THE SUPER IMPOSED LOAD, PROVIDE A 2HR FIRE RATING AND WORK FOR AN UNSHORED CONST. SPAN OF 13'-3", MORE THAN THE SPECIFIED 10'-0" SPAN.

$\boxed{\text{USE 3VLI20 (t=3.25) L.W. CONCRETE TOPPING @ SPAN OF 10'-0"}}$

↳ ASSEMBLY WEIGHT = 46 PSF

SUNY UPSTATE CANCER CENTER | BEAM SPOT CHECKS | MICHAEL KOSTICK | 2

CHECK COMPOSITE W16x26 [28]
- TRIB WIDTH = 10'-0"
- SPAN = 30'-0"
- CAMBER = 3/4"

★ # OF STUDS = EQUALLY SPACED 3/4" x 5" LONG ALONG BM CENTER LINE.

W16x26:
- $A_g = 7.68\ in^2$
- $I_x = 301\ in^4$
- $F_y = 50$ ksi [ALL BEAMS ARE SPECIFIED A992 GR. 50]

LOADING: DEAD LOAD:
- DECK = 46 PSF (FROM CATALOGUE)
- SI_{DL} = 25 PSF
- SELF WEIGHT = 26 lb/ft

LIVE LOAD:
TYP. FLOOR = 100 psf (NOT REDUCIBLE BY ASCE 7-10 SECT 4.7.3)

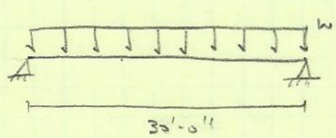
TRIB WIDTH = 10'

DL: 46 psf
25 psf
71 psf × 10' = 710 PLF
+ 26 PLF
736 PLF

LL: 100 psf
100 × 10' = 1000 PLF
1000 PLF

LOAD COMBINATION: 1.2DL + 1.6L.L. = w_u

$$w_u = 1.2(736) + 1.6(1000) = 2.483\ KLF$$



★ ASSUMING SIMPLE SPAN

$$M_u = \frac{w_u L^2}{8} = \frac{2.483(30)^2}{8} = 279.34\ ft\text{-}k$$

$$V_u = \frac{wL}{2} = \frac{2.483(30)}{2} = 37.25^k$$

$$b_R = b_L = \text{MIN OF} \begin{cases} \text{SPAN}/8 = 30(12)/8 = 45" & \leftarrow \text{CONTROLS} \\ 1/2\ \text{CLR SPAN} = 10(12)/2 = 60" \end{cases}$$

$\therefore$

$$B_{EFF} = 45 + 45 = \underline{\underline{90"}}$$

SUNY UPSTATE CANCER CENTER | BEAM SPOT CHECK | MICHAEL KOSTICK 3
AE THESIS

★ VALUES OBTAINED FROM AISC STEEL CONSTRUCTION MANUAL 14TH EDITION

ASSUME $a = 1''$

$\therefore$

$$Y2 = t - a/2 = 6.25 - 1/2 = 5.75''$$

TRY W16X26 → $\phi M_p = 166$ ft-k [FROM AISC 14TH ED TABLE 3-19]

↳ PNA = BFL $\phi M_n = 307.5$ FT-K

↳ $\Sigma Q_n = 194^k$

$\phi M_n = 307.5$ FT-K > 279 FT-K OK✓

CHECK a, Y2

$f'_c = 3000$ psi (LW CONC.)

$$a = \frac{\Sigma Q_N}{(.85) f'_c\, b_{eff}}$$

$$= \frac{194^k}{(.85)(3)(90)} \rightarrow a = .85 < 1.0 \therefore \text{OK}✓$$

$$Y2 = 6.25 - \frac{.85}{2} = 5.825 > 5.75 \text{ OK}✓$$

CHECK STUD #

$$\#\ \text{STUD}/\text{BM} = \frac{\Sigma Q_N}{Q_N}$$

$Q_n = 17.2^k$ [FROM TABLE 3-21]

↳ ASSUMING DECK IS ⊥ TO BM + 1 WEAK STUD PER RIB.

$$= \frac{194}{17.2} \rightarrow 11.28 \approx 12 \quad \therefore 24 \text{ STUDS REQ} < 28 \text{ STUDS PROVIDED}$$

OK✓

CHECK SHEAR

$\phi V_n = 106$ kips > $V_u = 37.25^k$ OK✓ [TABLE 3-2]

CHECK UNSHORED STRENGTH

$$W_u = \underbrace{1.2(46)(10)}_{\text{DECK WT}} + \underbrace{1.2(26)}_{\text{S.W.}} + \underbrace{1.6(20)(10)}_{\text{CONSTRUCTION WEIGHT}} = .903 \text{ KLF}$$

$$M_u = \frac{(.903)(30)^2}{8} = 101.6 \text{ FT-K} < 166 \text{ FT-K} \quad \text{OK}✓ \text{ FOR NO SHORING}$$

CHECK WET CONCRETE DEFLECTION

$$W_{wc} = 46(10) + 26 = .486 \text{ KLF} \qquad I = 301 \text{ in}^4$$

$$\Delta_{wc} = \frac{5wL^4}{384EI} = \frac{5(.486)(30)^4(1728)}{384(29000)(301)} = 1.015'' - .75'' = .265'' < 1.5''$$

OK✓ FOR W.C. DEFLECTION

$$\Delta_{WCMAX} = \frac{L}{240} = \frac{(30)(12)}{240} = 1.5''$$

SUNY UPSTATE CANCER CENTER | BEAM SPOT CHECK | MICHAEL KOSTICK | 4
AE THESIS

CHECK L.L. DEFLECTION

$W_{LL} = \underset{\uparrow L.L.}{(100)}(10) = 1\ KLF$

$I_{LB} = 777.5\ in^4 \rightarrow$ [FROM TABLE 3-20]
↳ CONSERVATIVE ASSUMING $Y2 = 5.75$, PNA = BFL

$$\Delta_{LL} = \frac{5(1)(30)^4(1728)}{384(29000)(777.5)} = .808'' - .75'' = .058'' < 1'' \text{ OK} \checkmark$$

FOR LIVE LOAD DEFLECTION

$$\Delta_{LL\,MAX} = \frac{(30)(12)}{360} = 1''$$

CHECK TOTAL LOAD DEFLECTION

$$W_{TL} = \underset{\uparrow DECK}{(46)(10)} + \underset{\uparrow L.L.}{(100)(10)} + \underset{\uparrow S.W.}{26} + \overset{ST_{DL}\downarrow}{25(10)} = 1.736\ KLF$$

$$\Delta_{TL} = \frac{5(1.736)(30)^4(1728)}{384(29000)(777.5)} = 1.403'' - .75'' = .653'' < 1.5''$$

$\therefore$ OK ✓

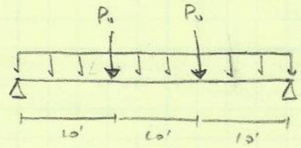
FOR TOTAL LOAD DEFLECTION

$$\Delta_{TL} = \frac{30(12)}{240} = 1.5''$$

$\boxed{\text{W16x26 w/ 28 STUDS AND 3/4'' CAMBER WORKS}}$

CHECK COMPOSITE GIRDER: W 24×68 [32] ★ 3/4", 5" LONG STUDS EQUALLY SPACED

- TRIB WIDTH = 30'-0"
- SPAN = 30'-0"
- NO CAMBER



W 24×68:
- $A_g = 20.1\ in^3$
- $F_y = 50$ ksi
- $I_y = 1830\ in^4$

$P_u = 2(37.25) = 74.5$ K

$V_U = P_U + \frac{w_u L}{2} = 74.5 + \left[\frac{(.068)(30)}{2}\right]1.2 = 75.72$ K

$M_U = P_a + \frac{wL^2}{8} = (74.5)(10) + \left[\frac{(.068)(30)^2}{8}\right]1.2 = 754.18$ FT-K

$b_R = b_L = $ MIN OF $\begin{cases} \text{SPAN}/8 = (30)(12)/8 = 45'' \leftarrow \text{CONTROLS} \\ \tfrac{1}{2}\ \text{CLR SPAN} = 30(12)/2 = 180'' \end{cases}$

∴

$b_{EFF} = 45 + 45 = \underline{90''}$

ASSUME $a = 1.0 \rightarrow Y2 = 6.25 - 1.0/2 = 5.75''$

TRY W 24×68 → $\phi M_P = 664$ FT-K [FROM TABLE 3-19]
PNA = 7 $\phi M_n = 937.5$ FT-K [INTERPOLATION]

$\phi M_N = 937.5$ FT-K > 754.18 FT-K <u>OK</u> ✓
$\Sigma Q_N = 251$ K

<u>CHECK a, Y2</u>

$a = \frac{251}{(.85)(3)(90)} = 1.094 > 1.0 \rightarrow$ RECALCULATE ϕM_N.

<u>USING a = 1.094</u>

$Y_2 = 6.25 - \frac{1.094}{2} = 5.703 \rightarrow$ TRY USING 5.5 (CONSERVATIVE)

W24×68 → $\phi M_p = 664$ FT-K [FROM TABLE 3-19]
PNA = 7 $\phi M_n = 933$ FT-K → USING Y2 = 5.5
$\Sigma Q_N = 251$

$\phi M_N = 933$ FT-K > 754.18 FT-K <u>OK</u> ✓

<u>CHECK STUD #</u>

$\#\text{STUDS}/_{BM} = \frac{\Sigma Q_N}{Q_N} = \frac{251}{17.2} = 14.6 \approx 15$

∴

30 STUDS/BM REQ. < 32 PROVIDED
<u>OK</u> ✓

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CHECK SHEAR

$\phi V_N = 295^k > 75.72^k$ OK✓ (TABLE 3-2)

CHECK UNSHORED STRENGTH

$P_U = [1.2(46)(10) + 1.2(68) + 1.6(20)(10)] \times 30 = 28.6^k$ (PT LOAD)

$M_U = P_U(a)$

$= 28.6(10^k) = 286$ k-ft < 664 k-ft OK✓ FOR NO SHORING

CHECK WET CONCRETE DEFLECTION

$P_{WC} = [(46)(10)(30)] = 13.8^k$ (PT-LOAD) $I = 1830\ in^4$

$w_{WC} = .068$ KLF

$$\Delta_{WC} = \frac{PL^3}{28EI} + \frac{5wL^4}{384EI} = \frac{(13.8)(30)^3(1728)}{28(29000)(1830)} + \frac{5(.068)(30)^4(1728)}{384(29000)(1830)}$$

$\Delta_{WC} = .457'' < 1.5''$ OK✓ FOR WET CONC DEFLECTION.

$$\Delta_{WC\,MAX} = \frac{(30)(12)}{240} = 1.5''$$

CHECK LIVE LOAD DEFLECTION

$P_{LL} = [100(10)(30)] = 30$ K (PT-LOAD)

$$\Delta_{LL} = \frac{(30)(30)^3(1728)}{28(29000)(3040)}$$

$I_{LB} = 3040\ in^4$ [TABLE 3-2a, PNA = 7, Y2 = 5.5]

$= .567'' < 1''$ OK✓ FOR LIVE LOAD DEFLECTION.

$$\Delta_{LL_{MAX}} = \frac{30(12)}{360} = 1''$$

CHECK TOTAL LOAD DEFLECTION

$P_{TL} = [(46)(10)(30) + (25)(10)(30) + (26)(30) + 100(10)(30)] = 52.1^k$

$w_{TL} = .068$ KLF

$$\Delta_{TL} = \frac{52.1(30)^3(1728)}{28(29000)(3040)} + \frac{(5)(.068)(30)^4(1728)}{384(29000)(3040)}$$

$\Delta_{TL} = 1.13'' < 1.5''$ OK✓ FOR TOTAL LOAD DEFLECTION.

$$\Delta_{TL\,MAX} = \frac{(30)(12)}{240} = 1.5''$$

$\boxed{\text{W 24x68 w/ 32 STUDS WORKS}}$

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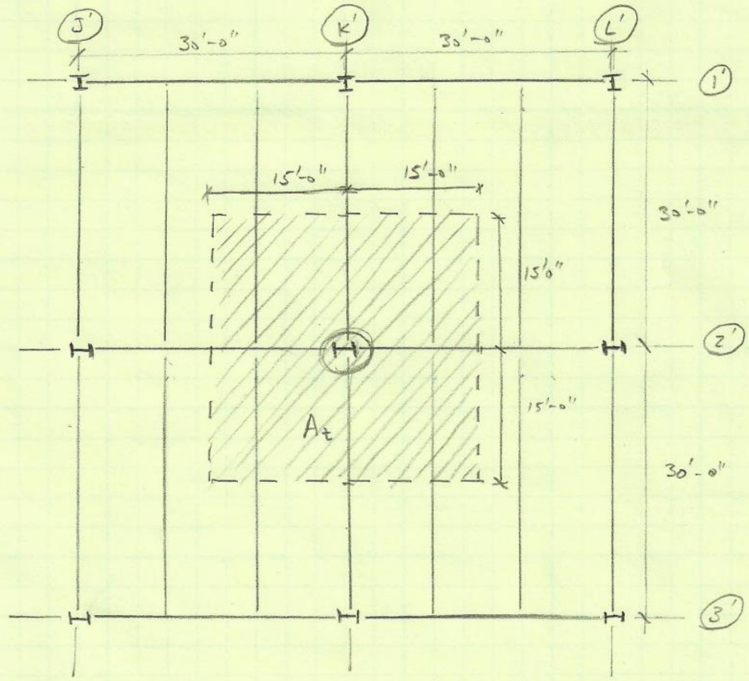
CHECK COLUMN K'2 SHOWN BELOW ON LEVEL 2 : W12×96



$A_t = 30' \times 30' = 900\ ft^2$

LOADS: PULLED FROM "BUILDING WEIGHTS" SPREADSHEET.

ROOF: 8 psf (BMS + GIRDER)
15 psf (ROOF ASSEMBLY)
25 psf (S.I. D.L.)

DL: 48 psf LL: 42 psf (SNOW LOAD)

TYP FLOOR: 6.5 psf (BMS + GIRDERS)
2.25 psf (COLUMNS)
46 psf (FLOOR ASSEMBLY)
25 psf (SI. D.L.)

DL: 80 psf LL: 100 psf (NON REDUCIBLE)

LOAD @ 2ND FLOOR = 3RD + 4TH + 5TH + ROOF → (3 FLRS + ROOF)

$P_L = (100)(900)(3\ \text{FLOORS}) = 270^k$

$P_S = (42)(900)(1\ \text{ROOF}) = 37.8^k$

$P_D = (48)(900)(1) + (80)(900)(3\ \text{FLOORS}) = 259.2^k$

$P_u = 1.2DL + 1.6LL + .5S$

$P_u = 1.2(259.2) + 1.6(270) + .5(37.8) = \underline{762^k}$

BECAUSE K'2 IS PART OF BRACED FRAME, CONSIDER PURE AXIAL STRENGTH.

$P_U = 762^k$

FLR TO FLR HEIGHT = 14'-0" (LEVEL 2)
↳ COLUMN DESIGNED AS PINNED AT EACH STORY LEVEL
↳ KL = 14'-0"

W12x96 : ϕP_n = 1020 KIPS @ KL = 14'-0"

$1020^k > 762^k$ ∴ $\boxed{\text{W12x96 IS ACCEPTABLE TO CARRY APPLIED LOADS}}$