SUNY

Upstate Cancer Center

Syracuse, New York

Final Report

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April 4th, 2012

SUNY Upstate Cancer Center | *Syracuse, New York*

Building Information

- **Occupancy Type:** Healthcare
- **Size:** 5 Stories
  90,000 Square Feet
- **Construction Dates:** Mar. 2011 – Sept. 2013
- **Cost:** $ 74 Million
- **Delivery Method:** Design - Bid - Build

Project Team

- **Owner:** SUNY Upstate Medical University
- **Architect/Engineer:** EwingCole
- **Civil Engineer:** Klepper, Hahn, & Hyatt
- **CM:** LeChase Construction, LLC
- **Traffic Consultant:** Fisher Associates

Structural System

- Foundation consists of cast-in-place concrete grade beams with a minimum 6" slab on grade. Grade beams sit atop drilled caissons which transfer load to bedrock.
- The superstructure is composed of structural steel.
- 30' x 30' typical bays composed of composite wide flange beams and girders.
- Floor construction is lightweight concrete topping on composite metal decking
- Lateral force resisting system is composed of ordinary braced frames in conjunction with moment connections.

Mechanical System

- Three 325 ton, electric, single compressor chillers with variable speed drives, used in conjunction with one triple cell cooling tower. (325 tons each cell)
- One 125 ton heat exchanger for winter use
- Three 88 BHP, natural gas / No. 2 fuel oil, hot water boilers. Preheat & Reheat Service
- Seven Custom industrial rooftop units with double wall construction provide 235,000 CFM.
- System consists of VAV boxes combined with a building automation system using Direct Digital Control Panels (DDCP) and Application Specific Controllers (ASC)

Electrical System

- Incoming service: 13.2 kV dual service stepped down via dual 5000kVA 13.2-4.16kV substation w/ secondary distribution switchgear at 5kV.
- A 3000kVA 480-208/120V transformer will provide distribution via a 1000A, 480/277V bus duct.

Architecture

- Five story central tower acts as a hub connecting the existing Upstate Medical University Hospital, Regional Oncology Center, and Gamma Knife Center.
- Features 27 private infusion rooms, three linear accelerator rooms, private counseling space, a personal boutique, meditation space, a family resource center, & a four seasons rooftop healing garden.
- Clad in white insulated metal paneling, interrupted with vision and spandrel glazing.
- 3-story North-East facing entrance atrium with entire façade enclosed by custom fritted glass curtain wall.
- Brick veneer and metal screening architectural accents.

Construction

- The Upstate Cancer Center will be construction in multiple phases including a partial demolition of the Regional Oncology Center and the University Hospital.

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Executive Summary

As part of the State University of New York (SUNY) Upstate Medical University's campus expansion, the SUNY Upstate Cancer Center will be a five story, seventy-two foot tall medical facility located in Syracuse, New York. A steel framing system supports the lightweight concrete and metal deck composite floor system, and lateral forces are resisted by ordinary steel braced frames. The structure sits atop concrete grade beams supported by drilled caissons.

The primary goal of this thesis was to redesign both the gravity and lateral structural systems of the Cancer Center, using reinforced concrete in place of structural steel, with the intentions of decreasing the cost of the structure. In addition, the building was also designed to resist disproportionate collapse in accordance with regulations set by the United State Department of Defense. Maintaining the progressive collapse theme, the building's site was redesigned to limit damage from exterior threats that could initiate a collapse. Carrying this ideology into the building envelope, the main façade of the Upstate Cancer Center was redesigned to accommodate effects from wind and blast pressures, impact of debris, and seismic movement. A heat transfer analysis was conducted to quantify the thermal performance of the new and redesigned glazing system

Building loads associated with the SUNY Upstate Cancer Center were determined in accordance with ASCE 7-10 and the New York State Building Code. Structural design for both the building's lateral and gravity force resisting systems was conducted using ACI 318-08. Progressive collapse design was conducted following protocol from the UFC 4-023-03. A two-way slab with concrete beams on all sides supported the gravity loads, while concrete moment frames resisted lateral loads acting on the building. Final design dimensions resulted in a 9 inch thick slab poured integrally with 22 inch wide by 24 inch deep typical support beams. Columns were chosen as square with dimensions of 24 inches by 24 inches. All perimeter beams were upsized to 22 inches by 28 inches deep and all perimeter columns on the first two stories above grade were upsized to 30 inches by 30 inches to meet progressive collapse requirements. The redesigned concrete structural system cost an estimated $415,644 more than the original steel structural system.

Using the Site Security Design Guide provided by the United State's General Services Administration, the site of the SUNY Upstate Cancer Center was modified to reduce or eliminate the risk of building and structural damage associated with a vehicular impact or exterior explosion. Bollards, planters, trees, and benches were used to disrupt a direct path from the roadway to the building. A plaza was created to increase the standoff distance of the building and therefore dampen the effects of an explosion.

New glazing and a mullion support system were designed to meet the maximum wind pressures, pressure resulting from a 70 pound explosion, impacts from airborne debris, and glazing movement from seismic activity. In addition to choosing an assembly with a low thermal conductance, the new glazing unit was selected with a low-e coating to help reduce heat gain from solar radiation. Heat transfer analyses were conducted for both glazing systems under summer and winter conditions. The proposed alternative glazing unit provided less heat gain in the summer months, however; it caused undesirable heat loss in the winter months.

Acknowledgements

To my parents, family, friends, professors, and colleagues; without whom I could not amount to who I am today.

I would like to extend my personal gratitude to the following individuals and associations for their knowledge, advice, and support throughout this year long project.

Penn State Architectural Engineering Faculty:
Thesis Advisor: Dr. Richard Behr
Dr. Ali Memari
Dr. Linda Hanagan
Professor Kevin Parfitt
Professor Robert Holland
Ryan Solnosky

SUNY Upstate Cancer Center:
Mr. Burton Thomas
Mr. Marius Dumitran

Ewing Cole:
Mr. Jason Wiley
Mr. Patrick Brunner

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)

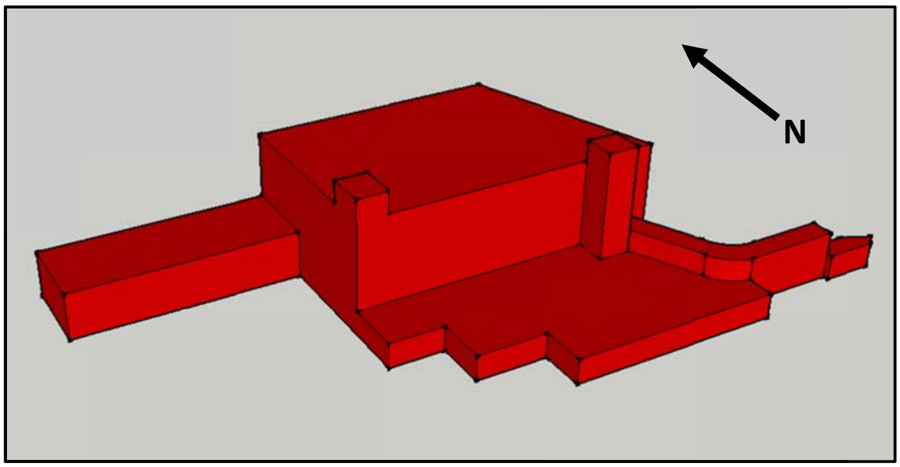
Figure 2 Exterior rendering of northeast entry façade. (Courtesy of EwingCole)

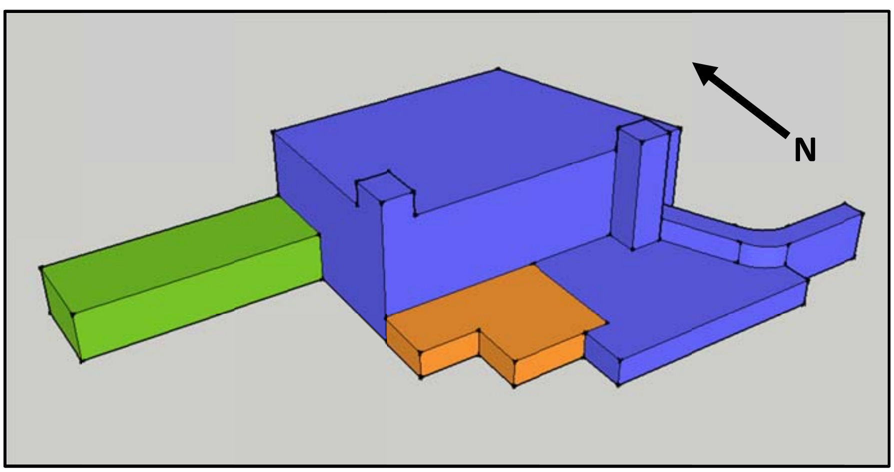
Upstate is committed to providing a comforting environment for its patients, providing amenities such as a meditation room, a boutique for gifts and apparel, and a four-season roof top healing garden. These gardens not only serve as a refreshing oasis, but also help to reduce the cooling costs for the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

Existing 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.

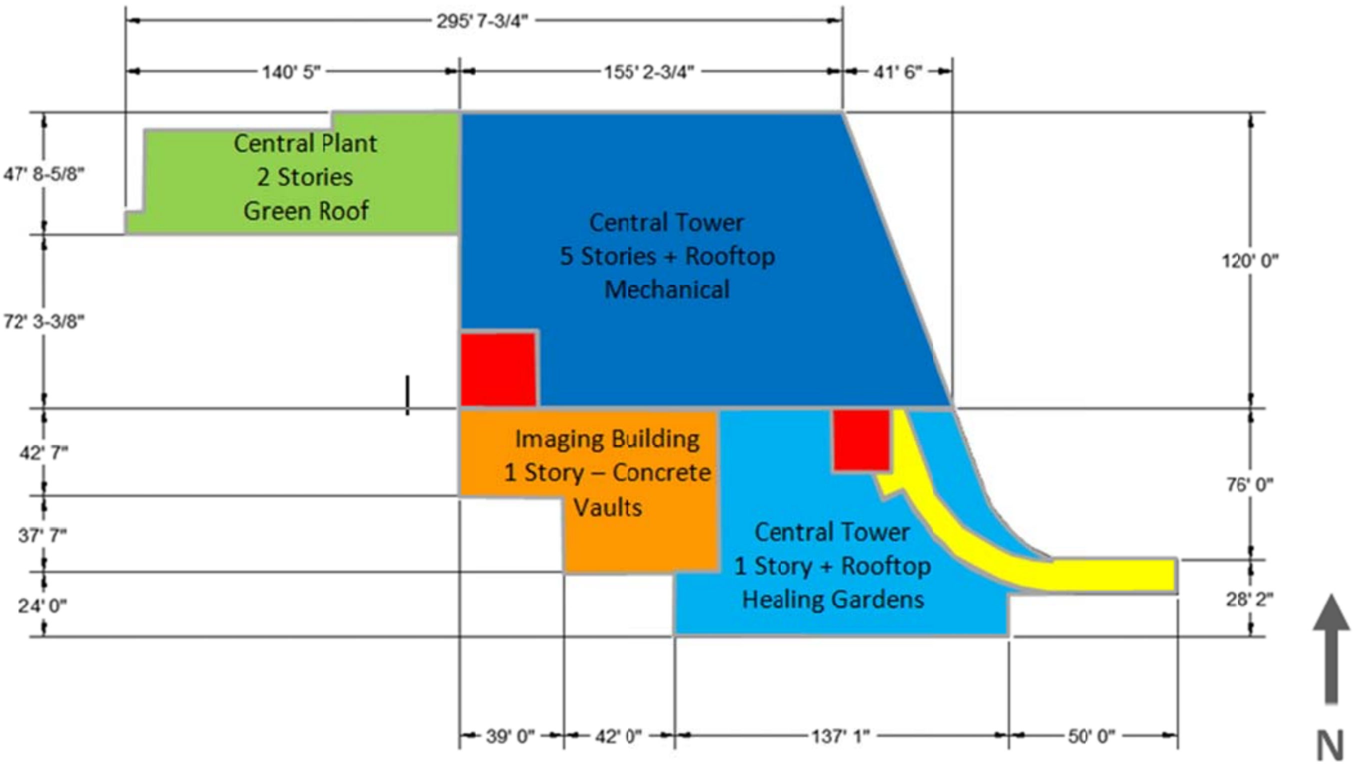


Figure 4 Two-dimensional building key plan showing main building divisions, dimensions, and description. Diagram key given below.

Diagram Key / Roof Elevations

- Central Tower – 72’-0”
- Central Plant – 30’-0”
- Connecting Corridor – 30’-0”
- Central Tower – 16’-0”
- Imaging Building – 16’-0”
- Elevator Core Shafts – 86’6”

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 within the site. 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 socketed 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)

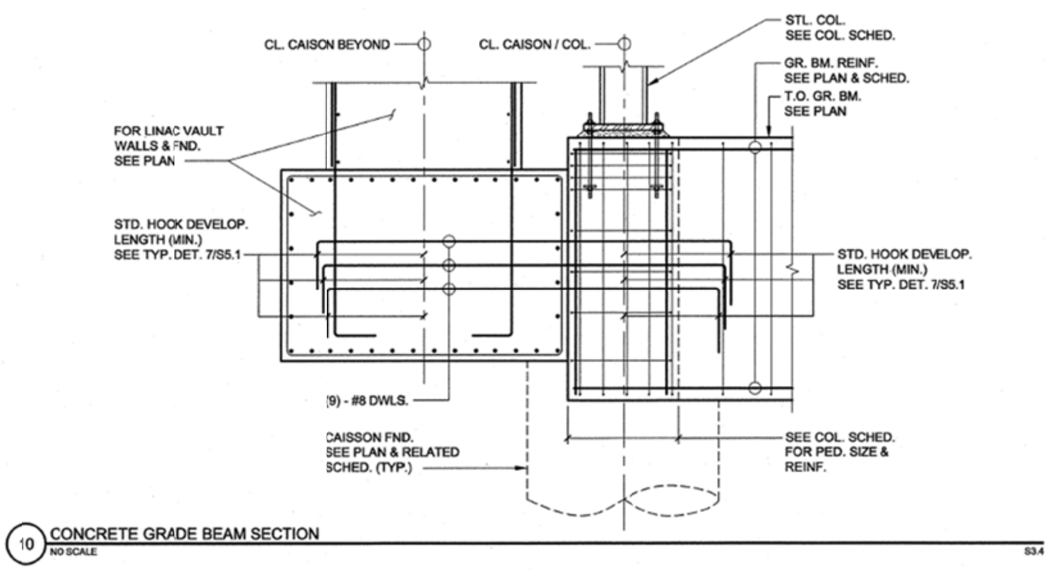


Figure 5 Typical grade beam section from sheet S3. 4
(Courtesy of EwingCole)

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.

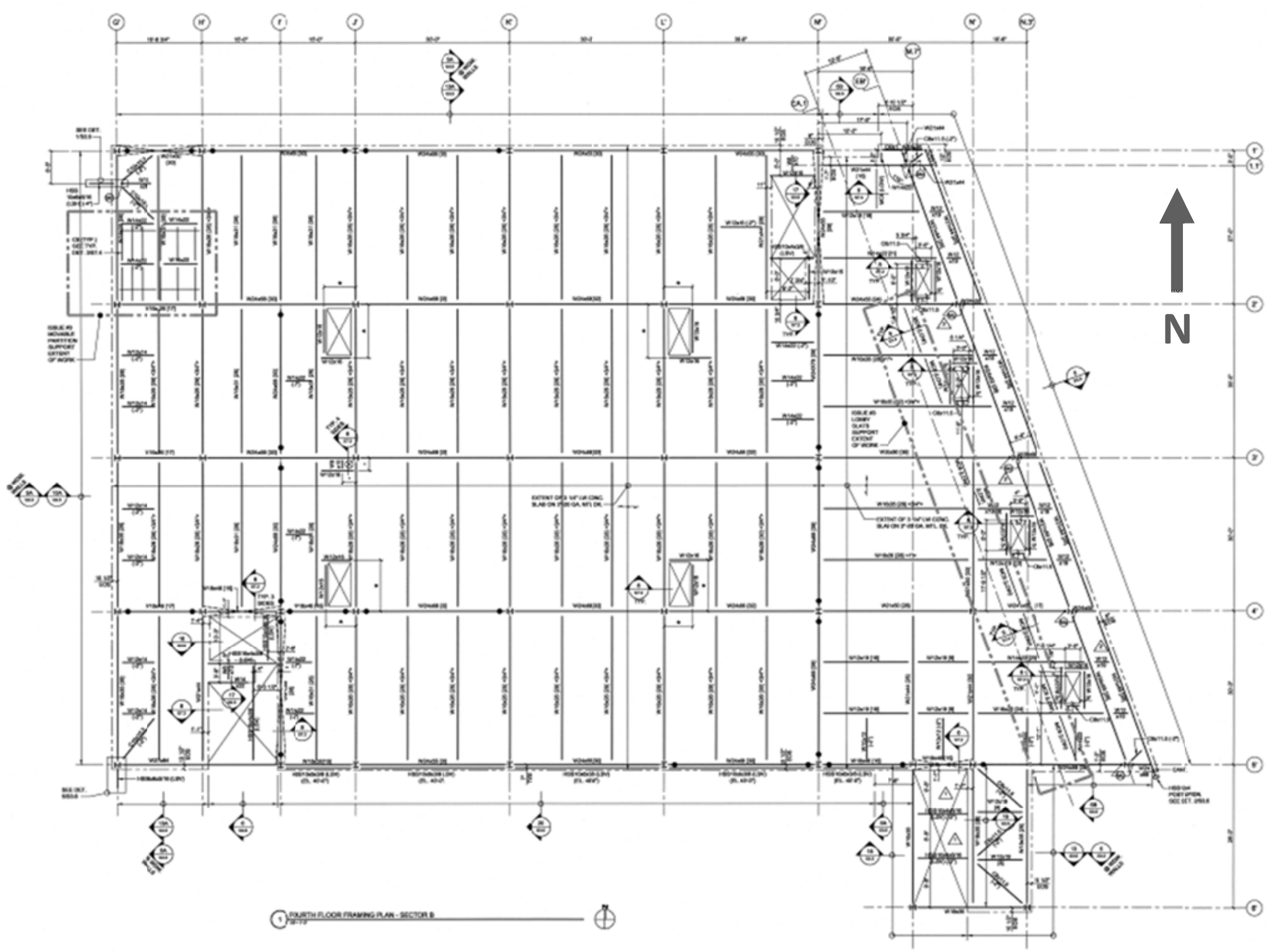


Figure 6 Typical framing layout (Central Tower) Floors two – four (Courtesy of EwingCole)

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 spans 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.

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 location of braced frames, highlighted in blue, within the building at the first story. Heavy black lines denote the location of building expansion joints.

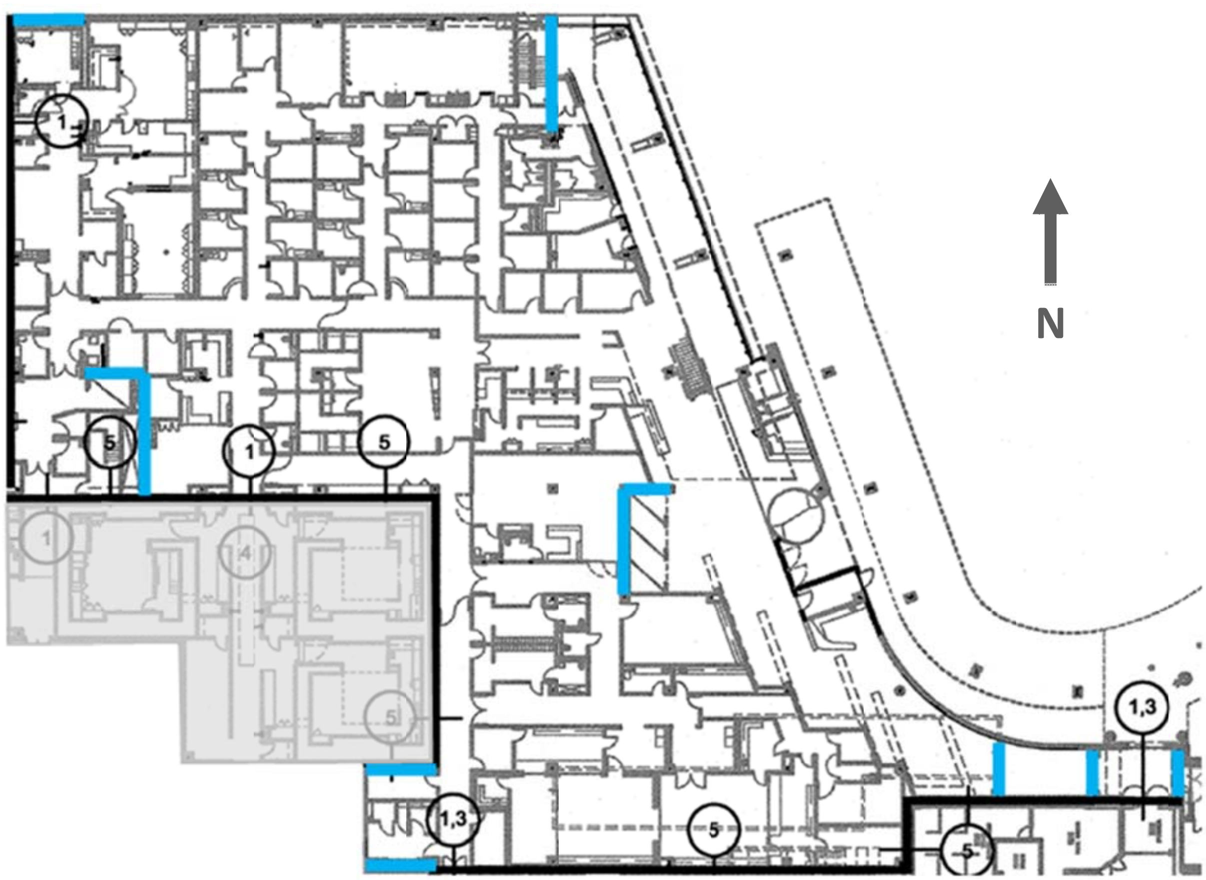


Figure 7 Location of braced frames in the Central Tower. (Courtesy of EwingCole)

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 noted in the structural drawings. Figure 8 below displays an elevation view of the braced frame located long grid line I' between lines 4' and 5'.

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

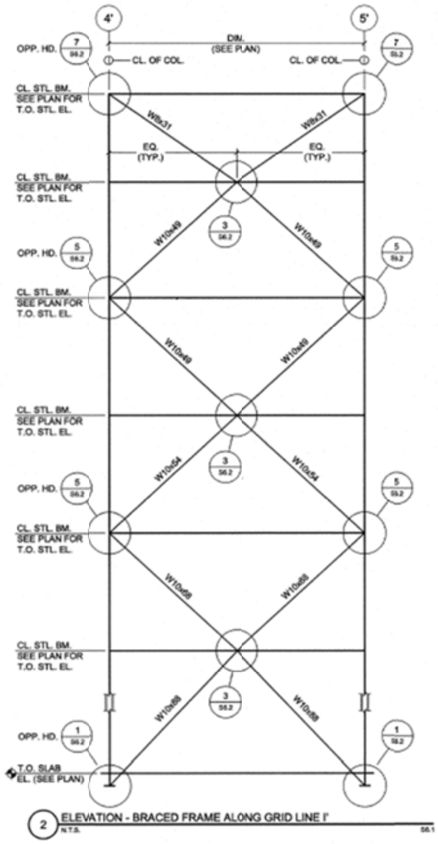


Figure 8 Braced frame elevation along grid line I' between lines 4' & 5' (Courtesy of EwingCole)

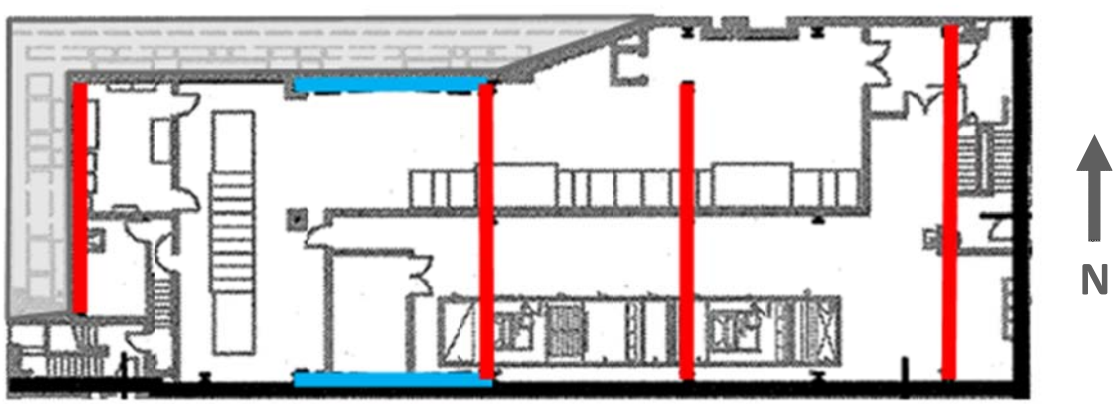


Figure 9 Floor plans showing braced (blue) and moment (red) frames locations in the central plant). (Courtesy of EwingCole)

Thesis Proposal

Structural Depth

Problem Statement

As concluded from previous technical reports, the SUNY Upstate Cancer Center adequately meets structural strength and serviceability requirements with its current design. Presently, the Cancer Center utilizes a steel superstructure, supported by cast-in-place concrete grade beams resting on drilled caissons driven into bedrock. A composite steel and concrete floor system spans the typical 30 foot by 30 foot bays.

In Technical Report 2 various alternative floor systems were explored to determine a suitable substitute for the existing system. Upon comparison of the systems, it was discovered that a reinforced concrete floor system was less expensive than the current composite floor system. Currently, braced frames used are used as the lateral force resisting system, and could potentially disrupt the layout of the undeveloped fourth and fifth floors.

A scenario has been created in which the SUNY Upstate Cancer Center must meet requirements to prevent disproportionate collapse due to the failure of a local structural member.

Problem Solution

Despite the fact that Syracuse, New York is known as a steel dominated city, reinforced concrete will be used to redesign the SUNY Upstate Cancer Center. Based on the cost data gathered from Technical Report 2 using reinforced concrete in place of the existing steel superstructure should reduce the cost of the building's structural system. Lateral forces will be resisted solely by concrete moment frames in the north-south and east-west directions, thus creating an open floor plan, as opposed to a system with concrete shear walls.

Utilizing codes and guidelines established by the United States Department of Defense and General Services Administration, the Cancer Center will be designed to resist progressive collapse from an exterior threat. It is the designer's intention to use the funds saved by switching building materials to compensate for the fee associated with the disproportionate collapse design. In addition to providing lateral resistance, concrete moment frames will be used in the disproportionate collapse design to bridge over missing structural elements and redistribute forces.

A pro / con review of all alternative flooring systems proposed in Technical Report 2 will be conducted to determine the best replacement for the current floor system. Gravity and lateral force resisting systems will be designed in accordance with industry accepted codes and standards. Changing the superstructure of the Cancer Center will surely cause repercussions to the foundation. A generalized foundation check will be conducted to ensure that the foundation can indeed support the load of the redesigned building. Design to resist progressive collapse will be in accordance particularly with the United Facilities Criteria. Because disproportionate collapse design applies only to structures of three or more stories, the five-story Central Tower will be the extent of this thesis' redesign.

Breadth 1 – Risk Mitigation & Site Redesign

Resisting progressive collapse is not accomplished solely by structural modification. Identifying and mitigating potential risks, such as explosions and vehicular impacts, by modifying exterior and landscape architecture is more effective in preventing disproportionate collapse than attempting to arrest the spread of initial structural failure. Adjustments will be made to the existing site of the SUNY Upstate Cancer Center, such as increasing stand-off distance, installing barriers, and employing energy deflection shields to reduce the effect and possibility of a potential threat. A modified site plan will be presented indicating strategies used to accomplish a safer building perimeter.

Breadth 2 – Building Envelope Analysis & Redesign

Presently, a full height glass curtain wall faces the northeast façade of the Cancer Center. Such a vast expanse of glazing presents an issue of heat loss in the winter and heat gain in the summer unless properly addressed. A heat transfer analysis will be conducted on the current façade system, and a proposed alternative system will be presented with aims of improving energy flow through the curtain wall. In addition, the glazing and mullion support system of this curtain wall will be designed to resist the effects of a 70 pound explosive at a distance of 50 feet as well as the impact of flying debris. The glazing pocket will be sized to prevent accidental fallout of the glass panel under structural movement.

MAE Requirements

In order to meet the MAE requirements for this thesis, knowledge and skills acquired from AE 597A, Computer Modeling of Building Structures, and AE 542, Building Enclosure Science and Design, will be applied. The gravity and lateral systems of the SUNY Upstate Cancer Center and the alternative path analysis will be modeled using ETABS and SAP computer modeling software respectively. In turn, data collected from the analyses will be used to design the structural systems of the cancer center. Material covered within AE 542, will be utilized to evaluate and redesign the glass curtain wall detailed in breadth topic two.

Design Codes & 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 Buildings and Other Structures, 2010 Edition
- ASCE 41-06 – Seismic Rehabilitation of Existing Buildings, 2006 Edition
- ACI 318-08 – Building Code Requirements for Structural Concrete and Commentary, 2008 Edition
- UFC 4-023-03 – Design of Buildings to Resist Progressive Collapse, 2009 Edition
- The Site Security Design Guide –General Services Administration
- ASTM E1300 – Standard Practice for Determining Load Resistance of Glass in Buildings
- ASTM F2248 – Standard Practice for Specifying an Equivalent 3s Duration Design Loading for Blast Resistant Glazing
- ASHRAE Handbook of Fundamentals, 2005 Edition

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

Materials

Concrete		
Item	**Weight (pcf)**	**Strength, f'c (psi)**
Piers / Caissons	Normal Weight (145)	5000
Slab on Grade (SOG)	Normal Weight (145)	4000
Beams / Columns / Floor Slabs / Equipment Pads / Sidewalks	Normal Weight (145)	4000
Lower Mechanical Roof Slab Deck	Normal Weight (145)	3500
Composite Floor Slabs	Light Weight (110)	3000
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
Reinforcing Steel	ASTM A615	60
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.

Load Cases & Combinations

ASCE 7-10 lists seven different basic load combinations as stated in section 2.3.2. All combinations were examined with the provided loads computed throughout this report. The selected three have been chosen as the most critical combinations for a typical floor given a specific loading scenario. Combination two (2) governs when considering the effects of gravity loads only. 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) ⟵ Controlling load combination for Gravity Loads
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

In coordination with the above selected load combination, the following wind cases, shown in the figure below, were taken into account during the redesign phase. Likewise, seismic forces acting in the north-south and east-west directions were also accounted for with the appropriate load combination.

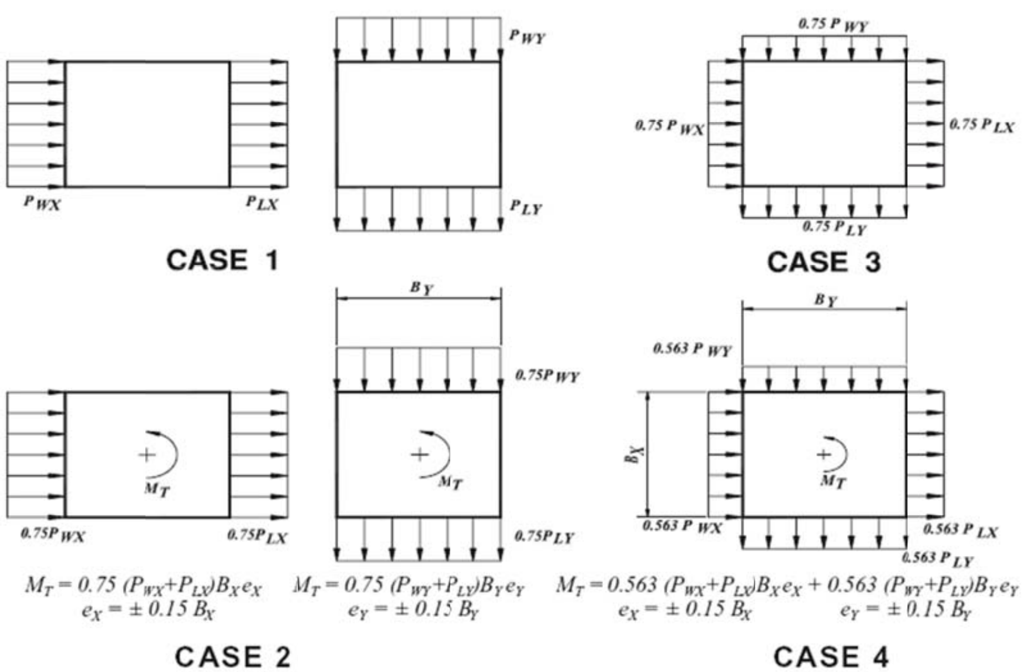


Figure 10 Design Wind Load Cases (Fig. 27.4-8) as specified in ASCE7-10 Chapter 27.

Structural Redesign

In order to determine the best alternative flooring system for the redesign of the SUNY Upstate Cancer Center, the four flooring systems, existing Composite Steel Deck; Precast Hollow Core Plank; Two-Way Flat Slab; and One-Way Pan Joists, explored in Technical Report 2 were further assessed. Several categories including weight, dimensions, cost, architectural considerations, structural consideration, and constructability were examined. A ranking ranging from one to four was assigned to the different floor systems for each category; where one represents the best rank and four represents the worst. The rankings were tallied and the lowest value corresponded to the most suitable, efficient substitute to the existing flooring system; Two-Way Flat Slab. Floor comparison can be found in Appendix A.

Using the Two-Way Flat Slab floor system will ultimately increase the overall weight of the building; however, the system's overall depth and cost will be reduced and leave the current 30 foot by 30 foot bays unaltered. In addition to the above stated, no additional fire-proofing will be needed to protect the concrete structure, and increased floor mass will provide better resistance to structural vibrations. Increased building weight, as a result of employing a heavier floor system will be addressed later in this report with a foundation check and redesign.

Gravity System

After selecting the Two-Way Flat Slab floor system for the structural redesign, a new column layout was established by modifying the existing one. Knowing that concrete moment frames were to be used to counteract lateral forces and assist in the progressive collapse design, the Two-Way Flat Slab was designed with beams spanning between supports in both directions. Initial trial dimensions were found based on code requirements and engineering heuristics. The modified column & beam layouts can be seen in Figure 11 below with a typical bay measuring 30 feet by 30 feet.

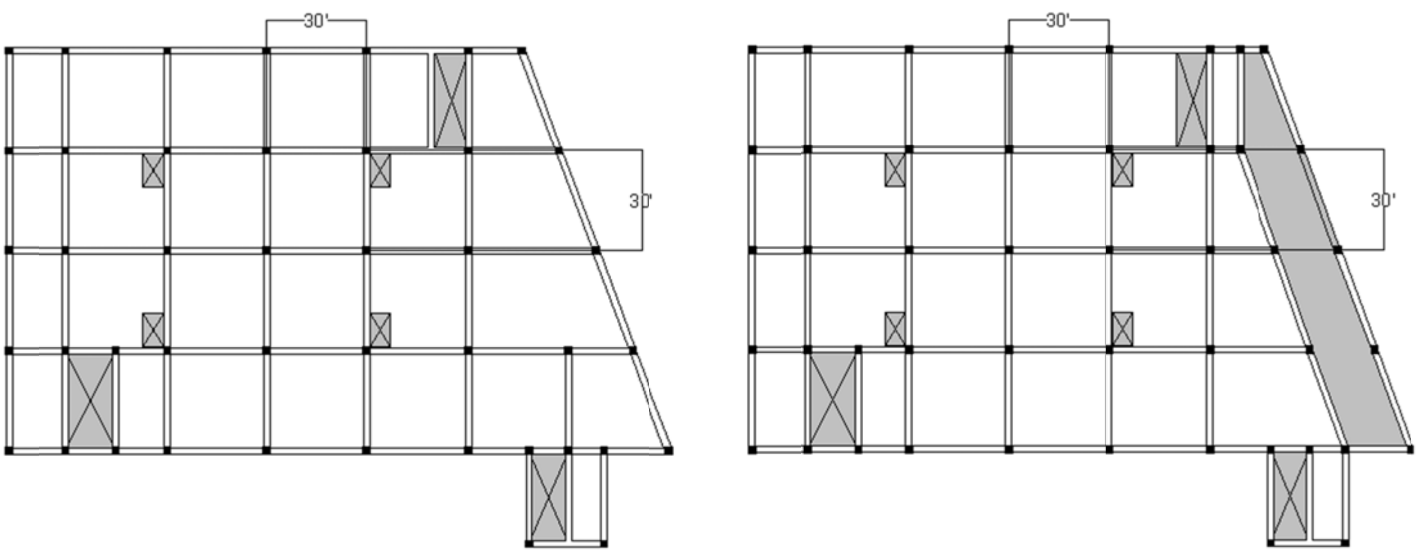


Figure 11 Modified Column / Beam layout for redesign. (Floors 4, 5, and Roof **(Left)**; Floors 2 and 3 **(Right)**
NOTE: Grey shading denotes openings in slab

An initial slab thickness for the flooring system was determined using Table 9.5(c) in ACI 318-08. Since this table corresponds to two-way slabs without interior beams, the obtained value would be slightly adjusted. Based on a 22 inch square concrete column, assuming edge beams in the exterior spans, Table 9.5(c) requires a minimum slab thickness of 10.24 inches. To account for the stiffness of beams spanning between the supports, this value was reduced by fifteen percent. **[1]** Rounding to nominal value, the chosen design slab thickness was 9 inches.

Beam depths were taken to be 2.5 times the design slab thickness. **[1]** Rounding the calculated value to an even depth resulted in 24 inches. Slabs, beams, and columns were assumed to be poured integrally with each other; therefore, the beam widths were taken as 22 inches. The resulting members were a 9 inch slab with 22 inch by 24 inch beams spanning between square columns measuring 22 inches by 22 inches.

Now that the trial dimensions were established, the design of the two-way slab with beams could begin. After reviewing the two design procedures discussed in ACI 318-08 Section 13.5, it was decided to follow the Equivalent Frame Method because of less stringent requirements. Initial trial dimensions were used to calculate the actual required slab thickness specified in Section 9.5.3.3 of ACI 318-08. Assuming the worst case scenario, the maximum required slab thickness was calculated as 8.62 inches, which is less than the trial dimension; therefore, the design moved forward using a nine inch slab. Design loads related to gravity forces were determined in order to proceed.

Dead Loads

Dead loads were calculated based on the loading that was considered permanent over the life of the life of the building. Items included in the calculation consisted of the self weight of the slab; beams; columns; exterior façade assemblies; mechanical, electrical, and plumbing (MEP) equipment; ceiling and floor finishes; and any specified permanent equipment. Weights of common building materials were gathered from literature or assumed based on engineering judgment, erring on the conservative side. Table 2 below lists the typical loads for various building components. It should be noted that “Super Imposed” encompasses MEP equipment and ceiling and floor finishes. No values are listed for permanent equipment. This table is only applicable for the Central Tower redesign.

Dead Loads	
Description	**Load**
Slab (9” Thick)	112.5 psf
Beams (22” x 24”)	23 psf
Columns (22” x 22”)	10 psf
Facades:	
Curtain Wall Glazing	15 psf
Insulated Metal Paneling	20 psf
Brick Veneer	40 psf
Super Imposed Dead Load:	
Floors	25 psf
Roof	10 psf

Table 2 Break down of typical dead load for Central Tower Redesign

[1] Wight, James K., and James G. MacGregor. *Reinforced Concrete Mechanics & Design*. 5th. Upper Saddle River: Pearson Prentice Hall, 2009. 734. Print

Live Loads

Existing 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 loads used in the Central Tower redesign 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 the existing steel structure versus the live load values used for the Central Tower redesign documented in this report.

Live Loads		
Occupancy Type	**Existing Design Live Load (psf) N. Y. State Building Code (2007)**	**Redesign Live Load (psf) IBC 2009 / ASCE 7-10**
Public Space / Typical Floor	100	100
Corridors	100	100
Mechanical Building Spaces	250	250
Typical Roof	45	20
Rooftop Gardens	100	100
Rooftop Mechanical Locations	150	125

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

Snow Loads

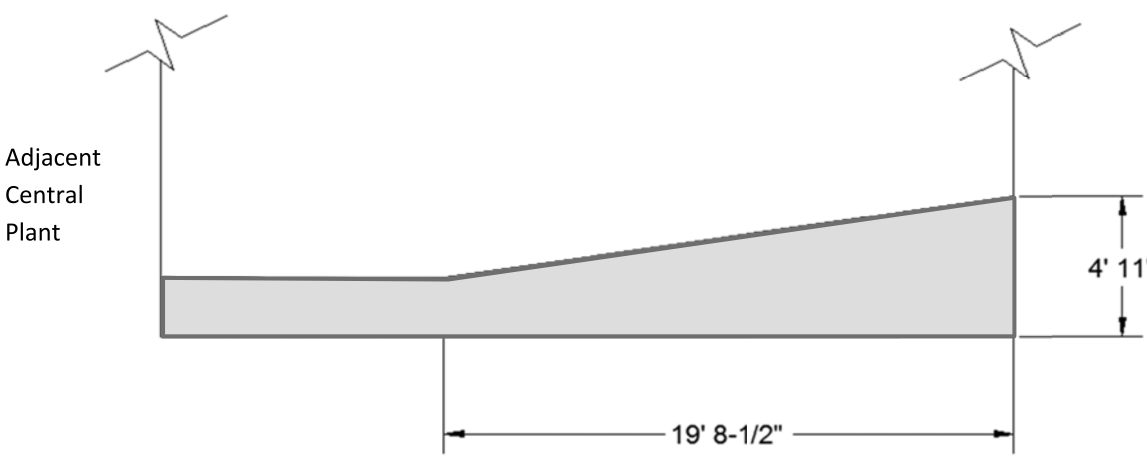
Snow Load was calculated for the Upstate Cancer Center using ASCE 7-10 Section 7.3, flat roof snow loads. Upon viewing the ground snow load map provided in ASCE 7-10 (Figure 7-1), it was determined that Syracuse, New York requires a case study ground snow load. Figure 1608.2 of the 2007 Building Code 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 B. 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.

nt



lower

Now that design loads have been determined, factored slab moments were found and distributed to the corresponding column and middle strips based on the provisions stated in Section 13.7 of ACI 318-08, Equivalent Frame Method. Positive and negative slab moments used for calculation were obtained from a frame analysis carried out using SAP 2000. Design moments were justified through moment coefficient calculations, outlined in section 8.3.3 of ACI 318-08. Primary reinforcing steel selected for the slab were #5 ASTM A615 60 ksi deformed bars in both directions with minimum clear cover of 0.75 inches to the edge of slab. Spacing of bars varied for different bays, but was limited to two times the depth of slab, or 18 inches, per code requirements. The following figures (Figures 13 & 14) illustrate the slab reinforcement scheme for the fourth floor in both plan and elevation. Sample calculations for the design of the two-way slab with beams can be found in Appendix C.

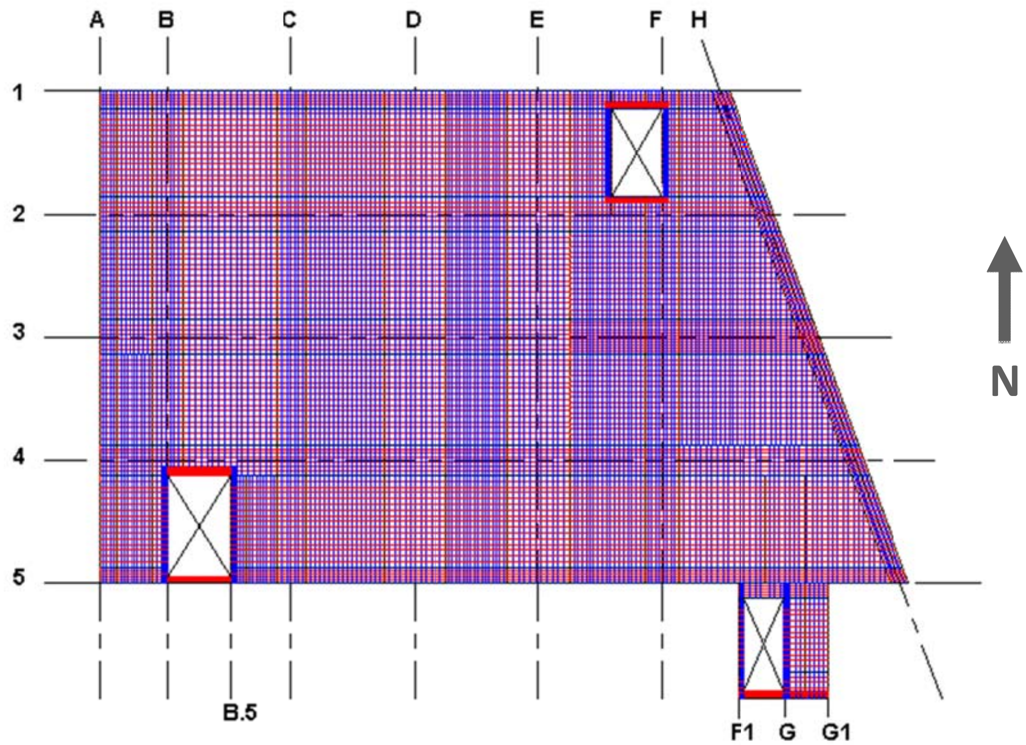


Figure 13 Slab reinforcement layout for gravity loads – 4th Floor – Plan view (E-W Reinforcement **(Red)**; N-S Reinforcement **(Blue)**)

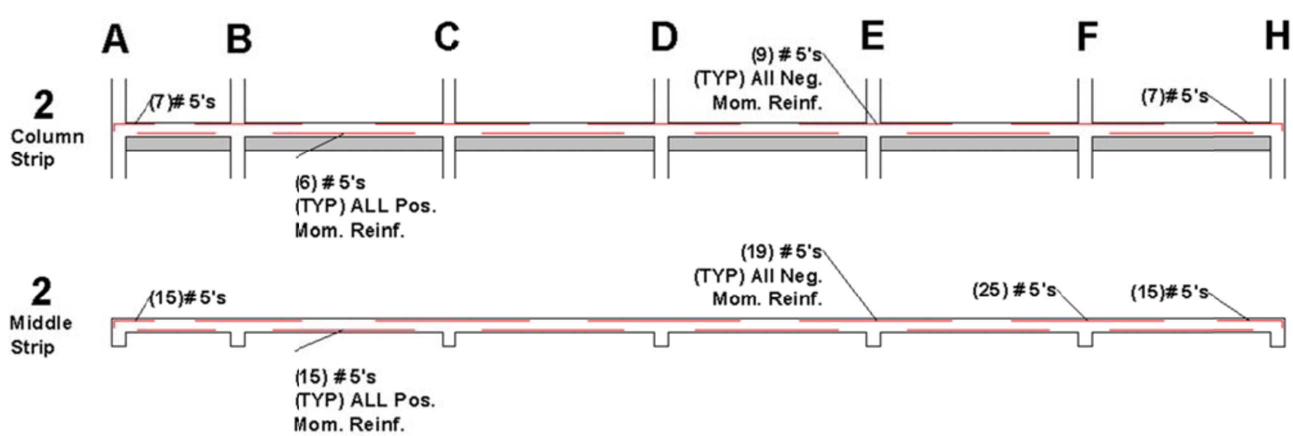


Figure 14 Slab reinforcement layout for gravity loads – 4th Floor – Elevation view (Column & Middle Strips along Column Line 2)

After detailing the slab flexural reinforcement, a two-way shear (punching shear) failure was addressed. Since the floor system was supported by concrete beams spanning between the columns in both directions, a two-way shear failure at the columns was assumed to be resisted by the beam framing elements. A check was conducted to ensure that excessive shear was not transferred to the beams, and the resulting forces could be resisted with the addition of reinforcing stirrups. Floor slabs were also checked for a one-way shear (wide beam shear) failure. The specified 9 inch slab proved to be more than adequate to carry the required loads. Beam design pertaining to gravity forces will be explained in the following paragraphs.

From the two-way slab design process, ACI 318-08 specifies that eighty-five percent of the factored moment transferred to the column strips is in turn transferred to the gravity beams. Using this methodology, positive and negative factored moments were determined, and beam longitudinal reinforcement was sized. Typical effective depth was taken as the distance from the top or bottom of the beam to the centroid of the reinforcing steel; 21.5 inches. Reinforcement was almost exclusively kept in a single layer with a few exceptions. Minimum reinforcing steel required by ACI 318-08 was governed by the quantity (200*b*d/fy), and the minimum spacing of the reinforcement was specified as the diameter of the reinforcing bar but not more than 1 inch. Typical longitudinal bars ranged from #6 to #9 ASTM A615 60 ksi deformed bars with a few exceptions.

Beams were also checked and designed for one-way shear accounting for shear from distributed loads as well as unbalanced moment between beam ends. Additional shear force needed to sustain the required loading was attained through the addition of shear reinforcement provided in the form of #3 stirrups. Spacing of stirrups varied from beam to beam but were limited to a maximum spacing of 10 ¾ inches. Calculations pertaining to beam design for gravity loads can be found in Appendix C. A typical beam reinforcement layout for gravity loads only can be seen the figure below.

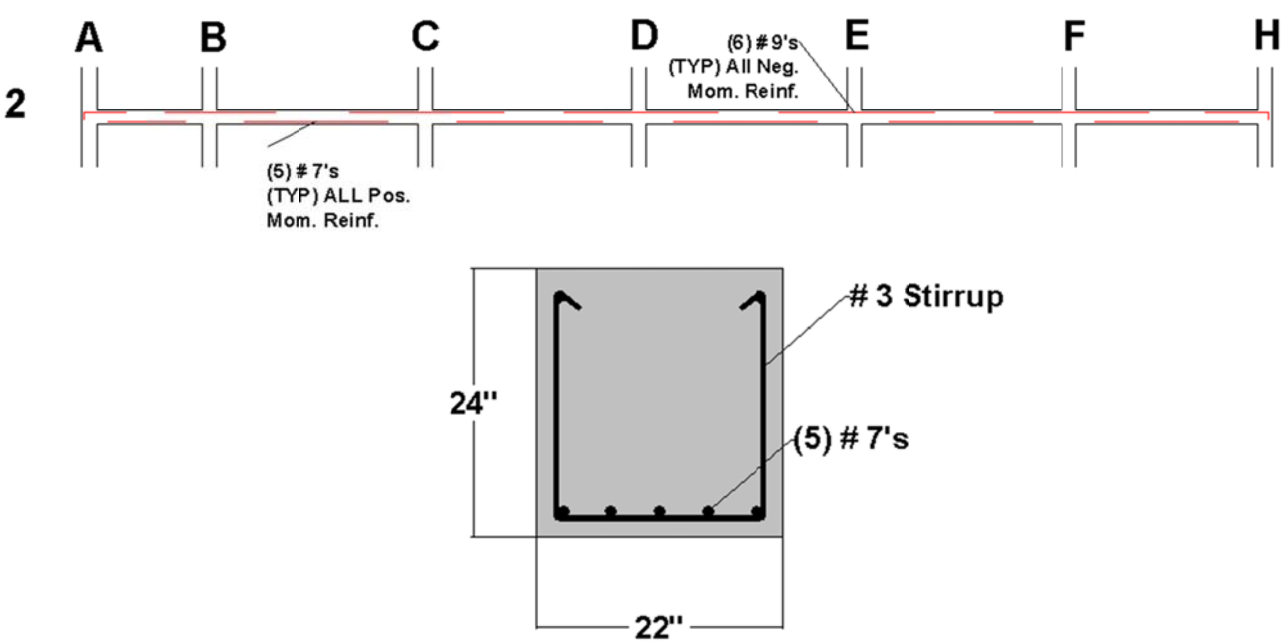
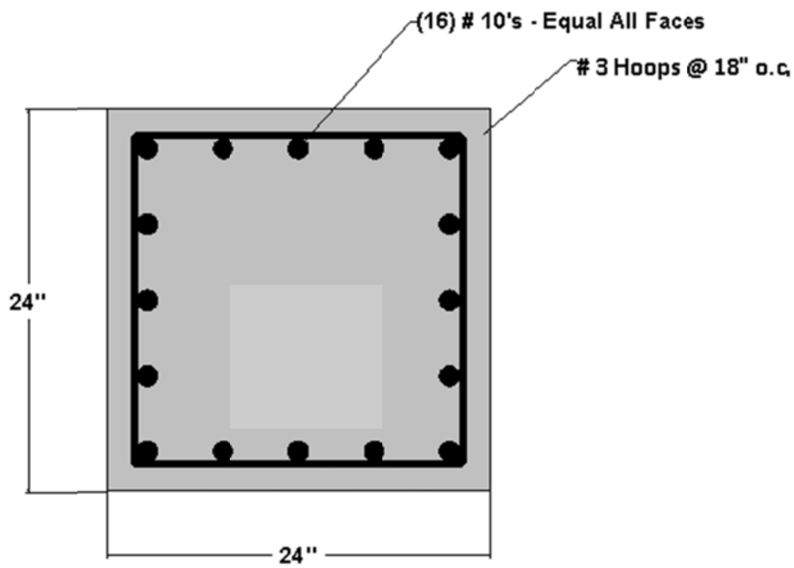


Figure 15 Reinforcement layout and cross-section of a typical beam for gravity loads – 4th Floor – Column Line 2

Finally, columns were designed for the associated gravity loads. An interior column at the first story was selected for design purposes because it carried the greatest axial load. Steel reinforcement was targeted between 1 and 8 percent of the total column cross-sectional area. Final design calculations resulted in a 24 inch by 24 inch column with (16) #10 ASTM A615 60 ksi deformed bars, five bars in each face, resulting in a reinforcement ratio of 3.5 percent. Confinement reinforcement was specified as #3 bars at a spacing of 18 inches. Because spacing between the bars was less than 6 inches, no cross-ties were required for the column. Figure 16 shows a cross section of the designed column.



Lateral System

Designing the building for gravity loading only, provided a basis from which the lateral system would develop. As stated previously, lateral forces acting upon the Central Tower of the SUNY Upstate Cancer Center will be countered by concrete moment frames running in both primary axes of the building. Lateral forces accounted for in this design will include both wind and seismic effects; these will be calculated in the following sections.

The lateral force resisting system will be modeled under the controlling lateral load case using ETABS computer modeling software. Design forces gathered from the analysis will then be used to design the beams and columns of the moment frames.

Wind Loads

Wind loads were calculated for the Central Tower of the Upstate Cancer Center using the Main Wind Force Resisting System (MWFRS) directional procedure for buildings of all heights outlined in ASCE 7-10 Chapter 27. Since this structural redesign only concerns the Central Tower of the Cancer Center, building expansion joints were placed to effectively isolate that portion of the building. Figure 17 shows the entire SUNY Upstate Cancer Center complex with the Central Tower, highlighted in blue, on the left. The image on the right shows the simplified geometry of the Upstate Cancer Center, which was used for calculating wind loads on the building.

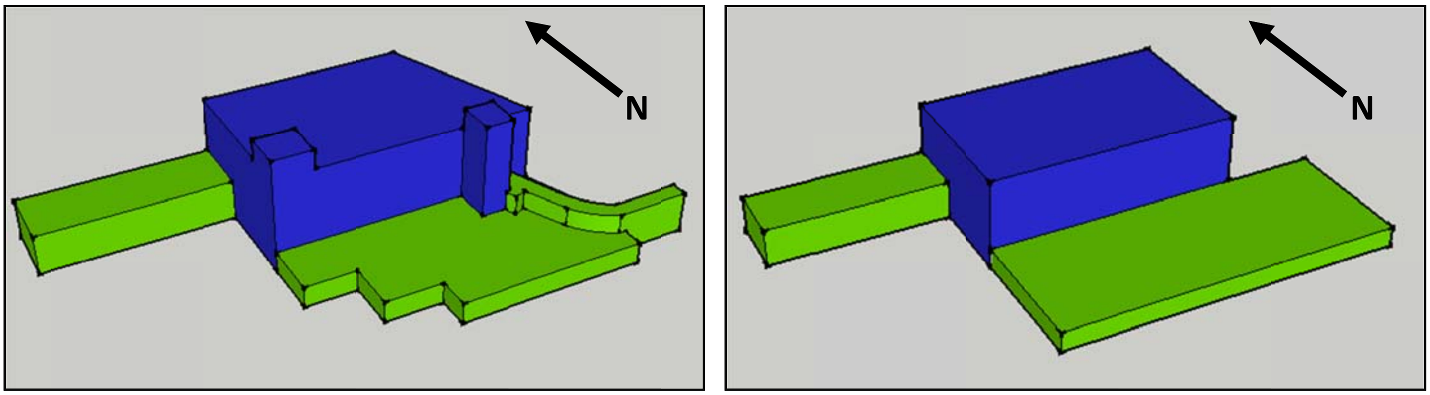


Figure 17 Google SketchUp model showing entire SUNY Upstate Cancer Center with Central Tower highlighted in blue **(Left)** and simplified geometries used for wind load determination. **(Right)**

Based on its location and urban setting, the Cancer Center was placed in Exposure Category B with a Basic Wind Speed of 120 miles per hour, in accordance with ASCE 7-10. Using section 26.9.3, the building's lower bound frequency was estimated to be 0.927 Hertz. Since this value is less than 1.0 Hertz, the building is classified as flexible by definition stated in Section 26.2. Gust factors were calculated for each major building direction following procedures outlined for flexible buildings. An internal pressure coefficient of +/- 0.18 was selected based on the assumption that windows were non-operable and glazing was impact resistant. Using the MWFRS, wind pressures were established for the windward, leeward, sidewalls, and roof locations in both the building's north-south and east-west directions. Detailed calculations used to determine wind pressures and forces can be found in Appendix B.

Wind hits the façade of the Cancer Center, with a parabolic pressures distribution, as shown in Figures 18 and 19 as well as Tables 5 and 6. As the façade fields the wind, pressures are transferred to the metal stud backup wall which is anchored to the floor slabs. Because the floor slabs are concrete and cast integrally with the beams and columns of the building, they act as a rigid diaphragm. Loads are transferred through the diaphragms to the moment frames perpendicular to the wind direction, with stiffer frames taking more share of the load. All the forces within a frame are resisted in the foundation through shear, uplift, and downward forces. Following this path, wind pressures were resolved into lateral forces acting at each story level. Visual representation of this data can be found in Tables 7 and 8 and Figures 20 and 21.

Wind Pressures (E-W Direction)						
Location	Level	Distance (ft)	K_z	q_z	q_h	Wind Pressure (psf)
Windward	Ground	0.0	0.57	18.01	28.20	17.54
	Two	16.0	0.59	18.34	28.20	17.77
	Three	30.0	0.70	21.95	28.20	20.27
	Four	44.0	0.78	24.49	28.20	22.02
	Five	58.0	0.85	26.50	28.20	23.42
	Roof	72.0	0.90	28.19	28.20	24.58
Leeward	All	0.0 - 72.0	0.90	28.19	28.20	-14.10
Side Walls	All	0.0 - 72.0	0.90	28.19	28.20	-22.15
Roof	-	0.0' - 36.0'	0.90	28.19	28.20	-27.02
	-	36.0' - 72.0'	0.90	28.19	28.20	-27.02
	-	72.0' - 144.0'	0.90	28.19	28.20	-17.27
	-	> 144.0'	0.90	28.19	28.20	-12.39

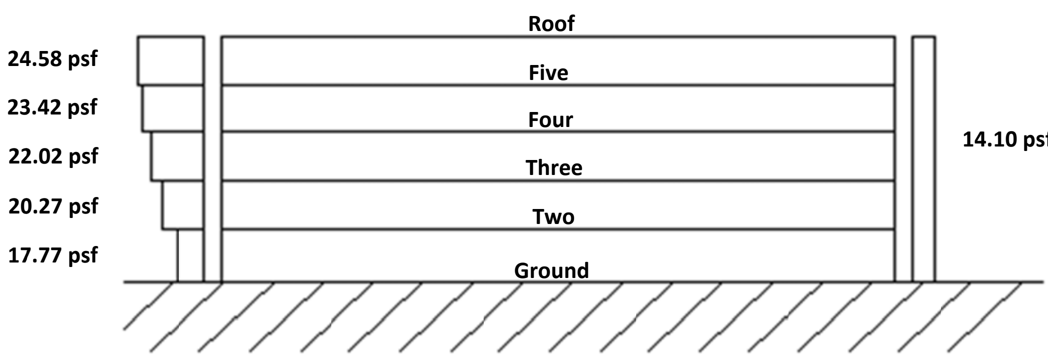


Table 5 / Figure 18 Table and Diagram of wind pressures in the East-West direction
NOTE: Roof uplift pressures displayed on the Story Force Diagram (Figure 20)

Wind Pressures (N-S Direction)						
Location	**Level**	**Distance (ft)**	**K_z**	**q_z**	**q_h**	**Wind Pressure (psf)**
Windward	Ground	0.0	0.57	18.01	28.20	17.19
	Two	16.0	0.59	18.34	28.20	17.42
	Three	30.0	0.70	21.95	28.20	19.85
	Four	44.0	0.78	24.49	28.20	21.55
	Five	58.0	0.85	26.50	28.20	22.91
	Roof	72.0	0.90	28.19	28.20	24.04
Leeward	All	0.0 - 72.0	0.90	28.19	28.20	-16.93
Side Walls	All	0.0 - 72.0	0.90	28.19	28.20	-21.67
Roof	-	0.0' - 36.0'	0.90	28.19	28.20	-28.31
	-	36.0' - 72.0'	0.90	28.19	28.20	-25.47
	-	72.0' - 144.0'	0.90	28.19	28.20	-17.88
	-	> 144.0'	0.90	28.19	28.20	-14.09

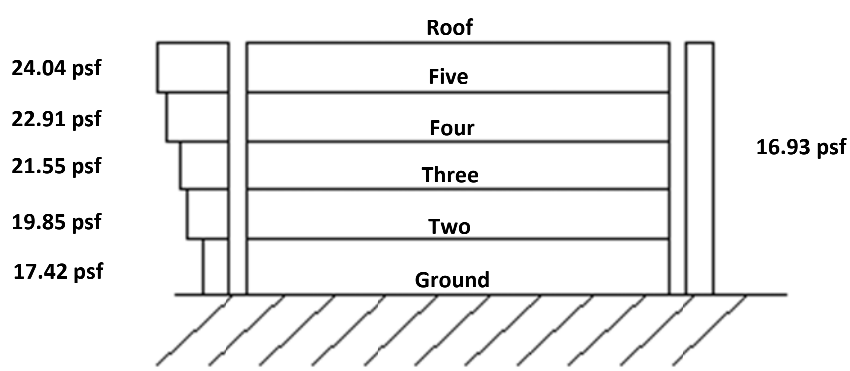


Table 6 / Figure 19 Table and Diagram of wind pressures in the North-South direction
NOTE: Roof uplift pressures displayed on the Story Force Diagram (Figure 21)

Wind Forces (E-W Direction)						
Floor Level	Elevation (ft)	Façade Area (ft^2)	Total Pressure (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.0	1184	31.64	37.46	372.07	0.00
Second	16.0	2220	31.87	70.75	334.61	1131.98
Third	30.0	2072	34.37	71.21	263.86	2136.20
Fourth	44.0	2072	36.12	74.85	192.65	3293.25
Fifth	58.0	2072	37.51	77.73	117.81	4508.38
Roof	72.0	1036	38.68	40.08	40.08	2885.48
			Total Base Shear =	372.07		
					Total Overturning Moment =	13955.30

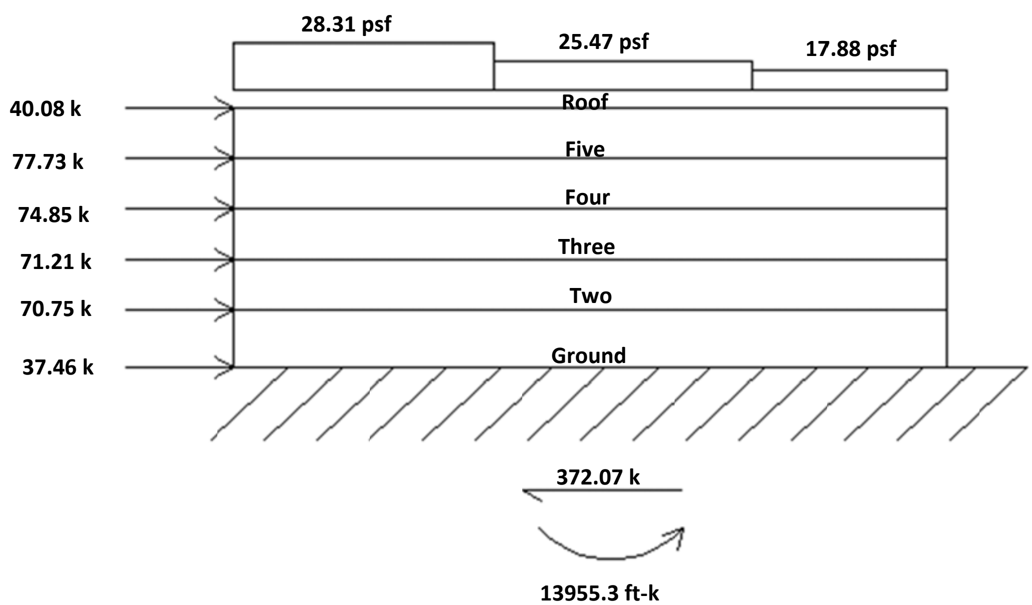


Table 7 / Figure 20 Table and Diagram of wind forces in the East-West direction

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	1574	34.12	53.71	528.48	0.00
Second	16.0	2951	34.35	101.36	474.77	1621.80
Third	30.0	2754	36.78	101.28	373.40	3038.48
Fourth	44.0	2754	38.48	105.99	272.12	4663.40
Fifth	58.0	2754	39.84	109.71	166.13	6363.37
Roof	72.0	1377	40.97	56.42	56.42	4062.33
			Total Base Shear =	528.48		
					Total Overturning Moment =	19749.38

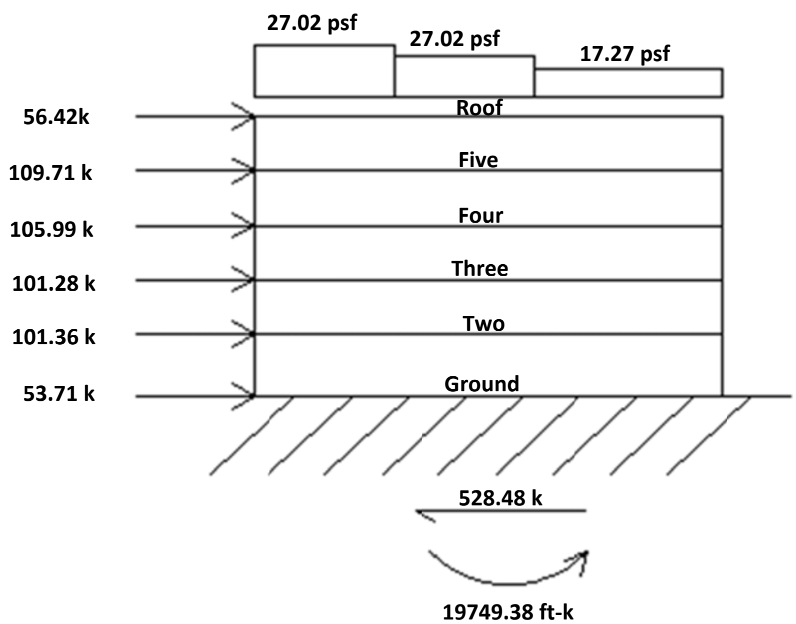


Table 8 / Figure 21 Table and Diagram of wind forces in the North-South direction

In summary, the wind analysis produced base shears of 372.07 kips and 528.48 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. Internal pressures were neglected in wind calculations because they are equal and oppose each other, essentially negating themselves.

Seismic Loads

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. Since the Central Tower is isolated from the rest of the Upstate Cancer Center through use of building expansion joints, seismic analysis for this portion of the building will be independent of the rest of the structure.

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 seismic base shear for the Central Tower, the building's weight needed to be calculated. Accounting for loads that were considered "permanent or attached" to the structure at stories elevated above ground level, the total building weight for the Central Tower was approximately 19759 kips. In comparison to the original steel structure, the weight of the redesigned Central Tower is nearly double.

To finish the base shear calculation and ultimately determine the lateral story forces, the seismic response coefficient, C_s, still needed to be obtained. Based on the Central Tower's Seismic Design Category, SDC C, ACI 318-08 requires that concrete moment frames be designed as Intermediate Moment Frames. This designation results in a response modification factor of 5.0; however, a response modification factor of 3.0 was used conservatively. The building period used for the seismic response coefficient calculation was taken as the smallest of the upper limit of calculated periods (ASCE 7-10 Section 12.8.2) and the period values obtained from the ETABS computer model, which will be discussed in the next section.

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 column. These floors act as rigid diaphragms transferring the generated seismic loads to the moment frames of the building which subsequently transfers the force to the foundation.

Forces were calculated for each floor using ASCE 7-10 Equation 12.8-11, Vertical Distribution of Forces, and are represented in Tables 9 and Figures 22. 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 during seismic force calculations assuming an offset of the center of mass equal to five percent of the building's dimension perpendicular to the load applied. Torsional amplification, Ax, was also taken into account and addressed. Calculations of accidental torsion can be found in tables 10 and 11. Due to the fact that not all lateral resisting frames are parallel to the major orthogonal axes of the building, the Central Tower has Horizontal Structural Irregularity Type 5. This issue will be addressed by adjusting the seismic loading, and will be discussed further later in this report. Calculations pertaining to seismic loads can be found in Appendix B.

Direct Shear

Seismic Forces (V_b = 765 kips, T = 1.28s, k= 1.39)								
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-kips)
Roof	14	72	3802	1451123	0.3617	277	765	19920
Fifth	14	58	4087	1154967	0.2878	220	488	12772
Fourth	14	44	4087	786690	0.1961	150	268	6599
Third	14	30	3835	433475	0.1080	83	118	2479
Second	16	16	3947	186206	0.0464	36	36	568
Totals			19758	4012461		765		42338

Table 9 Seismic forces calculated for the Central Tower.

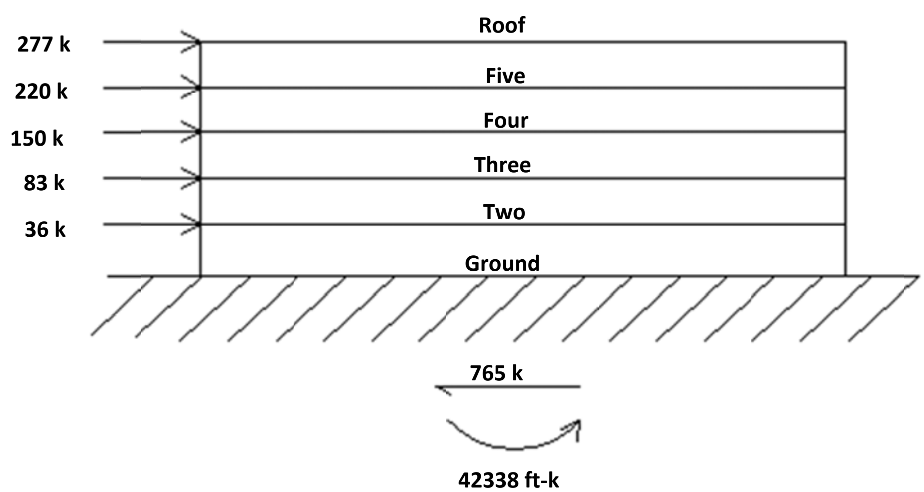


Figure 22 Diagram of Seismic forces acting on the Central Tower. (Both Directions)

Accidental Torsion

Accidental Torsion Due to Seismic Loads (E-W Directional Loading)								
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-kips)
Roof	14	72	277	765	120	6.00	1.00	1660
Fifth	14	58	220	488	120	6.00	1.00	1321
Fourth	14	44	150	268	120	6.00	1.00	900
Third	14	30	83	118	120	6.00	1.00	496
Second	16	16	36	36	120	6.00	1.00	213
Totals			765					4590

Table 10 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 Directional Loading)								
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-kips)
Roof	14	72	277	765	196.73	9.84	1.01	2749
Fifth	14	58	220	488	196.73	9.84	1.01	2188
Fourth	14	44	150	268	196.73	9.84	1.00	1475
Third	14	30	83	118	196.73	9.84	1.00	813
Second	16	16	36	36	196.73	9.84	1.00	349
Totals			765					7574

Table 11 Accidental torsion produced in the Central Tower 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 765 kips with an accidental torsion of 4590 foot-kips in the East-West direction and 7574 foot-kips in the North-South direction.

Computer Model (ETABS)

In order to analyze the lateral force resisting 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. Only lateral forces, wind and seismic, were considered in this analysis; therefore, modeling was limited only to those participating in the lateral system of the structure. When choosing the participating elements for the lateral system redesign of the Central Tower, only moment frames that spanned the full height of the building were selected to be modeled. This ideology assured that lateral forces would indeed reach the foundation of the building and not be distributed elsewhere. Figures 23 and 24 illustrate the proposed lateral system in three-dimensional and plan view.

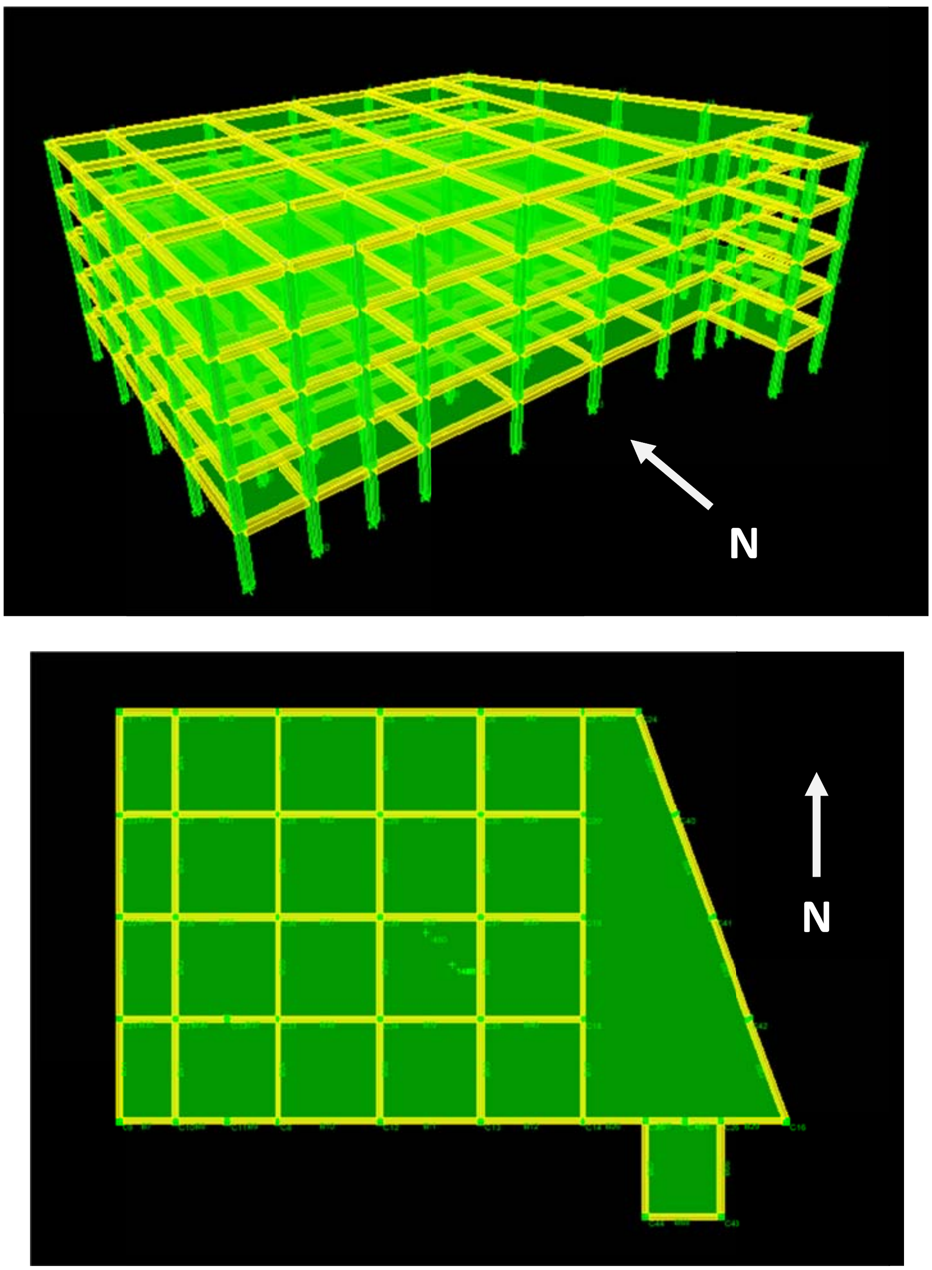


Figure 23/24 ETABS structural computer model of lateral force resisting system, (three-dimensional and plan views)

In best attempt to have the computer model behave as intended, the following assumptions and considerations were made:

- All elements were composed of 4000 psi concrete with E_c = 3600 ksi and a mass of 0. (Mass would be lumped at the center of mass of each floor)
- All Columns were 24 inches by 24 inches. All columns were assumed cracked in both axes. (Icr = 0.7*Ig)
- All beams were 22 inches by 24 inches deep. All beams were assumed cracked in the strong axis. (Icr = 0.35*Ig)
- All beams used Insertion Point of "8-Top-Center"
- Floor slabs were modeled as rigid diaphragms, such that all points would displace together. Depth of slab was ignored. Slab was assumed to have no out of plane resistance to lateral forces.
- All base joints were modeled as "Fixed" – All six degrees of freedom restrained.
- Beam-to-Column connections were modeled with Panel Zones with a Rigid End Offset of 0.5 for both beams and columns.

Considering the above, the resulting model produced periods of 1.83 seconds in the north-south direction, 1.64 seconds in the east-west direction, and 1.36 seconds as the first torsional mode. These values were used in the previous section to calculate the seismic response coefficient, C_s.

Now that the baseline model was created, the previously calculated wind and seismic loads could be applied and the member design phase could begin. Wind forces were applied at the center of pressure of the building, and seismic forces were applied at the diaphragms' center of mass at each floor including allowance for accidental torsion. As mentioned earlier, the Central Tower exhibits Horizontal Torsional Irregularity, Type 5 as specified in Table 12.3-1 of ASCE 7-10. To handle this concern, protocol discussed in Section 12.5.3 was followed. In compliance with this section, members of the structure must be designed for 100 percent of the seismic forces in one direction plus 30 percent of the forces for the perpendicular direction. The loading scenarios would be adjusted accordingly.

Thirteen different load cases were modeled in ETABS in order to determine the critical loads and deflections for wind and seismic forces. A summary of these cases can be seen in the table 12 below.

		Load Cases - Wind	
ASCE 7 - 10 Load Combination 4	1	1.0WX	1
	2	1.0WY	
	3	.75WX +.75WMX	2
	4	.75WX - .75WMX	
	5	.75WY + .75WMY	
	6	.75WY - .75WMY	
	7	.75WX + .75WY	3
	8	.563WX + .563WY + .563WMX + .563WMY	4
	9	.563WX + .563WY + .563WMX - .563WMY	
	10	.563WX + .563WY - .563WMX + .563WMY	
	11	.563WX + .563WY - .563WMX - .563WMY	
Comb. 5		Load Cases - Seismic	
	12	1.0EX + Accidental Eccentricity	
	13	1.0EY + Accidental Eccentricity	

Table 12 Wind and Seismic load cases considered for analysis of Central Tower

After running all cases in the computer program, it was determined that seismic loads corresponding to load cases 12 and 13 in Table 11 above controlled the redesign of the lateral system for the Central Tower.

Building Drift & Story Drift

Story drifts and overall building drifts were calculated for the Central Tower of the Upstate Cancer Center under the controlling wind and seismic design cases. From the previous section it was discovered that the controlling seismic load cases corresponded to equations 12 and 13 from Table 11. In addition, the controlling wind load cases were determined to correspond to equations 1 and 2 from Table 11. Seismic drift limitations are a code requirement and are needed to achieve strength, whereas wind drift limitations are a serviceability issue and mentioned in the ASCE 7-10 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 2.5 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 the Central Tower 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 adjacent buildings is 1.52”. I would recommend a building expansion joint of 2 inches to compensate for deflections and joint tolerances.

Seismic Drift: East - West Direction (Load Case 12)									
Level	Story Height (ft)	Story Drift Ratio (in/in)	(Cd/I) (2.5/1.5)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Allowable Total Drift (in)	
Roof	14	0.0016	1.7	0.449	1.68	OK	3.944	8.64	OK
5	14	0.0026	1.7	0.729	1.68	OK	3.495	6.96	OK
4	14	0.0035	1.7	0.982	1.68	OK	2.766	5.28	OK
3	14	0.0035	1.7	0.982	1.68	OK	1.784	3.60	OK
2	16	0.0025	1.7	0.802	1.92	OK	0.802	1.92	OK

Table 13 Drift values for the Central Tower considering seismic controlling load case 12

Seismic Drift: North - South Direction (Load Case 13)									
Level	Story Height (ft)	Story Drift Ratio (in/in)	(Cd/I) (2.5/1.5)	Story Drift (in)	Allowable Story Drift (in)		Total Drift	Allowable Total Drift (in)	
Roof	14	0.0028	1.7	0.786	1.68	OK	6.200	8.64	OK
5	14	0.0043	1.7	1.206	1.68	OK	5.415	6.96	OK
4	14	0.0054	1.7	1.515	1.68	OK	4.208	5.28	OK
3	14	0.0056	1.7	1.571	1.68	OK	2.693	3.60	OK
2	16	0.0035	1.7	1.122	1.92	OK	1.122	1.92	OK

Table 14 Drift values for the Central Tower considering seismic controlling load case 13

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	0.0002	0.040	0.42	OK	0.580	2.16	OK
5	14	0.0005	0.080	0.42	OK	0.540	1.74	OK
4	14	0.0009	0.149	0.42	OK	0.460	1.32	OK
3	14	0.0010	0.165	0.42	OK	0.311	0.90	OK
2	16	0.0008	0.146	0.48	OK	0.146	0.48	OK

Table 15 Drift values for the Central Tower considering wind controlling load case 1

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	0.0007	0.113	0.42	OK	1.285	2.16	OK
5	14	0.0012	0.207	0.42	OK	1.172	1.74	OK
4	14	0.0019	0.317	0.42	OK	0.965	1.32	OK
3	14	0.0022	0.362	0.42	OK	0.648	0.90	OK
2	16	0.0015	0.286	0.48	OK	0.286	0.48	OK

Table 16 Drift values for the Central Tower considering wind controlling load case 2

Member Design

Throughout the last few sections of this report it was established that seismic loading was the governing lateral force acting upon the Central Tower; however, both seismic and wind forces needed to be considered along with the previously determined gravity loading in order to ultimately determine the controlling load combination specified by Section 2.3.2 of ASCE 7-10. Calculations pertaining to this section can be found in Appendix D.

After finding the controlling load combination for each member, beams and columns were designed for the associated forces. Chapter 21 of ACI 318-08 specifies specific requirements that must be met when designing and detailing concrete moment frames for earthquake resistant structures. As stated previously, the lateral system of the Upstate Cancer Center would utilize intermediate moment frames because of it being located in Seismic Design Category C.

Design for this particular type of moment frame is outlined in Sections 21.2 and 21.3 of ACI 318-08. The most notable design requirements of theses sections is that two longitudinal reinforcing bars must continuous along both the top and bottom of all beams, and transverse reinforcement shall be provided in the form of hoops over a specified distance from the face of the support.

Using a previously devised spreadsheet for the design of the Two-Way Slab from earlier, the appropriate gravity loads were applied to the slab in order to determine the portion of the factored moment transferred to the beams. These values were then combined with the proper moments taken from the ETABS lateral system analysis. Moments due to lateral forces were justified by a portal method analysis conducted for select frames. Having determined the factored loads for the beam elements, longitudinal reinforcing steel would now be selected.

Typical effective depth was maintained as the distance from the top or bottom of the beam to the centroid of the reinforcing steel; 21 ½ inches. Reinforcement was almost exclusively kept in a single layer with a few exceptions. Minimum reinforcing steel required by ACI 318-08 was governed by the quantity (200*b*d/fy), and the minimum spacing of the reinforcement was specified as the diameter of the reinforcing bar but not more than 1 inch. Typical longitudinal bars varied based on floor level but were largely limited to #9 ASTM A615 60 ksi deformed bars. In order to ensure the ductile behavior of the beams, the reinforcement ratio was limited to 2.5 percent.

Beam shear reinforcement was specified as #5 closed hoops spaced at 3.5 inches for the first and last 48 inches of the beam in accordance with ACI 318-08. Stirrups of the same steel size and spacing were used in place of closed hoops where applicable. A typical beam reinforcement detail can be seen in Figure 25 below. Sample calculations can be found in Appendix D.

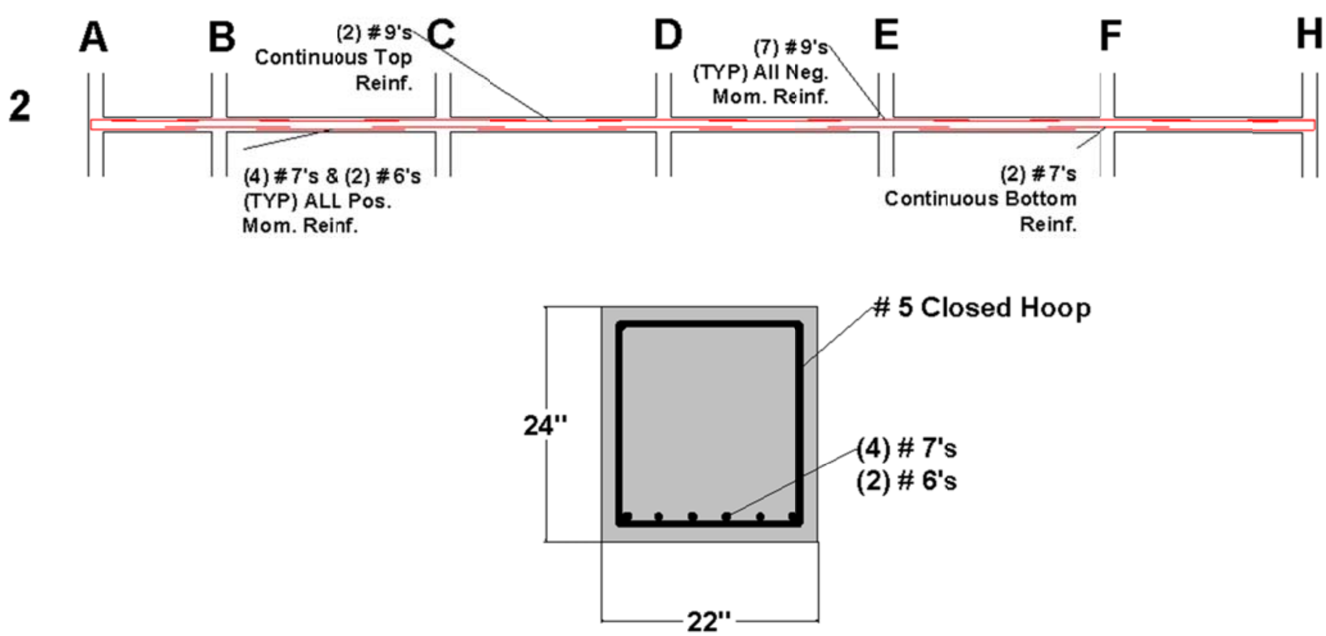


Figure 25 Typical beam reinforcement layout and cross section for gravity and lateral loads – 4th Floor – Column Line 2

Columns were redesigned considering combined axial and bending effects from gravity and lateral loads respectively at both the fourth floor and the ground level. Using the appropriate load combinations, the factored axial load and bending moments were determined for each column on the given floor level. Interaction diagrams were developed for proposed column sizes and reinforcing schemes using SpColumn computer software, and subsequently checked with a hand derived interaction diagram. Moment magnification issues were accounted for when checking column adequacy. Calculations and interaction diagrams pertaining to column design can be found in Appendix D.

Eventually it was determined that two column layouts would be used for the Central Tower redesign, and be spliced between the third and fourth floors. Bottom columns, associated with floors two to three, were specified as 24 inch by 24 inch with (16) #11 ASTM A615 60 ksi longitudinal bars distributed evenly in the four faces. Top columns, associated with floors four to the roof, were specified as 24 inch by 24 inch with (16) # 10 ASTM A 615 60 ksi longitudinal bars distributed evenly in the four faces. Transverse steel reinforcement was specified as #4 closed hoops spaced at 6 inches along the entire height of the column. Column cross-sections for the respective layouts can be seen in the figure below.

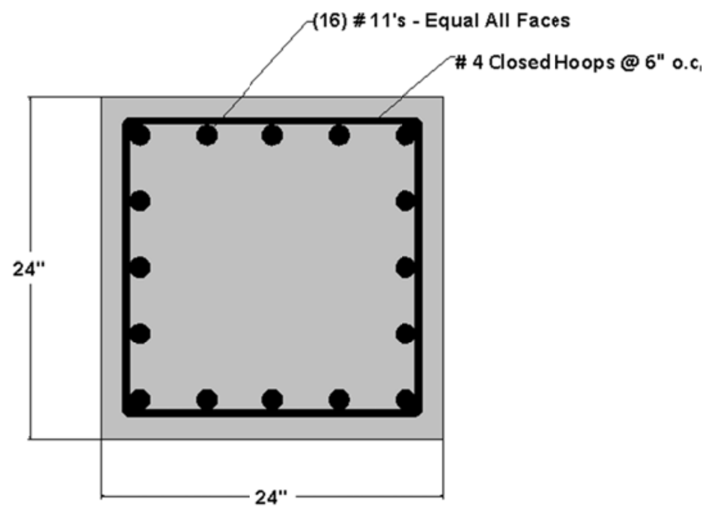


Figure 26 Typical cross-section for “Bottom Column”

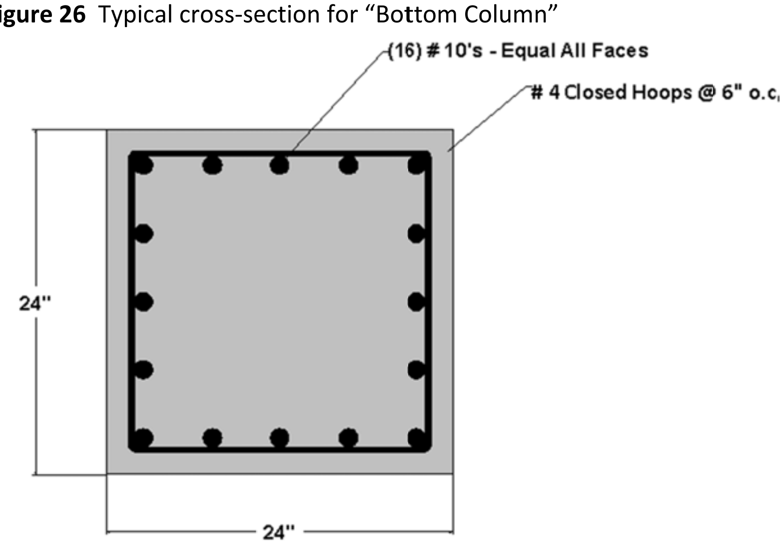


Figure 27 Typical cross-section for “Top Column”

Foundation Assessment & Alteration

Having completed the design of the alternative reinforced concrete structural system, the existing deep foundation consisting of drilled caissons would be evaluated and altered if necessary. From the geotechnical report provided by Atlantic Testing Laboratories, the allowable end bearing pressure and allowable skin friction for the dolostone soil beneath the site were given as 40 ksf and 10 ksf respectively. After reviewing the existing structural plans it was discovered that the average caisson diameter originally used was 48 inches. Using this dimension and the above pressures, it was figured that each pier or caisson was capable of carrying 628 kips of load. Dividing the building total weight by the load capacity of each caisson resulted in the need for 32 caissons to support the redesigned portion of the Upstate Cancer Center. In order to unitize the and simplified the layout location of the drill piers, 36 caissons measuring 48 inches in diameter would be placed along the major column lines of the redesigned Central Tower. Figure 28 illustrates the proposed layout.

A building subjected to lateral loading can have significant foundation issues due to overturning moment. Referencing Table 9, the greatest overturning moment is attributed to seismic loading and results in 42,338 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 foundation design. The resisting moment was found to be 1,185,540 foot-kips. The two-thirds fractions results in 790,360 foot-kips, which is far larger than the overturning moment of 42,338 foot-kips. Therefore, the foundation is perfectly suitable for the designed lateral loads. Complete calculations for the assessment and redesign of the foundation can be found in Appendix F.

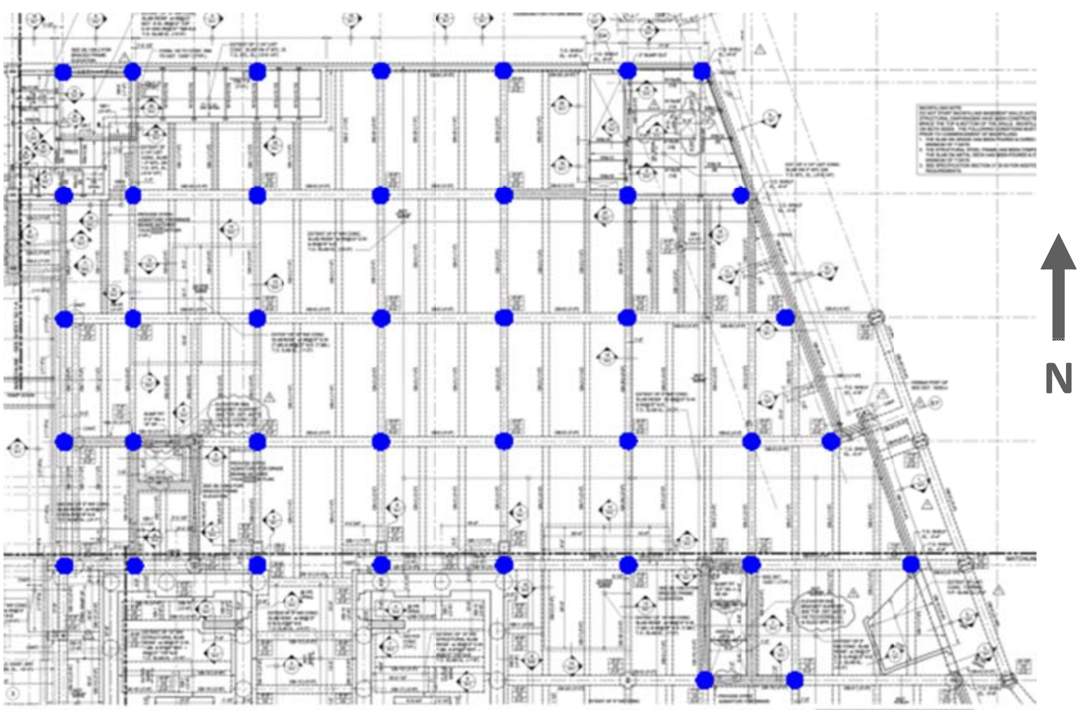


Figure 28 Proposed layout for caissons **(Blue)**, redesigned foundation. (Original image courtesy of EwingCole)

Progressive Collapse Analysis & Design

ASCE 7-10 defines progressive collapse as the spread of initial local failure from element to element, resulting eventually in the collapse of an entire structure or a disproportionately large part of it. Since the partial collapse of the Alfred P. Murrah building in Oklahoma City in 1995 due to the loss of an exterior support, design to resist progressive collapse has emerged as a serious consideration when designing or retrofitting a structure. Although progressive collapse, or disproportionate collapse, design requirements are most commonly seen in high security and governmental facilities, a scenario has been created in which the SUNY Upstate Cancer Center will be designed to meet such requirements.

There are multiple design guides and procedures available for use in the market; however, the analysis and design presented in this report will conform to the United Facilities Criteria 4-023-03 (UFC), backed by the United State's Department of Defense. The UFC discusses two design approaches: Direct Design and Indirect Design. Direct Design processes include: the Alternative Path Method, which requires the structure to be able to bridge over a missing structural element; and the Specific/Enhanced Local Resistance Method, which requires the building or parts of the building provide sufficient strength to resist a specific load or threat. Indirect Design processes consist of the Tie Force Method, in which minimum tensile forces must be met to mechanically tie the structure together and enhance its ductility, continuity, and redundancy.

Progress collapse design requirements vary by occupancy category. SUNY Upstate Cancer Center, a medical facility, is classified as Occupancy Category IV per ASCE 7-10, and therefore has the most stringent design criteria as stated by the UFC. To meet the code provisions, the Tie Force Method, Alternative Path Method, and Enhanced Local Resistance Method must be conducted. Design procedures will be conducted in order beginning with the Tie Force Method.

Tie Force Method

As stated previously, the Tie Force Method is used to mechanically tie the structure together allowing for the redistribution of forces due to the loss of a critical member. Section 3-1 of the UFC outlines the design procedure for the Tie Force Method. According to the code, the tie forces are to be carried within the reinforced concrete slab, already designed. Baseline design reinforcement within the slab can be used to fulfill the tie force requirements from this section, provided it is not located above a flexural element.

There are three types of tie forces that need to be designed for: internal, peripheral, and vertical. Although each tie force is calculated differently, the principle equation remains constant; the designed tie strength must be greater or equal to the tie force.

$$\boldsymbol{\phi R_n \geq F_T}$$

Where:

ϕR_n = Design Tie Strength

$= \phi\ \Omega\ A_s\ F_y$

F_T = Specified Tie Force

The design tie force is a function of the area of steel, A_s, its yield strength, F_y = 60 ksi, the appropriate strength reduction factor for the material, ϕ=.75 for tension, and the material over-strength factor, Ω, specified in ASCE 41-06. According to ASCE 41-06 Table 6-4, the appropriate over-strength factor associated with reinforcing steel is 1.25. A specific load combination is also considered for the calculation of tie forces:

$$\mathbf{W_F = 1.2D + 0.5L}$$

Required tie force is calculated differently for each type of tie. Internal tie force for both the longitudinal and transverse directions are determined with the following equation.

$\mathbf{F_i = 3*W_f *L_i}$ Where:

$\mathbf{L_i}$ = Greater distance between centers of columns supporting any two adjacent floor spaces in the considered direction

Peripheral tie forces account for the weight of building cladding, and ties must be located within three feet of the perimeter.

$\mathbf{F_p = 6*W_F * L_1*L_P}$ Where:

$\mathbf{L_1}$ = Greater distance between centers of columns at the perimeter of building in direction of loading

= Length of opening in direction under consideration

$\mathbf{L_P}$ = 3 feet

Vertical tie forces are to be resisted with the addition of vertical ties within the columns of the building. More often than not, these requirements are already met by the steel reinforcement used in the baseline design.

$\mathbf{F_V = A_T*W_F}$ Where:

$\mathbf{A_T}$ = Tributary area of specified column

After completing the tie force analysis for floors and columns of the Central Tower of the Upstate Cancer Center, it was discovered that the reinforcement required by the Tie Force Method was greater than the reinforcement needed for the two-way slab design. Therefore, the original slab reinforcement would be replaced with #6 ASTM A615 60 ksi deformed bars at 9 inches on center for internal ties, both directions, typical floor. Calculations pertaining to the Tie Force Method can be found in Appendix E. Figure 29 shows the tie layout for the fourth floor.

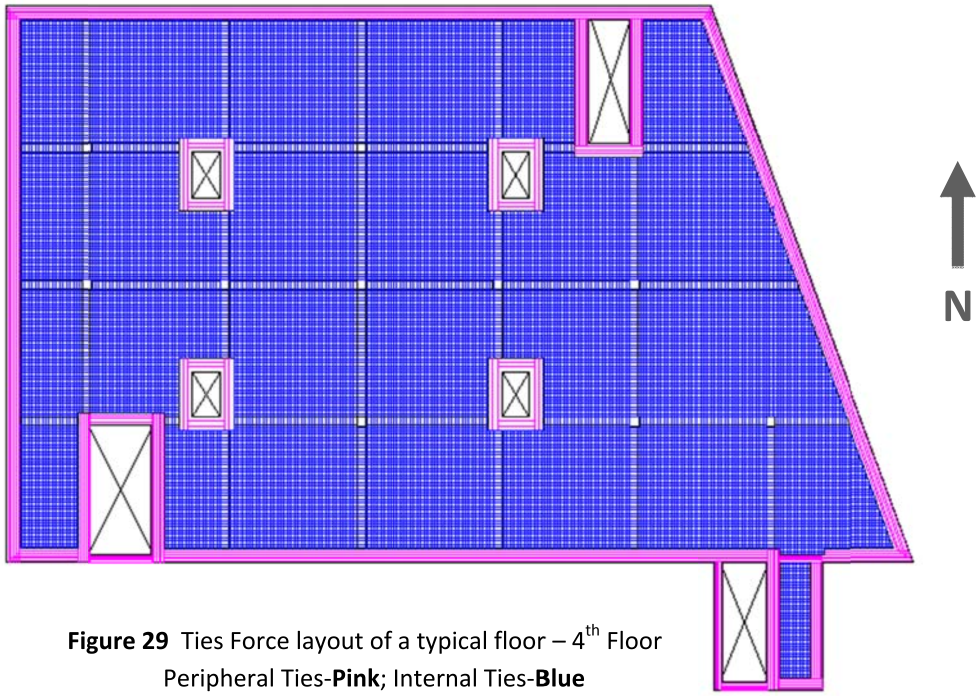


Figure 29 Ties Force layout of a typical floor – 4th Floor
Peripheral Ties-**Pink**; Internal Ties-**Blue**

Alternative Path Method

The Alternative Path Method essentially requires that flexural members can bridge over a missing support element; in the case of this report, a column. This process must be carried out at a minimum for three specific locations on the building perimeter: removal of a column at the middle of the long side of the building; removal of a column at the middle of the short side of the building; removal of a column at the corner of the building. In addition, the UFC mentions that special attention should be paid to columns at reentrant corners of the building as well as columns that are closely spaced.

Alternative Path Analysis must be carried out for the column locations described above at: the first story above grade (Story 1); the story directly below the roof (Story 5); the story at mid-height of the building (Story 3); and the story above a column splice location (Story 4). Column removal requirements are stated in the code as the clear height between lateral supports. It is crucial that beam-to-column joint is not removed when the column is removed. Figure 30 illustrates this point.

Figure 30 Images depicting the proper protocol required when removing a column for Alternative Path Analysis. (From UFC 4-023-03)

Within the Alternative Path Method, there are three analysis options: Linear Static, Nonlinear Static, and Nonlinear Dynamic. Nonlinear static analysis was chosen for the Upstate Cancer Center, mainly because it provides a better picture of member behavior without requiring a great deal of time to analyze. A particular load case is required per the UFC for the nonlinear static procedure. Bays that are directly adjacent to and above a removed column should be assigned the following load:

$$\mathbf{G_N = \Omega_N\ [(0.9\ or\ 1.2)*D + (0.5*L\ or\ 0.2*S)]}$$

Where:

G_N = Increased Gravity Loads = 350.8 psf [Central Tower]

Ω_N = Dynamic Increase Factor = 1.31 [Central Tower]
= $1.04 + 0.45/((\theta_{PRA}/\theta_Y) + 0.48)$ [Concrete]

All other bays of the building are to be assigned the following load:

$$\mathbf{G = (0.9\ or\ 1.2)*D + (0.5*L\ or\ 0.2*S)}$$

Where:

G = Gravity Loads = 267.8 psf [Central Tower-Typical Floor]

Lateral loads are applied to each face of the building, one at a time, thus creating four loading scenarios for each column removal. Lateral forces applied were determined with the following equation:

$$\mathbf{L_{LAT} = .002*\Sigma P}$$

Where:

L_{LAT} = Notional lateral load applied at each floor, each face of the building = 12.22 kips [Central Tower-Typical Floor]

ΣP = Sum of gravity loads (D+L) only acting at that specific floor, no dynamic increase factors. = 6109 kips [Central Tower-Typical Floor]

A three dimensional model of the Central Tower of the Upstate Cancer Center was created using SAP 2000 Nonlinear. Both primary and secondary structural elements were included in the analysis to best account for the true behavior of the building after losing a primary structural support. Frame elements were represented with centerline modeling, i.e. no rigid end offsets. All joints were considered as moment connections, but column to foundation connections were modeled as pinned, i.e. no rotational restraint. Floors were modeled as rigid diaphragms, all beams were assumed to have rectangular behavior, and material properties were the same as used in the earlier ETABS model.

Plastic hinges are allowed to form in both the beams and columns of nonlinear static model as long as they do not exceed the acceptance criteria provided in Chapter 4 of the UFC and Chapter 6 of ASCE 41-06. Hinges were assigned in the SAP model at the ends and mid-spans of all members. Auto-hinge properties were used and modified to reflect the proper behavior and acceptance criteria stated in the codes.

A "staged construction" load case was used in the SAP analysis to imitate the loading of the structure and the removal of a desired column. Loads were applied to the structure in steps for each stage of the analysis. This method allowed the model to converge at each step before continuing, thus helping to better identify the location and magnitude of failure. Failure of a member was based on the plastic hinge performance levels of Immediate Occupancy, Life Safety, and Collapse Prevention. If a hinge forms beyond the points just listed, the member has collapsed, thus failing.

The model was run for each of the required removal scenarios. Members that had failed were redesigned, either by increasing the amount of steel reinforcement, increasing the member cross section, or both. Once the model converged completely with plastic hinges within the accepted criteria, the structure was deemed adequate in accordance with the Unified Facilities Criteria. The figures below help illustrate the Alternative Path Analysis and Design of Column 1D at ground level. Calculations pertaining to the Alternative Path Method can be found in Appendix E.

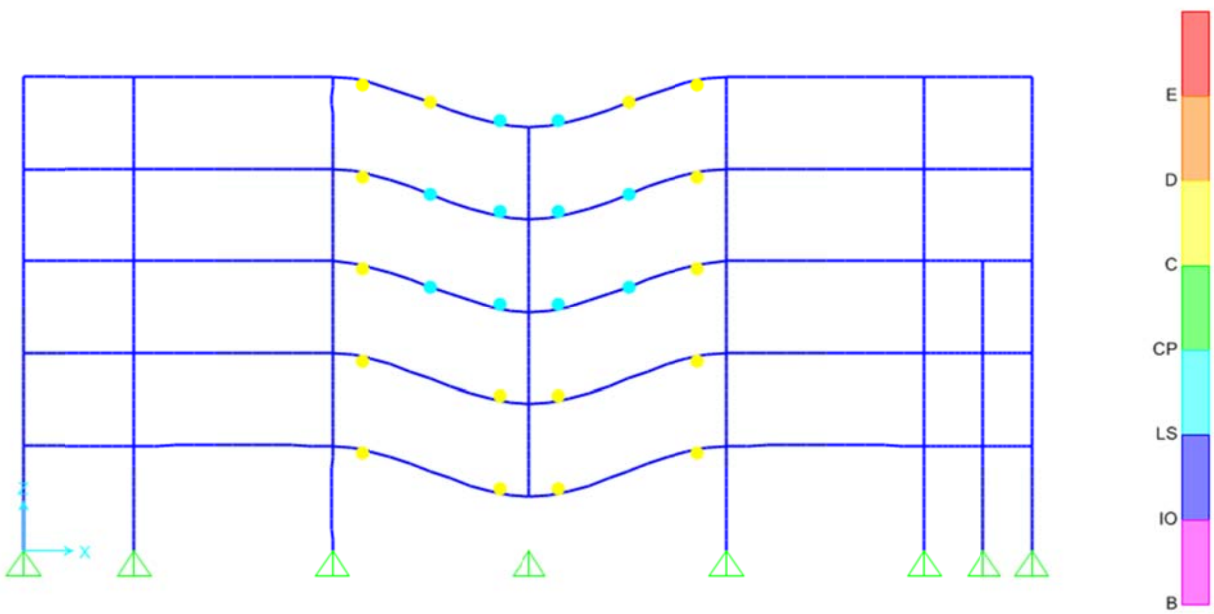


Figure 31 Alternative Path Analysis in SAP 2000 NL for removal of column D1. Structure has failed due to hinge failure. (C-Collapse **(Yellow)**)

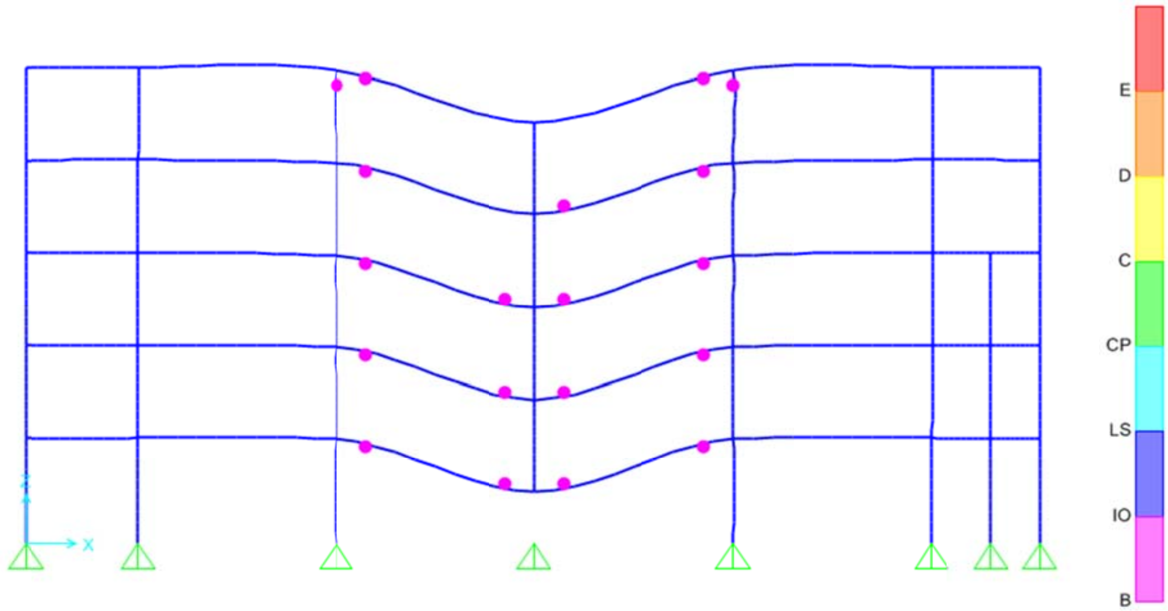


Figure 32 Alternative Path Analysis in SAP 2000 NL for removal of column D1 with redesigned members. Structure meets accepted criteria.

In summary of the Alternative Path Method, all exterior perimeter beams in the north-south and east-west directions were modified as 22 inch wide by 28 inch deep with (4) #8 and (5) #9 ASTM A615 60 ksi deformed bars in both the top and bottom of the member. Reinforcement in beams running north-south and framing into the exterior perimeter beams was modified to (4) 8 and (5) #9 ASTM A615 60 ksi deformed bars in both the top and bottom of the member. All beams running east-west and framing into the exterior perimeter beams remained unaltered. All columns in the building remain unaltered from the previous design.

Enhanced Local Resistance

According to the UFC, Enhanced Local Resistance criterion applies to all perimeter columns of a building in Occupancy Category IV for the first two stories above grade, corresponding to the "Bottom Columns" mentioned earlier. Enhanced Local Resistance criterion states that the specified columns must meet the enhanced flexural resistance (EFR). EFR is calculated as the larger of 2.0 times the baseline flexural resistance (initial design) or the existing flexural resistance (from Alternative Path Method). Since no columns were redesigned during the Alternative Path Analysis, the EFR was calculated as 2.0 times the baseline flexural resistance.

Baseline flexural resistance is taken as the nominal capacity, no ϕ factors, of a given column under the appropriate axial load; 1200 foot-kips for "Bottom Columns". Upsizing the given column to 30 inches by 30 inches with (20) #14 ASTM A615 60 ksi deformed bars placed evenly in all faces produces a nominal moment capacity of approximately 2500 foot-kips, which is larger than 2.0 times 1200 foot-kips; 2400 foot-kips. Therefore, all perimeter columns of the Central Tower on the first two floors above grade will be modified to 30 inches by 30 inches with (20) #14 ASTM A615 60 ksi deformed bars placed evenly in all faces. Calculations and interaction diagrams relating to Enhanced Local Resistance can be found in Appendix E.

Structural Redesign Summary

After multiple design iterations accounting for effects from gravity, lateral, and progressive collapse analyses, a final solution for the structural redesign of the Central Tower of the SUNY Upstate Cancer Center was reached. Typical slabs were ultimately established as two-way systems, 9 inches thick, with #6 ASTM A615 60 ksi deformed bars for flexural reinforcement spaced at 9 inches on center, as governed by the Tie Force Analysis within the progressive collapse design.

With limited exceptions, all interior beams were sized as 22 inches wide by 24 inches deep. Top and bottom flexural reinforcing steel varied depending on the beam's location. Tables summarizing beam longitudinal reinforcement can be found below. All exterior perimeter beams were sized as 22 inches wide by 28 inches deep and reinforced with (4) #8 and (5) #9 ASTM A615 60 ksi deformed bars , top and bottom for flexural reinforcement as governed by the Alternative Path Analysis within the progressive collapse design. Shear reinforcement was unitized for all beams, utilizing #5 closed hoops at 3.5 inches on center for the first and last 48 inches of the beam. Stirrups of the same size were used for the remainder of the beam shear reinforcement.

Floors 2 & 3			
Location	Size	Reinforcement	
		Top	Bottom
N-S			
Spandrels	22" x 28"	8.16	8.16
Interior Beams	22" x 24"	8.16	3.95
Single Spans	22" x 24"	5.95	3.00
Column Line X	22" x 24"	5.56	3.28
Interior P-C Beams	22" x 24"	8.16	8.16
E-W			
Spandrels	22" x 28"	8.16	8.16
Interior Beams	22" x 24"	7.58	3.60

Floors 4, 5, & Roof			
Location	Size	Reinforcement	
		Top	Bottom
N-S			
Spandrels	22" x 28"	8.16	8.16
Interior Beams	22" x 24"	8.16	3.95
Single Spans	22" x 24"	5.95	3.00
Interior P-C Beams	22" x 24"	8.16	8.16
E-W			
Spandrels	22" x 28"	8.16	8.16
Interior Beams	22" x 24"	7.00	3.28
Special Case (1)	22" x 28"	12.7	5.95

Table 17/18 Concrete beam reinforcement schedules for all floors – Final Design

Columns were broken down into two categories for design; top columns and bottom columns. Bottom columns extended from the building's base to the fourth floor where they were spliced with top columns to the roof level. Both top and bottom column dimensions were the same at 24 inches by 24 inches; however, top columns were reinforced with (16) #10 ASTM A615 60 ksi deformed bars and bottom columns were reinforced with (16) #11 ASTM A615 60 ksi deformed bars. All perimeter columns at the first two stories above grade were upsized to 30 inches by 30 inches with (20) #14 ASTM A615 60 ksi deformed bars to meet requirements established in the Enhanced Local Resistance Analysis within the progressive collapse design. Transverse column reinforcement was unitized utilizing #4 closed hoops at 6 inches on center along the full height of the column in accordance with ACI 318-08 requirements.

Finally the cost of the redesigned concrete superstructure would be compared to the original steel superstructure. A simplified cost analysis was based off a typical bay for both the original and redesigned superstructures of the Central Tower. Estimates for the concrete redesign were increased by 5 percent to account for progressive collapse design not addressed in a typical bay. Cost data was gathered from RS Means Costworks, a digital version of the tradition cost books. Factoring in location and current cost values, it was discovered that the redesigned concrete structural system was more expensive overall than the original steel superstructure, even without accounting for progressive collapse design. Accounting only for the cost of the structural systems, the existing steel structure was estimated at \$3,033,685 and the redesigned reinforced concrete system will allowances added for progressive collapse design was estimated at \$3,449,330; a difference of \$415,644. Initial cost comparison from Technical Report 2 was based off of assembly data only for floor construction. Because this cost estimate featured more detailed information and included all framing in addition to the floor assembly, it within reason that it does not reflect the original cost assumption. Cost data can be found in Appendix F.

Risk Mitigation & Site Redesign (Breadth 1)

Two design approaches are presented in the Unified Facilities Criteria in response to preventing progressive collapse: Direct Design and Indirect Design. Resisting disproportionate collapse solely through structural modification is not highly desired. Identifying and mitigating potential risks, such as explosions and vehicular impacts, by modifying exterior and landscape architecture is more effective in preventing disproportionate collapse than attempting to arrest the spread of initial structural failure Therefore, the UFC recommends a "belt and suspenders" approach integrating both Direct and Indirect Design approaches. Structural modifications relating to the Direct Design approach have already been carried out in the Structural Depth portion of this thesis. This breadth topic will focus on the Indirect Design approach, specifically site security and improvements.

Design practices and ideologies presented in this section are based on information provided by the United States General Services Administration's Site Security Design Guide. According to this guide, applicable threats and risks must first be identified and prioritized, and then site improvements will be implemented to eliminate or reduce these risks. For the SUNY Upstate Cancer Center, it was determined that the most prevalent risks associated with progressive collapse of the structure would result from a vehicular impact or explosion. In order to design a well protected site, improvements would be made to neighborhood region of the building, site standoff perimeter, site access and parking, building envelope, and building management. Figure 33 shows the existing site plan with area highlighted for improvement.

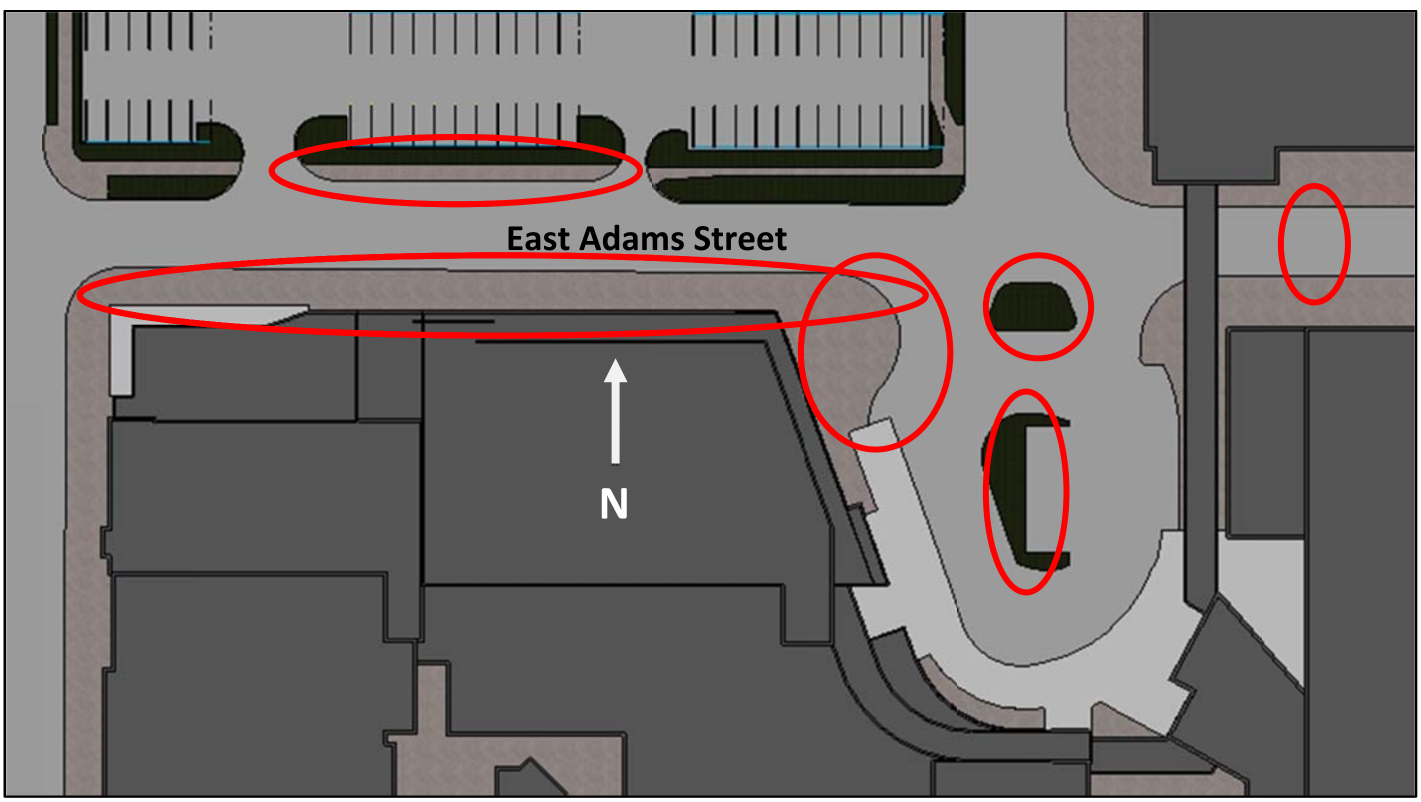


Figure 33 Existing site plan of the SUNY Upstate Cancer Center produced in Revit Architecture 2012

After reading through the GSA's suggestions for site improvements, an action plan was created to incorporate as many design features a possible to minimize the prescribed threats. Addressing neighborhood issues, East Adams Street was narrowed eliminating all on-street parking; increasing the standoff distance between the Central Tower and the roadway and decreasing the opportunity for a vehicle, possibly containing an explosive, to stop alongside the structure. Raised pedestrian cross-walks were also implemented in two locations along East Adams Street limiting the speed of vehicles passing traveling by the Cancer Center. This design practice has proved effective at various locations along Pollack and Curtin Roads on the Pennsylvania State University's campus.

To address site perimeter issues, specifically along East Adams Street, a combination of structural bollards, decorative planters, trees, and benches line the roadway. These obstructions protect the exterior of the building and perimeter structural elements from a direct impact of a traveling vehicle. In addition, these obstacles reside on brick pavers which are placed over a collapsible fill. Although unaffected by human foot traffic, this pavement will give way under the weight of a typical automobile, once again protecting the building from a direct impact.

Currently there is no restriction of vehicles allowed to enter or park within the building's site. To limit access to the site, the traffic circle was reduced in size and the existing on-site parking was removed. In addition, a guard house was placed at the entrance to the traffic circle to allow vehicle access to the site only to patients and Cancer Center personnel. Reducing the size of the existing traffic circle allowed for the addition of a fairly large plaza in front of the main north-east facing façade of the Upstate Cancer Center. Adorned with a decorative paving pattern, a fountain, landscaping rocks, benches, planters, and trees, this area was designed to provide site protection and security, through an obstructed stand-off perimeter, while maintaining a look that was aesthetically pleasing to patients and visitors. Site security would be maintained into the building with security check-point located at the building's entrance. The following figures illustrate the improvements to the Upstate Cancer Center

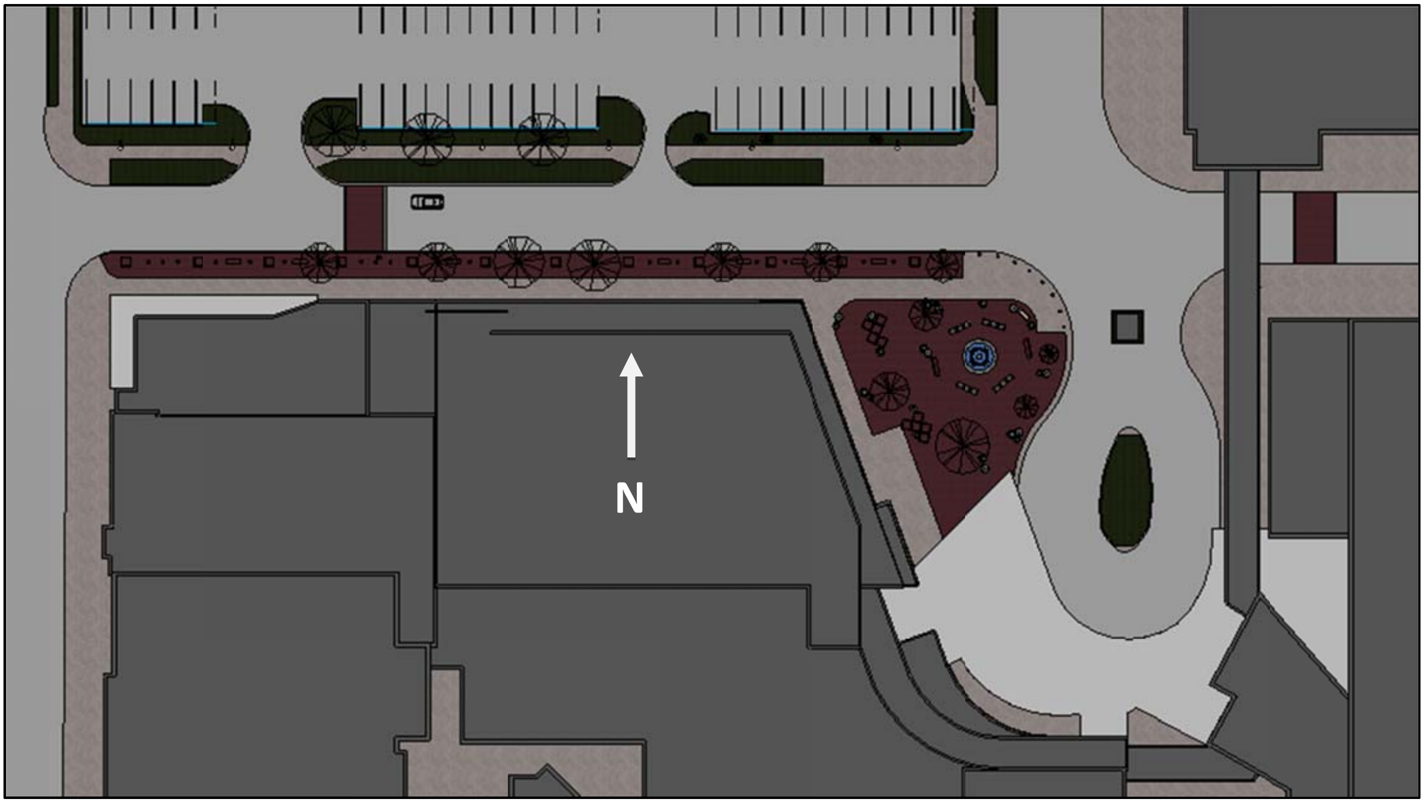


Figure 34 Redesigned Site Plan for the SUNY Upstate Cancer Center

Figure 35 Street level view of original site plan of the Upstate Cancer Center

Figure 36 Street level view of redesigned site plan of the Upstate Cancer Center

Figure 37 Rendered images showing site improvements along East Adams Street.

Figure 38 Rendered image showing the newly designed plaza area

Ultimately, the site modifications to the SUNY Upstate Cancer Center were designed to have the least impedance on the daily activities and operations of the building.

Building Envelope Analysis & Redesign (Breadth 2)

Presently, the main entrance façade of the SUNY Upstate Cancer Center, north-east, is faced with a full-height, glazed curtain wall system. Vast expanses of glass in a building façade usually lead to issues of building heating and cooling. Glass facades also exhibit vulnerability due to pressure distributions resulting from wind forces and explosions, impacts from flying projectiles, and structural movement associated with seismic forces. In addition to providing a better thermal performing glazing system, this section will address the design of the curtain wall to resist the abovementioned effects.

First, the glass panels of the system will be designed to resist the prescribed loads. The existing curtain wall system uses insulating glass units (IGU) with an overall thickness of 1 inch, made up of a 0.25 inch thick heat-strengthened outer light and a 0.25 inch thick annealed inner light separated by a 0.5 inch dry airspace. A 6063 T5 Aluminum mullion system with outer dimensions of 7.125 inches by 3.375 inches supports the IGU panels.

Being that IGU panels provide better thermal resistance than monolithic or laminated glass alone, the alternative curtain wall glazing will be designed as a an insulating glass unit. The unit assembly will consist of a laminated outer light, with both plies consisting of annealed glass, and a monolithic inner light, consisting of fully tempered glass, separated by a 0.5 inch dry air space. Having selected an assembly layout, the individual lights of glass would be sized for impact, wind, and blast loading.

Using the “Sacrificial Ply” ideology discussed in “Building Enclosures Science and Design” class, the outer, laminated, glass light was designed to resist the impact of a 2 gram steel ball traveling at a speed of 130 feet/sec, with the probability of breakage of the inner light equal to 8 in 1000. Using the design chart for sacrificial ply design, each ply of the laminated light was sized at 0.15625 inches, a total thickness of 0.3125 inches.

Next the glass panels would be sized using ASTM E1300 for wind loads assuming the maximum pressure determined from the previous wind calculations, 40.97 psf. Ignoring load sharing between the lights of the IGU, the Glass Type Factors (GTF) for the annealed and fully tempered lights were 1.0 and 4.0 respectively. For design purposes, IGU panels were dimensioned as 59 inches wide by 168 inches tall, specified by mullion spacing. Calculations resulted in both plies of the laminated outer light to be 0.25 inches, 0.5 inches total, and the fully tempered inner light to be 0.25 inches thick.

From the previous section, the redesigned site of the Cancer Center and addition of the plaza in front of the north-east façade created a standoff distance of 50 feet to the glazed curtain wall. Glass panels were designed based on the assumption of an explosion equivalent to 70 pounds of T-N-T; a small, concealed car-bomb. Using ASTM F2248 and the parameters described above, an equivalent 3 second pressure was found for the associated blast load. This equivalent 3 second pressure was used in combination with ASTM E1300 in order to size the glazing panels for blast loading. It was determined that each ply of the annealed laminated outer light needed to be 0.375 inches thick, 0.75 inches total, and the fully tempered inner light needed to be 0.5 inches thick in order to resist the prescribe blast loads. This loading scenario governed the design of the IGU panels.

Finally, the glazing pocket needed to be sized in order to prevent contact of the glass panels with the frame under seismic movement. Contact with the frame under such movement could cause it to accidentally fallout, injuring the building occupants. Clearance of 0.125 inches was originally provided on all sides of the glazing unit. Using the glass panel dimensions from before, drift values taken from the previous seismic analysis, and an importance factor of 1.50, it was found that the clearance from the glazing unit to the frame needed to be increased to 0.4375 inches.

Now that the glazing panels were designed for the given loading scenarios, the supporting mullions had to be redesigned to carry the required loads. According to established design principles, the mullions had to provide enough strength to carry 2.0 times the load resistance of the glazing units;230 psf. Mullions were designed as two continuous spans measuring 28 feet in length using 6063 T5 Aluminum. Following design procedures set forth by Wilson Curtain Wall Consultants, it was ultimately determined that the mullion size needed to be increased to 8 inches by 3.5 inches with a thickness of 1.2 inches. Mullion connections were detailed as having a minimum of (2) 0.5 inch diameter bolts.

A cross-section of the original and redesigned IGU's can be seen in figures 38 and 39. Calculations pertaining to the design of the glass panels and supporting mullions can be found in Appendix G.

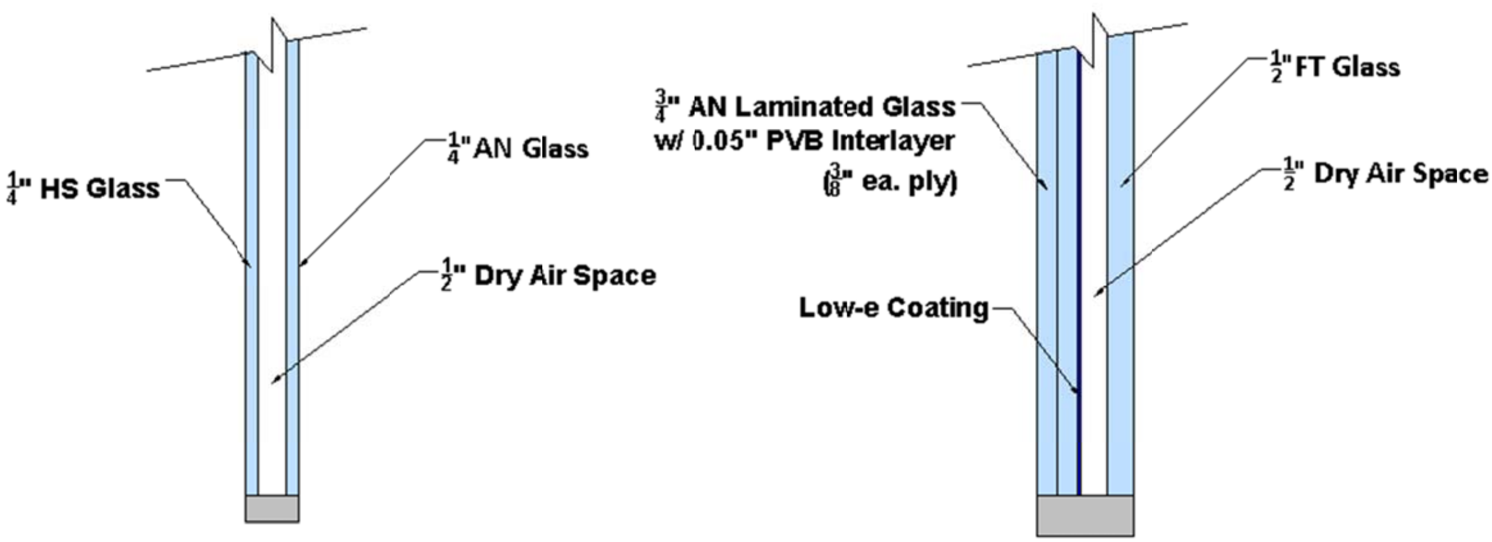


Figure 39/40 Cross-sections of the existing **(Left)** IGU and alternative **(Right)** IGU

The second portion of this breadth study will address the efficiency of the redesigned insulating glass unit against the original in terms of heat transfer through the curtain wall for both winter and summer conditions. Heat transfer was calculated using an equation in the ASHRAE Handbook of Fundamentals for energy flow through fenestration given by:

$$\mathbf{Q = U*A_{PF}*(T_{OUT} - T_{IN}) + (SHGC)*A_{PF}*E_T}$$

Where:

Q = Energy flow (Heat)
U = Thermal Resistance for the glazing unit
SHGC = Solar Heat Gain Coefficient for the glazing unit
$\mathbf{A_{PF}}$ = Glazing area
$\mathbf{T_{OUT}}$ = Outdoor temperature
$\mathbf{T_{IN}}$ = Indoor temperature
$\mathbf{E_T}$ = Total irradiance

Total irradiance was calculated for two particular days, June 21st at 1:00 p.m. (summer) and December 21st at 12:00 p.m. (winter), using the given latitude and longitude for Syracuse, New York and provisions provided in the ASHRAE Handbook of Fundamentals. An SHGC of 0.7 and U-values of 0.47 for winter and 0.50 for summer, for the current IGU's were found in the building specifications.

Reducing the existing SHGC and U-values, would lead to a reduced energy flow through the curtain wall. After researching several manufacturers, SunGuard's SuperNeutral 62 IGU was chosen as the alternative glazing, having the lowest SGHC and U-values; 0.32, 0.26(winter), 0.28(summer) respectively. A low-e coating was also used on the glazing helping to lower its solar heat gain coefficient by effectively blocking heat gain due to the introduction of ultraviolet radiation. Finally, the glazing unit was chosen because it had no effect on the existing appearance of the curtain wall.

Using the equation above, given assembly properties, and calculated irradiance values, heat transfer through the existing and redesigned curtain wall systems for summer and winter conditions was determined. For summer conditions, the existing IGU allows an average heat gain of 620,114 Btu/hr and the alternative IGU allows an average heat gain of 288,266 Btu/hr. For winter conditions, the existing IGU allows a heat gain of 72,683 Btu/hr; however, the alternative IGU actually has a heat loss of 36,505 Btu/hr. Although the SunGuard IGU provides desirable results for summer conditions, i.e. lower cooling costs, it requires a higher heating load and therefore high heating cost for winter conditions.

In summary, the proposed alternative IGU may not be the most thermally efficient curtain wall assembly. Addition analysis and a cost comparison related to heating and cooling loads, not part of this thesis, would need to be conducted to better assess the performance of the two glazing units. All calculations and cut sheets pertaining to the glazing design, mullion design, and heat transfer analysis can be found in Appendix G.

MAE Course Requirements

Knowledge attained throughout the several graduate level courses taken over the past year and a half greatly influenced the success of this thesis project. In particular material learned in AE 597: Computer Modeling of Building Structures was used to design and analyze the lateral structural model in ETABS and the alternative path analysis in SAP 2000 Nonlinear. Without sufficient background knowledge of these programs and basics of structural modeling, this project would have been nearly impossible to conduct. In addition, knowledge gathered through AE 542: Building Enclosures Science and Design, was used to conduct the entire portion of the Breadth 2: Building Envelope Analysis and Redesign. Independent research into the topics of progressive collapse was also conducted for this thesis. Although there is no association with a graduate level course, the extensive exploration into the topic and self-teachings required a more in depth thinking and knowledge base.

Conclusion

A concrete superstructure was successfully designed as an alternative structural system to the existing steel superstructure in place at the SUNY Upstate Cancer Center. Through comparison of multiple floor systems, a two-way slab with beams was chosen in hopes of decreasing the overall cost of the structural system. Gravity loads would be resisted through the reinforced concrete slab, beams, and columns, whereas lateral loads would be resisted through reinforced concrete intermediate moment frames. In addition, the Central Tower of the Upstate Cancer Center was designed to resist disproportionate collapse by meeting requirements established by the United States Department of Defense through the Unified Facilities Criteria.

All design work was conducted in accordance with local and national building codes and guidelines. After accounting for the worst case scenarios in all facets of design, a final solution was reached. The redesigned concrete structural system uses a 9 inch two-way reinforced concrete slab with inter-column beams in both orthogonal directions with a typical dimension of 22 inches wide by 24 inches deep. Spandrel, or perimeter, beams have dimensions of 22 inches wide by 28 inches in order to meet progressive collapse requirements. Columns are exclusively square in dimension with the majority of dimensions measuring 24 inches by 24 inches. Exterior perimeter columns for the first two stories above grade were upsized to 30 inches by 30 inches to meet progressive collapse requirements. Using a simplified cost analysis method, it was determined that the redesigned concrete structural system, including design for progressive collapse, cost $415,644 more than the existing steel structural system. Although it is more expensive, the concrete superstructure is still a viable alternative structural system.

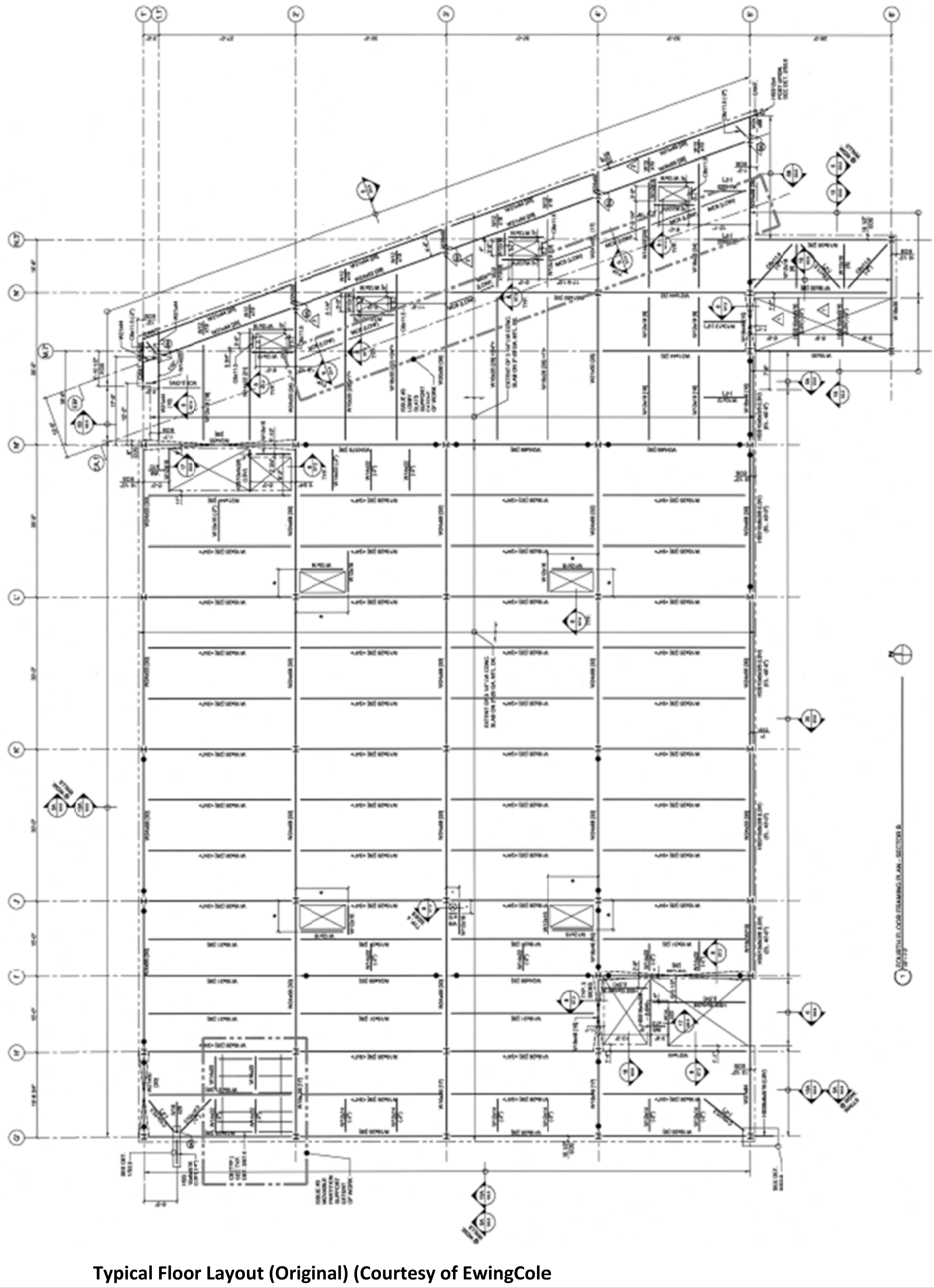
Supplementing the progressive collapse design portion of this report, an exploratory breadth was conducted with intentions of limiting or eliminating substantial risks that could initiate disproportionate collapse. Primary risks associate with the SUNY Upstate Cancer Center were structural damage due to vehicular impact and damage due to an exterior explosion. Following a guide supplied by the United States General Services Administration, site improvements were made surrounding the Central Tower to inhibit these events from occurring.

In addition to redesigning the site of the Upstate Cancer Center, the building's premiere façade was also redesigned to withstand the effects from an unwanted event. Using current design practices, the glazing units and pockets were sized to resist damage from impacts, wind pressures, blast pressures, and movement due to seismic effects. The redesign resulted in a more resilient and robust curtain wall system. Taking advantage of the redesign opportunity, glazing panels were selected with aims of reducing heat transfer through the curtain wall system. After selecting a glazing system with better thermal insulating properties, it was ultimately found that although the alternative fenestration unit provided less heat gain in the summer months, it caused heat loss during the winter months. The redesign resulted in a more resilient and efficient curtain wall system.

Appendices

Appendix A: Miscellaneous Design Criteria

Note: Appendices contain sample calculations only. To view the entire collection of data and design calculations, please contact the author.



Typical Floor Layout (Original) (Courtesy of EwingCole

		Systems			
		Existing	Alternatives		
Consideration		Composite Steel Deck on Composite Beams & Girders (2)	Precast Hollow Core Planks on Steel Girder (4)	Two-Way Flat Slab with Drop Panels (1)	One-Way Pan Joist System (3)
General Information					
	Weight	50.9 psf **(1)**	91 psf **(2)**	127.4 psf **(4)**	104.8 psf **(3)**
	Overall Depth	30" **(3)**	30" **(3)**	15.75" **(1)**	20.5" **(2)**
	Slab Depth	6.25" **(2)**	10" **(4)**	9.5" **(3)**	4.5" **(1)**
	Assembly Cost	20.04 $/sf **(3)**	25.96 $/sf **(4)**	17.44 $/sf **(1)**	18.33 $/sf **(2)**
Architectural					
	Bay Size	30'-0" x 30'-0" **(1)**	30'-0" x 20'-0" **(2)**	30'-0" x 30'-0" **(1)**	30'-0" x 30'-0" **(1)**
	Fire Rating	2 HR - UL Assembly **(3)**	2 HR - Unrestrained **(2)**	2 HR **(1)**	2 HR **(1)**
	Other	Requires Additional Fireproofing for Underside of Deck & Framing Members **(3)**	Fireproofing Needed for Exposed Framing Members **(2)** Change in Bay Size	Increase in Floor to Floor Height **(1)** Superstructure Changes to Concrete	Increase in Floor to Floor Height **(1)** Superstructure Changes to Concrete
Structural					
	Gravity System Alterations	No Change **(1)**	Increase Girder Size - Resize Columns Due to Altered Bay Sizes **(3)**	No Beams/Girders - Concrete Columns w/ Drop Panels **(2)**	Joists w/ Wide Beam Girders - Concrete Columns **(4)**
	Lateral System Alterations	No Change **(1)**	Possible Addition of Braced Frames **(2)**	Change From Braced Frames to Shear Walls **(3)**	Change From Braced Frames to Shear Walls **(4)**
	Foundation Alterations	No Change **(1)**	Alter Size and Location of Caissons & Grade Beams **(4)**	Increase Foundation Size to Carry Larger Building Weight **(3)**	Increase Foundation Size to Carry Larger Building Weight **(4)**
Construction					
	Formwork Required	Minimal **(2)**	None **(1)**	Yes **(3)**	Yes
	Constructability	Slightly Moderate **(2)**	Easy **(1)**	Moderate **(3)**	Slightly Difficult
	Lead Time	Moderate **(3)**	Long **(4)**	Moderate **(1)**	Moderate
Serviceability					
	Vibration Control	Moderate **(4)**	Slightly Moderate **(3)**	Good **(2)**	Great **(1)**
Feasible		**YES**	**NO**	**YES**	**YES**

Appendix B: Building Loads & Controlling Load Combinations

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)

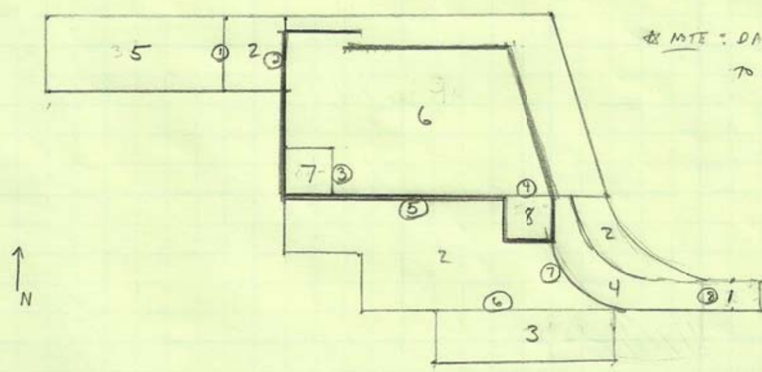
$C_t = 1.0$ (TABLE 7-3)

$I_s = 1.2$ (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

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

SNOW LOAD DUE TO DRIFTING



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

SIMPLIFIED ROOF PLAN

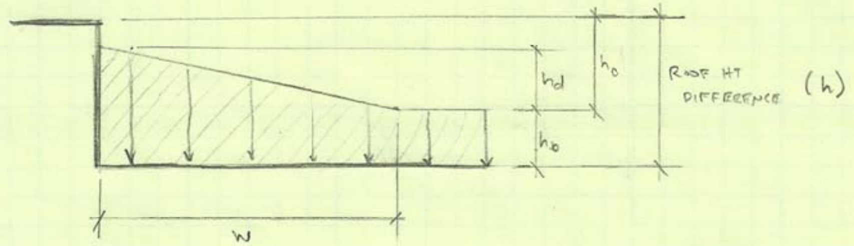
ROOF HEIGHTS
1 → 12'-9½"
2 → 17'-4"
3 → 21'-6"
4 → 28'-6"
5 → 30'
6 → 72'-¼"
7 → 82'-11½"
8 → 86'-6"

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

PARAPET LOCATIONS
(6 → 90'-0")

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

SNOW ACCUMULATION DIAGRAM (FIG 7-8)



SNOW DENSITY (γ)

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

$= .13(50) + 14$

$\gamma = 20.5\ pcf \le 30\ pcf$ OK ✓

BASE SNOW ACCUMULATION HEIGHT (ft)

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

$= (1.0)(42.0)$

$P_s = 42.0\ psf$

$h_b = 42.0/20.5 = 2.05\ ft$

CALCULATION FOR LOCATION [ROOF 5 TO ROOF 2]

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

$h_c = h - h_b$

$= 12.667' - 2.05' = 10.62'$

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

WINWARD DIR.

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

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

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

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 ✓

∴ $w_d = 4h_d = 4(4.24) = 16.96\ ft$

FIND DRIFT SNOW LOAD (pd)

$p_d = h_d \gamma$

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

$+ \ 42$ psf GROUND SNOW LOAD

129 psf

AE THESIS | LATERAL REDESIGN | MICHAEL KOSTICK

WIND LOAD

LOCATION : SYRACUSE, NY
BUILDING TYPE : HEALTHCARE
TOPOGRAPHY : HOMOGENOUS
TERRAIN : URBAN
MEAN ROOF HEIGHT : 72'-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 ASCE 7-10 26.7.3

TOPOGRAPHY FACTOR : $K_{zt} = 1.0$ (SECTION 26.8.1 - 26.8.2)

GUST FACTOR :

- 72'-0" > 60' → ∴ NOT LOW RISE [26.9.2]
  ↳ FLEXIBLE

- CHECK 26.9.3 PROVISIONS
  ↳ BLDG HEIGHT = 72' < 300' OK ✓ (26.9.2.1)

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

- E-W DIRECTION

$$L_{EFF} = \frac{(16+30+44+58+72)(196.73)}{16+30+44+58+72} = 196.73'$$

$$72' < 4(196.73) = 786.92' \text{ OK ✓}$$

- N-S DIRECTION

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

$$72' < 120(4) = 480' \text{ OK ✓}$$

∴ APPROXIMATE LOWER BOUND FREQUENCY BY EQ 26.9-3

$$n_a = \frac{43.5}{h^{.9}} = \frac{43.5}{(72)^{.9}} = .927 \text{ Hz} = 1.079 \text{ sec.}$$

[FREQ] [PERIOD]

.927 Hz < 1.0 ∴ BLDG IS FLEXIBLE

AE THESIS LATERAL ANALYSIS

GUST FACTOR → 26.9.5 - FLEXIBLE BLDGS. - 196.73' WIDTH

$$G_F = .925\left[\frac{1+1.7I_z\sqrt{g_Q^2Q^2+g_r^2R^2}}{1+1.7g_v I_z}\right]$$

- $g_v = \underline{3.4}$
- $g_Q = \underline{3.4}$

$$g_r = \sqrt{2\ln(3600 n_1)} + \frac{.577}{\sqrt{2\ln(3600 n_1)}} \qquad n_1 = .927$$

$$= \sqrt{2\ln(3600(.927))} + \frac{.577}{\sqrt{2\ln(3600(.927))}} = \underline{4.17} = g_r$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)}$$

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

$$N_1 = \frac{n_1 L_z}{V_z} = \frac{.927(350.06)}{84.72}$$

$$N_1 = 3.83$$

[26.9-16]

$$V_z = b\left(\frac{\bar{z}}{33}\right)^{\alpha}\left(\frac{88}{60}\right)V$$

$b = .45$

$\alpha = 1/4.0 = .25$

$$V_z = .45\left(\frac{43.2}{33}\right)^{.25}\left(\frac{88}{60}\right)120 = 84.72$$

$$L_z = \ell\left(\frac{z}{33}\right)^{\epsilon} = 320\left(\frac{43.2}{33}\right)^{.33} = 350.06$$

$$R_n = \frac{7.47(3.83)}{(1+10.3(3.83))^{5/3}}$$

$R_n = .060$

R_h, R_B, R_L

$$R_h \rightarrow \eta = 4.6\frac{n_1 h}{V_z} = \frac{.927(72)}{84.72}(4.6) = 3.62$$

$$R_B \rightarrow \eta = 4.6\frac{n_1 B}{V_z} = \frac{.927(196.73)}{84.72}(4.6) = 9.90$$

$$R_L \rightarrow \eta = \frac{15.4 n_1 L}{V_z} = \frac{.927(120)}{84.72}(15.4) = 20.2$$

$$R_h = \frac{1}{3.62} - \frac{1}{2(3.62)^2}\left(1-e^{-2(3.62)}\right) = .238$$

$$R_B = \frac{1}{9.90} - \frac{1}{2(9.90)^2}\left(1-e^{-2(9.90)}\right) = .096$$

$$R_L = \frac{1}{20.2} - \frac{1}{2(20.2)^2}\left(1-e^{-2(20.2)}\right) = .0483$$

AE THESIS

GUST FACTOR CONT – 196.73' WIDTH

$$R = \sqrt{\frac{1}{.02}(.06)(.238)(.096)(.53+.47(.0483))} = \underline{.195}$$

$$I_z = c\left(\frac{10}{z}\right)^{1/6} = .30\left(\frac{10}{43.2}\right)^{1/6} = \underline{.235}$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{B+h}{L_z}\right)^{.63}}} = \sqrt{\frac{1}{1+.63\left(\frac{196.73+72}{350.06}\right)^{.63}}} = \underline{.808}$$

$$\bullet\ G_F = .925\left[\frac{1+1.7(.235)\sqrt{(3.4)^2(.808)^2+(4.17)^2(.195)^2}}{1+1.7(3.4)(.235)}\right] =$$

$$\boxed{G_{F_{N-S}} = .841}$$

GUST FACTOR – 120' WIDTH [E-W]

- $g_v = 3.4$ • $L_z = 350.06$ • $I_z = .235$
- $g_a = 3.4$ • $V_z = 84.72$
- $g_r = 4.17$ • $R_n = .06$

$\eta \rightarrow R_h, R_B, R_L$

$$R_h \rightarrow \eta = \frac{4.6(.927)(72)}{84.72} = 3.62$$

$$R_B \rightarrow \eta = \frac{4.6(.927)(120)}{84.72} = 6.04$$

$$R_L \rightarrow \eta = \frac{15.4(.927)(196.73)}{84.72} = 33.2$$

$$R_h = .238$$

$$R_B = \frac{1}{6.04} - \frac{1}{2(6.04)^2}\left(1-e^{-2(6.04)}\right) = .152$$

$$R_L = \frac{1}{33.2} - \frac{1}{2(33.2)^2}\left(1-e^{-2(33.2)}\right) = .0297$$

$$R = \sqrt{\frac{1}{.02}(.06)(.238)(.152)(.53+.47(.0297))} = .243$$

AE THESIS

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

$$G_F = .925\left[\frac{1 + 1.7(.235)\sqrt{(3.4)^2(.836)^2 + (4.17)^2(.243)^2}}{1 + 1.7(3.4)(.235)}\right]$$

$$\boxed{G_{F\,E\text{-}W} = .865}$$

ENCLOSURE CLASSIFICATION: ENCLOSED [26.2 DEFINITIONS]
INTERNAL PRESSURE COEFFICIENTS: $\pm .18$ [TABLE 26.11-1]

USE MWFRS

VELOCITY PRESSURE COEFFICIENTS:

$K_z = 2.01\left(\frac{15}{z_g}\right)^{2/\alpha}$ $\quad z < 15'$ $\qquad z = 1200'$, $\alpha = 7.0$

$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}$ $\quad 15' \le z \le 1200'$

SAMPLE CALC - 4TH FLOOR

$$K_{44} = 2.01\left(\frac{44}{1200}\right)^{2/7.0} = .78$$

VELOCITY PRESSURE

$q_z = .00256\,K_z K_{zt} K_d V^2$ [Eq - 27.3-1]

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

$= 24.4$ PSF

EXTERNAL PRESSURE COEFFICIENTS $(C_p)(C_N)$

WINDWALL $\rightarrow$ $C_p = .8$ [USE q_z]
SIDEWALL $\rightarrow$ $C_p = -.7$ [USE q_h]
LEEWARD $\rightarrow$ $C_p = -.37$ [E-W] $\qquad L/B = \frac{196.73}{120} = 1.64$

$C_p = -.5$ [N-S] $\qquad L/B = \frac{120}{196.73} = .610$

ROOF PRESSURE

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

E-W $\quad h/L = 72/196.73 = .366$

$\theta = 0°$ (FLAT ROOF)

HORIZ. DIST FROM WINDWARD

$0 - h/2 = 0' - 36'$
$h/2 - h = 36' - 72'$
$h - 2h = 72' - 144'$
$> 2h = > 144'$

AE THIS IS

$h/L = .366$ [E-W]

ROOF COEFFICIENTS

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

$h/L = .6$ [N-S]

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

WIND PRESSURE → 44' E-W - WINDWARD

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

$p = (24.4)(.865)(.8) - (28.2)(\pm .18) = 21.96$ PSF

AE THESIS — LATERAL REDESIGN

SEISMIC

LOCATION : SYRACUSE, NEW YORK [LAT: 43.04°, LONG: -76.14°]
OCCUPANCY CATEGORY: IV - HEALTHCARE
SITE CLASS :
SPECTRAL RESPONSE ACCELERATION PARAMETERS (MCER)

- $S_{MS} = F_a S_S$ [Eq 11.4-1]
- $S_{M1} = F_v S_1$ [Eq 11.4-2]

$S_S = .143$ [USGS → .2s SPECTRAL RESP]
$S_1 = .062$ [USGS → 1s " "]
$F_a = 1.6$ [TABLE 11.4-1] } SITE CLASS D
$F_v = 2.4$ [TABLE 11.4-2] }

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

DESIGN SPECTRAL ACCELERATION PARAMETERS

- $S_{DS} = 2/3\, S_{MS} = 2/3(.2288) = .153$ (SHORT PERIOD) [Eq 11.4-3]
- $S_{D1} = 2/3\, S_{M1} = 2/3(.1488) = .099$ (1-SEC PERIOD) [Eq 11.4-4]

IMPORTANCE FACTOR
$I_e = 1.50$ [TABLE 1.5-2]

SEISMIC DESIGN CRITERIA

- $S_{DS} = .153 < .167 \rightarrow SDC = A$ [TABLE 11.6-1]
- $S_{D1} = .099 \rightarrow .067 < .099 < .133 \rightarrow SDC = C$ [TABLE 11.6-2]

USE SDC = C

ANALYSIS PROCEDURE
PER TABLE 12.6-1 USE EQUIVALENT LATERAL FORCE ANALYSIS

EFFECTIVE SEISMIC WEIGHT
W = 19759 kips [SPREAD SHEET]

PERIOD CALCULATION
COMPUTER MODEL → $T_x = 1.64s$
$T_y = 1.83s$

ESTIMATED → $T_a = C_t h_n^x$ - [CONCRETE MOMENT FRAMES]
$T_a = .016(72)^{.9} = .751s$

$C_u = 1.7$ B/c $.099 \leq .1$

$$T = \text{MIN OF} \begin{cases} C_u T_a = 1.7(.751) = 1.28s \leftarrow \text{CONTROLS} \\ T_x = 1.64s \\ T_y = 1.83s \end{cases}$$

AE THESIS | LATERAL

SPECTRAL RESPONSE COEFFICIENT → R=3 - ORDINARY CONCRETE MOMENT FRAMES

$$C_S = \text{MIN OF} \begin{cases} \dfrac{S_{DS}}{(R/I)} = \dfrac{.153}{\left(\frac{3}{1.5}\right)} = .0765 \\[2ex] \dfrac{S_{D1}}{T\left[\frac{R}{I}\right]} = \dfrac{.099}{1.28\left[\frac{3}{1.5}\right]} = .0387 \longleftarrow \text{CONTROLS} \\[2ex] \dfrac{S_{D1}\,T_L}{T^2\left[\frac{R}{I}\right]} = \dfrac{(.099)(6s)}{(1.28)^2\left[\frac{3}{1.5}\right]} = .181 \end{cases}$$

BASE SHEAR

$V = C_S W = .0387\,(19759)$ — BLDG WT = 19759 KIPS [SPREAD SHEET]

$V = 765$ KIPS

VERTICAL DISTRIBUTION OF FORCES

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

$k = 1.39$ [FROM INTERPOLATION] 12.8.3

SAMPLE CALC: FOURTH FLOOR → HEIGHT = 44' $w = 4087^k$

$$C_{v4} = \frac{(4087)(44)^{1.39}}{4012461} = .1961$$

$$F_4 = (765^k)(.1961) = 150 \text{ KIPS}$$

LOAD COMBINATION

DL: SLAB: $(\frac{9}{12})(150) = 112.5$ psf
BEAMS: $(\frac{15 \times 22}{144})(150) = 344$ PLF ≈ 23 psf
COLUMNS: $(\frac{24 \times 24}{144})(150) = 600$ PLF ≈ 10 psf
ST DL = 25 psf (FLOOR)
= 10 psf (ROOF)

LL: 100 psf (FLOOR)
L_r: 20 psf (ROOF)
SNOW: 42 psf (ROOF)
WIND: VARIABLE
QUAKE: VARIABLE

SLAB: GRAVITY ONLY

• $1.4D = 1.4[112.5 + 25] = 192.5$ psf (FLOOR)
$= 1.4[112.5 + 10] = 171.5$ psf (ROOF)

• $1.2D + 1.6L + .5(L_r \text{ or } S \text{ or } R) = 1.2(112.5+25) + 1.6(100) = 325$ psf (FLOOR)
$= 1.2(112.5+10) + .5(20) = 157$ psf (ROOF)
$= 1.2(112.5+10) + .5(42) = 168$ psf

• $1.2D + 1.0W + L + .5(L_r \text{ or } S \text{ or } R) = 1.2(112.5+25) + 100 = 265$ psf (FLOOR)
$1.2(112.5+25) + .5(20) = 157$ psf (ROOF)
$1.2(112.5+25) + .5(42) = 168$ psf

• $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } .5W) = 1.2(112.5+25) + 100 = 265$ psf (FLOOR)
$= 1.2(112.5+10) + 1.6(20) = 179$ psf (ROOF)
$= 1.2(112.5+10) + 1.6(42) = 214.2$ psf

• $1.2D + 1.0E + L + .2S = 1.2(112.5+25) + 100 = 265$ psf (FLOOR)
$= 1.2(112.5+10) + .2(42) = 155.4$ psf (ROOF)

• $.9D + 1.0W = .9(112.5+25) = 123.75$ psf (FLOOR)
$= .9(112.5+10) = 110.25$ psf (ROOF)

• $.9D + 1.0E = .9(112.5+25) = 123.75$ psf (FLOOR)
$= .9(112.5+10) = 110.25$ psf (ROOF)

FLOOR	ROOF
325 psf	214 psf

AMPAD

BEAMS: GRAVITY + LATERAL (DEPENDENT)

- $1.4D = 1.4(112.5 + 25 + 23) = 224.7$ psf (FLOOR)
  $1.4(112.5 + 10 + 23) = 203.7$ psf (ROOF)

- $1.2D + 1.6L + .5(L_r \text{ or } S \text{ or } R) = 1.2(160.5) + 1.6(100) = 353$ psf (FLOOR) ←
  $= 1.2(145.5) + .5(20) = 185$ psf (ROOF)
  $= 1.2(145.5) + .5(42) = 196$ psf

- $1.2D + 1.0W + L + .5(L_r \text{ or } S \text{ or } R) = 1.2(160.5) + 100 + 1.0W = 293$ psf $+ 1.0W$ (FLOOR) ←
  $= 1.2(145.5) + .5(20) + 1.0W$ 185 psf $+ 1.0W$ (ROOF)
  $= 1.2(145.5) + .5(42) + 1.0W$ 196 psf $+ 1.0W$ ←

- $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } .5W) = 1.2(160.5) + 100 = 293$ psf (FLOOR)
  $= 1.2(160.5) + .5\underline{W} = 193$ psf $+ .5W$
  $= 1.2(145.5) + 1.6(20) + .5W = 207$ psf $+ .5W$ (ROOF)
  $= 1.2(145.5) + 1.6(42) + .5W = 242$ psf $+ .5W$ ←

- $1.2D + 1.0E + L + .2S = 1.2(160.5) + 1.0E + 100 = 293$ psf $+ 1.0E$ (FLOOR) ←
  $= 1.2(145.5) + 1.0E + .2(42) = 183$ psf $+ 1.0E$ (ROOF) ←

- $.9D + 1.0W = .9(160.5) + 1.0W = 145$ psf $+ 1.0W$ (FLOOR)
  $= .9(145.5) + 1.0W = 131$ psf $+ 1.0W$ (ROOF)

- $.9D + 1.0E = .9(160.5) + 1.0E = 145$ psf $+ 1.0W$ (FLOOR)
  $= .9(145.5) + 1.0E = 131$ psf $+ 1.0W$ (ROOF)

FLOOR	ROOF
• 353 psf	• 242 psf + .5W
• 293 + 1.0W	• 183 psf + 1.0E
• 293 + 1.0E	• 196 psf + 1.0W

COLUMNS: GRAVITY + LATERAL (DEPENDENT)

- $1.4D = 1.4(112.5+25+23+10) = 239$ psf (FLOOR)
  $= 1.4(112.5+10+23+10) = 218$ psf (ROOF)

- $1.2D + 1.6L + .5(L_r \text{ or } S \text{ or } R) = 1.2(170.5) + 1.6(100) = 365$ psf (FLOOR) ←
  $= 1.2(155.5) + .5(20) = 197$ psf
  $= 1.2(155.5) + .5(42) = 208$ psf (ROOF)

- $1.2D + 1.0W + L + .5(L_r \text{ or } S \text{ or } R) = 1.2(170.5) + 1.0W + 100 = 305$ psf $+ 1.0W$ (FLOOR)
  $= 1.2(155.5) + 1.0W + .5(20) = 197$ psf $+ 1.0W$
  $= 1.2(155.5) + 1.0W + .5(42) = 208$ psf $+ 1.0W$ ← (ROOF)

- $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } .5W) = 1.2(170.5) + 100 = 305$ psf
  $= 1.2(170.5) + .5W = 305$ psf $+ .5W$ (FLOOR)
  $= 1.2(155.5) + 1.6(20) + .5W = 219$ psf $+ .5W$
  $= 1.2(155.5) + 1.6(42) + .5W = 254$ psf $+ .5W$ ← (ROOF)

- $1.2D + 1.0E + L + .2S = 1.2(170.5) + 1.0E + 100 = 305$ psf $+ 1.0E$ ←
  $= 1.2(155.5) + 1.0E + .2(42) = 195$ psf $+ 1.0E$ ←

- $.9D + 1.0W = .9(170.5) + 1.0W = 153.5$ psf $+ 1.0W$
  $= .9(155.5) + 1.0W = 140$ psf $+ 1.0W$

- $.9D + 1.0E = .9(170.5) + 1.0E = 153.5$ psf $+ 1.0E$
  $.9(155.5) + 1.0E = 140$ psf $+ 1.0E$

FLOOR	ROOF
• 365 psf	• 254 psf + .5W
• 305 psf + 1.0W	• 195 psf + 1.0E
• 305 psf + 1.0E	• 208 psf + 1.0W

Appendix C: Gravity Design Calculations

2-WAY SLAB DESIGN - EQUIVALENT FRAME METHOD

- USE ACI TABLE 9.5(c) FOR TRIAL THICKNESS
  - ↳ IF SLAB HAD ONLY EDGE BMS → $\ell_n/33 = h$
  - BASED ON 22" SQUARE COLUMN → $h = \dfrac{30 - 22/12}{33}(12) = 10.24"$
- REDUCE TRIAL h BY ≈ 15% TO ACCOUNT FOR BM STIFFNESS
  - ↳ $h = 8.7"$ → USE $\underline{\underline{9"}}$
- SIZE BEAM DEPTHS AS 2.5h [α_f VALUES → SLIGHTLY GREATER THAN 1.0]
  - $h_b = (2.5)(9) = 22.5"$ → USE 24"
- TRIAL DIMENSIONS
  - 22" x 22" SQR COLUMNS
  - 9" THICK SLAB ($h_f = 9"$)
  - 22" x 24" BEAM ($b_w = 22"$) + ($h_w = 24 - 9 = 15"$)
- DETERMINE α_f → $\alpha_f = \dfrac{E_{cb} I_b}{E_{cs} I_s}$ → $E_{cb} = E_{cs}$
- EDGE BEAMS

$$b_{eff} = b_w + h_w \le b_w + 4h_f$$
$$= 22 + 15 \le 22 + 4(9)$$
$$b_{eff} = 37"$$

$$k = \frac{1 + \left(\frac{37}{22} - 1\right)\left(\frac{9}{24}\right)\left[4 - 6\left(\frac{9}{24}\right) + 4\left(\frac{9}{24}\right)^2 + \left(\frac{37}{22} - 1\right)\left(\frac{9}{24}\right)^3\right]}{1 + \left(\frac{37}{22} - 1\right)\left(\frac{9}{24}\right)}$$

$$k = 1.27$$

$$I_b = k\frac{b_w h^3}{12} = (1.27)\left(\frac{22 \times 24^3}{12}\right) = 32187 \text{ in}^4$$

SAMPLE I_s VALUES

$$I_{s\,N\text{-}S\,A} = \frac{(8.36' \times 12)(9)^3}{12} = 6094 \text{ in}^4$$

$$I_{s\,E\text{-}W\,1} = \frac{(15 \times 12)(9)^3}{12} = 10935 \text{ in}^4$$

• INTERIOR BEAMS

$$b_{eff} = b_w + 2h_w \le b_w + 8h_f$$
$$= 22 + 2(15) \le 22 + 8(9)$$
$$b_{eff} = 52''$$

$$k = \frac{1 + \left(\frac{52}{22} - 1\right)\left(\frac{9}{24}\right)\left[4 - 6\left(\frac{9}{24}\right) - 4\left(\frac{9}{24}\right)^2 + \left(\frac{52}{22} - 1\right)\left(\frac{9}{24}\right)^3\right]}{1 + \left(\frac{52}{22} - 1\right)\left(\frac{9}{24}\right)}$$

$$k = 1.47$$

$$I_b = (1.47)\frac{(22 \times 24^3)}{12} = 37256 \text{ in}^4$$

SAMPLE I_s VALUES

$$I_{s_{N-S}}{}_{B} = \frac{(15 + 8.36)(12)(9)^3}{12} = 17029 \text{ in}^4$$

$$I_{s_{E-W}}{}_{2} = I_{s_{N-S}}{}_{C} = \frac{(30 \times 12)(9)^3}{12} = 21870 \text{ in}^4$$

• CALCULATE α

SAMPLE CALC. FOR PANEL A-B, 1-2

$$\alpha_{B_{A-B(1)}} = \frac{I_b}{I_s} = \frac{32187}{10935} = 2.94$$

$$\alpha_{B_{A-B(2)}} = \frac{37256}{21870} = 1.70$$

$$\alpha_{B_{1-2(A)}} = \frac{32187}{6094} = 5.28$$

$$\alpha_{B_{1-2(B)}} = \frac{37256}{17029} = 2.19$$

$$\alpha_{FM} = \frac{2.94 + 1.70 + 5.28 + 2.19}{4} = 3.03$$

CRITICAL SECTION → LOWEST α_{FM} VALUE → PANEL D-E, 1-2

$\alpha_{FM} = (1.39 + 1.7 + 1.7 + 1.7)/4 = 1.63$

MAX L_n $30 - 22/12 = 28.17'$

MIN L_n $30 - 22/12 = 28.17'$

β $28.17/28.17 = 1.00$

$h = 8.62" < 9"$ $\underline{\underline{OK}}$ ✓

α_{FM} 1.63

USE 9" SLAB

E.Q. $h = \dfrac{L_n(.8 + f_y/200000)}{36 + 5\beta(\alpha_{fm} - .2)}$

h FROM EQ 8.62

MIN h 5"

(2-WAY SHEAR)

• CHECK SHEAR FORCE IN BEAMS @ COLUMNS.

$DL = S.W$
$= .25 psf - SI_{DL}$

$LL = 100 psf$
* IGNORING S.W. OF BM.

$q_u = 1.2\left(\frac{9}{12}(.15) + .025\right) + 1.6(.100) = .325$ ksf

$d = 24" - \underbrace{1.5}_{C.C.} - \underbrace{.5}_{STIRRUP} - \frac{1.00}{2} = 21.5"$

$V_U = .325\left[30 \times 30 - \left(\frac{22 + 21.5}{12}\right)^2\right] = 288$ KIPS.

ϕV_c FOR 4 BMS FRAMING INTO COLUMN. $(\lambda = 1.0)$

$\Sigma\phi V_c = 4\left[.75 \times 2 \times 1\sqrt{1000} \times 22 \times 21.5\right] \times \frac{1K}{1000 lb} = 179.5^K$

$2\Sigma\phi V_c \geq V_U$ → ∴ USE STIRRUPS + SELECTED BMS FOR SHEAR.

COMPUTE MOMENTS IN SLAB

DL: SW
25 PSF SI_{DL}

LL: 100 psf

} TYPICAL FLOOR

$W_{D\,(SLAB-BEAM)} = \left[\frac{9}{12} \times .15 + .025\right] = .1375$

$W_{L\,(SLAB-BEAM)} = [.100] = .100$ ksf

B/C $w_L \leq 3/4\, w_D$ → PATTERN LOADING CAN BE IGNORED [13.7.6.2]

SAMPLE CALC → ALONG COLUMN LINE 2 → 30'

$W_{D\,SLAB} = 1.2[.1375][30] = 4.95$ K/FT

$W_{L\,SLAB} = 1.6[.100][30] = 4.80$ K/FT

$W_{TOTAL} = 9.75$ K/FT

LOADING DUE TO BEAM SELF WEIGHT

EXTERIOR BEAM – ADD FACADE WEIGHT [280 PLF]

$$w_D = 1.2\left[\frac{22\times15}{144}\times.15 + .280\right] = .75\ ^{K}/_{FT}$$

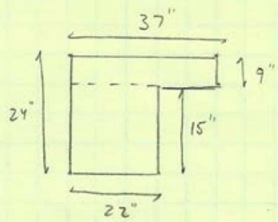
INTERIOR BEAM

$$w_D = 1.2\left[\frac{22\times15}{144}\times.15\right] = .413\ ^{K}/_{FT}$$

DISTRIBUTION OF MOMENTS TO COLUMN / MIDDLE STRIPS & BEAM/SLAB

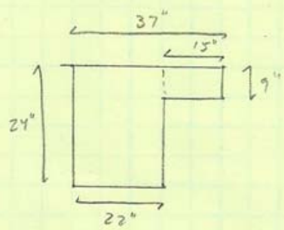
$$\beta_t = \frac{E_{cb}\,C}{2E_{cs}\,I_s} \qquad C = \Sigma\left[\left(1-.63\frac{x}{y}\right)\frac{x^3y}{3}\right]$$

EDGE BEAM



$$C = \left[\left(1-.63\left(\tfrac{9}{37}\right)\right)\left(\frac{37(9)^3}{3}\right)\right] + \left[\left(1-.63\left(\tfrac{22}{15}\right)\right)\left(\frac{15(22)^3}{3}\right)\right]$$

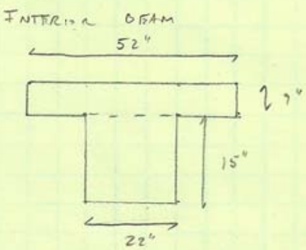
$C = 11659\ \text{in}^4$



$$C = \left[1-.63\left(\tfrac{9}{15}\right)\right]\left(\frac{15(9)^3}{3}\right) + \left[\left(1-.63\left(\tfrac{22}{24}\right)\right)\left(\frac{24(22)^3}{3}\right)\right]$$

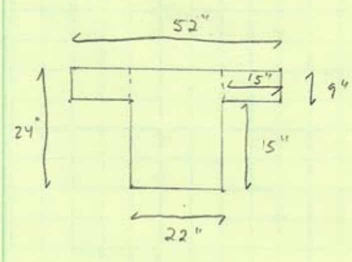
$C = 38257\ \text{in}^4$ ← CONTROLS

INTERIOR BEAM



$$C = \left[\left(1-.63\left(\tfrac{9}{52}\right)\right)\left(\frac{52(9)^3}{3}\right)\right] + \left[\left(1-.63\left(\tfrac{22}{15}\right)\right)\left(\frac{15(22)^3}{3}\right)\right]$$

$C = 15304\ \text{in}^4$



$$C = 2\left[\left(1-.63\left(\tfrac{9}{15}\right)\right)\left(\frac{15(9)^3}{3}\right)\right] + \left[\left(1-.63\left(\tfrac{22}{24}\right)\right)\left(\frac{24(22)^3}{3}\right)\right]$$

$C = 40525\ \text{in}^4$ ← CONTROLS

SLAB REINFORCEMENT → USE #5 BARS → .75" CLEAR COVER

$d_1 = 9 - .75 - \frac{.625}{2} = 7.94"$

$d_2 = 9 - .75 - .625 - \frac{.625}{2} = 7.31"$

$M_{uMAX} = -247.1$ → ASSUMING $jd = .95$

$A_s = \frac{247.1 \times 12000}{.9(60000)(.95)(7.94)} = 7.28 \text{ in}^2$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{7.28(60000)}{.85(4000)(15 \times 12)} = .71 \text{ in}$

$c = \frac{.71}{.85} = .84"$ $\frac{.003}{.84}(7.94 - .84) = .025 > .005$ → TCS $\phi = .9$

$jd = d - a/2 = 7.94 - \frac{.71}{2} = 7.59"$

$A_s = \frac{M_u \times 12000}{.9 \times 60000 \times 7.59} =$ $A_s = \frac{M_u \times 12000}{.9(60000) \times 6.96}$

$A_s = .0292 M_u$ [E-W] → $d = 7.94"$

$A_s = .032 M_u$ [N-S] → $d = 7.31"$

MIN A_s → $A_{s\,MIN} = .0018 bh$ [ACI 13.3.1]

MAX SPACING = $2h = 2(9) = 18"$

MIN # OF BARS SPACES = $\frac{b \times 12}{18}$

BEAM REINFORCEMENT

$A_s \geq \frac{M_u}{\phi f_y jd} = \frac{857 \times 12000}{.9(60000)(.95)(21.5)} = 9.32 \text{ in}^2$

$a = \frac{A_s f_y}{.85 f'_c b} = \frac{(9.32)(60000)}{.85(4000)(22)} = 7.48$ → $c = \frac{7.48}{.85} = 8.8"$

$\frac{.003}{8.8}(21.5 - 8.8) = .0043 < .005$ → $\phi = .65 + (.0043 - .002)\left(\frac{250}{3}\right) = .842$

$A_s = \frac{857 \times 12000}{.842(60000)(21.5 - \frac{7.48}{2})} = 11.46 \text{ in}^2$

$A_{s\,PROVIDED} = 5 \times 1.27 \times 2 = 12.7 \text{ in}^2 > 11.46 \text{ in}^2$ OKAY

$S_{MIN} = d_b$ FOR BAR TO BAR // + ⊥ [NO LESS THAN 1"]

ROOF SLAB DESIGN

↳ STILL MAINTAIN 9" SLAB

DL = SW + 10 psf SI_{DL} LL = 42 psf → (FLAT ROOF SNOW LOAD)

$DL = 1.2\left[\frac{9}{12}(.15) + .01\right] = .147$ ksf
$LL = 1.6[.042] = .067$ ksf
$\Big\}$ w = .214 KSF

$W_{DBEAM-EDGE} = .75$ KLF
$W_{DBEAM-INT} = .413$ KLF

(WIDE BEAM)

CHECK SHEAR IN SLAB → CRITICAL PANEL LOCATION → 30 × 30 d = 7.94

$\alpha \frac{L_2}{L_1} > 1.00$

$w = 1.2\left(\frac{9}{12} \times .15 + .025\right) + 1.6(.100) = .325$ ksf

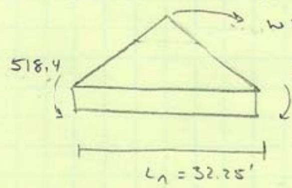
$V_u = (.325)(1)\left(15 - \frac{22/12}{2} - \frac{7.94}{12}\right) = 4.36^K$ / FT WIDTH

$$\phi V_c = \frac{(.75)(2)\sqrt{3000}\,(12)(7.94)}{1000} = 7.83^K \text{/ FT WIDTH}$$

$7.83^K > 4.36^K$ OK ✓

CHECK SHEAR IN BEAM → FRAME 3 F-H

w = 30(.325) = 9.75 KLF BM S.W = .413



$$V = \frac{1.15\, w_u \ell_n}{2}$$

$$V_u = 1.15(.413)\frac{32.25}{2} + 1.15 \times \frac{1}{2}(16.125 \times 9.75) + \frac{827.3 - 518.4}{32.25}$$

$V_u = 157.6^K$

MAX Vc w/o Vs

$$V_c = \frac{2\sqrt{4000}\,(22)(21.5)}{1000} = 59.8^K$$

$\phi V_c = (.75)(59.8) = 44.9^K < 157.6^K$ → NEED Vs

ADD STIRRUPS.

$$\text{MAX } s = \text{MIN OF} \begin{cases} d/2 = 21.5/2 = 10.75'' \\ 24'' \end{cases}$$

USE #3 STIRRUPS → AREA = .11 × 2 LEGS = .22 IN^2

$$\text{SPACING } (s) = \text{MIN OF} \begin{cases} \dfrac{(.22)(60000)}{50(22)} = 12'' \leftarrow \text{CONTROLS.} \\ \dfrac{(.22)(60000)}{.75\sqrt{4000}\,(22)} = 12.65'' \end{cases}$$

$$V_u/\phi = 107.6/.75 = 143.5^k \qquad V_c = 59.8^k$$

$$s = \frac{(.22)(60)(21.5)}{143.5 - 59.8}$$

$$= 3.39'' \rightarrow \text{SAY } 3'' \text{ O.C.}$$

USE (2) #3 STIRRUPS @ 3" O.C.

AE THESIS

COLUMN DESIGN – PURE GRAVITY

LOADS:

DL: SLAB $= \left(\frac{9}{12}\right)(150) = 112.5$ psf

LL: 100 psf – FLOOR
42 psf – ROOF SNOW

SI = 25 psf – FLOOR
10 psf – ROOF

BEAMS $= \left[\frac{22 \times 15}{144} \times 150 \times 30\right] 2$

$= \frac{20265}{(900)} = 23$ psf

30' × 30' BAY – 1ST STORY

10% COLUMN S.W.

$P_u = \Big[4\left[1.2(112.5+25+23) \times 1.10\right] + \left[1.2(112.5+10+23) \times 1.10\right]$
$+ 4\left[1.6(100)\right] + \left[1.6(42)\right]\Big] \times 900 / 1000$

$P_u = \underline{1572.03}$ KIPS → TRY A 24" × 24"

$.01 A_g < \rho < .08 A_g$

DESIGN

$\phi P_n = .8\phi\left[.85 f'_c (A_g - A_{ST}) + f_y A_{ST}\right]$

$P_u = 1572.03^k = .8(.65)\left[.85(4)(576 - A_{ST}) + 60 A_{ST}\right]$

$A_{ST} > 18.8 \text{ in}^2$

TRY (16) #10's $= 20.32 \text{ in}^2 > 18.8 \text{ in}^2$ OK ✓

$\rho = \frac{20.32}{24 \times 24} = .035 \begin{cases} < .08 \\ > .01 \end{cases}$ OK ✓

$\phi P_n = .8(.65)\left[.85(4)(576 - 20.32) + 60(20.32)\right]$

$1616.4^k > 1572.03^k$ OK ✓

- USE #3 TIES @ $S_{min} = \begin{cases} 16 \times 1.27 = 20.32" \\ 48 \times .375 = 18" \end{cases}$ ← CONTROLS

@ 18"

CHECK LONGITUDINAL BAR SPACING

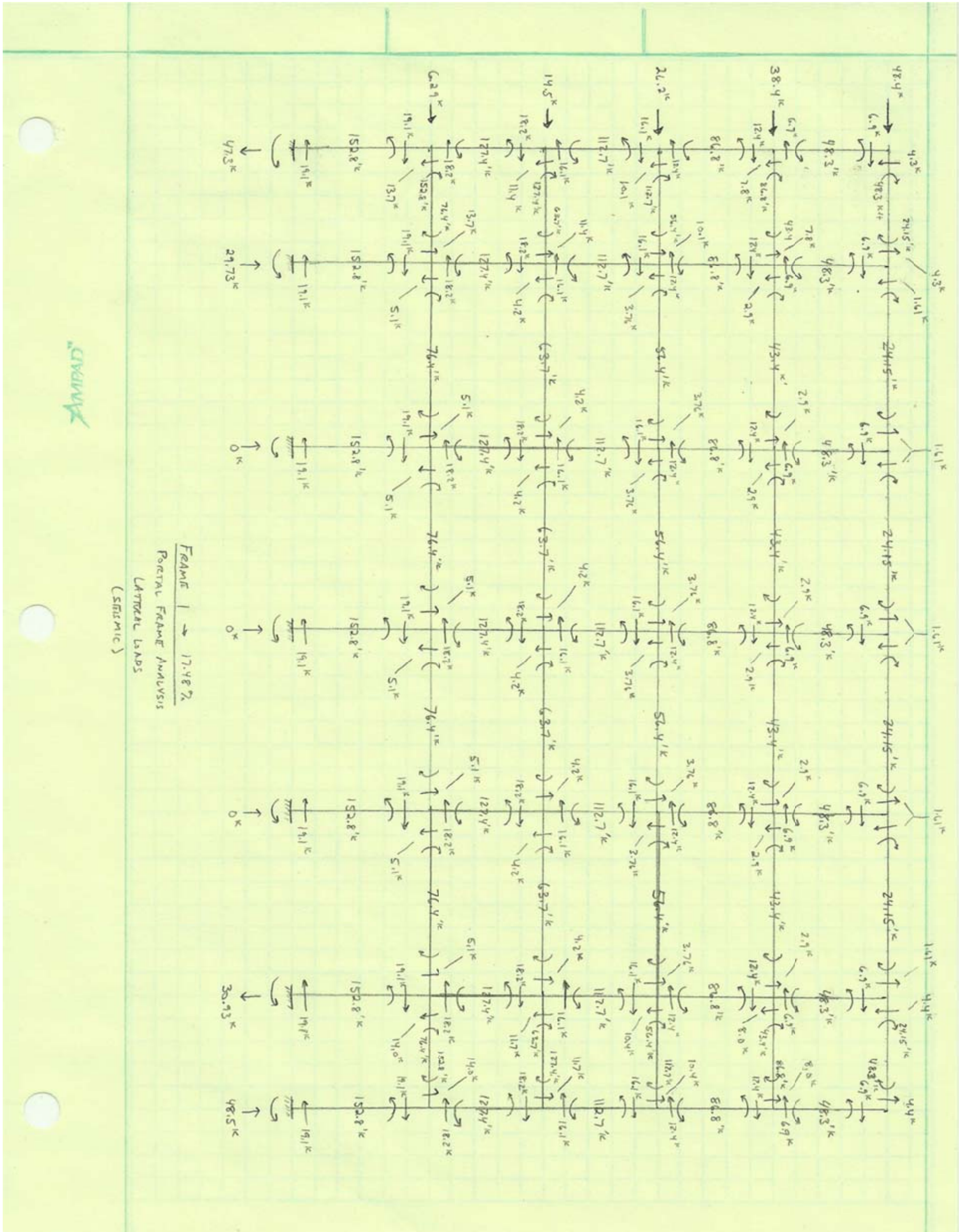
CLEAR COVER $= 1.5 + .375 + \frac{1.27}{2} = 2.51"$ EACH SIDE

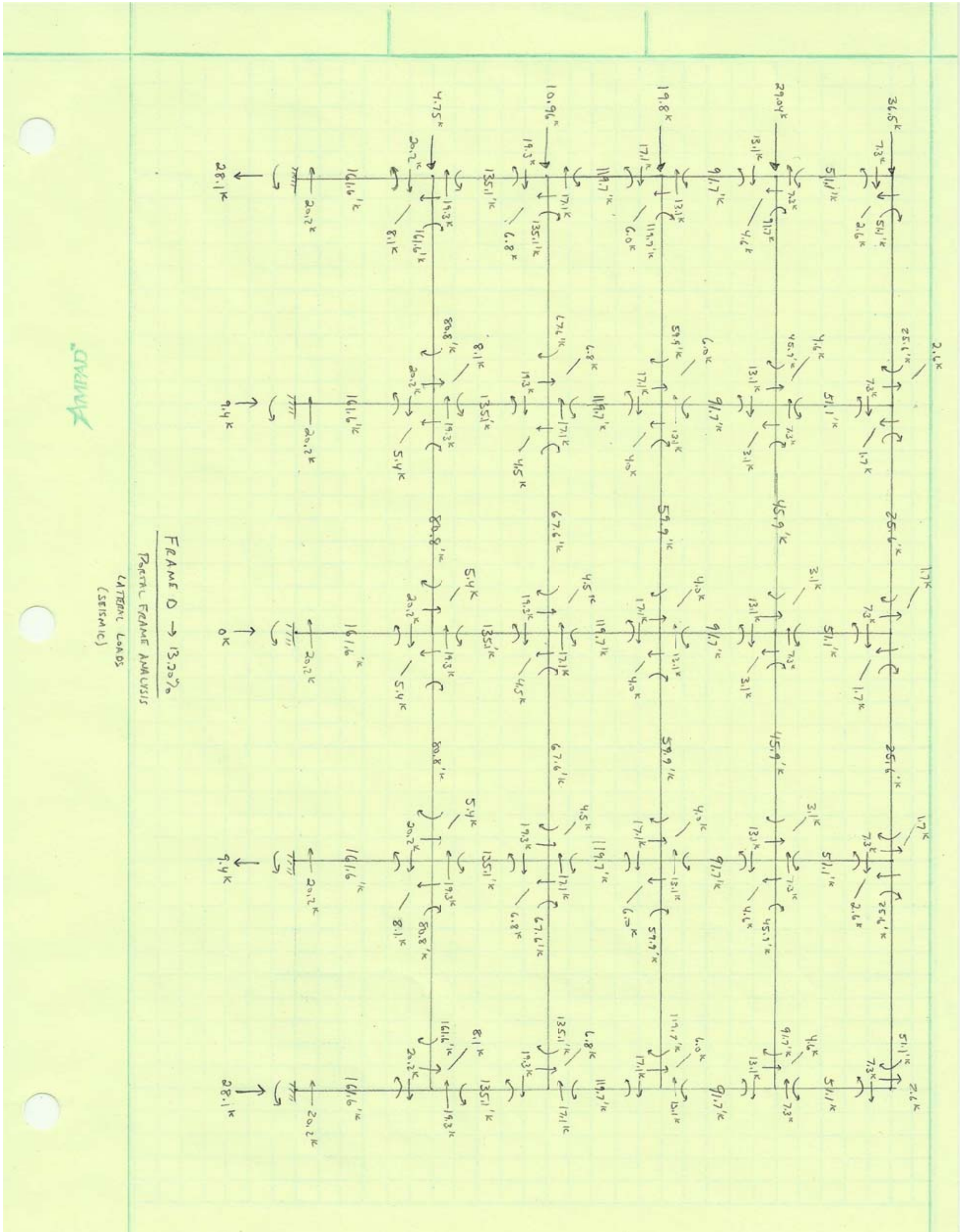
$24 - 2(2.51) = 18.98"$

CLEAR SPACE $= 18.98" - \underbrace{4}_{\text{BARS}}[1.27] = \frac{13.9"}{4 \text{ SPACES}} = 3.48" > d_b$

$< 6"$ → NO CROSS-TIES

Appendix D: Lateral Design Calculations


FRAME 1 → 17.48%
PORTAL FRAME ANALYSIS
LATERAL LOADS
(SEISMIC)


FRAME D → 13.20%
PORTAL FRAME ANALYSIS
LATERAL LOADS
(SEISMIC)

DESIGN OF BEAM - LAT + GRAV

FLEXURAL MEMBER - ACI 318-08 21.5.1

- COMPRESSION FORCE LESS THAN : $.1 A_g f'_c = .1(24 \times 22)(4) = 211.2^k$ OK ✓
- CLEAR SPAN > 4d → 4(21.5) = 86" OK ✓
- WIDTH > 10" OK ✓
  < COLUMN WIDTH OK ✓

DESIGN MOMENTS

$M_{U\ EXT\ NEG} = -288.3$ k-ft   d = 21.5"

$$A_s = \frac{288.3 \times 12}{.9 \times 60 \times .9 \times 21.5} = 3.31 \text{ in}^2$$

$20 d_b = 22$

$d_b \leq \frac{22}{20} = 1.1$ in (#9's MAX)

★ TRY USING (4) #7's + (2) #6's → $A_s = 3.28$ ★

$$a = \frac{3.28 \times 60}{.85 \times 4 \times 22} = 2.63 \rightarrow C = \frac{2.63}{.85} = 3.10$$

$$\varepsilon_s = \frac{.003}{3.10}(21.5 - 3.10) = .0178 > .005 \rightarrow \therefore \phi = .9$$

CALCULATE ϕM_n

$$\phi M_n = (.9)(3.28)(60)\left(21 - \frac{3.10}{2}\right)$$

$\phi M_n = 3533$ k-in
$= 294$ k-ft > 288.3 k-ft OK ✓

★ USE (4) #7's + (2) #6's

DESIGN MOMENTS

$$M_{U\ EXT\ POS} = \text{MAX OF} \begin{cases} 197.5 \text{ k-ft} & \leftarrow \text{CONTROLS} \\ .5(\phi M_n) = .5(294) = 147 \text{ k-ft} \end{cases}$$

197.5 k-ft → TRY (5) #6's → $A_s = 2.2$ in² → $\phi = .9$

$\phi M_n = 204$ k-ft > 197.5 k-ft OK ✓

CALCULATE PROBABLE MOMENTS $\phi = 1.0$ → EXT - NEG $\phi M_n = 294^{k\text{-}ft}$

↳ ADD FACTOR OF 1.25

$$M_{PR} = (1.0)(1.25 \times 60)(3.28)\left(21.5 - \left(\frac{1.25(2.63)}{2}\right)\right)$$

$M_{PR} = 407$ k-ft (SWAY RIGHT)

ACTS C.W. ON BEAM

DETERMINE STIRRUPS IN BEAMS → SIZE + SPACING
↳ REFER TO ACI 21.3 - INTERMEDIATE MOMENT FRAMES
DESIGN FOR WORST CASE

$V_u = \frac{M_{nL} + M_{nR}}{L_n} + \frac{W_u L_n}{2}$ → N-S FOURTH FLOOR COLUMN LINE H. → $M_{nL} = 874$ k-ft, $M_{nR} = 874$ k-ft, $w_u = .325 \times 30' = 9.75$ k/ft, $L_n = 30 - 24/12 = 28'$

$V_u = \frac{874 + 874}{28} + \frac{9.75(28)}{2} = 198.9 \rightarrow 199$ KIPS

$V_c = 2\lambda\sqrt{f'_c}\, b_w d = 2(1)\sqrt{4000}\,(22)(21.5) = 59.8$ KIPS

$V_s = \frac{199}{.75} - 59.8 = 205$ KIPS $< 8(1)\sqrt{4000}\,(22)(21.5) = 239$ KIPS OK ✓

$S = \frac{A_v f_y d}{V_s} = \frac{(2 \times .31)(60)(21.5)}{205.5} = 3.89$ → USE 3.5"

USE 2 LEGS #5 STIRRUP @ 3.5" STARTING 2" FROM COLUMN FACE

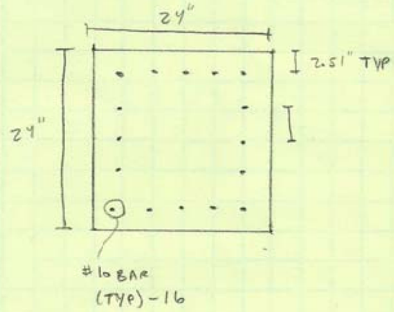
PROVIDE HOOPS OVER 2h DISTANCE FROM FACE OF COLUMN
[21.3.4.2]

$$\text{SPACING} = \text{SMALLEST OF} \begin{cases} d/4 = 21.5/4 = 5.38" \leftarrow \text{CONTROLS} \\ 8\,d_{b\,smallest} = 8 \times .75 = 6" \\ 24\,d_{b\,hoop} = 24 \times .625 = 15" \\ 12\text{ in} = 12" \end{cases}$$

USE HOOPS IN PLACE OF STIRRUPS FOR FIRST 2h FROM COLUMN FACE.
$2h = 2(24) = 48"$

HOOPS END @ 48"

PROVIDE #5 2-LEG STIRRUPS @ 3.5" O.C. FOR REST OF BEAM



a) AXIAL STRENGTH P_o

$\varepsilon_c = .003$ $\quad A_s = 16 \times 1.27 = 20.32\ in^2$

$\varepsilon_y = \frac{60}{29000} = .00207 < \varepsilon_{si}$

$P_o = .85 f'_c A_c + A_s f_y$

$= .85(4)((24 \times 24) - 20.32) + (20.32)(60)$

$P_o = 3108\ K$ $\quad \rightarrow \phi P_n = .8 \phi P_o = .8(.65)3108 = 1616\ K$

$\phi P_n = 2020$

b) BALANCED M_b, P_b

$c = \frac{\varepsilon_u}{\varepsilon_u + \varepsilon_y} d_{imax} = \frac{.003}{.003 + .00207}(24 - 2.51) = 12.72"$

$a = .85(12.72) = 10.81"$

$\varepsilon_{s1} = \frac{\varepsilon_u}{c}(c - d_i) = \frac{.003}{12.72}(12.72 - 2.51) = .0024 > .00207 \rightarrow f_{s1} = 60^{ksi}$ (C)

$\varepsilon_{s2} = \frac{.003}{12.72}(12.72 - 7.26) = .0013 < .00207 \rightarrow f_{s2} = .0013(29000) = 35.1^{ksi}$ (C)

$\varepsilon_{s3} = \frac{.003}{12.72}(12.72 - 12) = .00017 < .00207 \rightarrow f_{s3} = 4.92^{ksi}$ (C)

$\varepsilon_{s4} = \frac{.003}{12.72}(12.72 - 16.75) = -.00095$ $\quad f_{s4} = -27.6^{ksi}$ (T)

$\varepsilon_{s5} = \frac{.003}{12.72}(12.72 - 21.49) = -.00207 \rightarrow f_{s5} = -60^{ksi}$ (T)

$P_b = .85(4)(24)(.85)(12.72) + [(5)(1.27)(60 - 60) + 2(1.27)(35.1 + 4.92 - 27.6)]$

$P_b = 913.8^K \rightarrow \phi P_b = .65(913.8) = 594^K$

$M_b = .85(4)(24)(.85)(12.72)\left[\frac{24}{2} - \frac{.85(12.72)}{2}\right] + \left[(5)(1.27)\underset{\times(60)}{}\left(\frac{24}{2} - 2.51\right) + 5(1.27)\underset{\times(-60)}{}\left(\frac{24}{2} - 21.49\right)\right.$

$+ 2(1.27)\underset{\times(35.1)}{\left(\frac{24}{2} - 7.26\right)} + 2(1.27)\underset{\times(4.92)}{\left(\frac{24}{2} - 12\right)} + (2)(1.27)\underset{\times(-27.6)}{\left(\frac{24}{2} - 16.75\right)}$

$M_b = 13805\ K\text{-}IN$

$= 1150.4\ K\text{-}ft \rightarrow \phi M_n = 747.7\ K\text{-}ft$

c) PURE BENDING → ASSUME TRY $c = 5.97''$

$$f_{s1} = \frac{.003}{5.97}(5.97-2.51)(29000) = 50.4 \text{ ksi}$$

$$f_{s2} = \frac{.003}{5.97}(5.97-7.26)(29000) = -18.8 \text{ ksi}$$

$$f_{s3} = \frac{.003}{5.97}(5.97-12)(29000) = \quad \rightarrow -60 \text{ ksi}$$

$$f_{s4} = \frac{.003}{5.97}(5.97-16.75)(29000) = \quad \rightarrow -60 \text{ ksi}$$

$$f_{s5} = \frac{.003}{5.97}(5.97-21.49)(29000) = \quad \rightarrow -60 \text{ ksi}$$

$P_n = 0$

$$M_o = .85(4)(24)(.85)(5.97)\left[\frac{24}{2} - \frac{.85(5.97)}{2}\right] + \left[5(1.27)(50.4)\left(\frac{24}{2} - 2.51\right) + 2(1.27)(-18.8)\left(\frac{24}{2} - 7.26\right) + (2)(1.27)(-60)\left(\frac{24}{2} - 12\right) + (2)(1.27)(-60)\left(\frac{24}{2} - 16.75\right) + 5(1.27)(-60)\left(\frac{24}{2} - 21.49\right)\right]$$

$M_o = 922.4$ k-ft $\qquad \phi M_n = 830.2$ k-ft

d) PURE TENSION

$c = -\infty \qquad \varepsilon_s = -\varepsilon_y \qquad f_{si} = -f_y$

$$T_o = \Sigma A_{si} f_{si} = \left[2(5)(1.27)(-60) + (3)(2)(1.27)(-60)\right]$$

$T_o = -1219$ k

$\phi T_n = -1097$ k

e) $c = 16$

$$f_{s1} = \frac{.003}{16}(16-2.51)(29000) = \quad = 60 \text{ ksi}$$

$$f_{s2} = \frac{.003}{16}(16-7.26)(29000) = 47.5 \text{ ksi}$$

$$f_{s3} = \frac{.003}{16}(16-12)(29000) = 21.75 \text{ ksi}$$

$$f_{s4} = \frac{.003}{16}(16-16.75)(29000) = -4.08 \text{ ksi}$$

$$f_{s5} = \frac{.003}{16}(16-21.49)(29000) = -29.9 \text{ ksi}$$

$$P_n = (.85)(4)(24)(.85)(16) + \left[5(1.27)(60-29.9) + 2(1.27)(21.75 + 47.5 - 4.08)\right]$$

$P_n = 1466.4$ k

$\phi P_n = 953.2$ k

e)

$$M_n = .85(4)(24)(.85)(16)\left[\frac{24}{2} - \frac{(.85)(16)}{2}\right] + \left[5(1.27)(60)\left(\frac{24}{2} - 2.51\right) + 2(1.27)(47.5)\left(\frac{24}{2} - 7.26\right)\right.$$
$$\left. + 2(1.27)(21.75)\left(\frac{24}{2} - 12\right) + 2(1.27)(-4.08)\left(\frac{24}{2} - 16.75\right) + 5(1.27)(-29.1)\left(\frac{24}{2} - 21.49\right)\right]$$

$M_n = 984.1$ k-ft $\qquad \phi M_n = 639.7$ k-ft

f) $\varepsilon_t = .005$

$$C = \frac{.003}{.003 + .005}(21.5) = 8.06$$

$$f_{s1} = \frac{.003}{8.06}(8.06 - 2.51)(29000) = 59.9^{ksi}$$

$$f_{s2} = \frac{.003}{8.06}(8.06 - 7.26)(29000) = 8.6^{ksi}$$

$$f_{s3} = \frac{.003}{8.06}(8.06 - 12)(29000) = -42.5^{ksi}$$

$$f_{s4} = \frac{.003}{8.06}(8.06 - 16.75)(29000) = -60^{ksi}$$

$$f_{s5} = \frac{.003}{8.06}(8.06 - 21.49)(29000) = -60^{ksi}$$

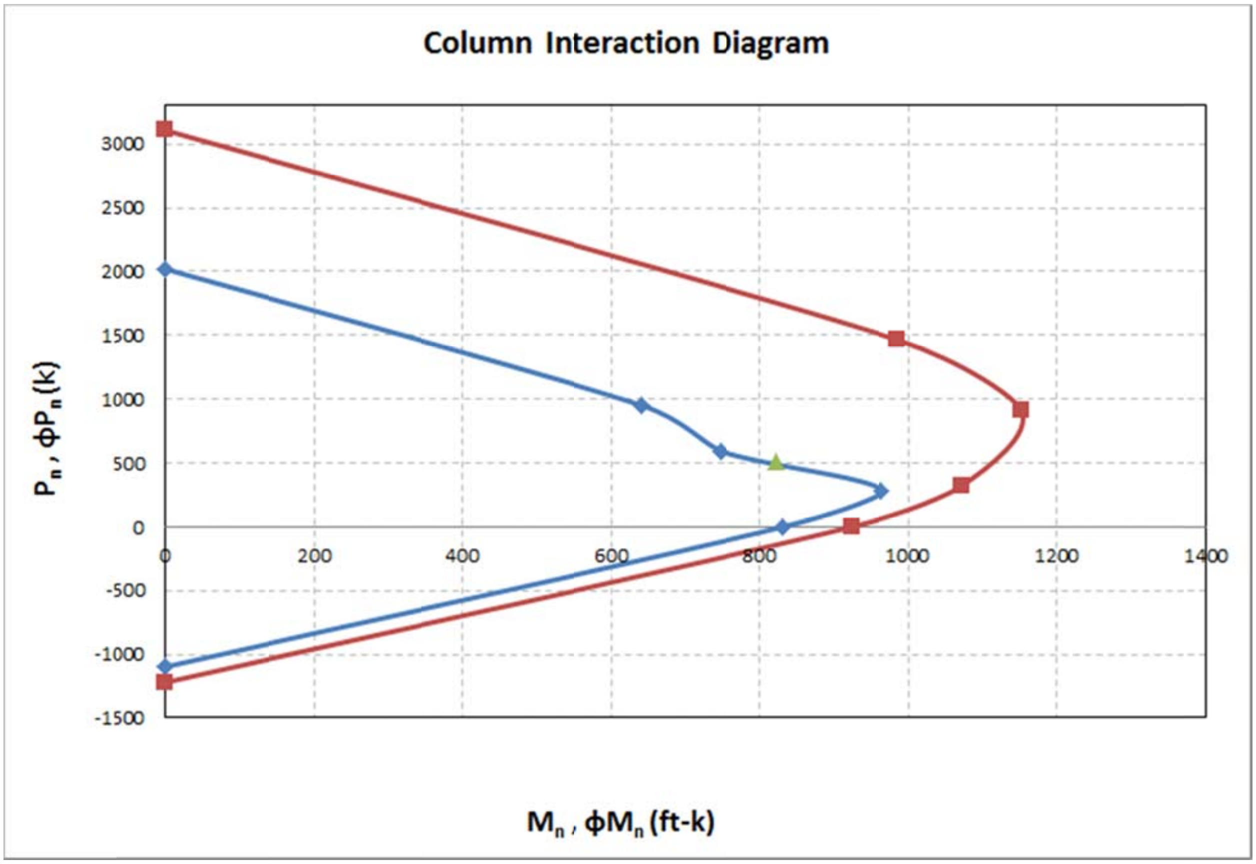
$$P_n = .85(4)(24)(.85)(8.06) + \left[(5)(1.27)(59.9 - 60) + (2)(1.27)(8.6 - 42.5 - 60)\right]$$

$P_n = 319.9 \qquad \phi P_n = 287.9^k$

$$M_n = .85(4)(24)(.85)(8.06)\left[\frac{24}{2} - \frac{(.85)(8.06)}{2}\right] + \left[5(1.27)(59.9)\left(\frac{24}{12} - 2.51\right) + 2(1.27)(8.6)\left(\frac{24}{12} - 7.26\right)\right.$$
$$\left. + (2)(1.27)(-42.5)\left(\frac{24}{12} - 12\right) + (2)(1.27)(-60)\left(\frac{24}{12} - 16.75\right) + (5)(1.27)(-60)\left(\frac{24}{12} - 21.49\right)\right]$$

$M_n = 1070.5$ k-ft $\qquad \phi M_n = 963.5$ k-ft

a) $\phi = .65$
b) $\phi = .65$
c) $\phi = .9$
d) $\phi = .9$
e) $\phi = .65$
f) $\phi = .9$


Column Interaction Diagram
3000
2500
2000
1500
1000
500
0
-500
-1000
-1500
0
200
400
600
800
1000
1200
1400
P_n , ϕP_n (k)
M_n , ϕM_n (ft-k)

CHECK COLUMN DESIGN → GRAVITY ONLY

IS STORY SIDESWAY / NON SIDESWAY

$$Q = \frac{\Sigma P_u \Delta_o}{V_{us} \ell_c} \le .05$$

$\Sigma P_u = 39813^k$
$\Delta_o = .802''$
$V_{us} = 765^k$ [SEISMIC STORY SHEAR / BASE SHEAR]
$\ell_c = 16' = 192''$

$$Q = \frac{39813(.802)}{(765)(192)} = .217 > .05 \rightarrow \therefore \text{ SWAY FRAME}$$

CHECK INTERIOR COLUMN G4 @ BASE STORY - SLENDERNESS →

$P_u = 1575^{kips}$ $M_u = 396$ k-ft

$\ell_u = 16(12) - 24'' = 168'' = 14'$
$r = .3h = .3(24) = 7.2''$ [ACI 10.10.1.2]
$k = 1.44$ [ACI FIG R10.10.1.1]

$$\frac{k \ell_u}{r} \le 22$$

$$\Psi_A = \frac{\Sigma I_c/\ell_c}{\Sigma I_b/\ell_b} = \frac{\left[\frac{27648}{192}\right]2}{\left[\frac{25344}{360}\right]} = 4.1$$

$$\frac{(1.44)(168)}{7.2} = 33.6 > 22$$

$\therefore$ COLUMN IS SLENDER

$\Psi_B = 0$ [FIXED SUPPORT]

FIND δ_{ns} (CONT)

$$\delta_{ns} = \frac{C_m}{1 - \frac{P_u}{.75 P_c}} \ge 1.0$$

$C_m = .6 + .4\left(\frac{M_1}{M_2}\right)$
$C_m(\text{int}) = .6 + .4\left(\frac{0}{396}\right) = .6$

$P_u = 1575$
$M_1 = 0$
$M_2 = 396$

$$P_c = \frac{\pi^2 EI}{(k \ell_u)^2} =$$

$$EI = \frac{.2 E_c I_g + E_s I_{se}}{1 + \beta_{dns}} =$$

☆ $I_g = \frac{(24)(24)^3}{12} = 27648 \text{ in}^4$

$E_c = 57000\sqrt{4000} = 3600$ ksi

$I_{se} = 2.06 \rho_g \gamma^2 I_g$ [TABLE 12-1] - McGREGOR → $\gamma = \frac{24 - 2.5 - 2.5}{24} = .79$

$= 2.06(.045)(.79)^2(27648)$

ASSUME $\rho_g \approx .045$

$I_{se} = 1599.5 \text{ in}^4$

$$\beta_{dns} = \frac{\text{MIN FACTORED SUSTAINED LOAD}}{\text{TOTAL FACTORED AXIAL LOAD}} = \frac{847}{1575} = .54$$

$$EI = \frac{.2(3600)(27648) + 29000(1600)}{1 + .54} = 43 \times 10^6 \text{ k-in}^2$$

$$P_c = \frac{\pi^2(43 \times 10^6)}{(1.44 \times 168)^2} = 10455 \text{ kips}$$

$$\delta_{ns} = \frac{.6}{1 - \frac{1575}{.75(10455)}} = .75 \rightarrow \text{USE } 1.0$$

CHECK SECTION FOR GRAVITY $\rightarrow M_c = M_2 = 316$

$$e = \frac{M_c}{P_u} = \frac{316 \times 12}{1575} = 3.02_{IN}$$

$$e/_h = \frac{3.02}{24} = .126$$

FROM FIG A-7b $\rightarrow$ (R-4-60-.75)

$\hookrightarrow \frac{\phi P_n}{A_g} = 2.85 \rightarrow A_g \geq \frac{1575}{2.85} = 552.6\ in^2 \rightarrow 576^{in^2} \geq 552.6\ in^2$ OK✓ (24X24 COLUMN)

FROM ACI 10.10.6.5

$M_{2min} = P_u(.6 + .03h) = 1575(.6 + .03(24)) = 2079\ k\text{-}in = 173.3\ k\text{-}ft$

$A_{ST} = \rho_g A_g = .045(576) = 25.92\ in^2$

$\hookrightarrow$ TRY (16) #11's $\rightarrow \rho = \frac{1.56 \times 16}{576} = .0433$

$\phi P_{n_{MAX}} = \phi(.8)[.85 f'_c (A_g - A_{st}) + f_y A_{st}]$

$= .65(.8)[.85(4)(576 - (16 \times 1.52)) + (60 \times (16 \times 1.52))]$

$\phi P_{n_{MAX}} = 1734$ KIPS > 1575 KIPS OK✓

COLUMN CHECK $\rightarrow$ GRAVITY + LATERAL

$P_u = 1575^k$

$M_2 = M_{2ns} + \delta_s \times M_{2s}$ $\rightarrow M_{n2s} = 68\ k\text{-}ft$

$\rightarrow M_{2s} = 231\ k\text{-}ft$

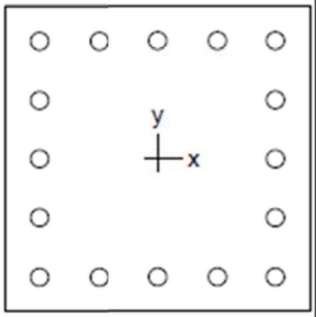
$$\delta_s = \frac{1}{1 - Q} \geq 1.0 \rightarrow Q = \frac{\Sigma P_u \times \Delta_o}{V_{us} \times l_c} = \frac{(32865)(.812)}{(765) \times (192)} = .182$$

$$\delta_s = \frac{1}{1 - .182} = 1.22$$

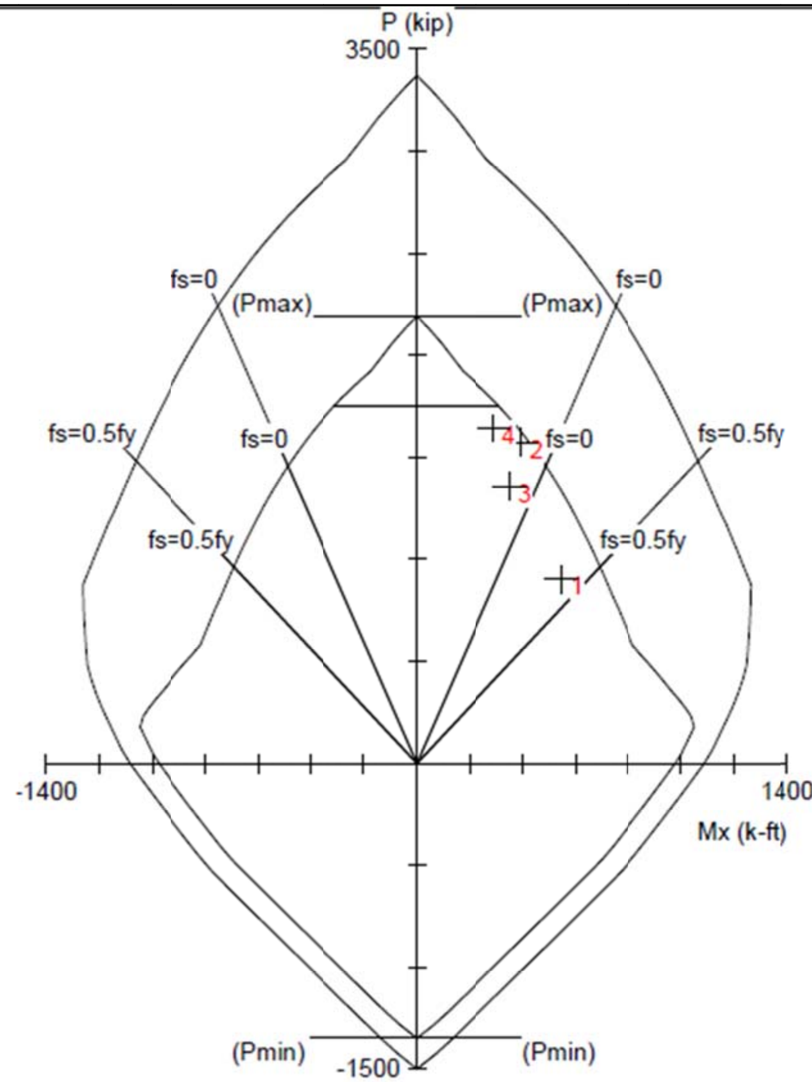
$M_2 = 68 + 1.22(231) = 350\ k\text{-}ft \rightarrow$ FROM INTERACTION DIAGRAM $\rightarrow \phi \frac{M_n}{bh^2} = .35$

$.35(24)(24)^2 = 4838\ in\text{-}k = 403\ ft\text{-}k > 350\ ft\text{-}k$ OK✓

SEE INTERACTION DIAGRAMS.



24 x 24 in



Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 04/03/12

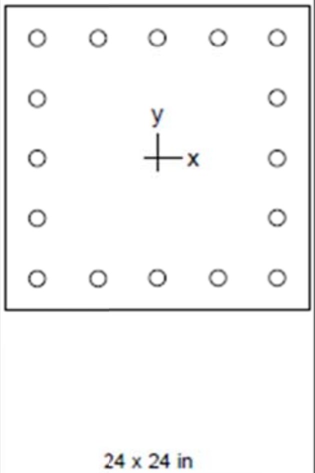
Time: 03:12:28

spColumn v4.80. Licensed to: Penn State University. License ID: 58318-1027155-4-22545-2CF68

File: Y:\Kostick\SpColumn\24x24 Column - Bottom.col

Project: AE Thesis

Column: Bott. Col.		Engineer: MK	
f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	16 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 24.96 in^2	rho = 4.33%
fc = 3.4 ksi		Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Min clear spacing = 3.24 in	Clear cover = 2.00 in
Confinement: Tied			
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65			



Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

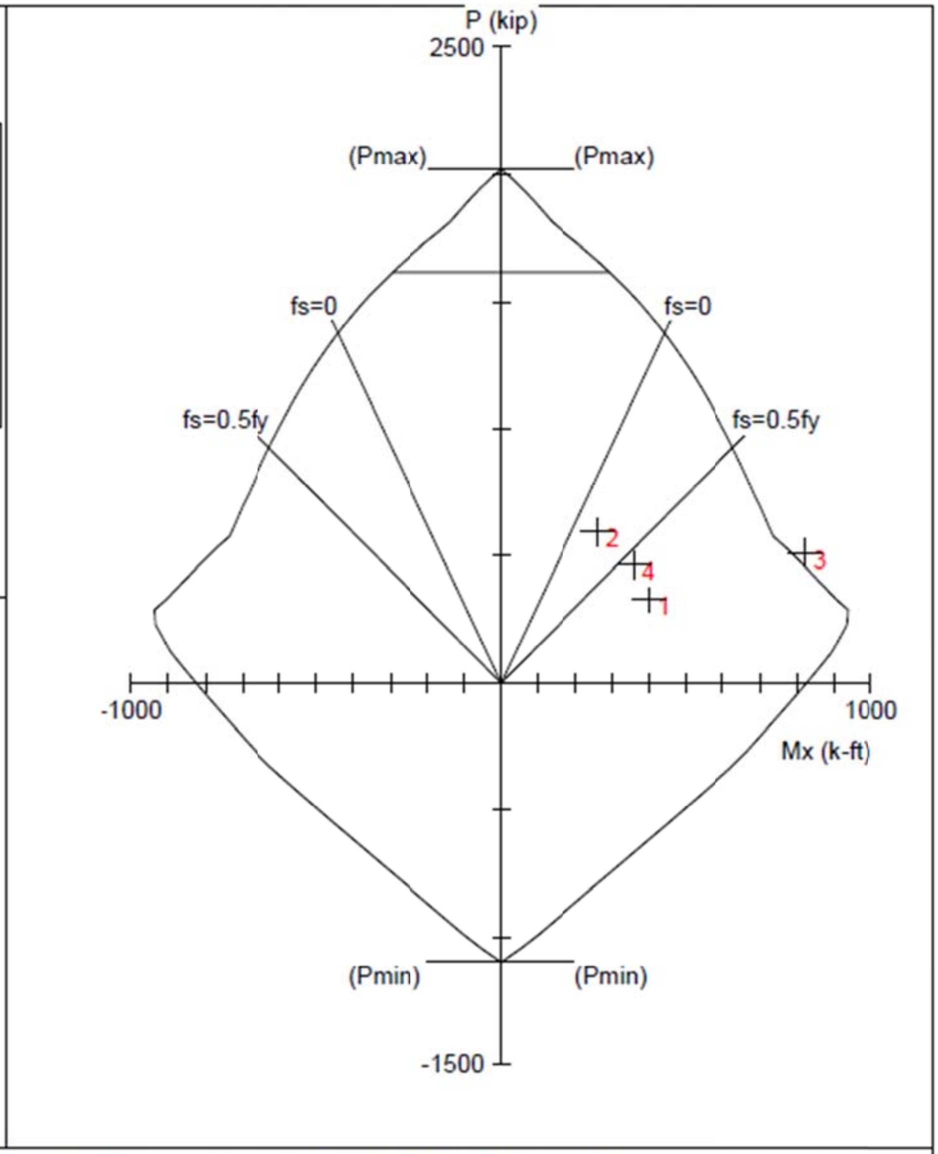
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 04/03/12

Time: 03:09:22



spColumn v4.80. Licensed to: Penn State University. License ID: 58318-1027155-4-22545-2CF68

File: Y:\Kostick\SpColumn\24x24 Column - Top.col

Project: AE Thesis

Column: Bott. Col.		Engineer: MK	
f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	16 #10 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 20.32 in^2	rho = 3.53%
fc = 3.4 ksi		Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Min clear spacing = 3.47 in	Clear cover = 1.88 in

Confinement: Tied

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

COLUMN REINFORCEMENT – TRANSVERSE

↳ PROVIDE HOOPS OVER l_o FROM EACH END OF COLUMN

$$l_o = \text{LARGEST OF} \begin{cases} h = 24'' \\ \tfrac{1}{6}h = 14\times12/6 = 28'' \leftarrow \text{CONTROLS} \\ 18'' \end{cases}$$

$$\text{SPACING} = \text{LEAST OF} \begin{cases} 8\,d_b = 8(1.27) = 10.2'' \leftarrow \text{CONTROLS} \\ 24\,d_{hoop} = 24(.5) = 12'' \\ \tfrac{1}{2}b = \tfrac{1}{2}(24) = 12'' \\ 12\text{ in} = 12'' \end{cases}$$

$$V_u = \frac{M_{nt} + M_{nb}}{L_u} = \frac{667 + 667}{14} = 123.9 \text{ KIPS}$$

$$V_c = 2\left(1 + \frac{N_u}{2000 A_g}\right)\lambda\sqrt{f'_c}\, b_w d$$

$$= 2\left(1 + \frac{352 \times 1000}{2000(24\times24)}\right)(1.0)\sqrt{4000}\,(24)(21.5) = 85.2 \text{ KIPS}$$

$$V_s = \frac{V_u}{\phi} - V_c = \frac{123.9}{.75} - 85.2 = 80 \text{ KIPS}$$

$$s = \frac{A_v f_y d}{V_s} = \frac{(.4)(60)(21.5)}{80} = 6.45 \text{ IN}$$

PLACE #4 HOOPS (2-LEGS) @ 6" ALONG 28" DISTANCE FROM ENDS OF COLUMNS

↳ PROVIDE HOOPS @ 6" O.C. THROUGHOUT ENTIRE COLUMN HEIGHT.

Appendix E: Progressive Collapse Design Calculations

P/C:
TIE FORCE METHOD

DEAD LOAD: SLAB S.W. = 112.5 psf
BEAMS ≈ 42.5 psf
COLUMNS ≈ 16.5 psf
SI DL = 20 psf (ROOF)
10 psf (MEP)
CLADDING = 20 psf (WALL AREA)

LIVE LOAD: L_r = 20 psf
FLOOR = 100 psf
MRI BAY = 125 psf

• $W_F = 1.2D + .5L$

ROOF

LONG./TRANS TIES (INTERNAL TIES)

$W_F = 1.2(112.5 + 42.5 + 16.5 + 20 + 10) + .5(20) = 251.8$ psf

$F_i = 3 W_F L_i$ → $L_i = 38.16'$
$= 3(251.8)(38.16)$
$F_i = 28.8$ k/ft

$\phi R_n = \phi \Omega A_s F_y$
$= (.75)(1.25)(A_s)(60)$
$\phi R_n = 56.25 A_s$

$\Omega = 1.25$ OVERSTRENGTH FACTOR FROM ASCE 41 TABLE 6-4
$\phi = .75$ → ACI MATERIAL PROP.
$F_y = 60$ ksi

$A_{s\,REQ} = \frac{28.8}{56.25} = .512\ in^2/ft$ → USE #6's @ 10" o.c. [.52 in²/ft]

PERIPHERAL TIES

$F_p = 6 w_F L_i L_p$

• PERIMETER → LONGITUDINAL + TRANSVERSE

$W_F = 1.2\left(112.5 + 42.5 + 16.5 + 20 + 10 + \frac{20 \times 7'}{3'}\right) + .5(20) = 307.8$ psf

$F_p = 6(307.8)(30)(3) = 166.2$ KIPS

$A_{s\,REQ} = \frac{166.2}{56.25} = 2.95\ in^2$ → USE (5) #7's (3.00 in²)

• SW STAIR → LONG DIR.

$W_F = 251.8$ psf

$F_p = 6(251.8)(30)(3) = 136$ KIPS

$A_{s\,REQ} = \frac{136}{56.25} = 2.42\ in^2$ → USE (6) #6's → (2.64 in²)

• SW STAIR → SHORT DIR.

$W_F = 251.8$ psf

$F_p = 6(251.8)(15)(3) = 68$ KIPS

$A_{s\,REQ} = \frac{68}{56.25} = 1.2\ in^2$ → USE (3) #6's (1.32 in²)

P/c :
TIE FORCE METHOD

- SE ELEVATORS → LONG DIR

$W_F = 251.8$ psf

$F_P = 6(251.8)(28)(3) = 127$ kips

$As_{REQ} = \frac{127}{56.25} = 2.26$ in^2 → USE (4) #7's (2.40 in^2)

- SE ELEVATORS → SHORT DIR

$W_F = 251.8$ psf

$F_P = 6(251.8)(11.5)(3) = 52.1$ kips

$As_{REQ} = \frac{52.1}{56.25} = .93$ in^2 → USE (2) #7's (1.2 in^2)

- MECHANICAL RUNS → ALL RUNS ; BOTH DIRECTIONS

$W_P = 251.8$ psf

$F_P = 6(251.8)(10)(3) = 45.3$ kips

$As_{REQ} = \frac{45.3}{56.25} = .81$ in^2 → USE (2) #6's (.88 in^2)

VERTICAL TIES → ALL COLUMNS / COLUMN DEPENDENT

COLUMN A1

AREA = 125.5 ft^2

WF = 251.8 psf

$F_V = (125.5)(251.8) = 31.6$ kips

$As_{REQ} = \frac{31.6}{56.25} = .56$ in^2 → NO ADDITIONAL REINFORCEMENT NEED.

ALTERNATIVE PATH LOADING

INCREASE GRAVITY LOADS

$$G_N = \Omega_N[(.9 \text{ or } 1.2)D + (.5L \text{ or } .2S)]$$

DEAD LOAD (D):
SLAB SW = 112.5 psf
BEAMS = 42.5 psf
COLUMNS = 16.5 psf
SI. DL = 10 psf (MEP)
= 20 psf (ROOF)
CLADDING = 280 plf (TYP FLR)

LIVE LOAD (L):
L_r = 20 psf
FLOOR = 100 psf
MRI BAY = 125 psf

SNOW LOAD:
S = 42 psf

$$\Omega_N = 1.04 + \frac{.45}{\left(\theta_{PRA}/\theta_y\right) + .48} \quad \text{[REINFORCED CONCRETE]}$$

$$\theta_y = \frac{Z F_{ye} l_b}{6 E I_b}$$

WHERE $EI_b = .5 E_c I_g = .5(3605 \text{ ksi})\left(\frac{22 \times 24^3}{12}\right) = 45682560 \text{ k-in}^2$

$$\theta_y = \frac{(3168)(60)(30)}{6(45682560)}$$

$$Z = \frac{bh^2}{4} = \frac{(22)(24)^2}{4} = 3168 \text{ in}^3$$

$F_{ye} = 60$ ksi

$\theta_y = .0208$ RADIANS

$\theta_{PRA} = .025$ [TABLE 4-1 UFC 4-023-03]

$$\Omega_N = 1.04 + \frac{.45}{\left(\frac{.025}{.0208}\right) + .48} = 1.31$$

$G_N = 1.31[1.2(112.5 + 42.5 + 16.5 + 10) + .5(100)]$ - [TYPICAL FLOOR LOAD]
$G_N = 350.8$ psf

GRAVITY LOADS

$G = (.9 \text{ or } 1.2)D + (.5L \text{ or } .2S)$
$= 1.2(112.5 + 42.5 + 16.5 + 10) + .5(100)$ - [TYP. FLOOR]
$G = 267.8$ psf

LATERAL LOADS

$L_{LAT} = .002 \Sigma P$

WHERE ΣP = SUM OF GRAVITY LOADS @ PARTICULAR LEVEL
$= (112.5 + 42.5 + 16.5 + 10 + 100) \times 21700 \text{ ft}^2$
$= 6108550$ lbs

$L_{LAT} = .002(6108550)$
$= 12.22$ KIPS

P-C ANALYSIS RESULTS.

E D C CP LS 10 B

LONG SIDE

155.03' → REMOVE COLUMN D
SIDE
(1ST FLR ABOVE GRADE) @ 1ST FLOOR → E-W → 22x28 w/ 8 in² TOP + BOTTOM } 1.0.
N-S → 22x24 w/ 8.16 in² TOP / 8 in² BOTTOM

(FLOOR DIRECTLY BELOW ROOF) @ 5TH FLOOR → E-W → 22x28 w/ 8 in² TOP + BOTTOM - 1.0.
N-S → 22x24 w/ 8.16 in² TOP / 8 in² BOTTOM - L.S.

(FLOOR @ MIDHEIGHT) @ 3RD FLOOR → E-W → 22x28 w/ 8 in² TOP + BOTTOM - 1.0.
N-S 22x24 w/ 8.16 in²/8 in² - 1.0.

(FLOOR DIRECTLY ABOVE COLUMN SPLICE) @ 4TH FLOOR → E-W → 22x28 w/ 8 in²/8 in² - 1.0.
N-S → 22x24 w/ 8.16 in²/8.16 in² - 1.0.

SHORT SIDE

120' → REMOVE COLUMN H3
SIDE CURTAIN WALL
@ 1ST FLOOR → N-S → 22x24 w/ 8 in²/6 in² - 1.0.
E-W → AS PROVIDED → 28x28 12.7/5.95 - 1.0.

@ 5TH FLOOR → N-S → 22x28 w/ 8.16 in²/6 in² } 1.0.
E-W → AS PROVIDED → 22x28 12.7/5.95

@ 3RD FLOOR → N-S → 22x28 w/ 8.16 in²/6 in² } 1.0.
E-W → AS PROVIDED → 22x28 12.7/5.95

@ 4TH FLOOR → N-S → 22x28 w/ 8.16 in²/8 in² } 1.0.
E-W → AS PROVIDED → 22x28 12.7/5.95

CORNER COLUMN

NE CORNER COLUMN H1
@ 1ST FLOOR → N-S, E-W } AS IS

@ 5TH FLOOR N-S, E-W } AS IS

@ 3RD FLOOR N-S, E-W } AS IS

@ 4TH FLOOR N-S, E-W } AS IS

REENTRANT CORNER → N-S, E-S } AS IS → ALL

LONG SIDE → N-S, E-W } SAME AS 155.03'

ENHANCED LOCAL RESISTANCE

FOR O.C. IV → APPLY ENHANCED FLEXURAL RESISTANCE TO ALL PERIMETER COLUMNS FOR THE 1ST 2 STORIES ABOVE GRADE.

FLEXURAL RESISTANCE OF BOTTOM COLUMNS → 24"x24" w/ (16) #11's [5 IN EACH FACE]

↳ FLEXURAL FAILURE OF COLUMN (M_n) UNDER AXIAL LOAD. (P_u)

CONSIDER 2 CASES

FLEXURAL RESISTANCE = GREATER OF { 2 × BASELINE F.R. → INITIAL DESIGN (FACTOR FOR COLUMNS IN O.C. IV); EXISTING F.R. → FROM ALTERNATIVE DESIGN }

★ SINCE NO COLUMNS NEEDED TO BE REDESIGNED FROM ALTERNATIVE PATH METHOD:

F.R. = 2 × BASELINE F.R.

BASELINE F.R. → ASSUME WORST CASE.

FIND M_n WHEN P_u = 968 KIPS. → USE INTERACTION DIAGRAM W/ NO ϕ FACTOR

↳ FROM DIAGRAM → 1200 K-FT

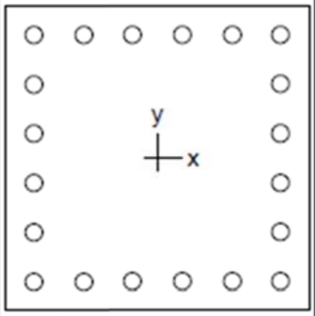
M_n = 1200 k-ft > M_n w/ NO AXIAL LOAD OK ✓

UPSIZE COLUMN TO 30"x30" w/ (20) #14s [6 IN EACH FACE]

M_n > 2 BASE M_n

2500 ft-k > 2(1200 ft-k)

2500 ft-k > 2400 ft-k OK ✓



30 x 30 in

Code: ACI 318-11

Units: English

Run axis: About X-axis

Run option: Investigation

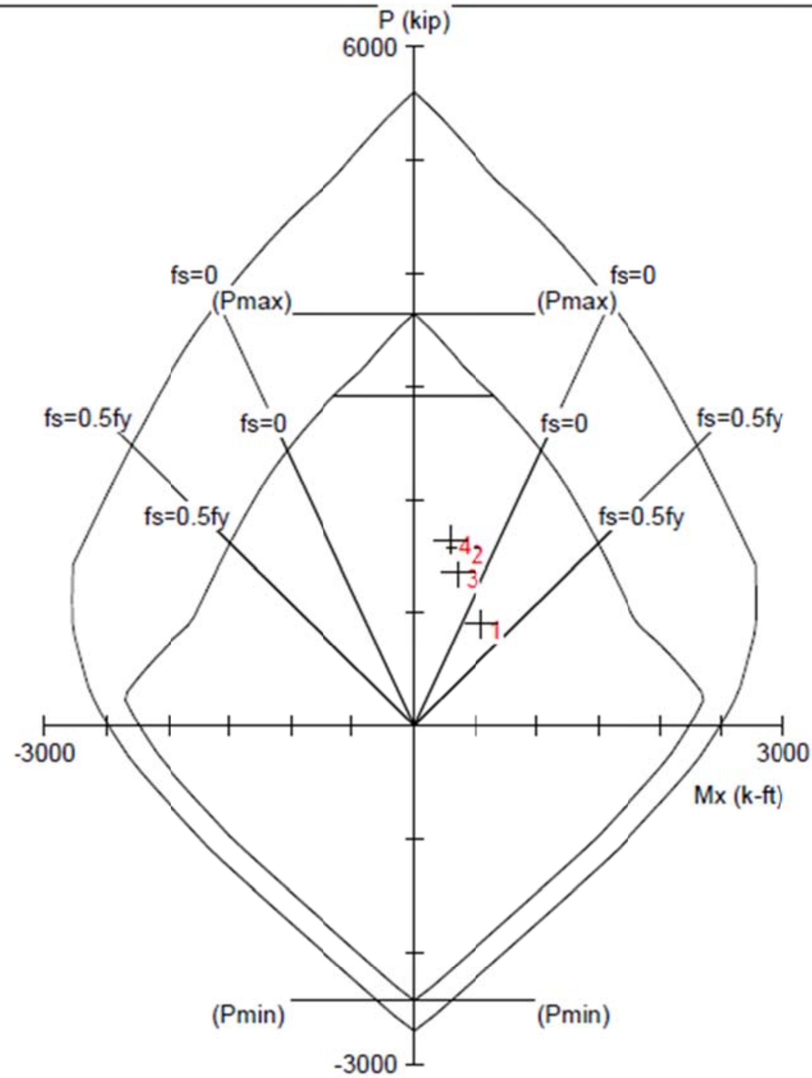
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 04/03/12

Time: 03:11:52



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File: Y:\Kostick\SpColumn\28x28 column - P-C.col

Project: AE Thesis

Column: Bott. Col.		Engineer: MK	
f'c = 4 ksi	fy = 60 ksi	Ag = 900 in^2	20 #14 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 45.00 in^2	rho = 5.00%
fc = 3.4 ksi		Xo = 0.00 in	Ix = 67500 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 67500 in^4
Beta1 = 0.85		Min clear spacing = 3.17 in	Clear cover = 2.00 in
Confinement: Tied			

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Appendix F: Computer Modeling Parameters, Foundation Analysis, & Cost Analysis Data

ETABS LATERAL MODEL

MATERIALS:
CONC 4 : MASS = 0
f'_c = 4 ksi
E_c = 3600 ksi
REINF = 60 ksi

FRAME SECTIONS:
COLUMNS : 24" x 24" (CONC 4)
$I_c = .70 I_g$ [ACI 318-08 SEC 10.10.4.1] - CRACKED

BEAMS : 24" x 22" (CONC 4) - STRONG AXIS BENDING (h = 24")
INSERTION POINT : 8 (TOP-CENTER)
$I_B = .35 I_g$ [ACI 318-08 SEC 10.10.4.1] - CRACKED

DIAPHRAM : CONCRETE SLAB - 9" THICK [IGNORE DEPTH]
- RIGID DIAPHRAGM w/ FLOOR MASS LUMPED @ C.O.M.

BASE JOINT RESTRAINT : FIXED
- ALL 6 DOF'S RESTRAINED

PANZEL ZONE / RIGID END OFFSETS
RIGID END OFFSET = .5
- COLUMNS
- BEAMS

MODE 1 : $T_y = 1.83_s$
MODE 2 : $T_x = 1.64_s$
MODE 3 : $T_z = 1.36_s$

EVALUATION OF FOUNDATION

FOUNDATION CONSISTS OF GRADE BEAMS ATOP DRILLED CAISSONS. CAISSONS ARE DRILLED 24" INTO DOLOSTONE BEDROCK

FROM GEOTECHNICAL REPORT:

- ALLOWABLE END BEARING PRESSURE = 40 ksf
- ALLOWABLE SKIN FRICTION = 10 ksf

ULTIMATE LOAD BEARING CAPACITY, q_u

$$q_u = 40 + 10 = 50 \text{ ksf}$$

FROM STRUCTURAL PLANS & GEOTECHNICAL REPORT
-TYPICAL PIER DIAMETER = 48" = 4'

CAPACITY OF SINGLE CAISSON

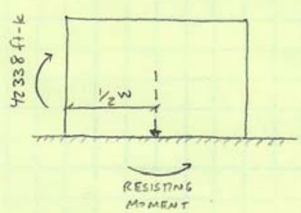
$$Q_u = \frac{\pi (4)^2}{4} \times 50 = 628 \text{ KIPS}$$

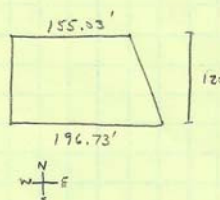
WEIGHT OF REDESIGNED STRUCTURE = 19759 KIPS

$$\#\text{ OF CAISSONS} = \frac{19759}{628} = 31.46$$

USE 36 CAISSONS PLACED AT MAJOR GRID LINES (SEE DIAGRAM)
↳ 48" DIAMETER

CHECK OVERTURNING MOMENT





$$E\text{-}W_{DIR} = \frac{155.03 + 196.73}{2} = 175.88'$$

$$N\text{-}S_{DIR} = 120'$$

BLDG WT = 19759 KIPS

$$\text{RESISTING MOMENT} = \text{BLDG WT} \times \tfrac{1}{2} \begin{bmatrix} 175.88' \\ 120' \end{bmatrix}$$

$$= 19759 \times \tfrac{1}{2}(120)$$
$$= 1185540$$

$$\tfrac{2}{3}(1185540) > 42338$$
$$790360 \text{ k-ft} \gg 42338 \text{ k-ft} \quad \text{OK}✓$$

Steel				
Quantity	Unit	Description	Unit Cost	Total
120	L.F.	w16x26 (4)	39.12	4694.40
60	L.F.	w24x68 (2)	95.15	5709.00
56	L.F.	w12x120 (4)	163.16	9136.96
9.72	C.Y.	Structural Concrete -Ready Mix (4000psi) L.W.C.	94.04	914.07
9.72	C.Y.	Structural Concrete Placing < 6"	22.78	221.42
900	S.F.	Metal Deck 3" 20 gauge	2.67	2403.00
9	C.S.F.	WWF, 6" x 6" - ASTM A185	45.48	409.32
72	Ea.	Bolts - A325 3/4"	9.02	649.44
176	Ea.	Shear Studs - 5" x 3/4"	2.1	369.60
900	S.F.	Concrete Finishing	0.73	657.00
			Total	25164.21
			Building Total	3033685.36

Concrete				
Quantity	Unit	Description	Unit Cost	Total
49.6	C.Y.	Structural Concrete (4000psi) N.W.C.	94.04	4664.38
25	C.Y.	Concrete Placing - Slab 6"-10" Thick	19.95	498.75
8.3	C.Y.	Concrete Placing - Column 24" Thick	34.45	285.94
16.3	C.Y.	Concrete Placing - Beam 24" Thick	35.58	579.95
448	S.F.C.A.	C.I.P. Column Forms - 24" square - 4 uses	5.81	2602.88
700	S.F.C.A.	C.I.P. Beam Forms - 24" deep - 4 uses	5.25	3675.00
900	S.F.C.A.	C.I.P. Elevated Slab Forms - 15'-20' - 4 uses	4.7	4230.00
0.901	Ton	Reinforcing steel in slabs - #4 - #7 -GR 60	1547.85	1394.61
0.93	Ton	Reinforcing steel in beams - #3 - #7 - GR 60	1899.24	1766.29
1.67	Ton	Reinforcing steel in beams - #8 - #18 - GR 60	1517.24	2533.79
0.26	Ton	Reinforcing steel in columns - #3 - #7 - GR 60	1966.09	511.18
2.38	Ton	Reinforcing steel in columns - #8 - #18 - GR 60	1617.52	3849.70
900	S.F	Concrete Finshing	0.73	657.00
			Total	27249.48
			Building Total	3449330.12

Appendix G: Curtain Wall Design Calculations

FACADE: ELEVATION (PARTIAL)

TRANSOMS = 4'-11" = 59" → PLACED @ 14' o.c. VERT.

MULLIONS = 14' LONG → ASSUME 2-SPAN CONTINUOUS FOR DESIGN.

- MIN THICKNESS = .125"
- DIMENSIONS = 7 1/8 × 3 3/8" [OUT-TO-OUT]

MATERIAL = 6063 T5 ALUMINUM. [MULLIONS]

CURRENT GLAZING = 4-SIDE S.S.G. IGU

↳ TOTAL THICKNESS = 1"

AIR SPACE = 1/2"

OUTER LITE = 1/4" H.S.

INNER LITE = 1/4" AN

CLEARANCE TO FRAME = 1/8"

DESIGN GLAZING FOR IMPACT, WIND, BLAST, & SEISMIC ; SIZE MULLIONS ; PERFORM HEAT TRANSFER

NEW GLAZING WILL BE IGU w/ LAMINATED OUTER LITE & SINGLE PLY INNER LITE.

IMPACT – BASED ON 8/1000 PROBABILITY OF INNER PLY BREAKAGE FROM "SMALL MISILE" (2g STEEL BALL @ 130 ft/s)

★ USE SACRIFICIAL PLY DESIGN CHART → 2g STEEL BALL @ 130 ft/s w/ PVB INTERLAYER = .06"

↳ USE ANNEALED (AN) → SAME THICKNESS FOR INNER/OUTER PLIES.

↳ CHOOSE 5/32" EACH PLY

↳ TOTAL THICKNESS = 5/16"

WIND – USING PRESSURES FROM WIND CALCS → SIZE GLAZING BASED ON ASTM E 1300 STANDARD

$p = 40.97$ psf → MAX PRESSURE

DESIGN IGNORING LOAD SHARING OF LITES. (CONSERVATIVE)

OUTER LITE → ANNEALED → SHORT TERM G.T.F. = 1.0

INNER LITE → FULLY TEMPERED (FT) → SHORT TERM G.T.F. = 4.0

DESIGN EACH LITE FOR FULL LOAD (CONSERVATIVE)

★ 1kPa = 20.9 psf

- PLATE DIMENSIONS = 168" × 59"

OUTER LITE → $LR = 40.97\text{ psf} = N.F.L. \times \underset{(GTF)}{1.0}$

$NFL \geq 40.97$ psf

[$NFL \geq 1.96$ kPa]

INNER LITE → $LR = 40.97\text{ psf} = NFL \times \underset{(GTF)}{4.0}$

$NFL \geq 10.24$ psf

[$NFL \geq .49$ kPa]

SIZE GLAZING FROM ASTM E1300.

WIND - CONT.

OUTER LITE → 2 PLIES → TOTAL THICKNESS = 1/2" (1/4" EACH PLY) → AN BOTH PLIES
↳ 2.25 kPa > 1.96 kPa OK✓

LR = (2.25 × 20.9)(1.0) = 47 psf ≥ 40.17 psf OK✓

INNER LITE → SINGLE PLY → THICKNESS = 1/4" → FT
↳ .70 kPa > .49 kPa OK✓

LR: (.70 × 20.9)(4.0) = 58.52 psf ≥ 40.97 psf OK✓

BLAST → BASED ON 70lb CHARGE @ 50' STANDOFF.
STANDOFF CHOSEN FROM SITE REDESIGN W/ PLAZA.

* GLAZING MUST BE 1/4" AT MINIMUM → ASTM F2248.
- USE ASTM F2248 TO CONVERT CHARGE TO 3s PRESSURE
- SIZE GLAZING W/ ASTM E1300.

EQUIVALENT 3-s PRESSURE OF 70lb CHARGE @ 50':
$P_{3}d = 112$ psf = 5.35 kPa

SIZE GLAZING W/ ASTM E1300
OUTER LITE → ANNEALED → GTF = 1.0
↳ 2 PLY → TOTAL THICKNESS = 3/4" → [3/8" EACH PLY]
↳ 5.5 kPa ≥ 5.35 kPa OK✓

$LR = (5.5 \times 20.9) \times \overset{(G.T.F)}{1.0} = 115$ psf > 112 psf OK✓

INNER LITE → FULLY TEMPERED → GTF = 4.0
↳ SINGLE PLY → $NFL = \frac{112}{4.0} = 28$ psf = 1.34 kPa

→ THICKNESS = 1/2" - [3/8" WAS ≈ 1.25 kPa
↳ 2.2 kPa > 1.34 kPa OK✓

$LR = (2.2 \times 20.9) \times \overset{(GTF)}{4.0} = 183.9$ psf > 112 psf OK✓

SUMMARY

USE IGU: OUTER LITE → LAMINATED ANNEALED:
3/4" THICK → (3/8" EACH PLY) → BLAST CONTROLLED.
INNER LITE → FULLY TEMPERED → SINGLE PLY.
1/2" THICK → BLAST CONTROLLED.

DESIGN GLAZING POCKET FOR SEISMIC MOVEMENT

PANEL DIMENSIONS = 168" × 59" $I = 1.50$ [ASCE 7-10] – TABLE 1.5-2

STORY DRIFT → 1.57" [FROM SEISMIC CALCS]

$$\Delta_{FALLOUT} \geq 1.25 \times I \times D_p$$
$$= 1.25\,(1.5)\,(1.57)$$
$$= 2.94"$$

$$D_{CLEAR} = 2c_1\left(1 + \frac{h_p c_2}{b_p c_1}\right) \qquad c_1 = c_2 \rightarrow \text{SAME CLEARANCE ALL AROUND.}$$

$$2.94 = 2c\left(1 + \frac{168(c)}{59(c)}\right)$$

$c = .38"$ → USE $\underline{c = 7/16"}$ ALL AROUND TO PREVENT CONTACT OF GLASS AND FRAME.

DESIGN OF MULLIONS FOR BLAST LOADING (CONTROLLING LOAD)

T5 Al → $7\tfrac{1}{8} \times 3\tfrac{3}{8} \times t$

→ 2 SPAN CONTINUOUS = 28'

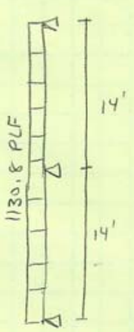
TRIB WIDTH = 59"

CONTROLLING LOAD → BLAST = 2× LR = 2×115 psf = 230 psf

$w = 230 \times \frac{59}{12} = 1130.8$ PLF → CAN ASSUME DISTRIBUTED LOAD B/C DIFFERENCE IN LOADING SCENARIOS < 4%

USE "WILSON HANDOUT" (AE542) → w/ 2 EQ SPANS DIST. [TABLE 3-2]

MOMENTS:
- MAX POS. = $\alpha_m \times q \times L^2 = .0703\,(1130.8)(14)^2 = 15.9$ k-ft
- MAX NEG = $-.125 \times 1130.8 \times 14^2 = -27.7$ k-ft ← CONTROLS
- MAX SHEAR = $.625 \times 1130.8 \times 14 = 9.9$ k



- MAX DEFLECTION : $\frac{\alpha_m q L^4}{100EI} = \frac{.521\,(1130.8)(14)^4}{100\,EI} = \frac{226327}{EI}$ (lb-ft³)

ALLOWABLE STRESSES → T5 ALUMINUM.

$\sigma_{AXIAL} = 62000$ kPa = 1296 ksf = 9 ksi

$\sigma_{BENDING} = 69000$ kPa = 1442 ksf = 10 ksi

$\sigma_{SHEAR} = 37000$ kPa = 773 ksf = 5.4 ksi

$\sigma_{BEARING} = 117000$ kPa = 2445 ksf = 17 ksi

MAX DEFLECTION = L/180 OR 20mm (.787")

BENDING:

$$10 = \frac{(27.7 \times 12)\left(\frac{7.125}{2}\right)}{I_{REQ}} \qquad I_{REQ} \geq 118.4\ \text{in}^4$$

$$I_{REQ} = \frac{1}{12}(3.375)(7.125)^3 - \frac{1}{12}(3.375 - 2t)(7.125 - 2t)^3 = 118.4\ \text{in}^4$$

NO GOOD

↓ ↓

INCREASE MULLION SIZE TO 3.5" × 8" DEEP

MULLION DESIGN CONT.

$$10 = \frac{(27.7 \times 12)\frac{8.0}{2}}{I_{REQ}} \rightarrow I_{REQ} > 132.96 \text{ in}^4$$

$$I_{REQ} = \frac{1}{12}(3.5)(8)^3 - \frac{1}{12}(3.5-2t)(8-2t)^3 = 132.96 \text{ in}^4$$

$$t \geq 1.2''$$

SHEAR

$$5.4 = 1.5\, V/A = 1.5\,(9.9/A_{REQ})$$

$$A_{REQ} = 2.75 \text{ in}^2 \rightarrow 2.75 = 8(3.5) - [(3.5-2t)(8-2t)]$$

$$t \geq .122''$$

DEFLECTION

$$\frac{168}{180} = .93 \rightarrow .787'' \text{ CONTROLS} \qquad E = 70000\ N/mm^2 = 10160 \text{ ksi}$$

$$f_{MAX} = \frac{(226327 \times 1728)/1000}{10160\left(\frac{1}{12}(3.5)(8)^3 - \frac{1}{12}(3.5-2(1.2))(8-2(1.2))^3\right)}$$

$$= .281'' < .787'' \quad \underline{\text{OK}} \checkmark$$

BEARING - MAX REACTION IS @ MIDDLE SUPPORT.

MIDDLE SUPPORT REACTION $= (.625+.625)qL$

$= 1.25(1130.8)(14) = 19.8$ kips

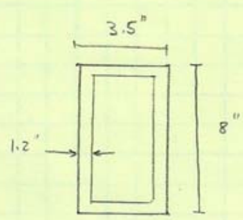
USING 2 BOLTS → 9.9^k EACH BOLT

REQ DIAMETER (D)

$$7 \text{ ksi} = \frac{9.9}{2(1.2)(D)} \rightarrow D \geq .242 \text{ in}$$

2 BEARING SURFACES

USE $\frac{1}{2}''\ \phi$ BOLTS.



USE (2) $\frac{1}{2}''\ \phi$ BOLTS FOR CONNECTIONS.

HEAT TRANSFER

ENERGY FLOW THROUGH FENESTRATION

$$Q = UA_{pf}(t_{out} - t_{in}) + (SHGC)A_{pf}E_t$$

$A_{pf} = (72' + 3') \times 145' = 10875\ ft^2$

$t_{out} = 82°F$ - SUMMER AVERAGE

$= 16°F$ - WINTER AVERAGE

$t_{in} = 68°F$

U, SHGC } MATERIAL DEPENDANT

$E_t = \underbrace{E_{DN}\cos\theta + E_d + E_R}_{\text{CALCULATE FOR WINTER \& SUMMER CONDITIONS.}}$

E_t CALCULATIONS

SUMMER - 1:00 pm (EST); JUNE 21ST, SYRACUSE, NY → [43° 02' 49" N, 76° 08' 39" W]

(75° W) LSM; [43.05°, 76.14°] LAT, LON

STANDARD TIME = 12:00 pm (ADJUST FOR DAYLIGHT SAVINGS)

LST = 12.0

ET = -1.4 MIN (JUNE) - ASHRAE TABLE 7

$$AST = LST + \frac{ET}{60} + \frac{(LSM - LON)}{15}$$

$$= 12.0 + \frac{-1.4}{60} + \frac{(75 - 76.14)}{15} = 11.901$$

$H = 15(AST - 12)$

$= 15(11.901 - 12) = -1.49°$

$\delta = 23.45°$ - (JUNE) - TABLE 7 ASHRAE

$\sin\beta = \cos(LAT)\cos(\delta)\cos(H) + \sin(LAT)\sin(\delta)$

$\sin\beta = \cos(43.05)\cos(23.45)\cos(-1.49) + \sin(43.05)\sin(23.45) = .942$

$\beta = 70.36°$

$$\cos\phi = \frac{\sin(\beta)\sin(LAT) - \sin(\delta)}{\cos(\beta)\cos(LAT)} = \frac{\sin(70.36)\sin(43.05) - \sin(23.45)}{\cos(70.36)\cos(43.05)} = .997$$

$\phi = 4.14$

GLAZING FACE IS 20° N OF E.

$\psi = -110°$ - TABLE 9 ASHRAE

$\gamma = \phi - \psi$

$= 4.14 - (-110) = 114.14° > 90$ ∴ SURFACE IS IN SHADE.

$\cos\theta = \cos\beta\cos\gamma$

$= \cos(70.36)\cos(114.14) = -.1375$

$\theta = 97.9°$

Find E_{DN}, E_d, E_r ; E_t

$A = 346 \text{ BTU}/hr \cdot ft^2$ $B = .185$ - TABLE 7

$$E_{DN} = \frac{A}{Exp\left(\frac{B}{\sin\beta}\right)} = \frac{346}{Exp\left(\frac{.185}{\sin(70.36)}\right)} = 284.3 \text{ BTU}/h \cdot ft^2$$

• $E_{DN} \cos\theta = 284.3 \cos(97.9) = -39.07 \text{ BTU}/h \cdot ft^2 < 0$ ∴ DOES NOT AFFECT E_t

$C = .137$ - TABLE 7

$Y = .55 + .437 \cos\theta + .313 \cos^2\theta$

$= .55 + .437 \cos(97.9) + .313(\cos(97.9))^2 = .496$

• $E_d = CYE_{DN}$

$= (.137)(.496)(284.3) = 19.32 \text{ BTU}/h \cdot ft^2$

• $E_r = E_{DN}\,(C + \sin\beta)\rho_g \,\frac{1 - \cos\Sigma}{2}$ → ASSUME .34 (NEW CONCRETE) $= \rho_g$

$= 284.3(.137 + \sin(70.36))(.34)\left(\frac{1-0}{2}\right)$

$= 52.14 \text{ BTU}/h \cdot ft^2$

• $E_t = 19.32 + 52.14 = \underline{\underline{71.46}} \text{ BTU}/hr \cdot ft^2$

WINTER - 12:00pm ; DECEMBER 21ST , SYRACUSE N.Y. → [43° 02' 47" N, 76° 08' 31" W]

EST (75° W) LSM [43.05° N, 76.14° W] LAT , LON

STANDARD TIME = 12:00 PM - DAYLIGHT SAVINGS ADJUSTED

LST = 12.0

ET = 1.6 MIN (DEC) - ASHRAE TABLE 7

$$AST = 12 + \frac{1.6}{60} + \frac{(75 - 76.14)}{15} = 11.95$$

$H = 15(11.95 - 12) = -.739°$

$\delta = -23.45°$ (DEC) - ASHRAE TABLE 7

$\sin\beta = \cos(43.05)\cos(-23.45)\cos(-.739) + \sin(43.05)\sin(-23.45) = .399$

$\beta = 23.49°$

$$\cos\phi = \frac{\sin(23.49)\sin(43.05) - \sin(-23.45)}{\cos(23.49)\cos(43.05)} = .9998$$

$\phi = 1.25°$

For 20° N of E

$\psi = -110°$

$\gamma = 1.25 - (-110) = 111.25° > 90 \rightarrow$ SURFACE IS IN SHADE

$\cos\theta = \cos(23.49)\cos(11.25) = -.332$

$\theta = 109.4°$

E_{ON}, E_d, E_r; E_t

$A = 382\ BTU/h \cdot ft^2 \qquad B = .141$

$$E_{ON} = \frac{382}{\exp\left(\frac{.141}{\sin(23.49)}\right)} = 268.3\ BTU/h \cdot ft^2$$

- $E_{ON} \cos\theta = 268.3 \cos(109.4) = -89.11\ BTU/h \cdot ft^2 < 0$ ∴ DOES NOT AFFECT E_t

$C = .103$

$Y = .55 + .437(-.322) + .313(-.322)^2 = .439$

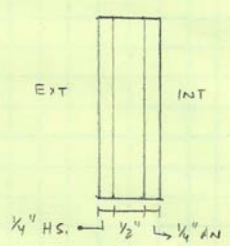
- $E_d = (.103)(.439)(268.3) = 12.13\ BTU/h \cdot ft^2$
- $E_r = (268.3)(.103 + \sin(23.49))(.34)(\frac{1}{2}) = 22.88\ BTU/h \cdot ft^2$
- $E_t = 12.13 + 22.88 = \underline{\underline{35.01}}\ BTU/h \cdot ft^2$

CURRENT GLAZING → IGU → 1" TOTAL

1/4" H.S. / 1/2" AIR SPACE / 1/4" AN.

U VALUE = .47 WINTER, = .50 SUMMER $\Big\}$ $BTU/h \cdot ft^2 \cdot °F$

SHGC = .7



$Q = UA_{pf}(T_{out} - T_{in}) + (SHGC)A_{pf}E_t$

$Q = (.47)(10875)(16 - 68) + (.7)(10875)(35.01)$

$Q = 728.63\ BTU/h$ [WINTER] [GAIN]

$Q = (.50)(10875)(82 - 68) + (.7)(10875)(71.46)$

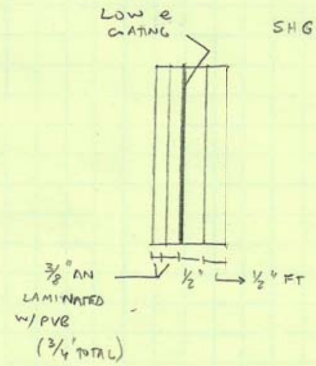
$Q = 620114\ BTU/h$ [SUMMER] [GAIN]

PROPOSED GLAZING ALTERNATIVE → IGU → 1.75" TOTAL

3/4" LAMINATED ANNEALED / LOW e COATING / 1/2" AIR SPACE / 1/2" FULLY TEMPERED

U-VALUE = .26 WINTER
= .28 SUMMER } BTU/h·ft²·°F

SHGC = .32



$Q = UA_{pf}(T_{OUT} - T_{IN}) + (SHGC)A_{pf}E_t$

$Q = .28(10875)(46-68) + (.32)(10875)(35.01)$

$\dot{Q} = -36505$ BTU/HR [WINTER] [LOSS]

$Q = .26(10875)(82-68) + (.32)(10875)(71.46)$

$\dot{Q} = 288266$ BTU/HR [SUMMER] [GAIN]

COMPARE

NEW = 1/2 ORIGINAL [SUMMER]

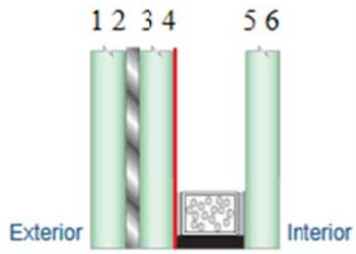
NEW → LOSES HEAT IN WINTER

OLD → GAINS HEAT IN WINTER

SuperNeutral 62 (#4)

SuperNeutral

Outboard Substrate:	Lami Glass (UltraWhite / UltraWhite)
Inboard Substrate:	Clear
Exterior Appearance:	Ultra Clear
Transmission	
Visible Light %:	62
UV %:	0
Solar Energy %:	27
Reflectivity	
Visible Light Out %:	11
Visible Light In %:	13
Solar Energy %:	29
U-Value	
Winter Nighttime Argon:	0.00
Winter Nighttime Air:	0.28
Summer Daytime Air:	0.26
Relative Heat Gain:	76
Shading Coefficient:	0.36
Solar Heat Gain Coefficient:	0.32
Light-to-Solar Gain:	1.96



THERMAL STRESS GUIDELINES

Outboard Lite:

Inboard Lite:

Key:

- This SunGuard lite must be tempered or heat-strengthened
- Exercise caution when using annealed SunGuard products, heat-strengthening or tempering may be required
- Go with annealed SunGuard products

NOTE: The thermal stress guideline is only a rough guide to the thermal safety of a glazing. Other factors such as large glass areas, shapes and patterns, thick glass, glass damaged during shipping, handling or installation, orientation of the building, exterior shading, overhangs/fins that reduce wind speed, and areas with high daily temperature fluctuations can all increase the probability of thermal breakage. The results shown are not for any specific glazing installation and do not constitute a warranty against glass breakage.