SUNY

Upstate Cancer Center

Syracuse, New York

Technical Report 3

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Lateral System Analysis

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November 16, 2011

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Executive Summary

The intent of this report is to analyze the existing lateral force resisting system of the SUNY Upstate Cancer Center located in Syracuse, New York. Although spoken of as one building entity, the Upstate Cancer Center is really broken down into three subsections which are separated by building expansion joints. Because these joints penetrate all the way through the building, each subsection of the building is analyzed independently of each other. This report will look at only two of the three building sections, the Central Tower and Central Plant.

Lateral forces in the Central Tower are resisted by ordinary steel braced frames running both the North-South and East-West directions. In the Central Plant braced frames running in the East-West direction work in conjunction with moment frames running in the North-South direction.

Two, separate, three-dimensional structural models were created using ETABS, one for the Central Tower and one for the Central Plant. To best represent the model as it was intended to function some considerations were made to the model. All column bases were modeled as fixed, all braces and horizontal beam elements were released of moment at the member ends, and the floor structure was modeled as a rigid diaphragm with building mass lumped at its center of mass.

The computer models were used to find relative stiffness of each individual frame. To accomplish this, a 1 kip force was applied at a given story level. By calculating the amount of force each frame resists and its relative displacement, and by utilizing the relationship ($k = p / \delta$), the stiffness of each frame, and therefore the relative stiffness of each frame could be found. This helped to establish the distribution of direct shear and torsional shear to the lateral elements at a given story level.

Seismic forces were modified from the previous Technical Report 1 to include accidental torsion. Of the seven load combinations listed in ASCE7-10 Section 2.3.2, combination 4 controlled for wind loads and combination 5 controlled for seismic loads. From here, thirteen load cases, eleven for wind and two for seismic, were analyzed in ETABS to determine the governing case. Seismic loads controlled the North-South and East-West directions of the Central Tower and Central Plant. The controlling seismic load case and controlling wind load case were analyzed in ETABS considering story drifts and overall building drifts. Total drift and story drift values fell within the allowable requirements for seismic, $0.010h_{sx}$, and wind, $H/400$.

The controlling load cases were also used to determine the effects on the foundation of the Upstate Cancer Center due to overturning moment introduced through the lateral resistance system. It was found that for the controlling case, the resisting moment was much greater than the overturning moment, therefore dismissing any issues with the buildings foundation.

Finally, two bracing members and two columns were chosen for spot checks to ensure the adequacy of the members. Although bracing members were checked for purely axial load, the columns had to be checked for combined loading due to the introduction of moment from the lateral forces. All four members proved to be more than sufficient to carry the required loading.

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 formed 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)

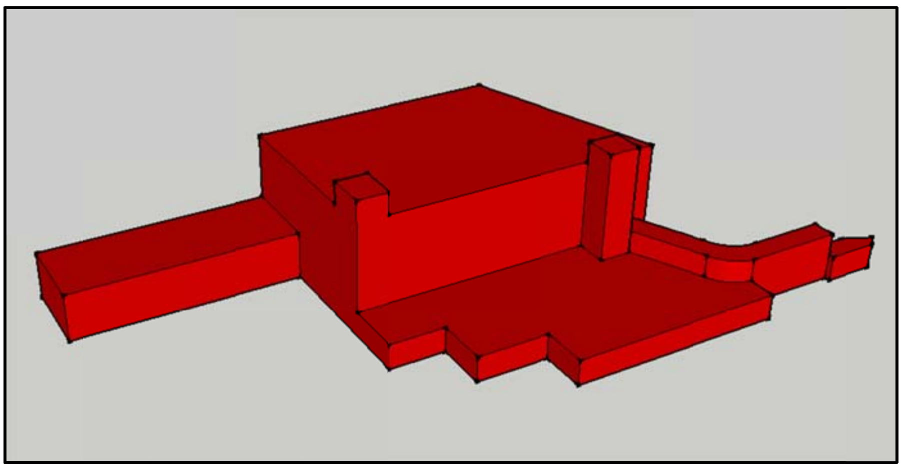
Figure 2 Exterior rendering of northeast entry façade. (Courtesy of EwingCole)

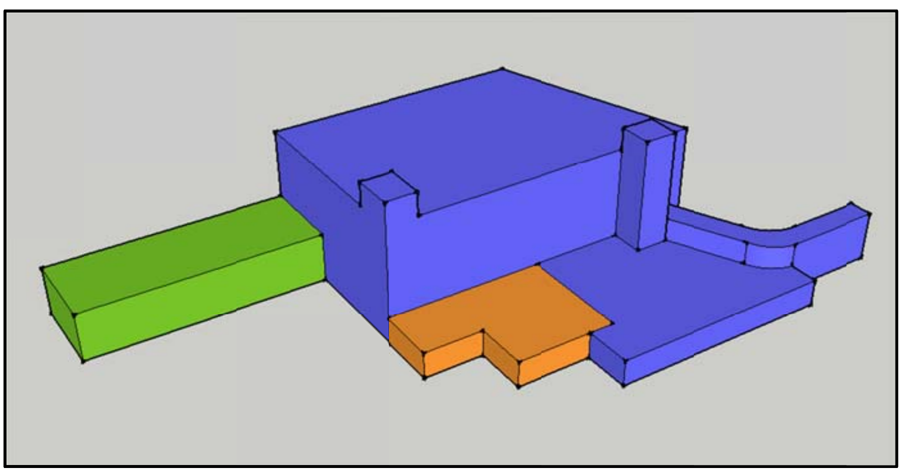
Upstate is committed to providing a comforting environment for its patients, providing amenities such as a meditation room, a boutique for gifts and apparel, and a four-season roof top healing garden. These gardens not only serve as a refreshing oasis, but also help to reduce the cooling costs for the Upstate Cancer Center, adding to Upstate's 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

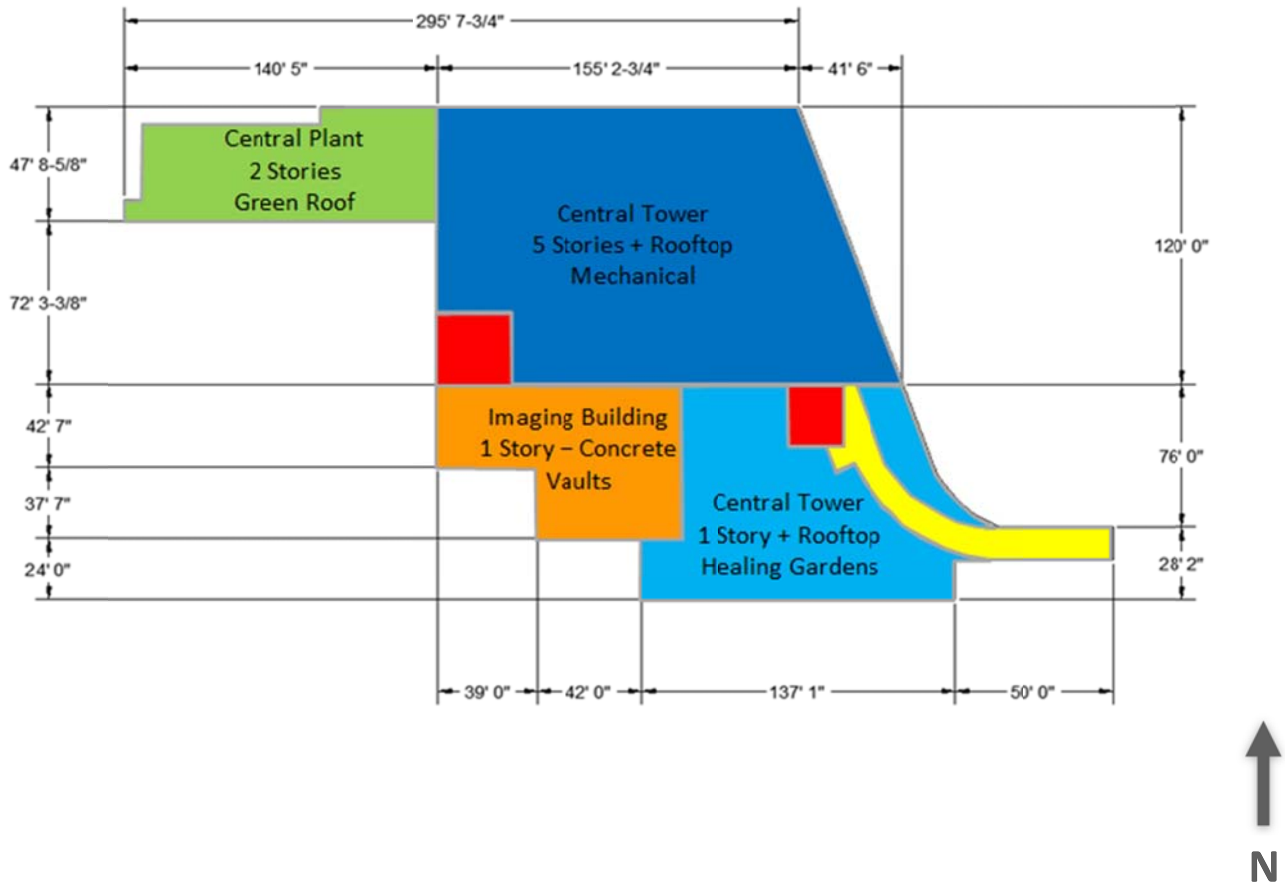
Building Key

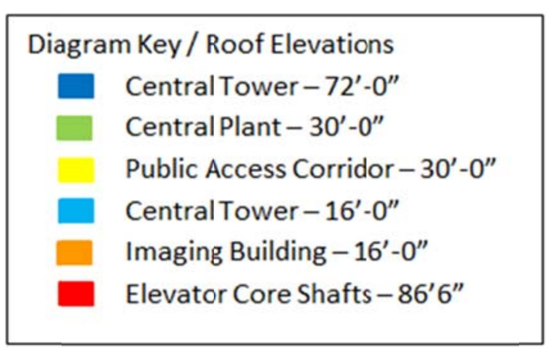
In an attempt to better represent the building geometries, a three-dimensional Google SketchUp model and a two-dimensional building plan have been created. Main divisions of the building were divided and designated based on the location of expansion joints specified on Sheet A.3.7.4. (See Appendix A) The three-dimensional model below shows the entire SUNY Upstate Cancer Center in red. Directly beneath this is a similar model displaying the three major sections of the building: the Central Tower, the Central Plant, and the Imaging Building.





Below is a two dimensional representation of the building key. Color coding has been used to distinguish between different portions of the building as well as differing roof elevations. In addition, relevant building data such as story counts and basic dimensions have also been included. Building names assigned in this section will apply to data, calculations, and descriptions later in this report.





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)

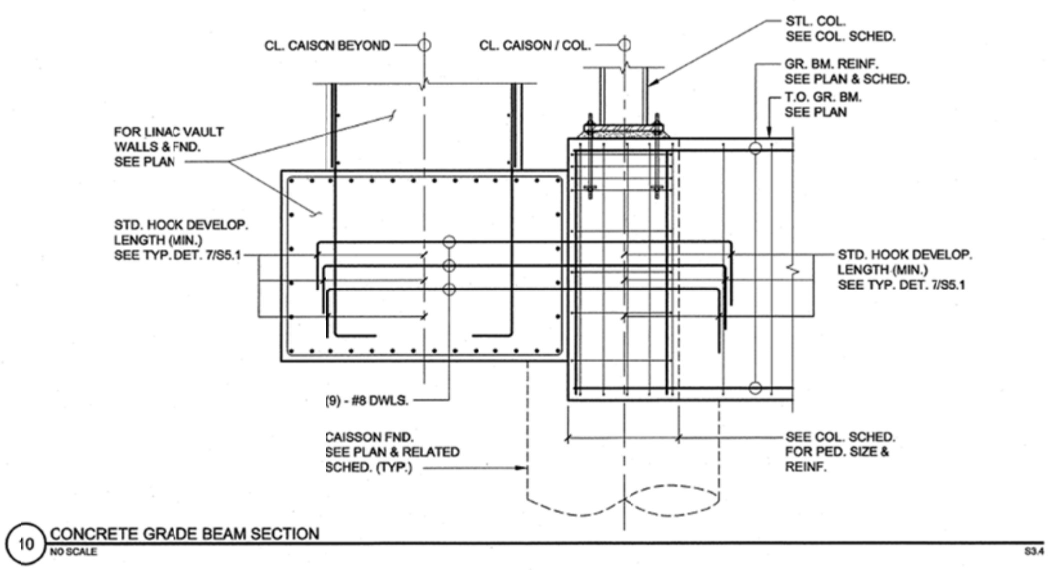


Figure 5 Typical grade beam section from sheet S3. 4
(Courtesy of EwingCole)

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 in the Central Tower.

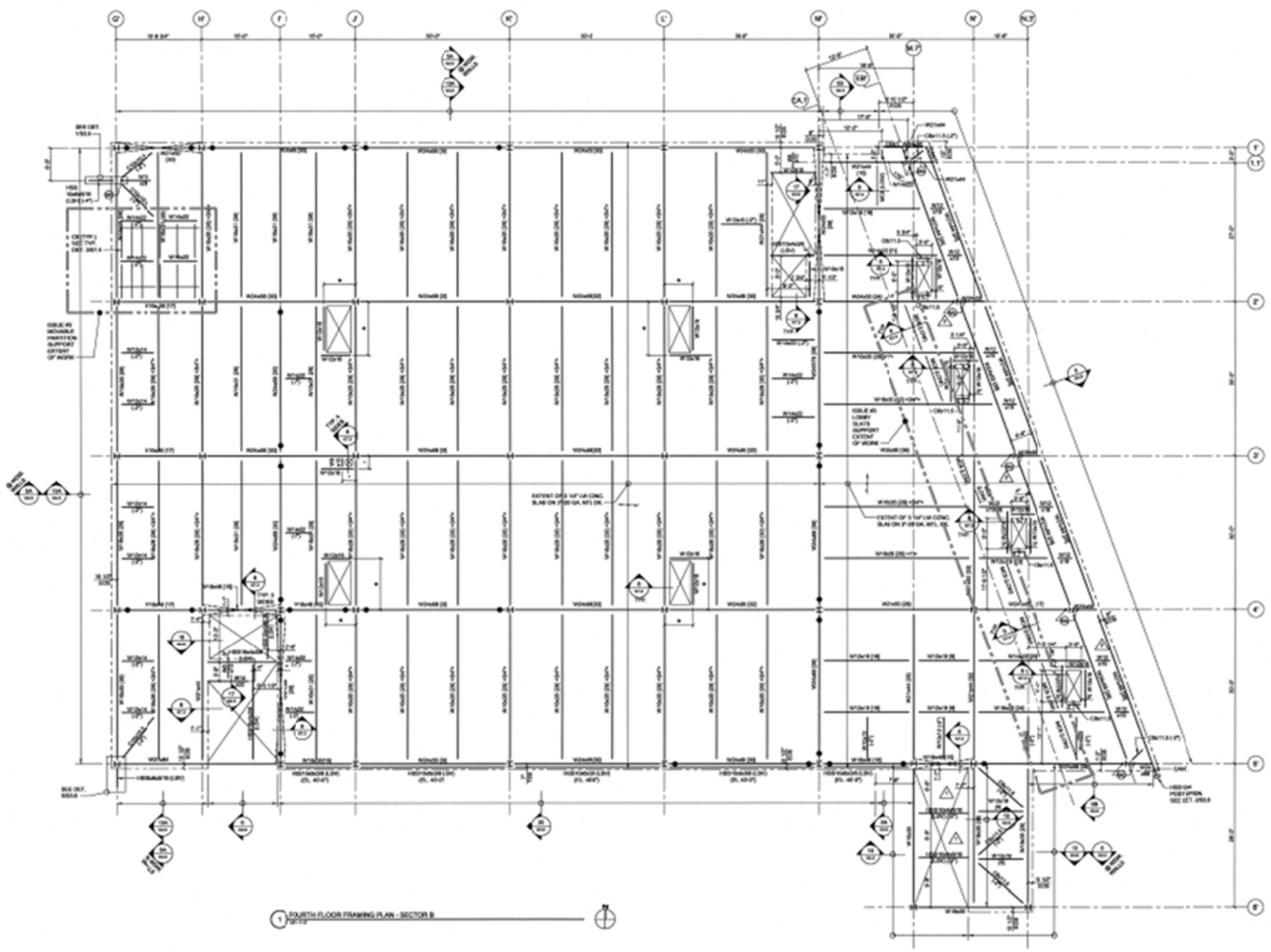


Figure 6 Typical framing layout (Central Tower) Floors two – four (Courtesy of EwingCole)

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, making a total 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

Full building expansion joints exist in the Upstate Cancer Center, effectively separating the Central Plant and Imaging Building from the Central Tower. Because of this, it is reasonable to assume that each portion of the cancer center behaves independently of each other under lateral loading, and therefore has its own unique lateral force resisting system. This report will only consider the analysis of the lateral systems in the Central Tower and Central Plant.

Lateral forces acting on the Central Tower are opposed by a series of ordinary steel braced frames running in the East-West and North-South directions. These braced frames generally run the full height of the building, from ground level to the roof. Braced frames are located, surrounding the elevator cores, along the exterior walls of the building, and along interior framing lines. Figure 7 shows the Central Tower and the location of braced frames, highlighted in blue, within the building at the first story. Heavy black lines denote the location of building expansion joints.

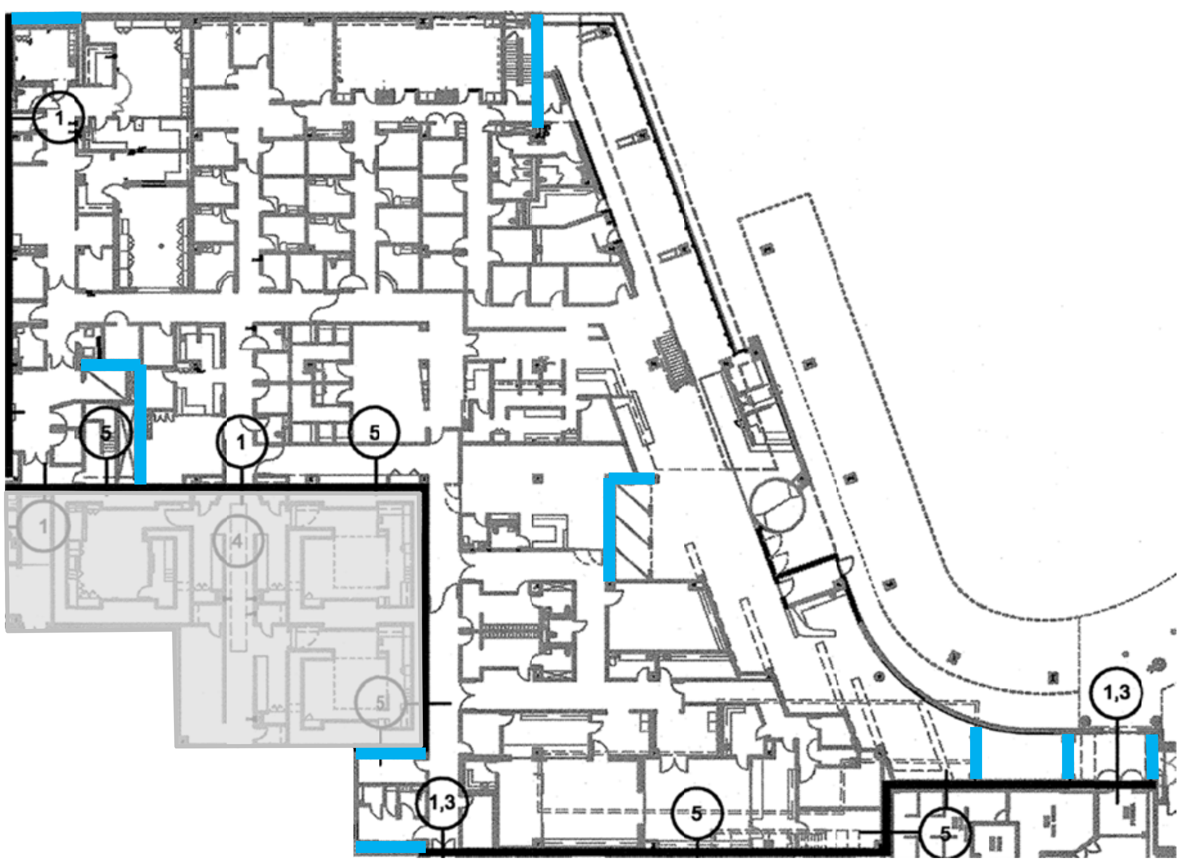


Figure 7 Location of braced frames in the Central Tower. (Courtesy of EwingCole)

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 view of the braced frame located along 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. Figure 9 shows the Central Plant with the locations of braced frames, highlighted in blue, and moment frames, highlighted in red at the first story. Heavy black lines denote the locations of expansion joints.

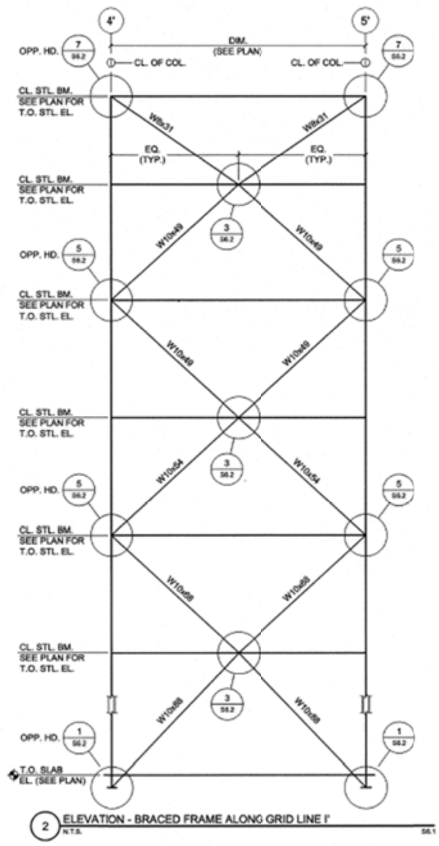


Figure 8 Braced frame elevation along grid line I' between lines 4' & 5' (Courtesy of EwingCole)

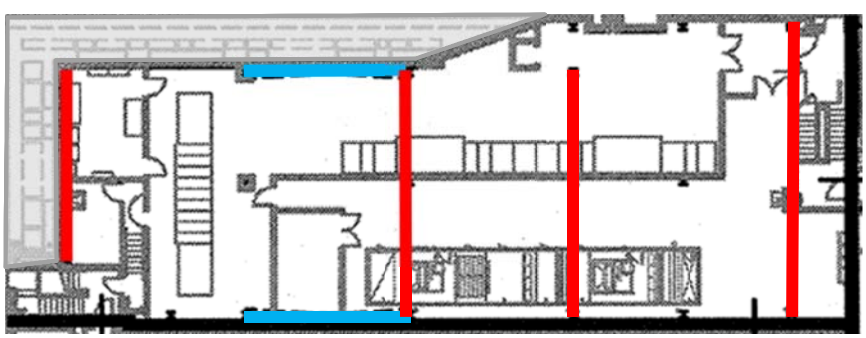


Figure 9 Floor plans showing braced (blue) and moment (red) frames locations in the central plant). (Courtesy of EwingCole)

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
  - Allowable Building Drift (Wind) = H/400 [ASCE Commentary Appendix C Section CC.1.2]
  - Allowable Story Drift (Seismic) = $0.010h_{sx}$ [ASCE Table 12.12-1]
- AISC Manual of Steel Construction, 14th Edition, Load Resistance Factor Design (LRFD)
- ACI 318-08, Building Code Requirements for Structural Concrete and Commentary
- Vulcraft Steel Roof and Floor Deck 2008

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

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 Formed 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	20 psf
Brick Veneer	40 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, in psf, were multiplied by their respective tributary area and summed for a grand total, in lbs.

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

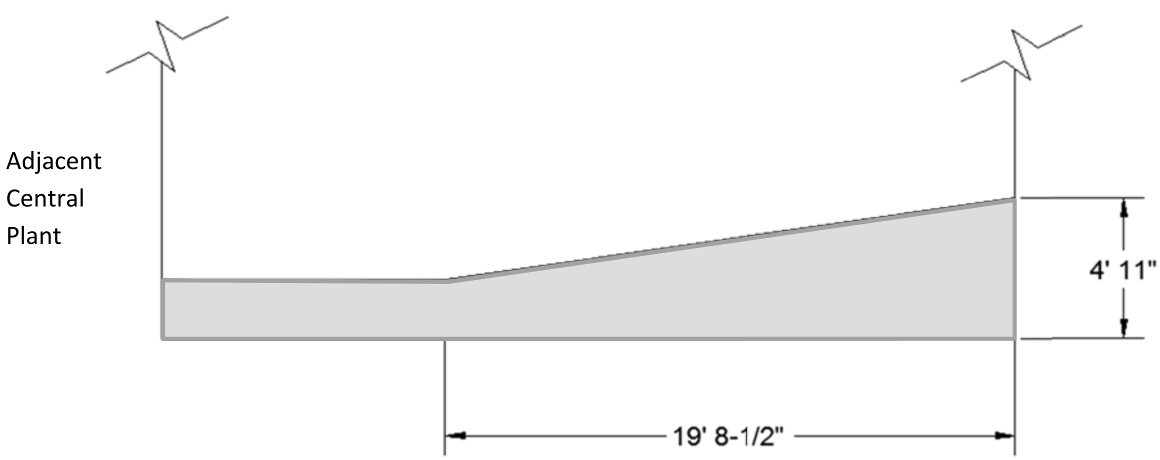
Snow Load

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.



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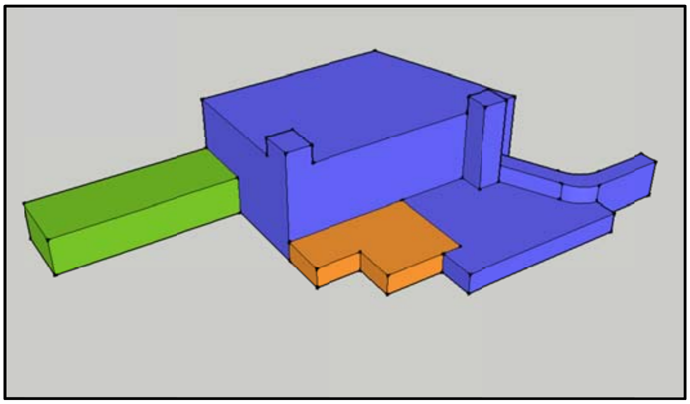
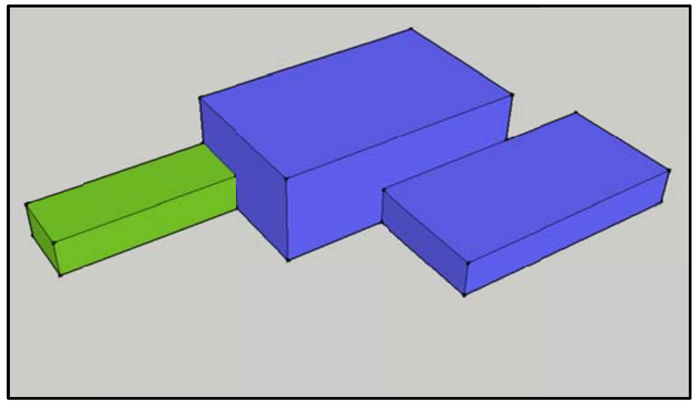
Wind Load

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. Since the building was irregular in shape, it was broken into two pieces for analysis, the Central Plant, green, and the Central Tower, blue. The geometries were further simplified by assigning a mean roof of 30'-0" to the lower level of the Central Tower and the Central Plant. The upper portion of the Central Tower had a mean roof height of 72'-0". A Google SketchUp model, provided in Figure 11, represents the original and simplified building geometries.





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 greater than 1.0 Hertz, the building can be classified as rigid by definition stated in Section 26.2. 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 C.

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 which is anchored to the floor slabs. The floor slab acts as a rigid diaphragm to transfer the load to the vertical frames of the building. Shear forces in the frames are then resisted by the foundation of the structure. 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 through 10 and Figures 14 through 17.

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. Since these parapets are not rigidly connected to any lateral elements, their forces are neglected in this lateral analysis.

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

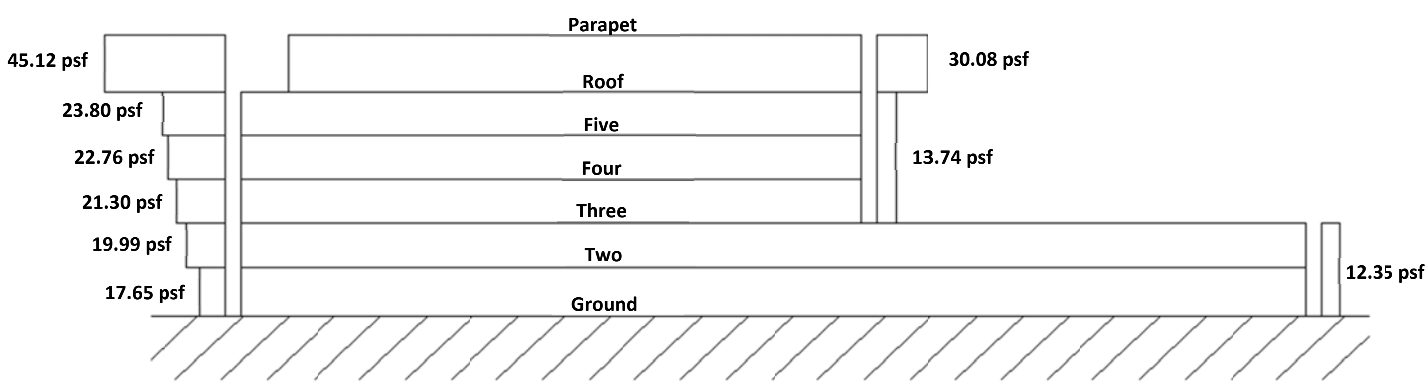


Table 5 / Figure 12 Table and Diagram of wind pressures in the East-West direction
NOTE: Roof uplift pressures displayed on the Story Force Diagram (Figure 14)

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

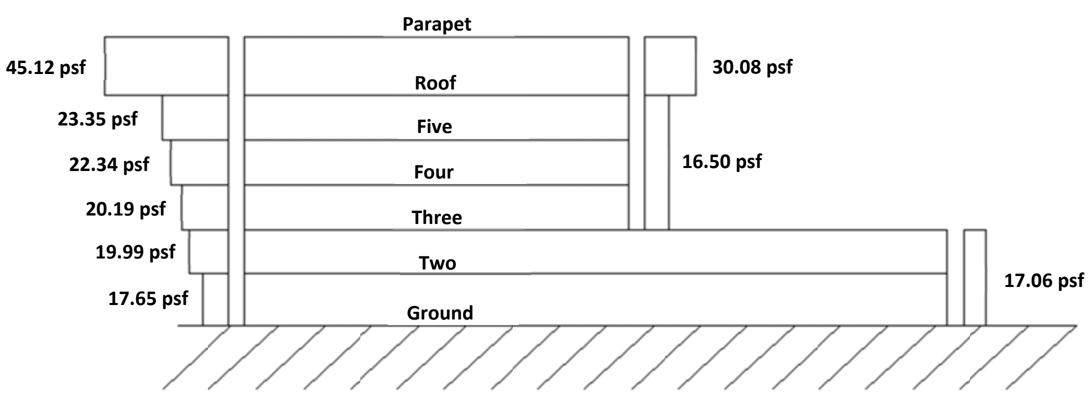


Table 6 / Figure 13 Table and Diagram of wind pressures in the North-South direction
NOTE: Roof uplift pressures displayed on the Story Force Diagram (Figure 15)

Wind Forces (E-W Direction) – Central Tower						
Floor Level	Elevation (ft)	Façade Area (ft^2)	Total Pressure (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.0	1792.0	29.6	52.99	383.36	0.00
Second	16.0	3360.0	30.0	100.78	330.37	1612.52
Third	30.0	2408.0	32.3	77.87	229.59	2336.15
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 =	383.36		
					Total Overturning Moment =	12365.62
Parapet	90.0	2160.0	75.2	162.44	-	-
Mech. Equip.	90.0	-	-	6.50	-	-

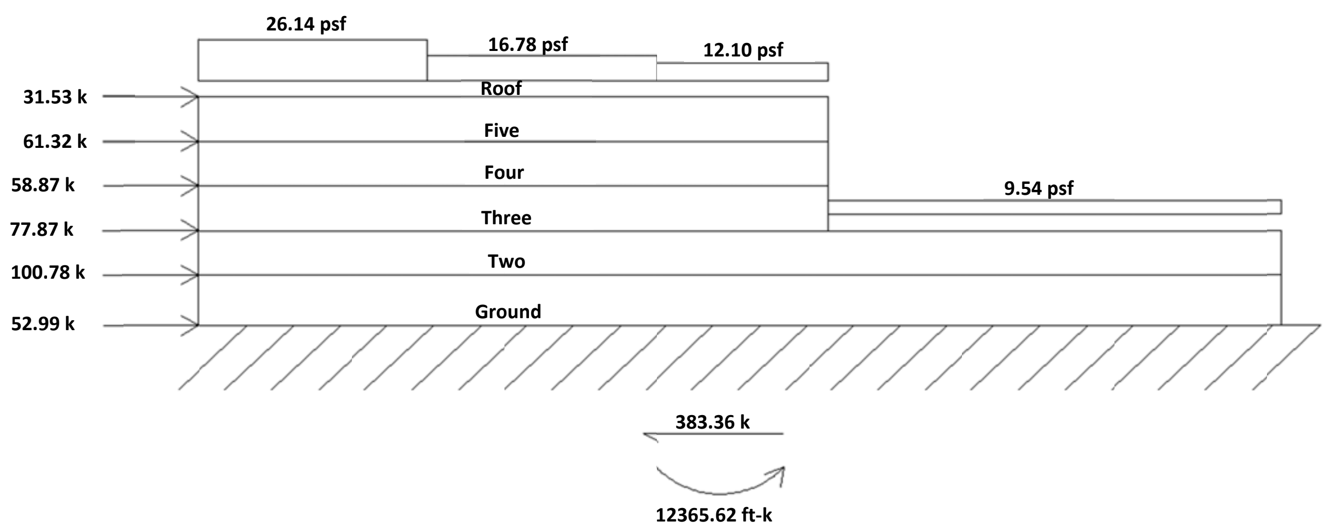


Table 7 / Figure 14 Table and diagram of wind forces in the East-West direction for the Central Tower

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	2376.0	34.3	81.46	604.29	0.00
Second	16.0	4455.0	34.7	154.63	522.83	2474.07
Third	30.0	2779.0	37.1	102.97	368.20	3089.10
Fourth	44.0	2758.0	37.4	103.18	265.23	4539.95
Fifth	58.0	2758.0	38.8	107.10	162.05	6211.84
Roof	72.0	1379.0	39.8	54.95	54.95	3956.42
			Total Base Shear =	604.29		
					Total Overturning Moment	20271.38
Parapet	90.0	2160.0	75.2	162.44	-	-
Mech. Equip.	90.0	-	-	22.50	-	-

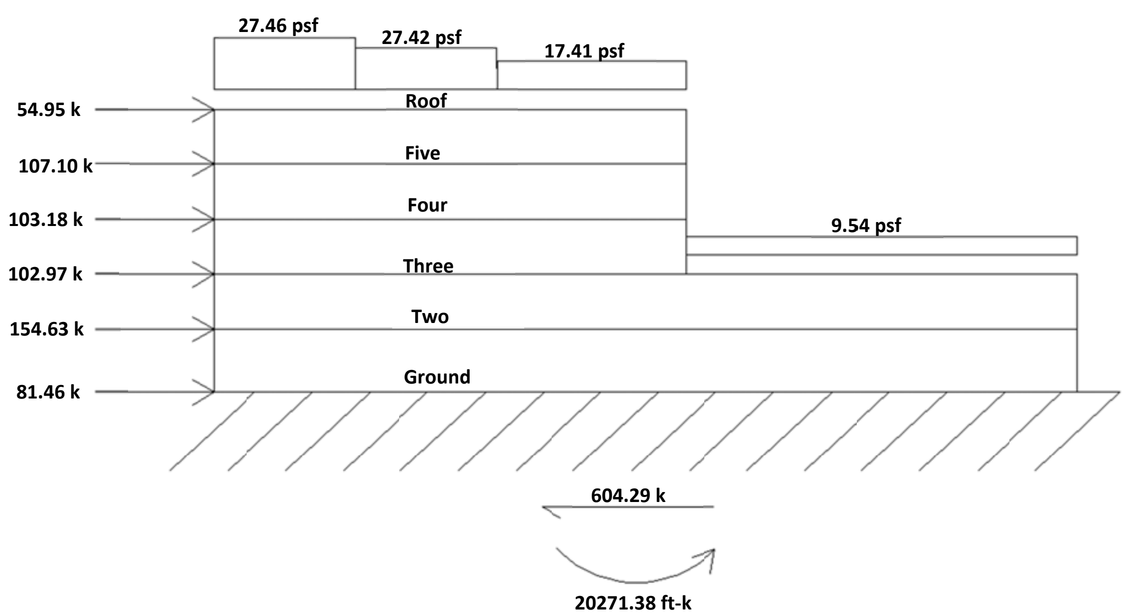


Table 8 / Figure 15 Table and diagram of wind forces in the North-South direction for the Central Tower

Wind Forces (E-W Direction) – Central Plant						
Floor Level	Elevation (ft)	Façade Area (ft^2)	Total Pressure (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.0	384.0	29.6	11.35	43.82	0.00
Second	16.0	720.0	30.0	21.60	32.46	345.54
Roof	30.0	336.0	32.3	10.87	10.87	325.97
			Total Base Shear =	43.82		
				Total Overturning Moment =		671.51
Mech. Equip.	90.0	-	-	6.50	-	-

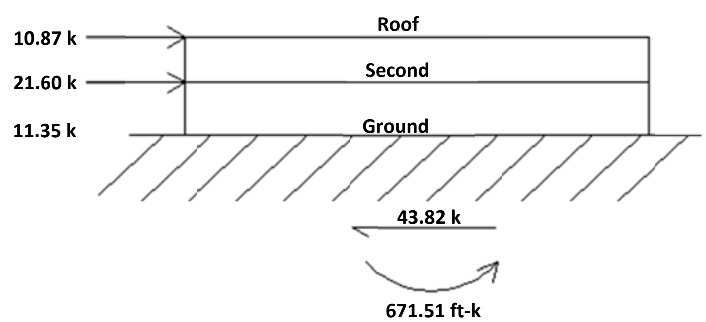


Table 9 / Figure 16 Table and diagram of wind forces in the East-West direction for the Central Plant

Wind Forces (N-S Direction) – Central Plant						
Floor Level	Elevation (ft)	Façade Area (ft^2)	Total Pressure (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.0	1056.0	34.3	36.20	139.16	0.00
Second	16.0	1980.0	34.7	68.72	102.96	1099.58
Roof	30.0	924.0	37.1	34.24	34.24	1027.11
			Total Base Shear =	139.16		
				Total Overturning Moment =		2126.69
Mech. Equip.	90.0	-	-	22.50	-	-

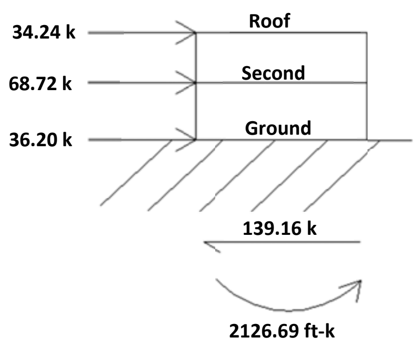


Table 10 / Figure 17 Table and diagram of wind forces in the North-South direction for the Central Plant

In summary, the wind analysis produced base shears of 383.36 kips and 604.29 kips in the East-West and North-South directions respectively of the Central Tower. 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. Base shears in the Central Plant were much less than those found in the Central Tower. The base shears were 43.82 kips and 139.16 kips in the East-West and North-South directions respectively.

Internal pressures were neglected in wind calculations because they are equal and oppose each other, essentially negating themselves. 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. Nonetheless, the calculated wind pressures and resulting forces were taken to be acceptable.

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 9999 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.

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 act as rigid diaphragms transferring the generated seismic loads to the braced frames of the building which subsequently transfers the force to the foundation.

Seismic forces were altered slightly from the values obtained in Technical Report 1. The Seismic Response Coefficient, C_s, was recalculated considering the upper limit for calculated periods, ASCE 7-10 Section 12.8.2, and the period values obtained from the computer model. Forces were calculated for each floor using Equation 12.8-11, Vertical Distribution of Forces, and are represented in tables 11-12 and figures 18-19. Because the Seismic Response Coefficient is the same for both directions of loading, only one set of calculations needed to be performed. Accidental torsion was considered for each building assuming an offset of the center of mass from its actual location. This offset was taken at five percent of the building dimension perpendicular to the applied forced. Torsional amplification, Ax, was also taken into account and addressed. Calculations of accidental torsion can be found in tables 13-14 and 15-16. All hand derived seismic calculations can be found in Appendix D.

Direct Shear

Seismic Forces -Central Tower (V_b = 652 kips, T=.8398s, k=1.17)								
Story Level (i)	**Story Height (h_i) ft**	**Floor 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	1345	200355.5	0.2728	177.8	177.8	12804.72
Fifth	14	58	1777	205540.9	0.2798	182.4	360.3	10581.87
Fourth	14	44	1730	144839.1	0.1972	128.6	488.9	5656.85
Third	14	30	1862	99589.04	0.1356	88.4	577.3	2651.97
Second	16	16	3285	84208.47	0.1146	74.7	652.0	1195.95
Totals			9999	734533		652.0		32891.36

Table 11 Seismic forces for the Central Tower. (Both directions)

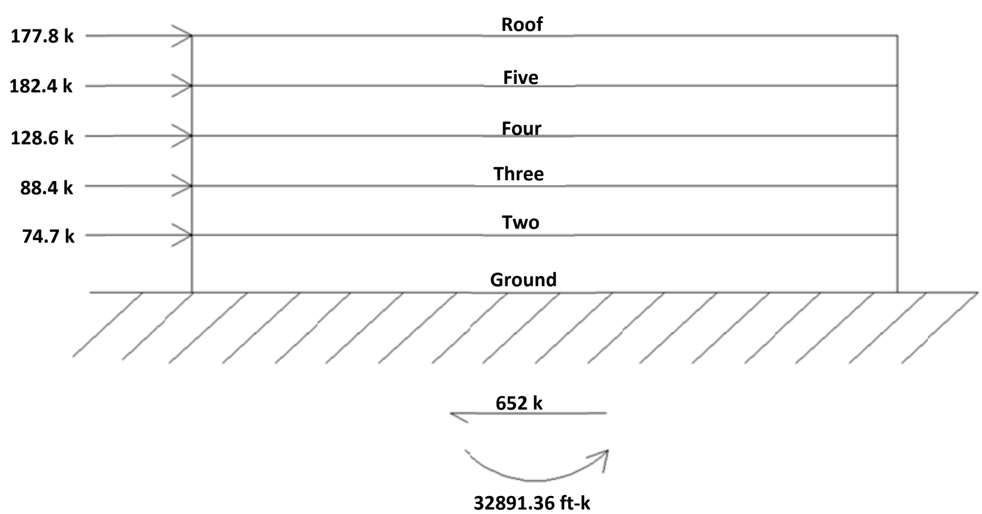


Figure 18 Diagram of Seismic forces for the Central Tower. (Both directions)

Seismic Forces - Central Plant (V_b = 212.8 kips, T=0.256s, k=1.0)								
Story Level (i)	Story Height (h_i) ft	Floor 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	156.7	156.7	4700.13
Second	16	16	1120	17920	0.2645	56.3	213.0	901.26
Totals			2781.4	67762		213.0		5601.40

Table 12 Seismic forces for the Central Plant. (Both directions)

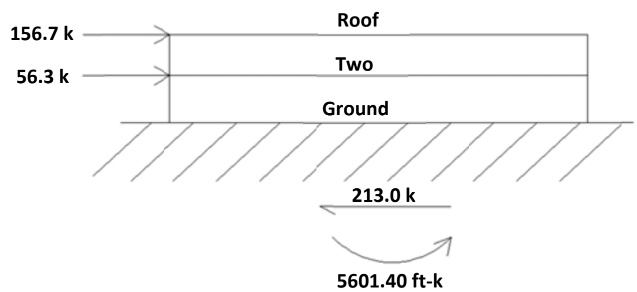


Figure 19 Diagram of Seismic forces for the Central Plant. (Both directions)

Accidental Torsion

Accidental Torsion Due to Seismic Loads (E-W Direction Loading) – Central Tower								
Story Level (i)	Story Height (h_i) ft	Floor Height (h) ft	Story Forces (f_i) kips	Story Shear (V_i) kips	By (ft)	5% By (ft)	Ax	Mz (ft-k)
Roof	14	72	177.8	177.8	197	9.85	1.00	1751.8
Fifth	14	58	182.4	360.3	197	9.85	1.00	1797.1
Fourth	14	44	128.6	488.9	197	9.85	1.00	1266.4
Third	14	30	88.4	577.3	297	14.85	1.00	1312.7
Second	16	16	74.7	652.0	297	14.85	1.00	1110.0
Totals			360.3					7237.9

Table 13 Accidental torsion produced in the Central Tower at 5% offset of Center of Mass due to E-W Seismic Loading

Accidental Torsion Due to Seismic Loads (N-S Direction Loading) – Central Tower								
Story Level (i)	**Story Height (h_i) ft**	**Floor Height (h) ft**	**Story Forces (f_i) kips**	**Story Shear (V_i) kips**	**Bx (ft)**	**5% Bx (ft)**	**Ax**	**Mz (ft-k)**
Roof	14	72	177.8	177.8	120	6	1.00	1068.1
Fifth	14	58	182.4	360.3	120	6	1.00	1094.7
Fourth	14	44	128.6	488.9	120	6	1.00	771.4
Third	14	30	88.4	577.3	224	11.2	1.02	1009.9
Second	16	16	74.7	652.0	224	11.2	1.00	837.2
Totals			360.3					4781.2

Table 14 Accidental torsion produced in the Central Tower at 5% offset of Center of Mass due to N-S Seismic Loading

Accidental Torsion Due to Seismic Loads (E-W Direction Loading) – Central Plant								
Story Level (i)	**Story Height (h_i) ft**	**Floor Height (h) ft**	**Story Forces (f_i) kips**	**Story Shear (V_i) kips**	**By (ft)**	**5% By (ft)**	**Ax**	**Mz (ft-k)**
Roof	14	30	156.7	156.7	132	6.6	1.00	1034.03
Second	16	16	56.3	213.0	132	6.6	1.01	375.49
Totals			213.0					1409.52

Table 15 Accidental torsion produced in the Central Plant at 5% offset of Center of Mass due to E-W Seismic Loading

Accidental Torsion Due to Seismic Loads (N-S Direction Loading) – Central Plant								
Story Level (i)	**Story Height (h_i) ft**	**Floor Height (h) ft**	**Story Forces (f_i) kips**	**Story Shear (V_i) kips**	**Bx (ft)**	**5% Bx (ft)**	**Ax**	**Mz (ft-k)**
Roof	14	30	156.7	156.7	48	2.4	1.00	376.01
Second	16	16	56.3	213.0	48	2.4	1.00	135.19
Totals			213.0					511.20

Table 16 Accidental torsion produced in the Central Plant at 5% offset of Center of Mass due to N-S Seismic Loading

The resulting base shear calculated through analysis for the Central Tower was 652 kips with an accidental torsion of 7237.9 foot-kips in the East-West direction and 4781.2 foot-kips in the North-South direction. The base shear calculated for the Central Plant was 212.8 kips with an accidental torsion of 1409.52 foot-kips in the East-West direction and 511.20 foot-kips in the North-South direction.

Computer Model

In order to analyze the lateral system of the SUNY Upstate Cancer Center, a three-dimensional structural model was created with the use of ETABS, a Computer and Structures Inc. modeling and analysis program. Since only lateral forces were considered in this analysis, modeling of members was limited only to those participating in the lateral system of the structure. Models for the Central Tower and Central Plant were created in separate files under the assumption that they act independently of each other. Their displacements will be checked in relation to each other and the limitations of the building expansion joint that divides them. These models were used to obtain the data located later in this report such as controlling load cases, story drifts, and member forces. Figure 20 shows the three-dimensional models in ETABS.

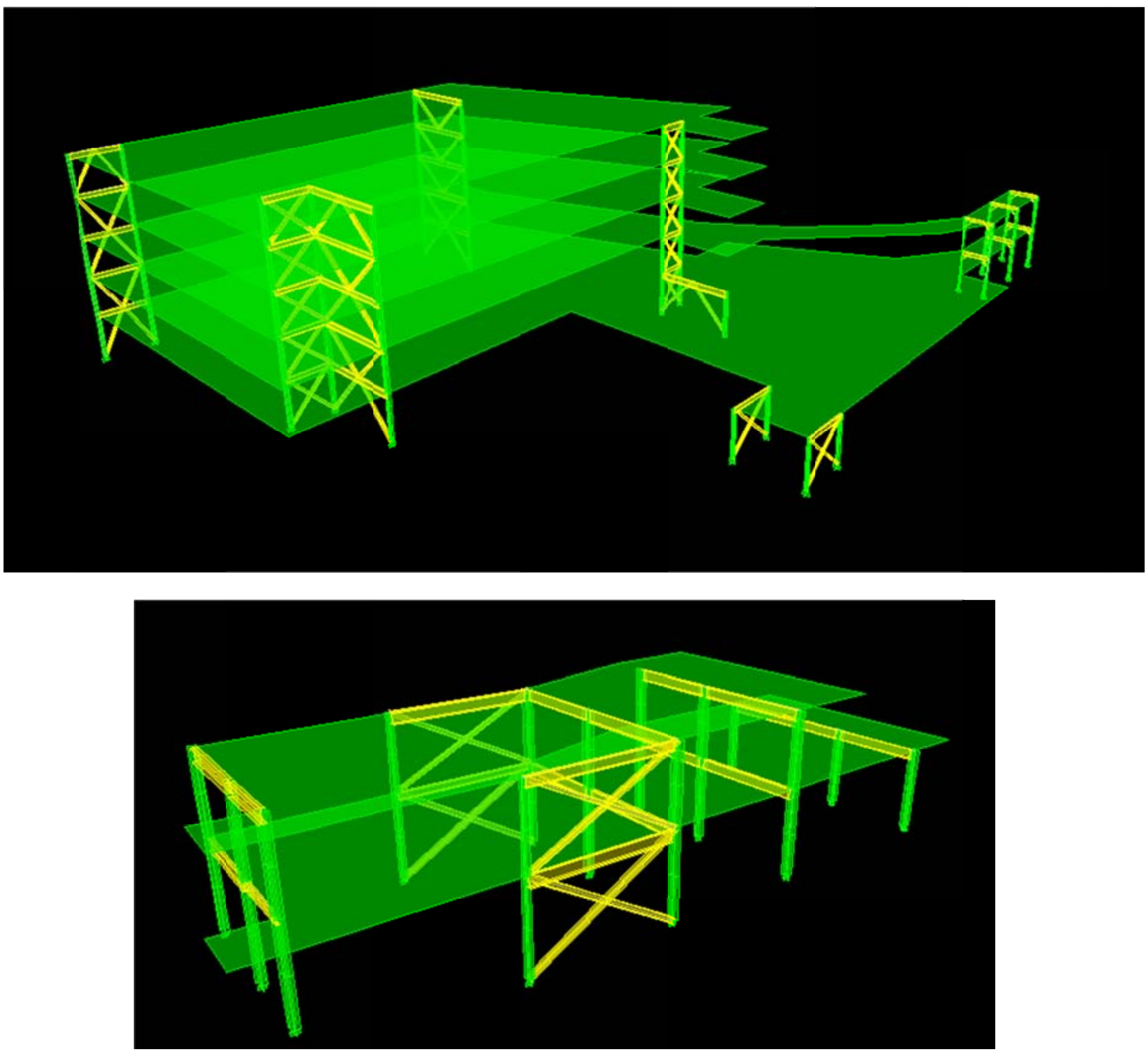


Figure 20 ETABS three-dimensional models looking northeast. **Top**: Central Tower, **Bottom**: Central Plant

In best attempt to recreate the lateral system of each building effectively, the following assumptions and considerations were made:

- Column splices were modeled at the floor level rather than at inner story locations.
- Diagonal bracing and horizontal framing members were specified to have moment releases at their ends to conform to the connection details specified in the structural drawings. This ensured that these members would purely carry axial loads.
    - **<u>Exception:</u>** In the case of the moment frames in the Central Plant, no releases were applied to the horizontal beam members.
- Intersection of bracing at mid story level was modeled without releases. (Model was run with and without releases at these points and resulted in minimal or no change) See Figure 21 for connection detail.
- Since no lateral forces were tabulated for the elevator core shafts, braced frames were terminated at the roof level of the Central Tower, elevation: 72’-0”.
- Each floor level was modeled as a rigid diaphragm, such that all points at that level would displace together.
- After careful consideration, all column fixities were modeled as fixed, due to flange and web welding of the wide flange members to the base plates and the anchor bolt configuration. (See Figure 22)

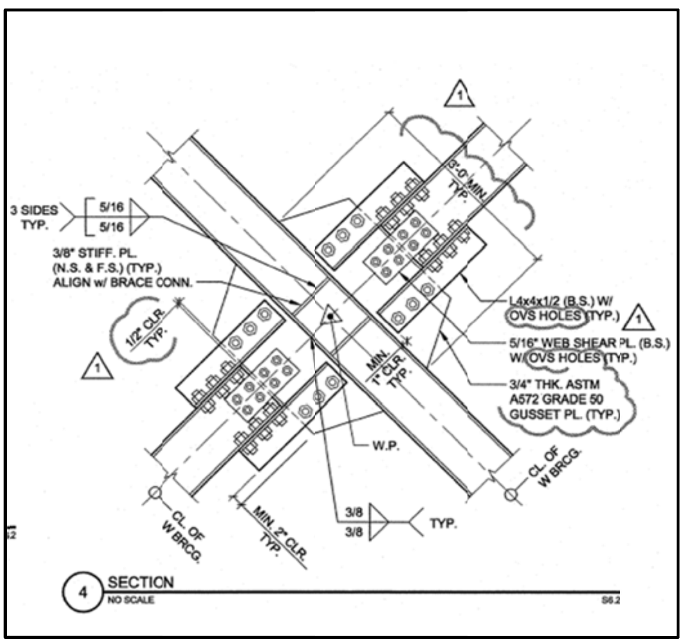


Figure 21 Intersection of bracing at mid-story detail. (Courtesy of EwingCole)

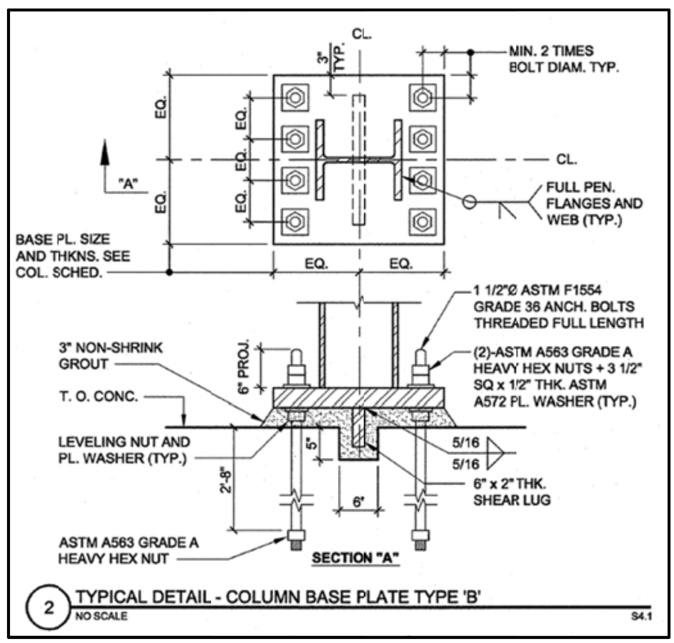


Figure 22 Typical column-to-baseplate connection detail. (Courtesy of EwingCole)

Analysis

Relative Stiffness

Lateral forces acting at a particular level in a structure must be resisted by the lateral elements at that particular level. Using the structural engineering principle that load follows stiffness; one can see that the stiffest element, the element with the highest relative stiffness, will resist the most lateral load. Relative stiffnesses were calculated for both the Central Tower and Central Plant at each floor level of the building. This was accomplished by placing a one kip load at a particular story level and measuring the relative displacement of the frame in question, between the story where the load was applied and the story beneath it. The shear force resisted by that particular frame at that level would then be divided by its relative displacement to find the frame's stiffness at that particular level. In more general terms, this ideology was based on the principle equation that stiffness is equal to force divided by displacement. ($k = p / \delta$).

It was expected that the relative stiffness of the frames in the Central Tower would be consistent for floor four, floor five, and the roof. This was assumed based on that fact that these upper three floors are nearly identical in plan. However, the first two floors differ in plan from each other and the rest of the floors; therefore, a shift in the frames' relative stiffness would be expected.

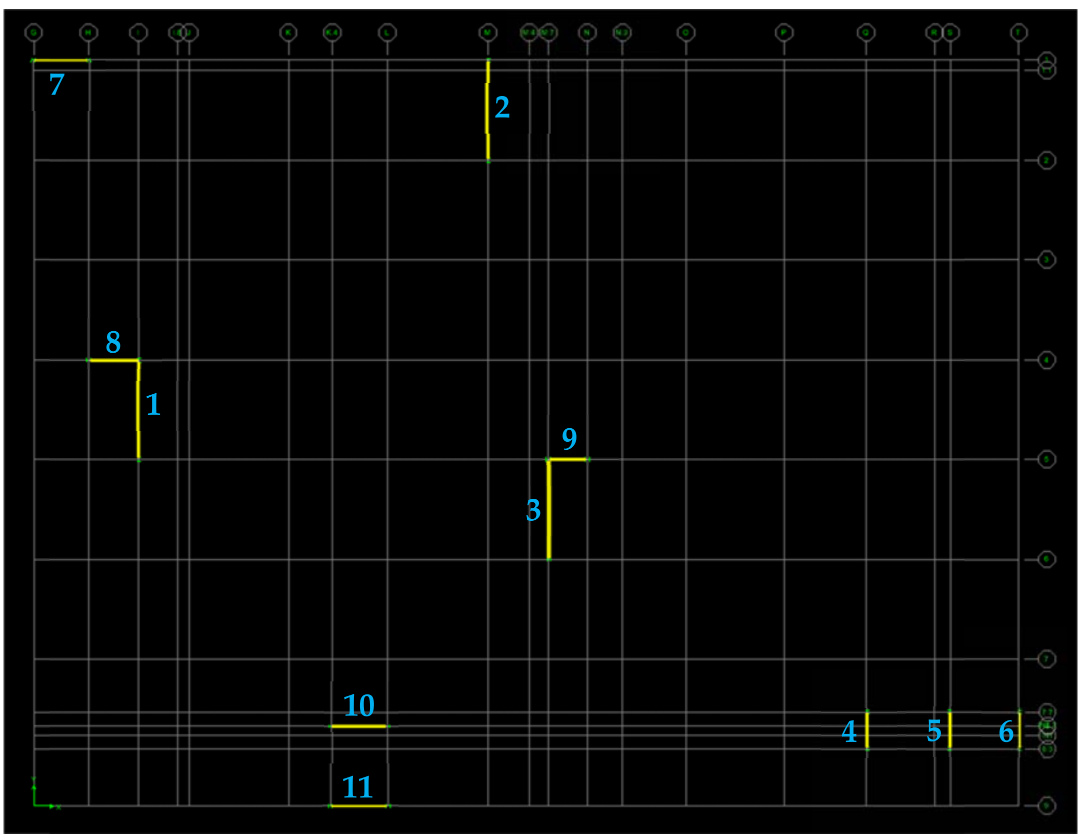


Figure 23 Location and numbering of frames on ground floor of Central Tower

Central Tower - Frame Stiffnesses					
Frame		Force - p [kips]	Δ - [in]	k - [kips/in]	Relative k [%]
At Roof Level					
Y	1	0.4009	0.000491	816.5	56.25
	2	0.5989	0.000943	635.1	43.75
	3	-	-	-	-
	4	-	-	-	-
	5	-	-	-	-
	6	-	-	-	-
X	7	0.3175	0.001712	185.5	29.84
	8	0.4953	0.001576	314.3	50.56
	9	0.1864	0.00153	121.8	19.60
	10	-	-	-	-
	11	-	-	-	-
At Fifth Floor					
Y	1	0.4149	0.000398	1042.5	55.29
	2	0.5807	0.000689	842.8	44.71
	3	-	-	-	-
	4	-	-	-	-
	5	-	-	-	-
	6	-	-	-	-
X	7	0.2978	0.001074	277.3	28.08
	8	0.451	0.000992	454.6	46.04
	9	0.2466	0.000965	255.5	25.88
	10	-	-	-	-
	11	-	-	-	-
At Fourth Floor					
Y	1	0.4409	0.000297	1484.5	56.75
	2	0.5533	0.000489	1131.5	43.25
	3	-	-	-	-
	4	-	-	-	-
	5	-	-	-	-
	6	-	-	-	-
X	7	0.3056	0.000597	511.9	30.32
	8	0.4211	0.000589	714.9	42.34
	9	0.2705	0.000586	461.6	27.34
	10	-	-	-	-
	11	-	-	-	-
At 3rd Floor					
Y	1	0.1829	0.000089	2055.1	50.56
	2	0.7218	0.000378	1909.5	46.98
	3	-	-	-	-
	4	0.0135	0.000691	19.5	0.48
	5	0.0298	0.000759	39.3	0.97
	6	0.0337	0.000817	41.2	1.01
X	7	0.194	0.000293	662.1	24.62
	8	0.4509	0.000382	1180.4	43.90
	9	0.3487	0.000412	846.4	31.48
	10	-	-	-	-
	11	-	-	-	-
At 2nd Floor					
Y	1	0.2269	0.000082	2767.1	38.04
	2	0.4435	0.000162	2737.7	37.64
	3	0.2887	0.000176	1640.3	22.55
	4	0.0106	0.000248	42.7	0.59
	5	0.0114	0.000267	42.7	0.59
	6	0.0121	0.000283	42.8	0.59
X	7	0.2048	0.000171	1197.7	20.36
	8	0.2818	0.000168	1677.4	28.51
	9	0.2191	0.000168	1304.2	22.17
	10	0.1413	0.000166	851.2	14.47
	11	0.1408	0.000165	853.3	14.50

Table 17 Relative Stiffness of braced frames in Central Tower under 1 kip loading (X & Y Directions)

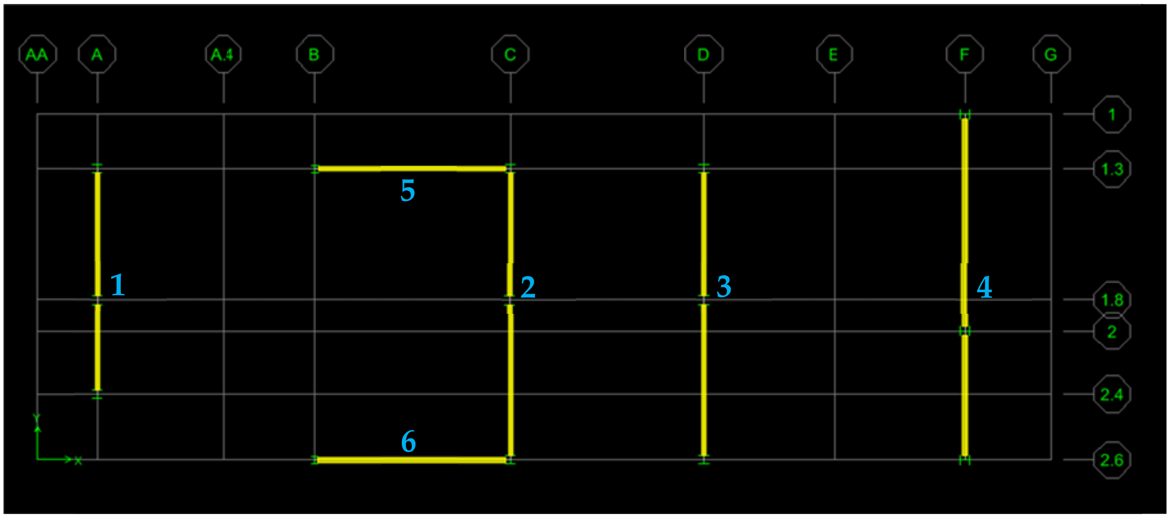


Figure 24 Location and numbering of frames on ground floor of Central Plant.

Central Plant - Frame Stiffness					
Frame		**Force - p [kips]**	**Δ - [in]**	**k - [kips/in]**	**Relative k [%]**
At Roof Level					
Y	1	0.3514	0.002753	127.6	34.53
	2	0.3234	0.002707	119.5	32.32
	3	0.329	0.002685	122.5	33.15
	4	-	-	-	-
X	5	0.5555	0.000561	990.2	50.77
	6	0.4379	0.000456	960.3	49.23
At 2rd Floor					
Y	1	0.2368	0.00171	138.5	34.91
	2	0.2835	0.001784	158.9	29.69
	3	0.2898	0.001819	159.3	29.77
	4	0.1464	0.001866	78.5	14.66
X	5	0.5387	0.000258	2088.0	50.33
	6	0.4203	0.000204	2060.3	49.67

Table 18 Relative Stiffness of braced frames and moment frame in Central Plant under 1 kip loading (X & Y Directions)

After calculating the relative stiffness values of each frame at every floor, the center of rigidity was calculated for the first story of the Central Tower and compared to values from ETABS. Hand calculations for the location of the center of rigidity can be found in Appendix F.

Listed in the tables below are the global coordinates for the center of rigidity, center of mass, and center of pressure obtained from ETABS for both the Central Tower and Central Plant. These positions are vital for distributing lateral loads throughout the building. All lateral loads are resisted through the center of rigidity, however seismic forces are applied through the center of mass and wind forces are applied at the center of pressure. If there exists eccentricity between the center of mass and center of rigidity, or center of pressure and center of rigidity, then torsional shear will exist and needs to be distributed the lateral elements. (See Appendix F for calculation of torsional rigidity, distribution of direct shear, and distribution of torsional shear.)

Central Tower - C.O.M. / C.O.R. / C.O.P. - Global Coordinates						
Level	XCOM (ft)	YCOM (ft)	XCOR (ft)	YCOR (ft)	XCOP (ft)	YCOP (ft)
Roof	90.756	159.607	75.081	140.571	98.3646	149.1667
5	90.756	159.607	77.669	141.664	98.3646	149.1667
4	90.756	159.607	82.094	140.518	98.3646	149.1667
3	95.127	150.262	91.393	133.567	148.4063	120.6771
2	115.350	115.686	102.971	110.609	148.4063	112.0834

Table 19 Center of mass, center of rigidity, & center of pressure global coordinates obtained from ETABS for the Central Tower

Central Plant - C.O.M. / C.O.R. / C.O.P. - Global Coordinates						
Level	XCOM (ft)	YCOM (ft)	XCOR (ft)	YCOR (ft)	XCOP (ft)	YCOP (ft)
Roof	60.231	21.093	60.681	20.464	55.1667	23.8594
2	76.219	21.772	66.791	20.323	70.2084	23.8594

Table 20 Center of mass, center of rigidity, & center of pressure global coordinates obtained from ETABS for the Central Plant

Load Cases and Combinations

ASCE7-10 lists seven different basic load combinations as stated in section 2.3.2. Since only lateral loads were considered for analysis, combinations that considered the highest wind or seismic loads were picked to control. With the incorporation of dead load, live load, and snow load, it was determined that combination four (4) controlled when considering wind load, and combination five (5) controlled when considering seismic loads. The combinations are as follows:

1. 1.4D
2. 1.2D + 1.6L + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or 0.5W)
4. 1.2D + 1.0W + L + 0.5(L_r or S or R) ← Controlling load combination for Wind
5. 1.2D +1.0E + L + 0.2S ← Controlling load combination for Seismic
6. 0.9D + 1.0W
7. 0.9D + 1.0E

Considering the controlling load combination as specified above, thirteen different load cases were created in order to determine the max design force that the lateral members needed to carry. Eleven of these cases were taken from ASCE7-10 figure 27.4-8 depicted below.

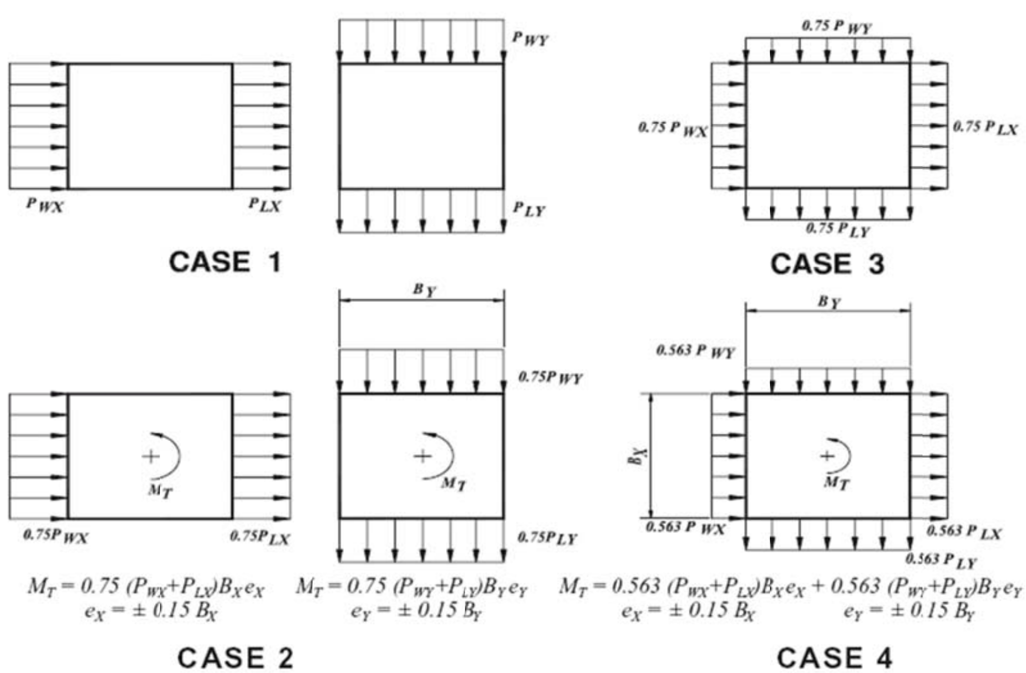


Figure 25 Design Wind Load Cases (Fig. 27.4-8) as specified in ASCE7-10 Chapter 27.

The remaining two load cases consider seismic forces with accidental torsion acting in the East-West and North-South directions. A summary of the load cases used for analysis for both buildings is given below.

<table>
<tr><td rowspan="12">ASCE 7 - 10 Load Combination 4</td><td colspan="2">Load Cases - Wind</td><td></td></tr>
<tr><td>1</td><td>1.0WX</td><td rowspan="2">1</td></tr>
<tr><td>2</td><td>1.0WY</td></tr>
<tr><td>3</td><td>.75WX +.75WMX</td><td rowspan="4">2</td></tr>
<tr><td>4</td><td>.75WX - .75WMX</td></tr>
<tr><td>5</td><td>.75WY + .75WMY</td></tr>
<tr><td>6</td><td>.75WY - .75WMY</td></tr>
<tr><td>7</td><td>.75WX + .75WY</td><td>3</td></tr>
<tr><td>8</td><td>.563WX + .563WY + .563WMX + .563WMY</td><td rowspan="4">4</td></tr>
<tr><td>9</td><td>.563WX + .563WY + .563WMX - .563WMY</td></tr>
<tr><td>10</td><td>.563WX + .563WY - .563WMX + .563WMY</td></tr>
<tr><td>11</td><td>.563WX + .563WY - .563WMX - .563WMY</td></tr>
<tr><td rowspan="3">Comb. 5</td><td colspan="2">Load Cases - Seismic</td><td rowspan="3"></td></tr>
<tr><td>12</td><td>1.0EX + Accidental Torsion</td></tr>
<tr><td>13</td><td>1.0EY + Accidental Torsion</td></tr>
</table>

Table 21 Load case considered for analysis of Central Tower and Central Plant

Where:

<table>
<tr><td rowspan="8">WMX = WX*.15*Bx</td><td rowspan="8">WMY = WY*.15*By</td><td colspan="2">Central Tower</td></tr>
<tr><td>Bx=224'</td><td>FLR 2-3</td></tr>
<tr><td>Bx=120'</td><td>FLR 4-Roof</td></tr>
<tr><td>By=297'</td><td>FLR 2-3</td></tr>
<tr><td>By=197'</td><td>FLR 4-Roof</td></tr>
<tr><td colspan="2">Central Plant</td></tr>
<tr><td>Bx=48'</td><td>FLR 2-Roof</td></tr>
<tr><td>Bx=132'</td><td>FLR 2-Roof</td></tr>
</table>

Table 22 Building data used to create load cases.

Each case was run in ETABS to determine which one controlled the design of the lateral system of each building. The means for evaluating each load case reverts back to the equation that stiffness is equal to force divided by displacement. Since the stiffness of each floor remains constant, it was determined that the load case that caused the greatest deflection would create the largest force in the lateral resisting members and therefore be the controlling load case. This ideology was carried out on a floor to floor basis. A summary of the findings can be found in Appendix H. It was determined that load case twelve and thirteen, seismic loading, controlled the strength design of both the Central Tower and the Central Plant, in the East-West and North-South directions respectively.

Drift and Story Drifts

Story drift as well as overall building drift were calculated for the Central Tower and Central Plant, under the controlling load cases. As stated in the previous section, load case twelve and thirteen controlled strength design; they also control seismic drift. The controlling wind load cases were load case one and two in the East-West and North-South directions respectively. Seismic drift limitations are a code requirement and needed to achieve strength, while wind drift limitations are a serviceability issue and mentioned in the ASCE7 commentary.

Story drift ratio values were obtained from ETABS and adjusted to compare to allowable limits. In order to do so, the story drift ratio was multiplied by its respective story height. It should be noted that seismic story drift values were adjusted by a factor of (C_d / I) as specified by ASCE7-10 section 12.8.6. A deflection amplification factor of 3.0 was used in conjunction with an importance factor of 1.5. Seismic drifts were compared to $0.010h_{sx}$ as stated in Table 12.12-1 for risk category IV, while wind drifts were limited to H/400 as suggested in the commentary. The following tables display the drift values for each building under the controlling load cases. From inspection, it can be seen that all drifts and story drifts are within the prescribed limitations. It should be noted that the max deflection possible between the Central Tower and Central Plant is 1.01" which is less than the 2.0" allowable by the building expansion joint. Therefore the joint size is proper.

Central Tower Seismic Drift: East - West Direction (Load Case 12)									
Level	Story Height (ft)	Story Drift Ratio (in/in)	(Cd/I) (3/1.5)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Allowable Total Drift (in)	
Roof	14	0.0032	2.0	1.070	1.68	OK	4.545	8.64	OK
5	14	0.0032	2.0	1.085	1.68	OK	3.474	6.96	OK
4	14	0.0026	2.0	0.870	1.68	OK	2.390	5.28	OK
3	14	0.0025	2.0	0.839	1.68	OK	1.520	3.60	OK
2	16	0.0018	2.0	0.682	1.92	OK	0.682	1.92	OK

Table 23 Drift values for the Central Tower Considering Seismic Controlling Load Case 12.

Central Tower Seismic Drift: North - South Direction (Load Case 13)									
Level	Story Height (ft)	Story Drift Ratio (in/in)	(Cd/I) (3/1.5)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Allowable Total Drift (in)	
Roof	14	0.0020	2.0	0.678	1.68	OK	3.060	8.64	OK
5	14	0.0023	2.0	0.770	1.68	OK	2.383	6.96	OK
4	14	0.0021	2.0	0.718	1.68	OK	1.612	5.28	OK
3	14	0.0017	2.0	0.566	1.68	OK	0.894	3.60	OK
2	16	0.0009	2.0	0.328	1.92	OK	0.328	1.92	OK

Table 24 Drift values for the Central Tower Considering Seismic Controlling Load Case 13.

Central Plant Seismic Drift: East - West Direction (Load Case 12)									
Level	Story Height (ft)	Story Drift Ratio (in/in)	(Cd/I) (3/1.5)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Allowable Total Drift (in)	
Roof	14	0.0006	2.0	0.200	1.68	OK	0.388	3.60	OK
2	16	0.0005	2.0	0.188	1.92	OK	0.188	1.92	OK

Table 25 Drift values for the Central Plant considering Seismic Controlling Load Case 12.

Central Plant Seismic Drift: North - South Direction (Load Case 13)									
Level	Story Height (ft)	Story Drift Ratio (in/in)	(Cd/I) (3/1.5)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Allowable Total Drift (in)	
Roof	14	0.0028	2.0	0.926	1.68	OK	2.167	3.60	OK
2	16	0.0032	2.0	1.241	1.92	OK	1.241	1.92	OK

Table 26 Drift values for the Central Plant considering Seismic Controlling Load Case 13.

Central Tower Wind Drift: East - West Direction (Load Case 1)								
Level	Story Height (ft)	Story Drift Ratio (in/in)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Total Allowable Drift (in)	
Roof	14	0.0009	0.146	0.42	OK	0.706	2.16	OK
5	14	0.0009	0.153	0.42	OK	0.560	1.74	OK
4	14	0.0008	0.138	0.42	OK	0.407	1.32	OK
3	14	0.0008	0.137	0.42	OK	0.270	0.90	OK
2	16	0.0007	0.133	0.48	OK	0.133	0.48	OK

Table 27 Drift values for the Central Tower considering Wind Controlling Load Case 1.

Central Tower Wind Drift: North - South Direction (Load Case 2)								
Level	Story Height (ft)	Story Drift Ratio (in/in)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Total Allowable Drift (in)	
Roof	14	0.0010	0.163	0.42	OK	0.870	2.16	OK
5	14	0.0012	0.196	0.42	OK	0.708	1.74	OK
4	14	0.0012	0.199	0.42	OK	0.512	1.32	OK
3	14	0.0010	0.176	0.42	OK	0.313	0.90	OK
2	16	0.0007	0.137	0.48	OK	0.137	0.48	OK

Table 28 Drift values for the Central Tower considering Wind Controlling Load Case 2.

Central Plant Wind Drift: East - West Direction (Load Case 1)								
Level	Story Height (ft)	Story Drift Ratio (in/in)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Total Allowable Drift (in)	
Roof	14	0.000037	0.006	0.42	OK	0.019	0.90	OK
2	16	0.000066	0.013	0.42	OK	0.013	0.48	OK

Table 29 Drift values for the Central Plant considering Wind Controlling Load Case 1.

Central Plant Wind Drift: North - South Direction (Load Case 2)								
Level	Story Height (ft)	Story Drift Ratio (in/in)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Total Allowable Drift (in)	
Roof	14	0.000705	0.118	0.42	OK	0.382	0.90	OK
2	16	0.001373	0.264	0.42	OK	0.264	0.48	OK

Table 30 Drift values for the Central Plant considering Wind Controlling Load Case 2.

Overturning Moment & Impact on Foundation

Overturning moment sometimes occur in a building that is subjected to lateral loads, and can have significant effect on the building's foundation. Referencing table 11, the greatest overturning moment is attributed to seismic loading and results in 32891.36 foot-kips. This moment is resisted by the weight of the building acting at a moment arm of half the building width. Two-thirds of this value must be greater than the overturning moment to assure there are no issues with the current foundation. The resisting moment was found to be 807919.2 foot-kips. The two-thirds fractions results in 538612.8 foot-kips, which is far larger than the overturning moment of 32891.36 foot-kips. Therefore, the foundation is perfectly suitable for the designed lateral loads. Complete calculations of the resisting moment can be found in Appendix E.

Strength Check

Four members were chosen for spot checks to ensure that the member sizing was indeed adequate for the applied load. Two columns and two braces, indicated in figure 25, were chosen from Frame 2 at both the first story and fourth story.

The load path was checked by distributing the direct and torsional shears to the appropriate frame. Applied force was then dispersed to the appropriate members. Axial forces found in the brace matched those displayed by ETABS. Brace members were checked solely for axial load capacity, while columns were checked for combined loading effects in addition to axial capacity. All members were beyond adequate taking into consideration their unbraced length. Detailed calculations for these members can be found in Appendix G.

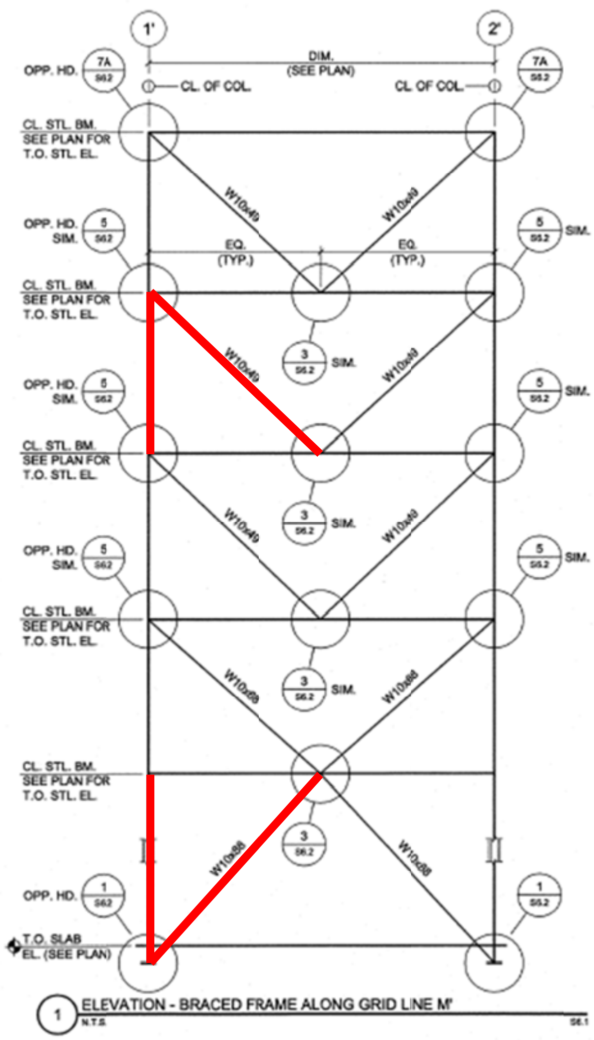


Figure 26 Braced frame #2 showing members selected for strength checks in red.

Conclusion

In summary, the intention of this report was to analyze the existing lateral system of the SUNY Upstate Cancer Center considering both strength and serviceability requirements. The cancer center uses a series of braced frames to resist the lateral loads within the Central Tower while the Central Plant resists lateral loads with a combination of braced frames and moment frames.

Lateral loads, in particular seismic loading, were modified from the previous Technical Report 1 to better reflect the actual story forces. A three-dimensional structural model was created using ETABS software. In order to accurately represent the structure, columns were fixed at the base, beams and braces were released of end moments, and the floor system was modeled as a rigid diaphragm capable of distributing the lateral forces to the proper later elements. Hand calculations were conducted to both complement and verify the results from the computer model.

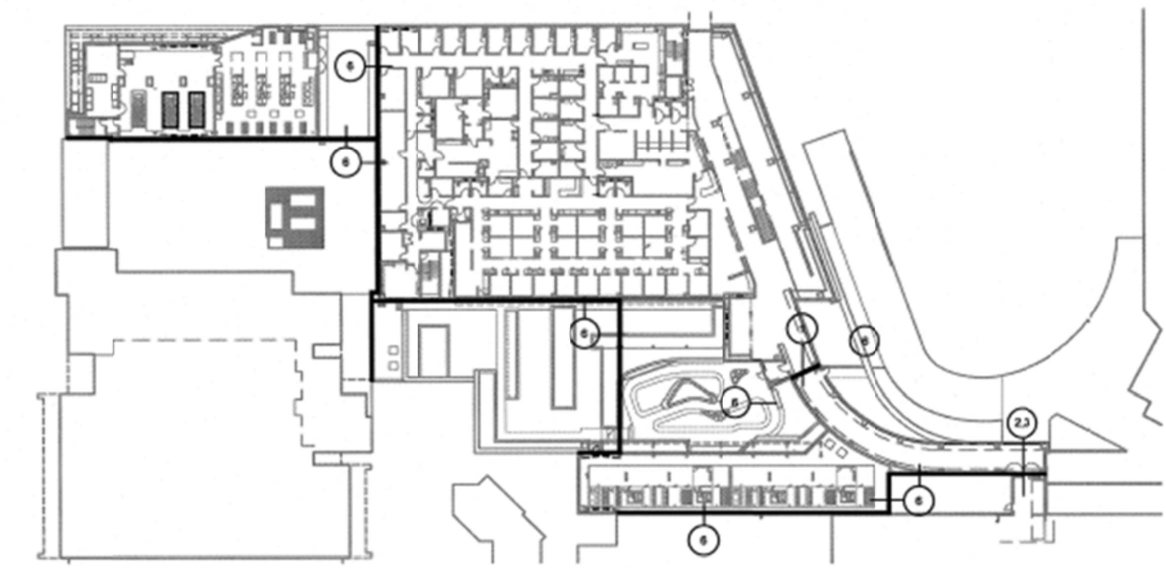
Load paths and distribution were determined to be based on the stiffness of a particular frame at a story level. Each frame's stiffness was determined by applying a 1 kip load at a particular story level, measuring the appropriate force and displacement, and then calculated by using the relation $k=p/\delta$. Considering two types of lateral loading meant analyzing multiple load combinations and cases to determine the governing one. Overall a total of thirteen load cases were entered into ETABS to analyze. Seismic loading controlled both the North-South and East-West directions of the Central Tower and Central Plant.

Using the controlling seismic and wind load cases, story drifts and overall building drifts were calculated using story drift ratios obtained from ETABS. Seismic drift values were checked for strength against ASCE7-10 code requirements of $0.010h_{sx}$ for risk category IV. Wind drift values were limited to H/400, a serviceability threshold suggested in the commentary of ASCE7-10. All story drifts and total building drifts were deemed acceptable having met the requirements just stated.

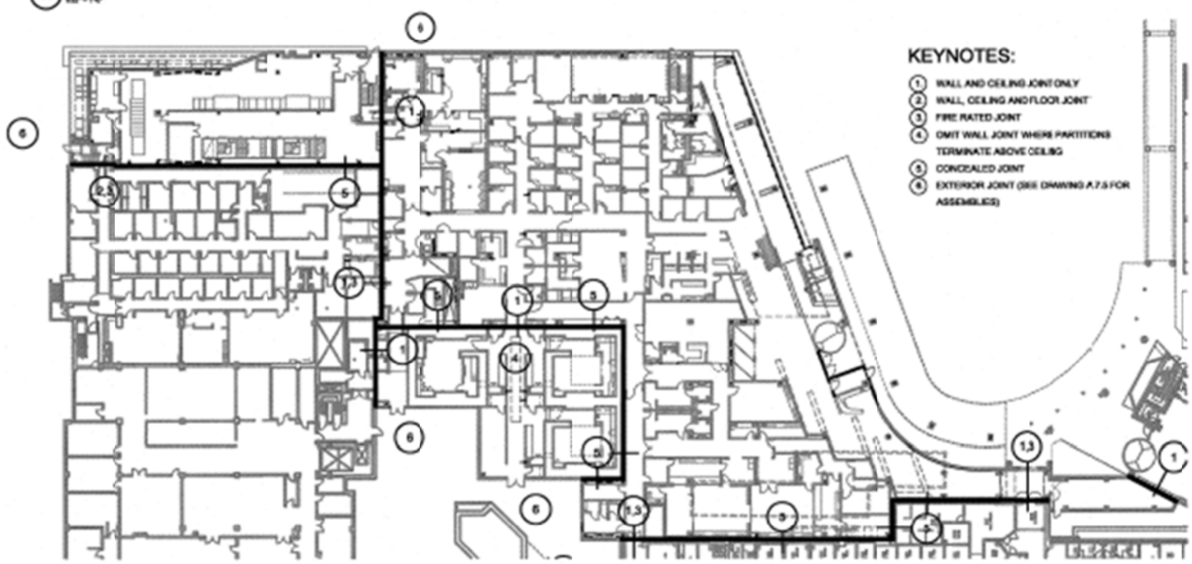
Lateral forces were checked to see if they had any effect on the foundation of the Upstate Cancer Center, in particular any issue with overturning moment. The controlling overturning moment was found to be less than two-thirds the resisting moment; therefore, any issues with the foundation in relation to the later system were dismissed.

Finally, spot checks were conducted on two bracing members and two columns found in braced frame number two. Using the controlling lateral load case for strength, the members were checked for adequate capacity. The braced members were checked against pure axial while the columns were checked for combined loading. All members chosen for strength checks proved to be more than sufficient to carry the required loads.

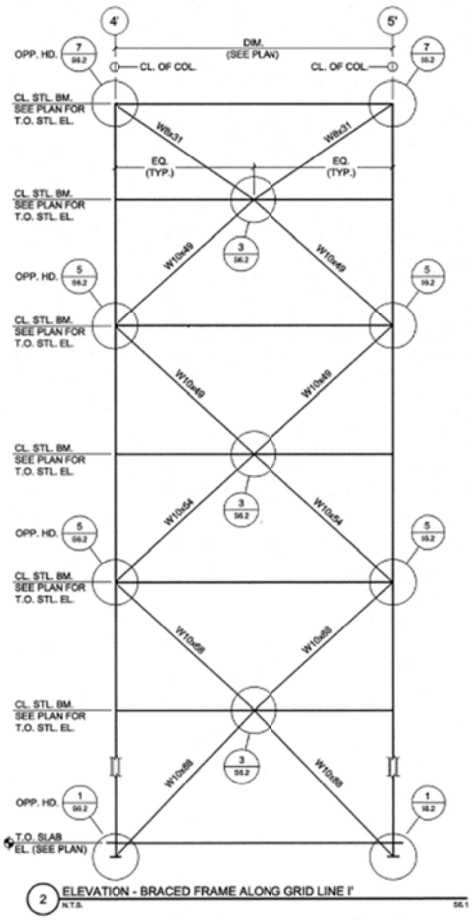
Appendix A: Miscellaneous Drawings



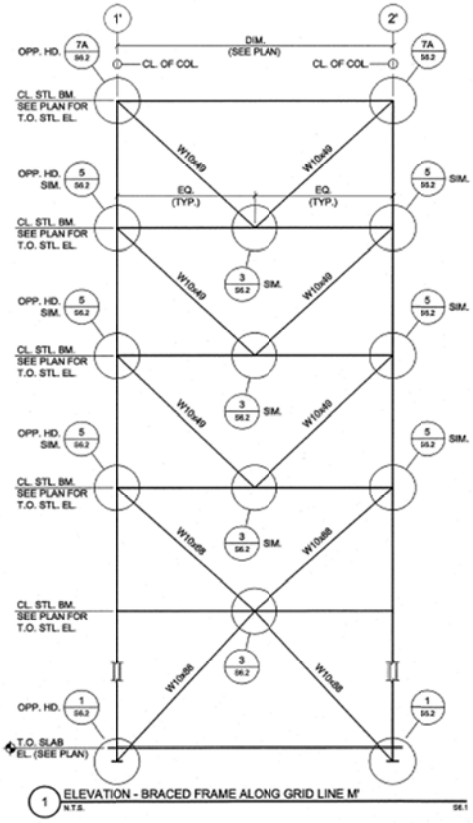
11 EXPANSION JOINT LOCATION AT SECOND FLOOR



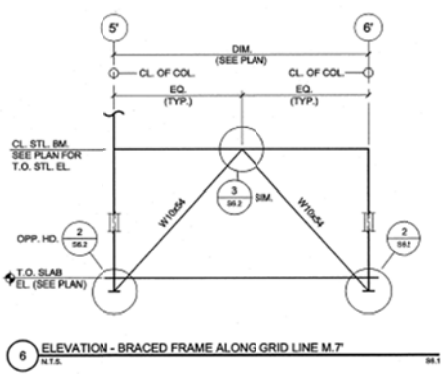
10 EXPANSION JOINT LOCATION AT FIRST FLOOR



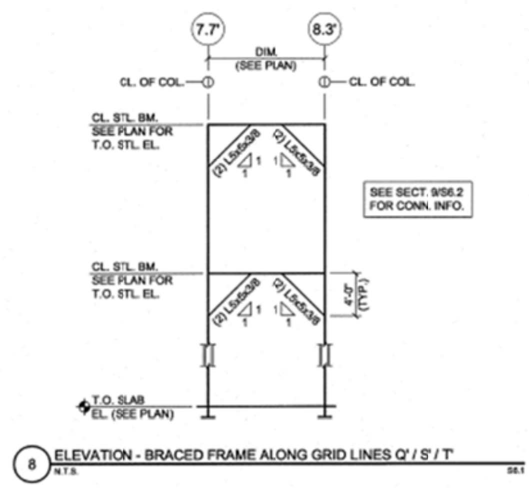
Frame # 1



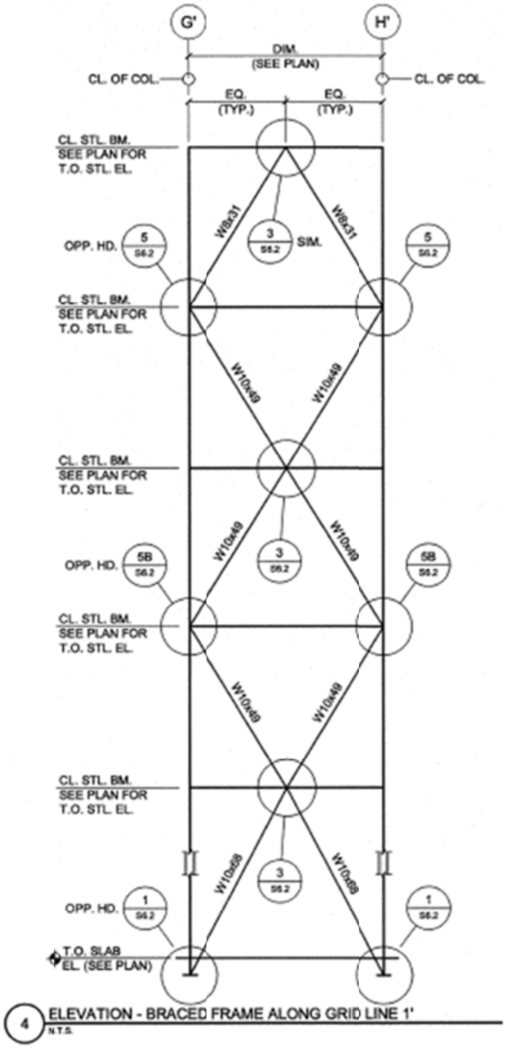
Frame # 2



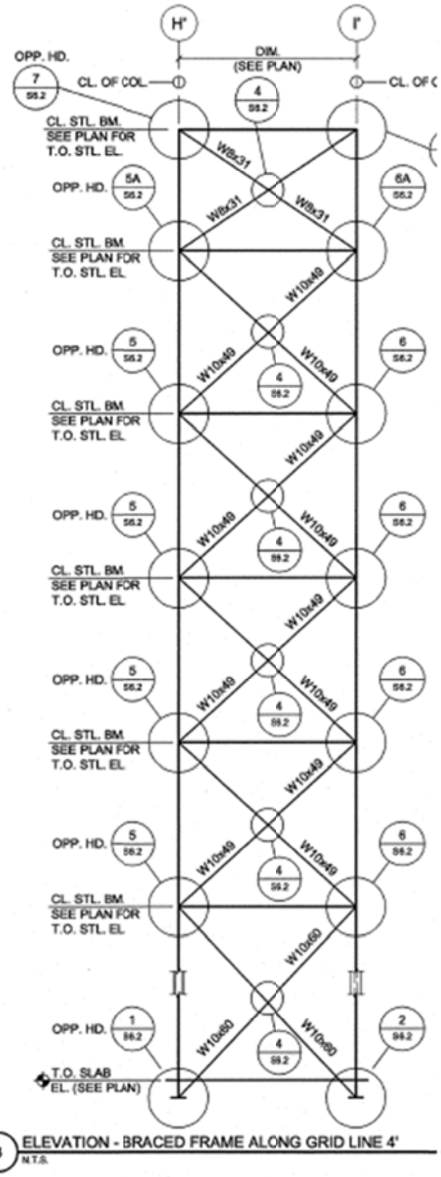
Frame # 3



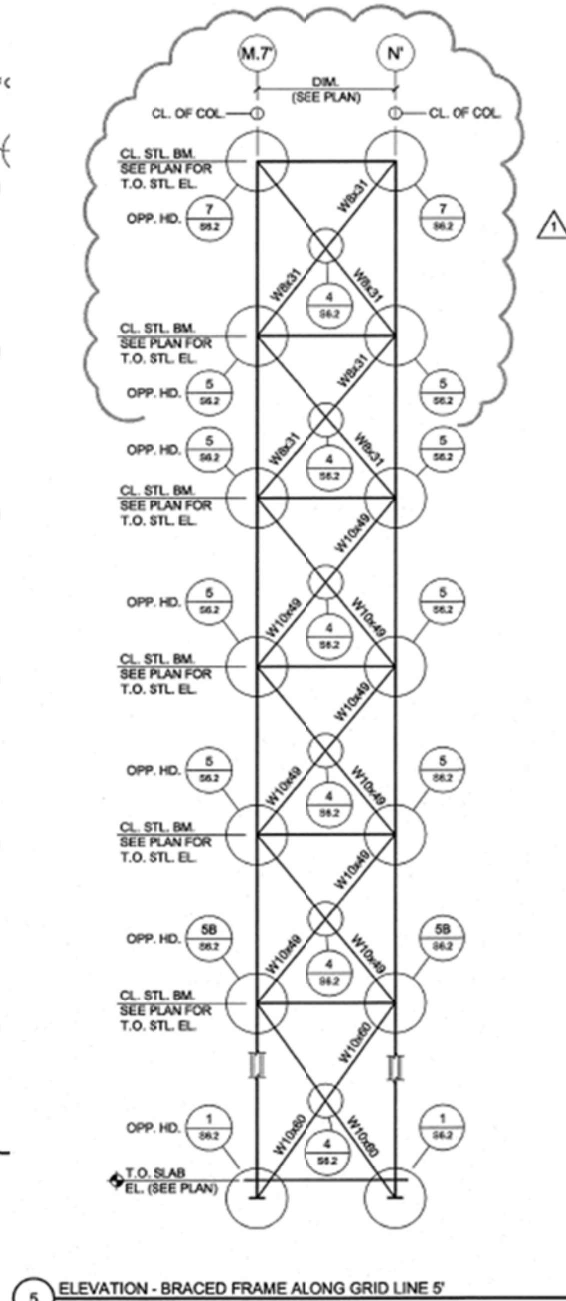
Frames # 4,5,6



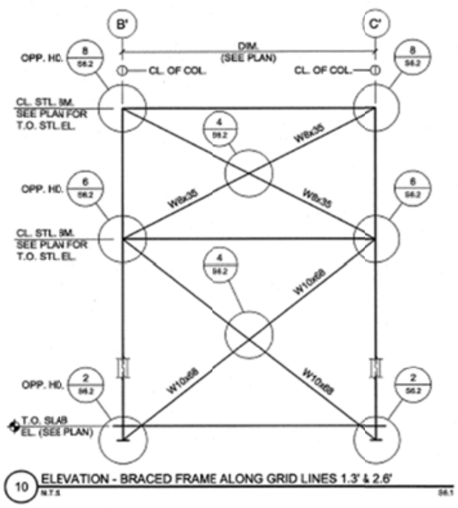
Frame # 7



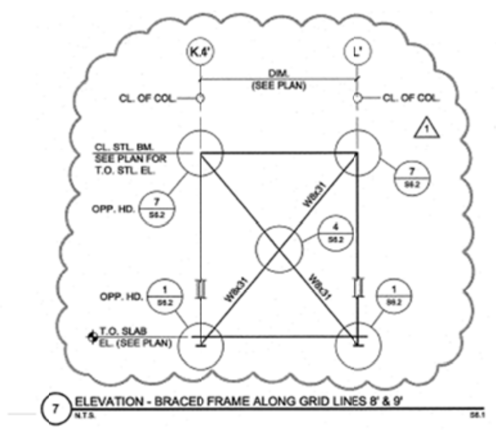
Frame # 8



Frame # 9



Frame # 5, 6 –
Central Plant



Frame # 10, 11

Appendix B: Snow Load

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

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

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

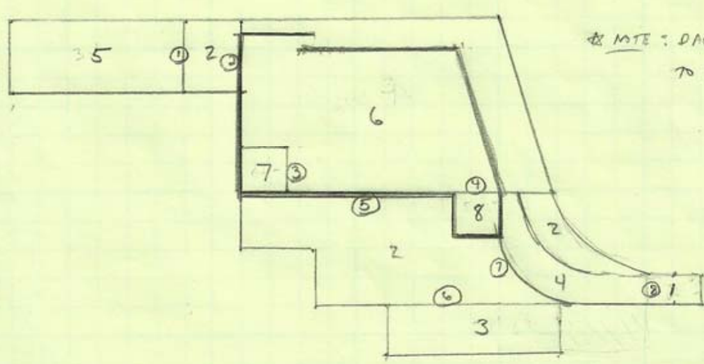
C_t = 1.0 (TABLE 7-3)

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")

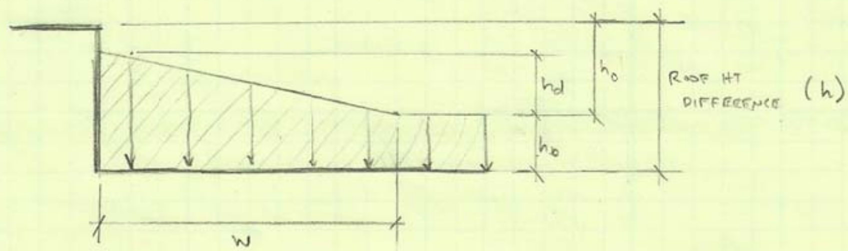
SIMPLIFIED ROOF PLAN

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")

SNOW ACCUMULATION DIAGRAM (FIG 7-8)



SNOW DENSITY (γ)

$\gamma = .13 p_s + 14 \leq 30$ pcf (EQ 7.7-1)

$= .13(50) + 14$

$\gamma = 20.5$ pcf ≤ 30 pcf OK ✓

BASE SNOW ACCUMULATION HEIGHT (ft)

$h_b = P_s / \gamma$ WHERE $P_s = C_s P_f$ $C_s = 1.0$ (FIGURE 7-2 a) 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 $\frac{h_c}{h_b} < .2$ $h = 30' - 17.333' = 12.667'$

$h_c = h - h_b$
$= 12.667' - 2.05' = 10.62'$

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

WINWARD DIR.

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

$l_u = 22'-10'' > 20'$ ∴ $l_u = 22'-10''$

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

NOTE: SEE SPREAD SHEET FOR ADDITIONAL SNOW LOAD CALCULATIONS

LEEWARD DIR

$l_u = 110'-4'' > 20'$ ∴ $l_u = 110'-4''$

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

• LEEWARD CONTROLS → $h_d = 4.24$ ft $< h_c$ OK ✓

∴ $w_d = 4 h_d = 4(4.24) = 16.96$ 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	12.67	10.62	5.18	22.83	1.42	110.33	4.24	4.24	16.96
2	6 to 2	54.69	52.64	25.68	22.83	1.42	155.25	4.93	4.93	19.73
3	7 to 6	10.94	8.89	4.34	155.25	3.70	31.73	2.29	3.70	14.80
4	8 to 6	14.48	12.43	6.06	120.00	3.30	28.00	2.13	3.30	13.21
5	6 to 2	54.69	52.64	25.68	68.00	2.54	120.00	4.40	4.40	17.61
6	3 to 2	4.17	2.12	1.03	68.00	2.54	31.40	2.28	2.12	8.47
7	2 to 4	11.27	9.22	4.50	213.67	4.24	20.00	1.75	4.24	16.96
8	1 to 4	15.71	13.66	6.66	48.88	2.16	71.00	3.46	3.46	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)	17.98	15.93	7.77	177.25	3.92	3.92	15.67		
	6 to P (N-S)	17.98	15.93	7.77	120.00	3.30	3.30	13.21		
	7 to P	7.04	4.99	2.44	31.73	1.72	1.72	6.87		
	8 to P	3.50	1.45	0.71	28.00	1.60	1.45	3.97		

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 C: Wind Loading

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

LOCATION: SYRACUSE, NEW YORK
BUILDING TYPE: HEALTH CARE
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

$$L_{EFF} = \frac{(16')(217.92') + (30)(217.92) + (44+58+72)(155.25')}{16+30+44+58+72}$$

$= 185.08'$

$90' < 4(185.08')$

$90 < 740.32'$ 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

· B/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"

$= 75/72 = 1.042\ H_z > 1.0$

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

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

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

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)

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

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

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

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

$0'-36' \rightarrow C_p = -.9$

$36'-72' \quad C_p = -.9$

$72'-144' \quad C_p = -.5$

$>144' \quad C_p = -.3$

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

ROOF COEFFICIENTS

$0'-36' \rightarrow C_p = -.98$

$36'-72' \rightarrow C_p = -.86$

$72'-144' \rightarrow C_p = -.54$

$>144' \rightarrow C_p = -.38$

THROUGH INTERPOLATION

CALCULATE WIND PRESSURE

→ ENCLOSED AND PARTIALLY ENCLOSED RIGID BUILDINGS

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

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

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

$\underline{P_4} = 21.3$ psf

WIND PRESSURE DETERMINATION ON ROOF TOP 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)

$q_P = .00256\,(.96)(1.0)(.85)(120)^2$

$q_P = 30.1$ psf

WINDWARD DIRECTION

$P_P = 30.1\,(1.5)$

$P_P = 45.15$ psf

LEEWARD DIRECTION

$P_P = 30.1\,(1.0)$

$P_P = 30.1$ 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

$$F = q_z G C_f A_f \quad (\text{Eq } 29.5\text{-}1)$$

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)
$A_f = 710 \text{ ft}^2$ (N-S)
200 ft^2 (E-W)

- $F = 30.1(.83)(1.3)(200)$
  $F = \underline{6496 \text{ lbs}}$ (E-W DIR)

$F = 30.1(.81)(1.3)(710)$
$F = \underline{22504 \text{ lbs}}$ (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 D: Seismic Loading

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 (SECTION 12.8)

EFFECTIVE SEISMIC WEIGHT = 11058 KIPS = W [SPREADSHEET]

FROM ETABS MODEL
↳ $T_X = 1.2763_s$
$T_Y = .9498_s$

$T_a = C_t h_n^x \rightarrow h_n = 72'\text{-}0''$
$C_t = .02 \quad x = .75$ [TABLE 12.8-2 → "OTHER" STRUCTURAL SYSTEMS]

$T_a = .02(72)^{.75} = .494s$
↳ $C_u = 1.7$ B/c $\overset{S_{D1}}{.099} \le .1$

$$T = \min \text{ of } \begin{cases} C_u T_a = 1.7(.494_s) = .8398_s \leftarrow \text{CONTROLS} \\ 1.238_s, \; .9071_s \end{cases}$$

$$C_s = \min \text{ of } \begin{cases} S_{DS}/(R/I) = \dfrac{.153}{3/1.5} = .0765 \\ \dfrac{S_{D1}}{T\left[\frac{R}{I}\right]} = \dfrac{.099}{.8398\left(\frac{3}{1.5}\right)} = .0589 \leftarrow \text{CONTROLS} \\ \dfrac{S_{D1}\,T_L}{T^2\left(\frac{R}{I}\right)} = \dfrac{(.099)(6)}{(.8398)^2\left(\frac{3}{1.5}\right)} = .421 \end{cases}$$

BASE SHEAR
$V = C_s W$
$= (.0589)(11058) = 652$ kips

VERTICAL DISTRIBUTION OF FORCES

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

$.5_s < .8398 < 2.5_s$
↳ INTERPOLATE BTWN 1 & 2 TO FIND K
$k = 1.17$

CENTRAL PLANT
SEISMIC WEIGHT = 2781 kips
$T_x = .3819s \qquad T_a = .02(72)^{.75} = .256s$
$T_y = .9767s \qquad$ ↳ $C_u = 1.7$

$$T \le \begin{cases} (1.7)(.256) = .4352 \\ .3819, \; .9767 \end{cases} \quad C_s \min = .0765$$

$V = (.0765)(2781) = 212.7$ KIPS

✱ $k = 1.0$ B/c $\left.\begin{matrix} .4352 \\ .3819 \end{matrix}\right\} \le .5_s$

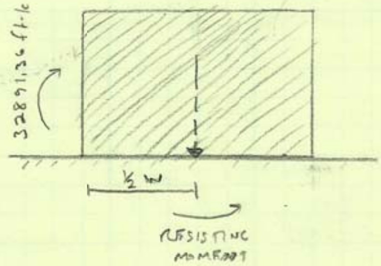
Seismic Design Criteria		
Parameter	Value	Source
Site Class	D	Geotechnical Report
Short Spectral Response Acceleration (S_S)	0.143g	USGS DesignMaps
1-sec. Spectral Response Acceleration (S_1)	0.062g	USGS DesignMaps
Site Coefficient (F_a)	1.6	ASCE 7-10:Table 11.4-1
Site Coefficient (F_V)	2.4	ASCE 7-10:Table 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 - Central Tower	
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)	0.0589
C_u	1.7
$T_{model - x}$	1.2963
$T_{model - y}$	0.9498
T	0.8398

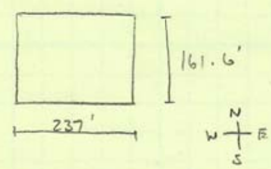
Appendix E: Overturning Moment

SUNY UPSTATE CANCER CENTER	SEISMIC OVERTURNING MOMENT	
AE THESIS		MICHAEL KOSTICK

CENTRAL TOWER
FROM TABLE
↳ 32891.36 ft-kips



AVG WIDTH OF BLDG



RESISTING MOMENT = BLDG WT × ½ LESS OF $\left\{ \begin{array}{l} 161.6' \\ 237' \end{array} \right\}$

$= (9999)(\tfrac{1}{2}(161.6))$

$= 807919.2$ ft-k
↳ MULTIPLY BY $2/3$

$(2/3)(807919.2) = 538612.8$ ft-k $\gg 32891.36$ ft-k
↳ ∴ OK TO RESIST OVERTURNING MOMENT

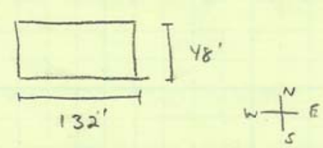
E-W DIR = $\dfrac{120+120+120+224+224}{5 \text{ FLOORS}}$

$= 161.6'$

N-S DIR = $\dfrac{197+197+197+277+277}{5 \text{ FLOORS}}$

$= 237'$

CENTRAL PLANT
FROM TABLE
↳ 5601.4 ft-k



RESISTING MOMENT =

$= (2784)(\tfrac{1}{2}(48))$

$= 66816$ ft-k
↳ MULTIPLY BY $2/3$

$2/3\,(66816) = 44544$ ft-k $\gg 5601.4$ ft-k
↳ ∴ OK TO RESIST OVERTURNING MOMENT

Appendix F: Lateral Load Distribution

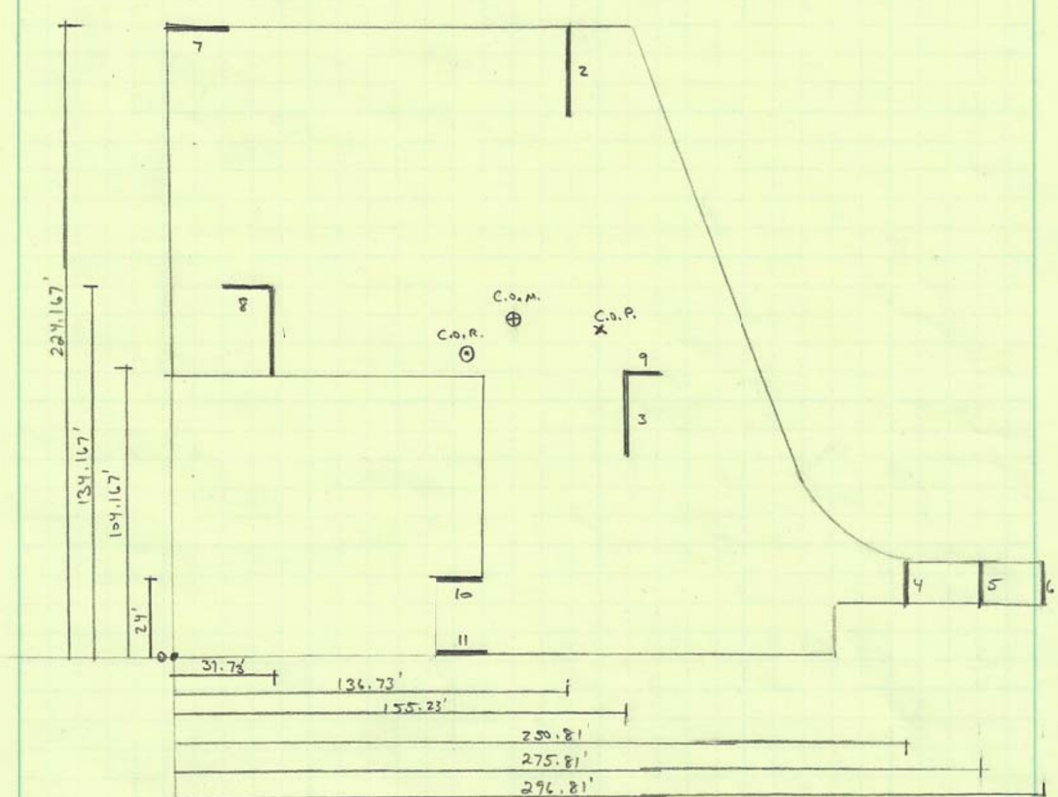
- LOAD FOLLOWS STIFFNESS → STIFFEST FRAME RESISTS MOST LOAD
  ↳ [BASED ON RELATIVE STIFFNESS]

DIRECT SHEAR → $V_i^d = \frac{R_i}{\sum R_i} \cdot V$

TORSIONAL SHEAR → $V_i^t = \frac{V e d_i R_i}{J}$

J = TORSIONAL MOMENT OF INERTIA = $\sum R_i d_i^2$

CENTRAL TOWER → 2ND FLOOR



FRAME STIFFNESSES (k/in)

1: 2767.1	4: 42.7	7: 1197.7	10: 851.2
2: 2737.7	5: 42.7	8: 1671.4	11: 853.3
3: 1640.3	6: 42.8	9: 1304.2	

UPSTATE CANCER CENTER | LATERAL LOAD DISTRIBUTION | MICHAEL KOSTICK | 2
AE THESIS

CENTER OF RIGIDITY

$$X_R = \frac{\Sigma R_i x_i}{\Sigma R_i} = \frac{(31.73)(2767.1)+(136.73)(2737.7)+(155.23)(1640.3)+(250.81)(42.7)+(275.81)(42.7)+(296.81)\times(42.8)}{2767.1+2737.7+1640.3+42.7+42.7+42.8}$$

$$= 103.2' \rightarrow \text{ETABS} = 102.97' \therefore \text{OK}$$

$$Y_R = \frac{\Sigma R_i y_i}{\Sigma R_i} = \frac{(1197.7)(224.167)+(1677.4)(134.167)+(1304.2)(164.167)+(851.2)(24)+(853.3)(0)}{1197.7+1677.4+1304.2+851.2+853.3}$$

$$= 110.44' \rightarrow \text{ETABS} = 110.61' \therefore \text{OK}$$

CENTER OF MASS
FROM ETABS
C.O.M. → X-DIR = 115.35'
C.O.M. → Y-DIR = 115.69'

CENTER OF PRESSURE
FROM ETABS
C.O.P. → X DIR = 148.4
C.O.P. → Y DIR = 112.08

TORSIONAL MOMENT OF INERTIA (J)

$$J = \Sigma R_i d_i^2$$

$$J = (2767.1)(71.47)^2 + (2737.7)(33.53)^2 + (1640.3)(52.03)^2 + (42.7)(147.6)^2 + (42.7)(172.61)^2 +$$
$$+ (42.8)(193.61)^2 + (1197.7)(113.73)^2 + (1677.4)(23.73)^2 + (1304.2)(6.27)^2 +$$
$$+ (851.2)(86.44)^2 + (853.3)(110.44)^2 =$$

$$J = 58714690.7 \text{ k/in } (ft^2)$$

ECCENTRICITY → EARTHQUAKE LOADING

- SEISMIC LOADS ACT THROUGH CENTER OF MASS (C.O.M)
  ↳ RESISTED THROUGH CENTER OF RIGIDITY
- WIND LOADS ACT THROUGH CENTER OF PRESSURE (C.O.P.)
  ↳ RESISTED THROUGH CENTER OF RIGIDITY

SEISMIC

$e_x = X_{COR} - X_{COM} = 103.2' - 115.35' = 12.15'$

$e_y = Y_{COR} - Y_{COM} = 110.44' - 115.69' = 5.25'$

WIND

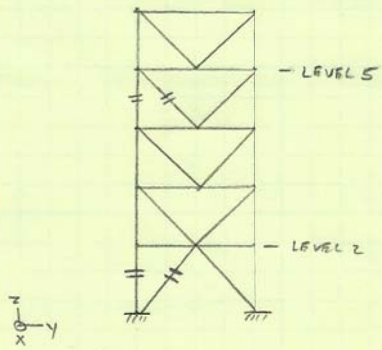
$e_x = X_{COR} - X_{COP} = 103.2' - 148.4' = 45.2'$

$e_y = Y_{COR} - Y_{COP} = 110.44' - 112.08' = 1.64'$

Appendix G: Strength Check / Member Check

SUNY UPSTATE CANCER CENTER | STRENGTH / SPOT CHECK | MICHAEL KOSTICK | 1

FRAME #2 ALONG COLUMN LINE M.



FRAME #2

CONTROLLING LOAD CASE = 1.0 Ey + ACCIDENTAL TORSION
[COMBO 13]
@ LEVEL 2 → 74.7^k + 837.2 ft-k
ACTING @ C.O.M.

FRAME 2 → RELATIVE STIFFNESS - FLR 2
↳ = 37.64 %

$V_D = (74.7^k)(.3764) = \underline{28.12^k}$ ← DIRECT SHEAR

TOTAL TORSIONAL SHEAR = DIRECT SHEAR × e + ACCIDENTAL TORSION

TOTAL TORSIONAL SHEAR

$$V_T = \frac{Ve(d_a R_A)}{J} + \frac{M_E(d_a R_A)}{J}$$

$$= \frac{(74.7)(12.15)[(136.73-103.2)2737.7]}{58714690.7} - \frac{(837.2)[(136.73-103.2)2737.7]}{58714690.7}$$

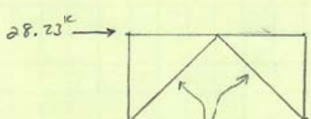
$V_T = 1.419 - 1.309$

$V_T = .11$ ← → ∴ TOTAL SHEAR = 28.12 + .11 = $\underline{28.23^k}$ (← DIR)

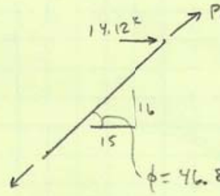
ETABS = 28.4924^k (← DIR)

∴ OK

CONSIDERING ONLY LOAD @ 2^{ND} LEVEL



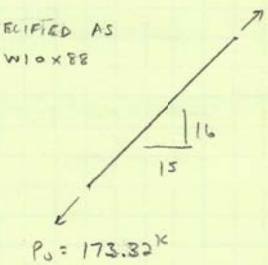
DISTRIBUTED EVENLY → $\frac{28.23}{2} = 14.12^k$



$P_u = \frac{14.12}{\cos 46.85} = \underline{20.64^k}$ → ETABS = 20.6^k OK

CHECK BRACE FOR ENTIRE LOAD COMBINATION 13 BELOW LEVEL 2

SPECIFIED AS W10x88



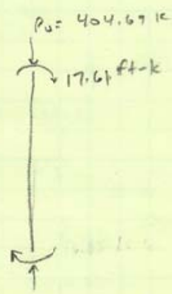
$P_u = 173.32^k$
$L_b = \sqrt{15^2+16^2} = 21.93'$

FROM TABLE 4-1 : $\phi P_n = 558^k$ @ $L_b = 22'$
(AISC STEEL MANUAL)

$558^k > 173.32^k$ OK ✓

CHECK COLUMN BELOW LEVEL 2 → LOAD COMBINATION 13

W12x152
$L_b = 16'$



$P_u = 404.69^k$
$M_u = 17.61$ ft-k
PURE AXIAL → TABLE 4-1: $\phi P_n = 1540^k$ @ $L_b = 16'$

$1540^k > 404.69^k$ OK ✓

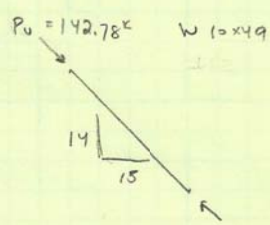
CHECK INTERACTION → TABLE 6-1

$KL: (16)\left(\frac{r_x}{r_y}\right) = (16)(1.77) = 28.32'$

@ 30' $\begin{cases} p \times 10^{-3} = 1.26 \\ b \times 10^{-3} = 1.12 \end{cases}$

$$\frac{(404.69)(1.26)}{1000} + \frac{(1.12)(17.61)}{1000} = .53 < 1.0 \text{ OK ✓}$$

CHECK BRACE BELOW LEVEL 5 → LOAD COMBINATION 13



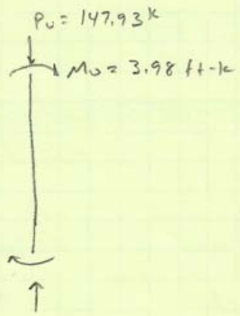
$P_u = 142.78^k$
$L_b = \sqrt{14^2+15^2} = 20.52'$

FROM TABLE 4-1 : $\phi P_n = 295^k$ @ $L_b = 22'$
(AISC STEEL MANUAL)

$295^k > 142.78^k$ OK ✓

CHECK COLUMN BELOW LEVEL 5

W12x96
$L_b = 14'$



$P_u = 147.93^k$
$M_u = 3.98$ ft-k

PURE AXIAL → TABLE 4-1 : $\phi P_n = 957^k$ @ $L_b = 16'$

$957^k > 147.93^k$ <u>ok</u> ✓

<u>CHECK INTERACTION</u>

$KL = (14')\left(\frac{r_x}{r_y}\right) = (14)(1.76) = 24.64'$

@ 26' $\begin{cases} p = 1.66 \\ b = 1.92 \end{cases}$

$$\frac{147.93(1.66)}{1000} + \frac{(3.98)(1.92)}{1000} = .253 < 1.0 \quad \text{ok} ✓$$

Appendix H: Controlling Load Case

Central Tower Diaphragm Displacements Roof		
Load Combination	Displacement	
	UX	UY
COMB 1	0.6264	0.0688
COMB 2	0.058	0.5362
COMB 3	0.4487	0.0626
COMB 4	0.4917	0.0407
COMB 5	0.0096	0.4330
COMB 6	0.0966	0.3715
COMB 7	0.5137	0.4539
COMB 8	0.3293	0.372
COMB 9	0.4091	0.3258
COMB 10	0.3616	0.3555
COMB 11	0.4413	0.3094
COMB 12	1.9916	0.2197
COMB 13	0.1605	0.9974

Central Tower Diaphragm Displacements - Fifth Floor		
Load Combination	Displacement	
	UX	UY
COMB 1	0.4872	0.0487
COMB 2	0.0354	0.4432
COMB 3	0.3483	0.0440
COMB 4	0.3832	0.0291
COMB 5	0.0161	0.3536
COMB 6	0.0692	0.3113
COMB 7	0.3923	0.3690
COMB 8	0.2491	0.2985
COMB 9	0.3131	0.2667
COMB 10	0.2753	0.2873
COMB 11	0.3393	0.2555
COMB 12	1.4880	0.1529
COMB 13	0.1082	0.7905

Central Tower Diaphragm Displacements - Fourth Floor		
Load Combination	Displacement	
	UX	UY
COMB 1	0.3407	0.0291
COMB 2	0.0162	0.3225
COMB 3	0.2424	0.0257
COMB 4	0.2690	0.0179
COMB 5	0.0194	0.2533
COMB 6	0.0438	0.2306
COMB 7	0.2679	0.2638
COMB 8	0.1672	0.2094
COMB 9	0.2147	0.1924
COMB 10	0.1871	0.2036
COMB 11	0.2346	0.1866
COMB 12	0.9778	0.0878
COMB 13	0.0638	0.5392

Central Tower Diaphragm Displacements Third Floor		
Load Combination	Displacement	
	UX	UY
COMB 1	0.2054	0.0134
COMB 2	0.0097	0.1917
COMB 3	0.1477	0.0116
COMB 4	0.1606	0.0085
COMB 5	0.0074	0.1480
COMB 6	0.0219	0.1395
COMB 7	0.1615	0.1538
COMB 8	0.1052	0.1198
COMB 9	0.1273	0.1134
COMB 10	0.1149	0.1175
COMB 11	0.1369	0.1111
COMB 12	0.5408	0.0388
COMB 13	0.0368	0.2911

Central Tower Diaphragm Displacements - Second Floor		
Load Combination	Displacement	
	UX	UY
COMB 1	0.0722	0.0015
COMB 2	0.0044	0.0753
COMB 3	0.054	0.0004
COMB 4	0.0543	0.0026
COMB 5	0.0032	0.0597
COMB 6	0.0035	0.0532
COMB 7	0.0575	0.0554
COMB 8	0.0429	0.0451
COMB 9	0.0431	0.0402
COMB 10	0.0431	0.0429
COMB 11	0.0433	0.0381
COMB 12	0.1610	0.0045
COMB 13	0.0096	0.0917

Combination 12	
Level	Force (kips + ft-k)
Roof	177.8 + 1751.8
5	182.4 + 1979.1
4	128.6 + 1266.4
3	88.4 + 1312.7
2	74.7 + 1110.0

Combination 1 - 1.0WX	
Level	Force (kips)
Roof	31.53
5	61.32
4	58.87
3	77.87
2	100.78

Combination 2 - 1.0WY	
Level	Force (kips)
Roof	54.87
5	107.1
4	103.18
3	102.97
2	154.63

Combination 13	
Level	Force (kips + ft-k)
Roof	177.8 + 1068.1
5	182.4 + 1094.7
4	128.6 + 771.4
3	88.4 + 1009.9
2	74.7 + 837.2

Central Plant Diaphragm Displacements - Roof		
Load Combination	Displacement	
	UX	UY
COMB 1	0.0146	0.0004
COMB 2	0.0003	0.333
COMB 3	0.0109	0.0002
COMB 4	0.011	0.0007
COMB 5	0.0005	0.246
COMB 6	0.0009	0.254
COMB 7	0.0112	0.2503
COMB 8	0.0078	0.1846
COMB 9	0.0088	0.1907
COMB 10	0.0079	0.1853
COMB 11	0.0089	0.1913
COMB 12	0.1379	0.0009
COMB 13	0.0002	0.9244

Central Plant Diaphragm Displacements - Second Floor		
Load Combination	Displacement	
	UX	UY
COMB 1	0.0081	0.0004
COMB 2	0.0002	0.2015
COMB 3	0.006	0.0004
COMB 4	0.0062	0.001
COMB 5	0.0007	0.1569
COMB 6	0.0011	0.1453
COMB 7	0.0062	0.1508
COMB 8	0.0039	0.1181
COMB 9	0.0053	0.1094
COMB 10	0.0041	0.1171
COMB 11	0.0054	0.1084
COMB 12	0.0556	0.0052
COMB 13	0.0005	0.4643

Combination 12 - 1.0EX	
Level	Force (kips + ft-k)
Roof	156.5 +1034.0
2	56.3 + 375.5

Combination 1 - 1.0WX	
Level	Force (kips)
Roof	10.87
2	21.6

Combination 13 - 1.0EY	
Level	Force (kips + ft-k)
Roof	156.5 + 376.0
2	56.3 + 135.2

Combination 2 - 1.0WY	
Level	Force (kips)
Roof	34.24
2	68.72