FINAL REPORT – AE SENIOR THESIS

ORCHARD PLAZA

SOUTHWEST PENNSYLVANIA

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Advisor: Dr. Thomas Boothby

April 9th, 2014

ABSTRACT

ORCHARD PLAZA

Architectural Engineering Senior Thesis

GENERAL INFORMATION

Occupancy: Retail & Office
Competed: December 2006
Size: 144,000 SF
Levels: 6
Cost: $18,500,000
Delivery Method: Design-Bid-Build

PRIMARY PROJECT TEAM

Owner: Millcraft Investments
Architect: STRADA
MEP: Allen & Shariff Corporation
Structural Engineer: Barber & Hoffman, Inc.
Civil Engineer: GAI Consultants Inc.

STRUCTURE

Deck: 2"18 Gage (Composite deck)
Floor System: 4" Reinforced Concrete
Lateral System: Eccentrically Braced Frames
Framing System: W-shape columns, girders, beams
Foundation: Caissons & Grade Beams

ARCHITECTURE

Façade: Brick & Limestone veneer
Glazing: Individual windows & Curtain walls

MEP SYSTEMS

Lighting: Fluorescent, Halogen, & Incandescent lamps
HVAC System: Variable Air Volume with Rooftop Units

CHRISTOPHER DUARTE - STRUCTURAL

http://www.engr.psu.edu/ae/thesis/portfolio/2014/cjd5213

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EXECUTIVE SUMMARY

This thesis report presents and investigates three structural redesign concepts for the Orchard Plaza office building. First, the initial gravity structure of composite beams and girders satisfactorily supports the prescribed design loads, but prohibits integration between the floor structure and HVAC systems. An open web joist system will be used in attempt to integrate these systems. Cross sections of each system will be taken to compare the integration of existing HVAC ducts.

Second, the existing eccentric frames nearly perform like moment frames, but require eleven full building height bays of frames in order to control the building's drift limitations. A concentrically braced frame system will be modeled and compared conceptually to a model of the existing lateral system. Since the façade's architecture drove the decision to use the original eccentric frames, allowing for windows to be placed within each bay, architectural implications of using concentric frames will be analyzed and potential solutions will be presented.

Third, two auxiliary subjects will be explored regarding Orchard Plaza. The building will be modeled and tested under summer and winter daylighting conditions, as the current building provides little acknowledgement to solar concerns. Once the buildings shadow patterns are found, solutions to the various solar scenarios will be investigated and elaborated upon. A green roof will also be considered for the western half of Orchard Plaza's roof. Implications regarding system type, loading considerations, cost and maintenance will be presented along with the benefits of implementing a green roof system.

ACKNOWLEDGEMENTS

I would like to acknowdlege the following people for their contributions in aiding in the initiation and completion of this report.

MILLCRAFT INVESTMENTS

Thank you to Chad Wheatley of Millcraft Investments for your guidance in selecting the appropriate structure for my analysis. Your timely assistance in acquiring the approval and drawings needed for this report is greatly appreciated.

THE PENNSYLVANIA STATE UNIVERSITY

Thank you Dr. Thomas Boothby and Professor Kevin Parfitt for your consultation throughout the selection of my building and analysis throughout my entire senior year.

The enitre Acrchitectural Engineering Department has provided me with an unmatched education that has well prepared me to be a professional practicing engineer

FAMILY & FRIENDS

The unwavering support from my family and friends has been pivotal in my success in the rigorous Architectural Engineering curriculum. With their help, I have overcome the many set-backs I have had through my five years at Penn State.

INTRODUCTION

Orcard Plaza is a six-story office structure situated in an urban environment in Southwest Pennsylvania. The building was completed in December of 2006 and resides on the corner of a city block one road away from the town's main thoroughfare. An existing public parking garage adjacent to Orchard Plaza serves the parking needs of the office building.

Completed at a cost of $18.5 million, Orchard Plaza totals 144,000 square feet of leaseable space. The façade is adorned with a limestone, brick and metal panels and is complimented with green glass windows and curtain wall. The architect's goal for this structure was to maximize the openness of the floor plan while simultaneously incorporating ample natural daylighting.

Given it's location in the northeastern United States, a steel structure was determined to be most economical and best satisfy the desire for an open floorplan and continuous spread of exterior windows. The building has no concrete or masonry shear walls, allowing for maximized floor plan flexibility.

EXISTING STRUCTURAL OVERVIEW

FOUNDATION

The foundation for Orchard Plaza consists of a series of grade beams that rest on a total of forty-one caissons. Slabs on grade of varying thicknesses form the first floor with expansion joints at structural gridlines and column bases. Details of each foundation element can be seen below.

CAISSONS

Caissons ranging from thirty to seventy-six inches in diameter secure the columns to the soil. The caisson notes specify that the caisson depth must extend a minimum of one foot into limestone bedrock. Longitudinal rebar extends a minimum of ten feet below the top of each caisson.

Caisson caps serve as column base plate bolt anchors. Their height varies per column. Details of caissons and caisson caps can be seen in Figures 1 and 2 respectively.

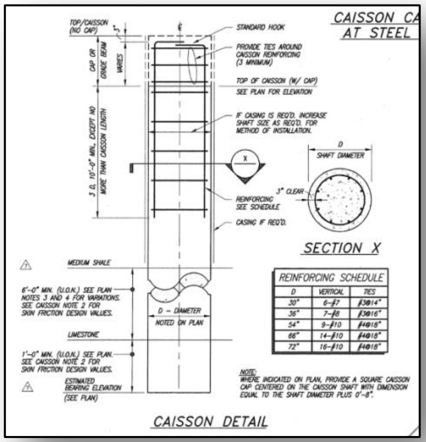


Figure 1: Caisson Detail – S0.00

Courtesy of STRADA

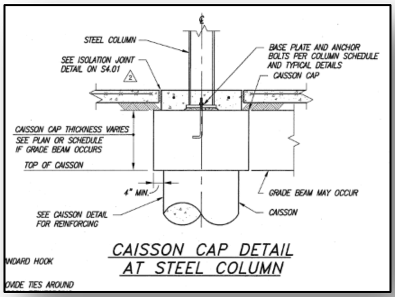


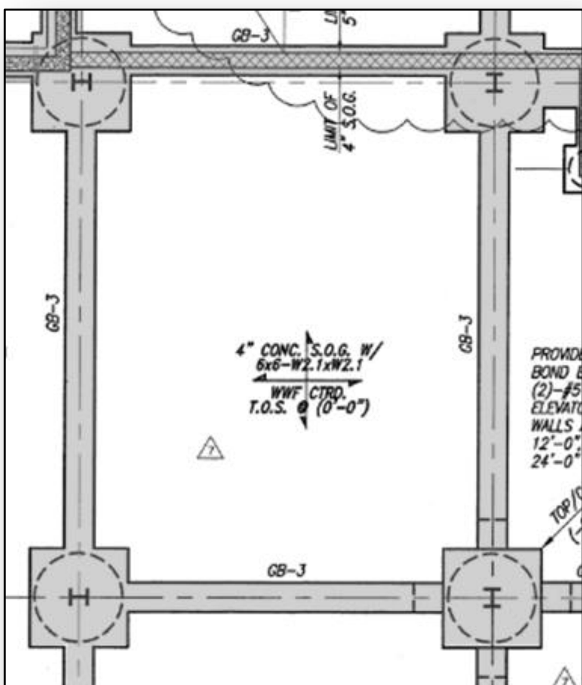
Figure 2: Caisson Cap Detail – S0.00

Courtesy of STRADA

GRADE BEAMS & SLABS ON GRADE

Grade beams of widths varying from eighteen to thirty-two inches and depths up to three feet provide a grid of foundation between most columns. Slabs on grade with expansion joints between grade beams and slabs on grade and between adjacent slabs compose the first floor (ground floor) of the building. Figure 3 shows the interaction of the grade beams with the caisson caps/column bases and the slabs on grade.

Figure 4 shows an example of the relationship of expansion joints to the slabs on grade, grade beams, and column bases.



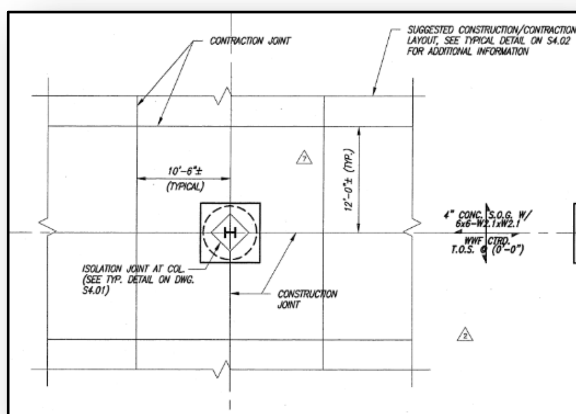


Figure 3: Grade Beams – S1.00

Courtesy of STRADA

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Figure 4: Expansion Joints – S1.00

Courtesy of STRADA

FLOOR FRAMING & TYPICAL BAYS

Typical floor framing consists of beams and girder construction of varying sizes. Figure 7 shows a typical beam and girder layout for the first floor. Floors two through six follow a very similar design. Beams range in size from W16x31 to W21x44 while girders vary from W24x68 to W 30x99 with exceptions for both beams and girders surrounding floor openings.

Figure 7: First Floor Framing Plan – S1.01

Courtesy of STRADA

35" x 42' Bay area coverage

35' x 28' Bay area coverage

FLOOR SYSTEM DETAILS

Floors two through five utilize a composite decking system comprised of normal weight concrete, two inch 18 gauge composite decking, and welded wire framing placed one inch from the top of the slab. Where exterior brick veneer requires support, deeper beams run the length of the exterior with 3/8” plate welded perpendicular to of the beam. A system of HSS tubing, shims, and angle form the brick veneer support while an angle brace runs up to the beam behind (Figure 8) or is joined directly with a double angle connection (Figure 9). Similar connections are done for masonry veneer facades on the lower floors. Some exterior edges also include small cantilevers.

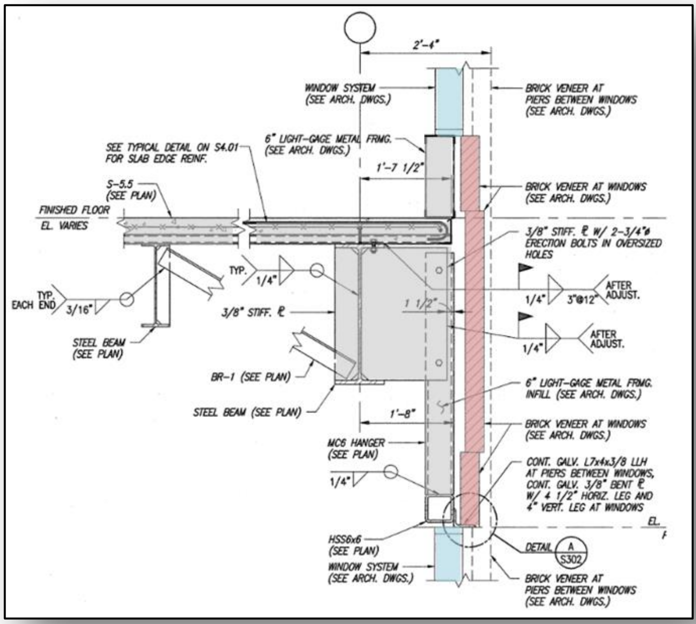


Figure 8:

Floor to Exterior connection with brace – S3.02

Courtesy of STRADA

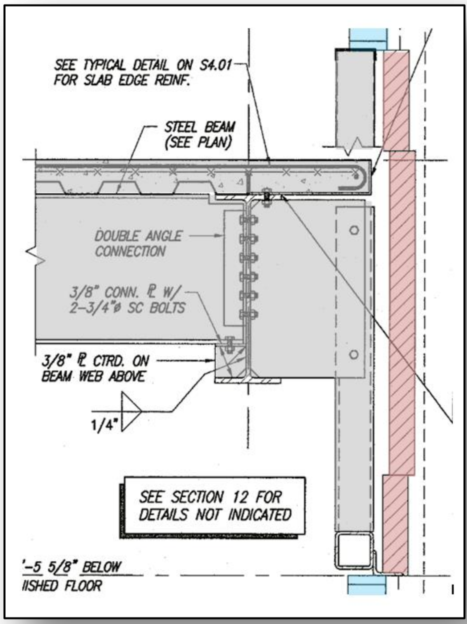


Figure 9:

Floor to Exterior connection– S3.02

Courtesy of STRADA

COLUMNS

All columns rest on caissons or grade beams as described earlier. Column base plates are typically mounted to caissons with four anchor bolts as shown in gray in Figure 10. Additional base plates and anchor bolts are added are added for any base joints with the lateral system (shown in blue in Figure 11).

Column splices occur four feet above the floor slab of the first, third, and fifth floor unless required to be at a different height to avoid brace connections. Base columns range from W14x99 on the exterior to W14x257 on the interior. See Appendix A for column schedule.

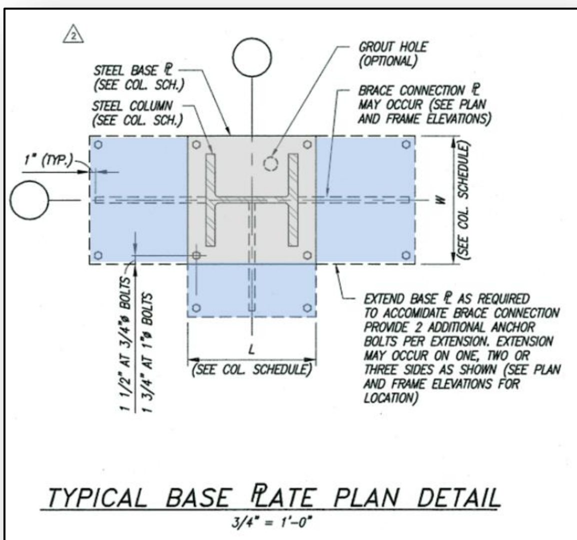


Figure 10:

Typical Base Place Elevation – S2.02

Courtesy of STRADA

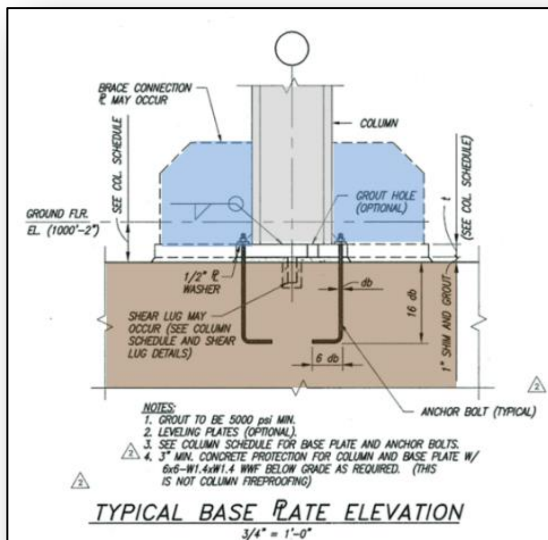


Figure 11:

Typical Base Place Plan Detail – S2.02

Courtesy of STRADA

LATERAL SYSTEM

The primary lateral load resisting elements are eccentrically braced frames formed from W-shape beams and HSS tubing. The location of all moment framing elements is shown in blue in Figure 12 below. The orientation of these frames is distributed relatively evenly between the north-south and east-west direction to adequately accommodate lateral loading from all directions.

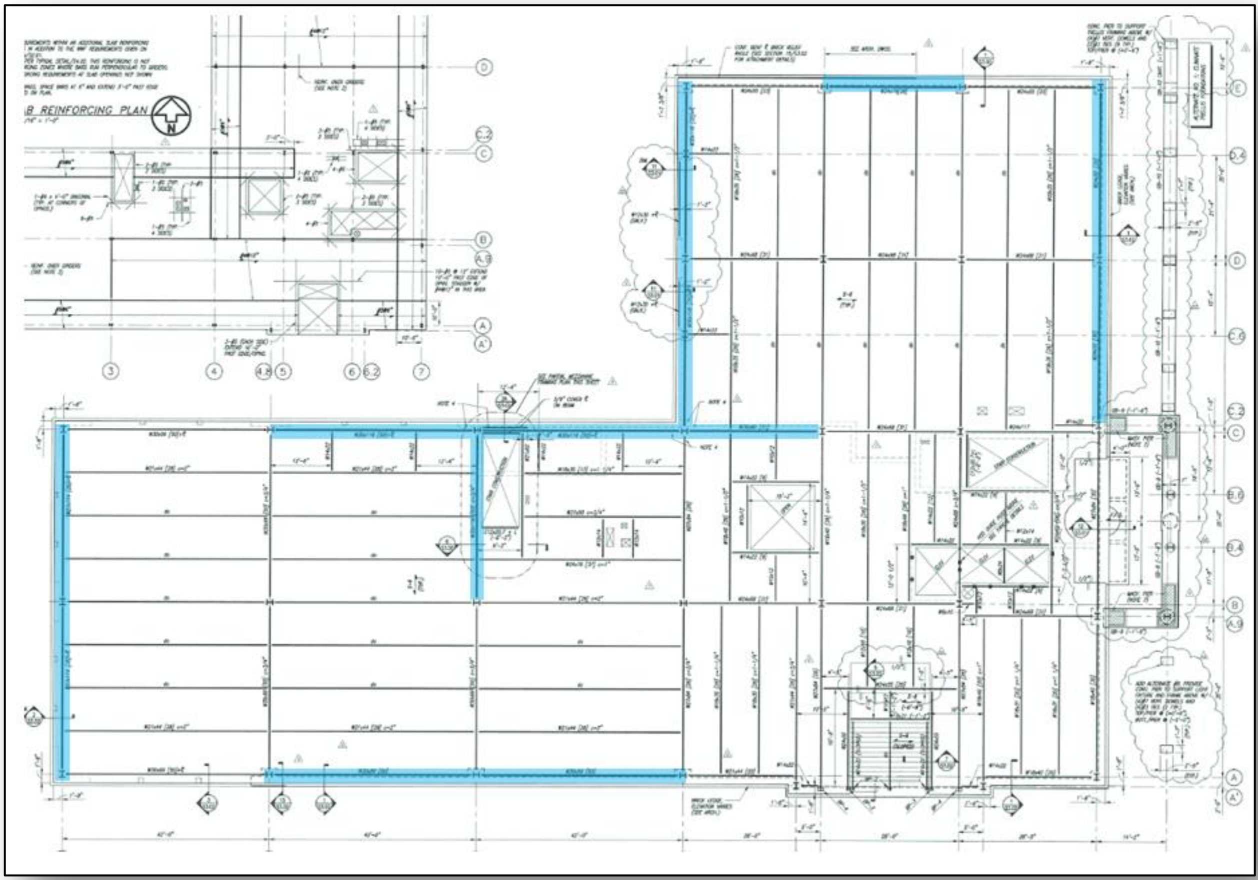


Figure 12: First Floor Framing – S1.01

Courtesy of STRADA

Pictured below is the left-most frame highlighted in Figure 12 on the previous page. All eccentric bracing is constructed using HSS tubing and ranges in wall thickness from ½" to ¼". This configuration is used to provide maximum flexibility with glazing placement on exterior frames and office floor space flexibility within interior frames. Orchard Plaza contains no concrete shear walls or concrete central core.

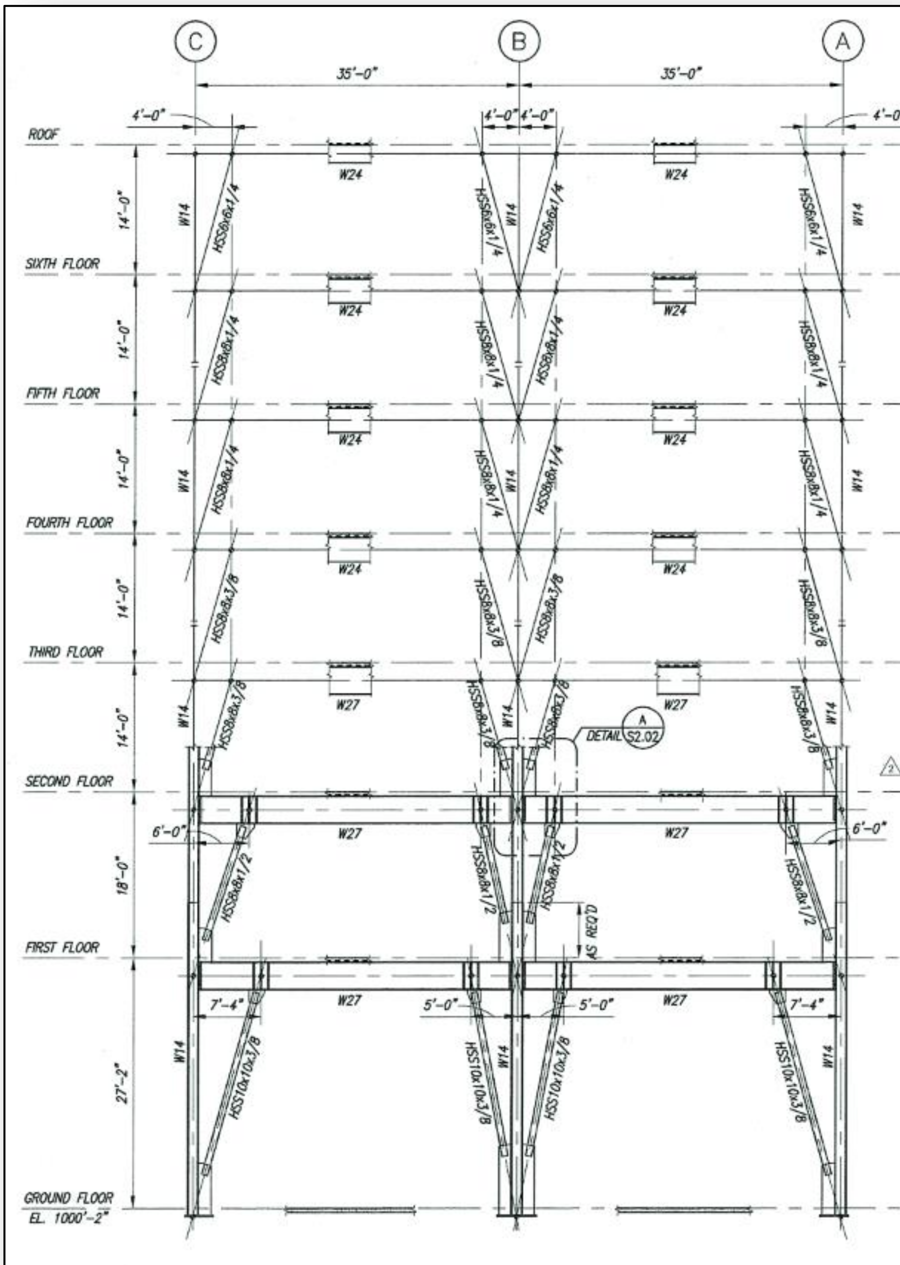


Figure 13: Line 1 Lateral Frame – S2.01

Courtesy of STRADA

Lateral frame connections are characterized by welded plates at both ends of the HSS tube, shown in purple, and are welded to columns and girders as seen in Figure 14 below. This connection requires a significant amount of prefabricated welding and field welding. Stiffener plates must also be added on both sides girder webs at the upper connection of the HSS tube and respective connection plate.

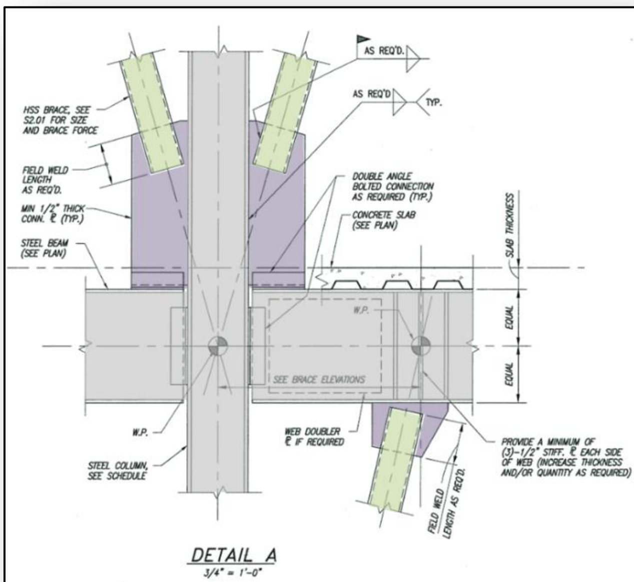


Figure 14: Interior Lateral Frame Joint– S2.02

Courtesy of STRADA

CODES & REFERENCES

Structural designs will follow criteria perscribed by the following standards and codes:

- AISC Steel Construction Manual 14th Edition
- ASCE 7-10
- International Building Code 2009
- AISC Design Guide 11 for Vibrations

Additional references include:

- Vulcraft Steel Joist and Decking Catalog
- AISC Design Examples
- Steel Joist Institute Catalog
- Reducing Urban Heat Islands: Compendium of Strategies – Green Roof
- Various imgaes cited within report

PROBLEM STATEMENT

After analyzing the gravity system, lateral system, and loading factors the structure of Orchard Plaza was found to be acceptable under the codes used for design. Since no critical structural improvements are necessary, the structure will be modified for aesthetic enhancement and system integration and analyzed to ensure practicality and cost effectiveness.

The existing floor system is comprised of composite beams and girders. This system is very viable for material cost control and strength. One downside of this system is that mechanical and electrical systems must pass below the solid floor gravity system. This leaves a void of unused space between the floor decking and bottom flange of beams and girders. An additional dropped acoustic ceiling tile grid is hung below all the mechanical systems.

As mentioned in the structural overview, Orchard Plaza's existing lateral system is comprised of six primary eccentrically braced frames. While structurally sufficient, the frames are not as easy to construct as their counterparts, concentrically braced frames.

To mitigate some of these construction and space utilization concerns, alterations have been made to the existing gravity and lateral systems. It is believed that concentrically braced frames will significantly increase the stiffness of the structure, allowing for a possible reduction in the total number of frames needed. Consequently, having fewer frames will reduce the cost of the lateral system.

PROPOSED SOLUTION

A solution to the integration concern between the floor gravity system and the mechanical and electrical components is to use an open-web joist system in place of the existing beams. Joists allow for ductwork and conduit to pass through the depth of the floor structure. This was done in order to maximize space utilization and integration between floors. This should allow for the acoustic ceiling grid to be hung slightly higher, providing a more open workspace.

The gravity system will be modeled using the two typical bay sizes of 42' x 35' and 35' x 28' to determine if additional girders to control the depth of the joists. With the intention of raising the acoustic ceiling grid, controlling the depth of the joists is critical. The column layout is planned to remain the same as the existing structure but column sizes and strength will be analyzed and modified as necessary. I will attempt to accommodate all existing HVAC duct sizes and layouts. Based on the orientation of the joists, necessary openings, spacing, and X-bracing will be provided. The integration of the new floor structure and existing HVAC system will be presented.

The lateral system was modified to concentrically braced frames to increase stiffness. This increased stiffness will be used to help convert some of the two bay braced frames into one bay frames in order to decrease material and construction costs. Individual bays will be analyzed using RISA to compare structural effectiveness and full height frames will be analyzed in RAM to determine overall drifts and deflections. The effect of these frames on the architecture will also be researched with possible solutions provided

BREADTH STUDIES

DAYLIGHTING

With the introduction of a new curtain wall system and a much higher amount of daylight entering the space, an analysis will be performed to determine the natural light paths throughout the year and proposed systems to mitigate excess sunlight when necessary. Factors such as glazing type and shading devices, both active and passive, will be discussed. Once all factors influencing natural lighting on Orchard Plaza are researched, an ideal system will be proposed as well as details for integrating relevant systems into the space.

GREEN ROOF

It is proposed that a green roof be added to the western side of the building. A vacant plot measuring 84' x 70' is currently unoccupied by any rooftop equipment and is an ideal location for a green roof. Structural concerns such as additional rooftop and column loading will be explored with needed accommodations presented. Logistics such as drainage, maintenance access, and fall protection will be researched and presented along with practical solutions.

BUILDING LOADS

DEAD LOADS

The dead loads used for Orchard Plaza were derived both from the structural drawings and from hand calculations. A new 2" non-composite deck was sized and the weight of the concrete slab system was deterimined from the Vulcraft catalog. The weight of the existing masonry façade was calculated and applied as a linear load around the perimiter of each floor of the structure.

Dead Loads	
Description	**Load**
Superimposed	12 psf
Exterior Walls	784 pfl
Floor Slab - Level 1	68 psf
Floor Slab - Levels 2-6	68 psf
Roof	30 psf

LIVE LOADS

Live loads shown below were determined using the design loads from the structural drawings and verified using ASCE 7-10. For this project, floor live loads were simplified to 100psf of Floor 1 and 80psf of Floors 2-6.

Live Loads	
Description	**Load (psf)**
Lobbies & Corridors	100
Office Areas	80
Main Corridors Above Ground Level	80
Electrical & Mechanical Rooms	200
Stairs & Landings	100
Light Storage	125
General File Areas	175
Heavy Storage	250
Roof Live Load	30

SNOW LOADS

Snow loads for Orachard Plaza were taken from the structural drawing's general notes. These values were verified using ASCE 7-10. A summary of the factor considered are listed in the table below.

Snow Loads	
Description	**Value**
Ground Snow Load P_g	25 psf
Flat-Roof Snow Load P_f	18 psf
Snow Exposure Factor C_e	1
Snow Importance Factor I_e	1
Thermal Factor	1
Wind Directionality Factor K_d	0.85

WIND LOADS

Wind loads for Orchard Plaza were analyzed in RAM in accordance with Chapter 26 and 27 of ASCE 7-10. Based on an Occupancy category of II, a basic wind speed of 90mph was used given the structure's location in Southwestern Pennsylvania. The structure was considered as flexible with rigid diaphragms and Gust factors were calculated accordingly in RAM and verified through hand calculations.

For wind casses, as shown in Figure 15, were considered when applying wind loads to the structure. Eccentricities and torsional moments were considered for Case 2 and Case 4. Wind pressures for all four cases were calculated and can be found in Appendix A. For simplification, Orchard Plaza was assumed to be a regular-shaped building. Using commentary from ASCE 7-10, the applicable Gust Factor was found. Detailed calculations can also be found in Appendix A.

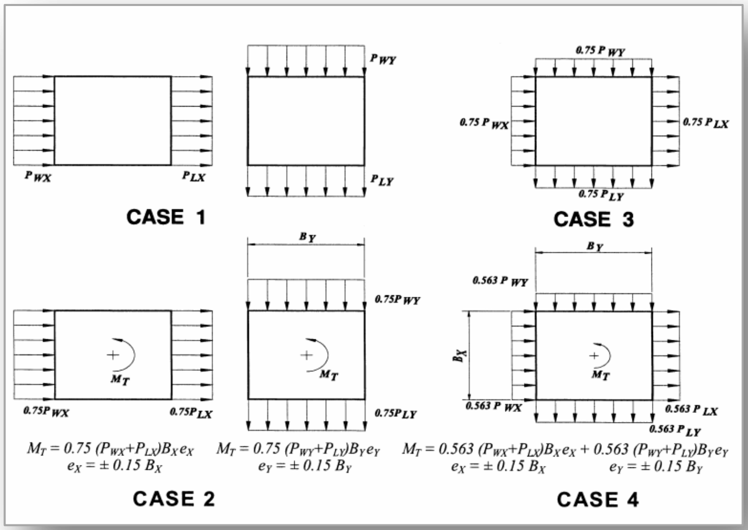


Figure 15: Design Wind Load Cases (ASCE 7-10 Figure 27.4-8)

Figure 16 and 17 show the total combined wind pressures (Windward+Leeward) and resulting shear forces applied in the North-South and East-West direction respectively. The North-South direction results in a larger base shear. This is logical as the North and South faces are larger and consequently experience a higher wind load.

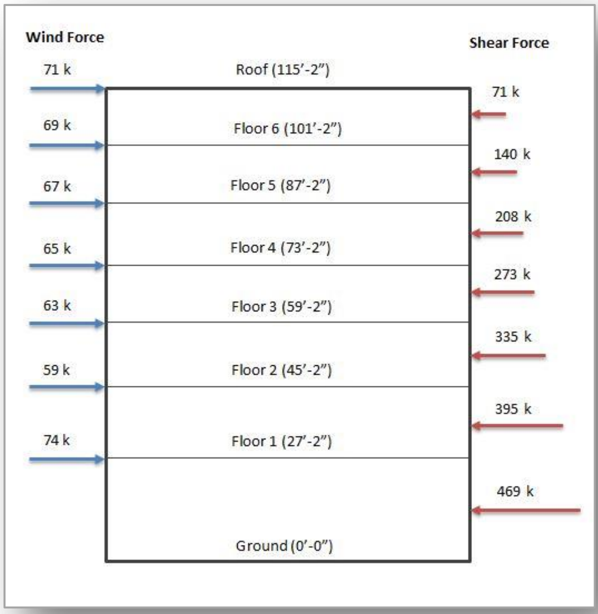


Figure 16: Story Forces and Shear (North-South)

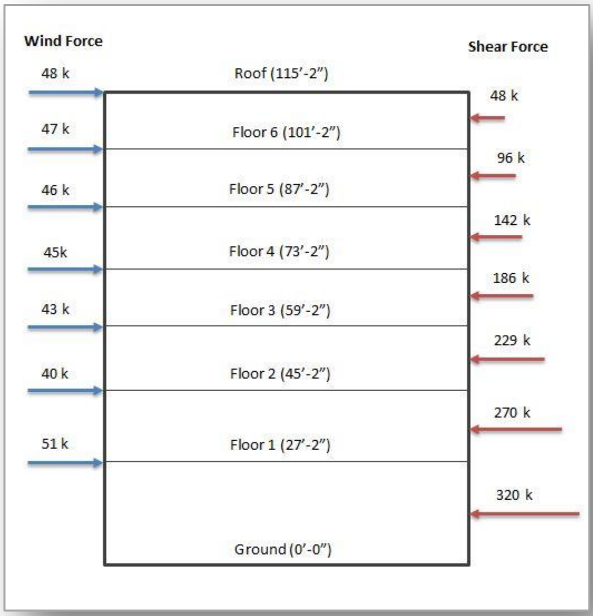


Figure 17: Story Forces and Shear (East-West)

SEISMIC LOADS

Seismic analysis of Orchard Plaza was completed through referencing Chapter 11 and 12 of ASCE 7-10. The geometric footprint of the building was assumed to be identical in the North-South and East-West direction. First, each floor and ultimately the entire building weight was founding using the dead loads applied to the structure. Next, the Equivalent Lateral Force Procedure was used to determine the total base shear incurred at the ground floor (0'-0") and ultimately redistributed to each floor as seen in the table below. A total base shear of 367 kips and total overturning moment of 29248 foot-kips was calculated.

Seismic Loads							
Level	**h_x (ft)**	**hx^k (ft)**	**w_x (k)**	**c_{vx}**	**wx*hx^k**	**F_v (k)**	**Overturning Moment (ft-k)**
1	27.17	34.69	1646.4	0.0578	57114.66	21	577
2	45.17	59.88	1646.4	0.0998	98592.61	37	1656
3	59.17	80.03	1646.4	0.1333	131756.6	49	2897
4	73.17	100.53	1646.4	0.1675	165511.8	62	4502
5	87.17	121.33	1646.4	0.2021	199751.3	74	6472
6	101.17	142.37	1646.4	0.2372	234401.8	87	8816
Roof	115.17	163.64	617.4	0.1023	101028.8	38	4328
		Total	10495.8	1	988157.6	367	29248
$\Sigma(w_i)(h_i)^k$ =988157							
Base Shear (k) = 367							
Total Overturning Moment (ft-k) = 29248							

LOAD COMBINATIONS

Listed below are the thirteen load combinations considered for analysis per ASCE 7-10 section 2.3.2.

1. 1.4D
2. 1.2D + 1.6L + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + L
4. 1.2D + 1.6(L_r or S or R) + 0.5W_x
5. 1.2D + 1.6(L_r or S or R) + 0.5W_y
6. 1.2D + 1.0W_x + L + 0.5(L_r or S or R)
7. 1.2D + 1.0W_y + L + 0.5(L_r or S or R)
8. 1.2D + 1.0E_x + L + 0.2S
9. 1.2D + 1.0E_y + L + 0.2S
10. 0.9D + 1.0W_x
11. 0.9D + 1.0W_y
12. 0.9D + 1.0E_x
13. 0.9D + 1.0E_y

Considering both the X and Y direction, it expected that the Y direction will control for wind as wind from the Y direction induces a greater base shear. The roof live load is greater than the snow load and will control all (L_r or S or R) scenarios.

GRAVITY SYSTEM REDESIGN

OPEN WEB JOISTS

After exploring three alternative floor systems, an open web joist system was determined to be the most practical alternative to the existing composite beam and girder gravity system. Open web joists were chosen for their flexibility with MEP systems, allowing ducts and conduit to pass through the depth of the structure instead of having to pass underneath as seen in Figure 18.

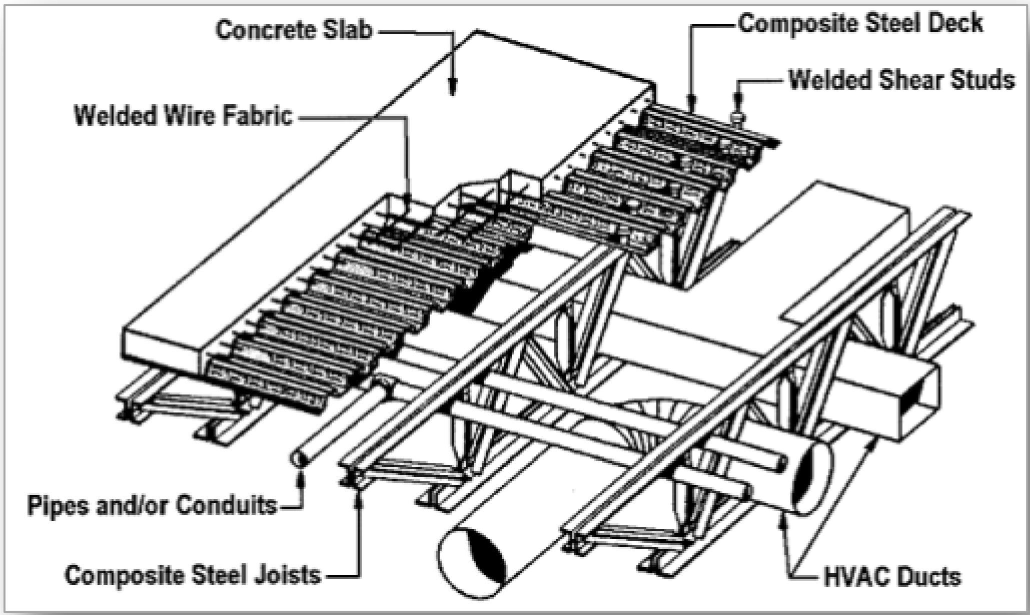
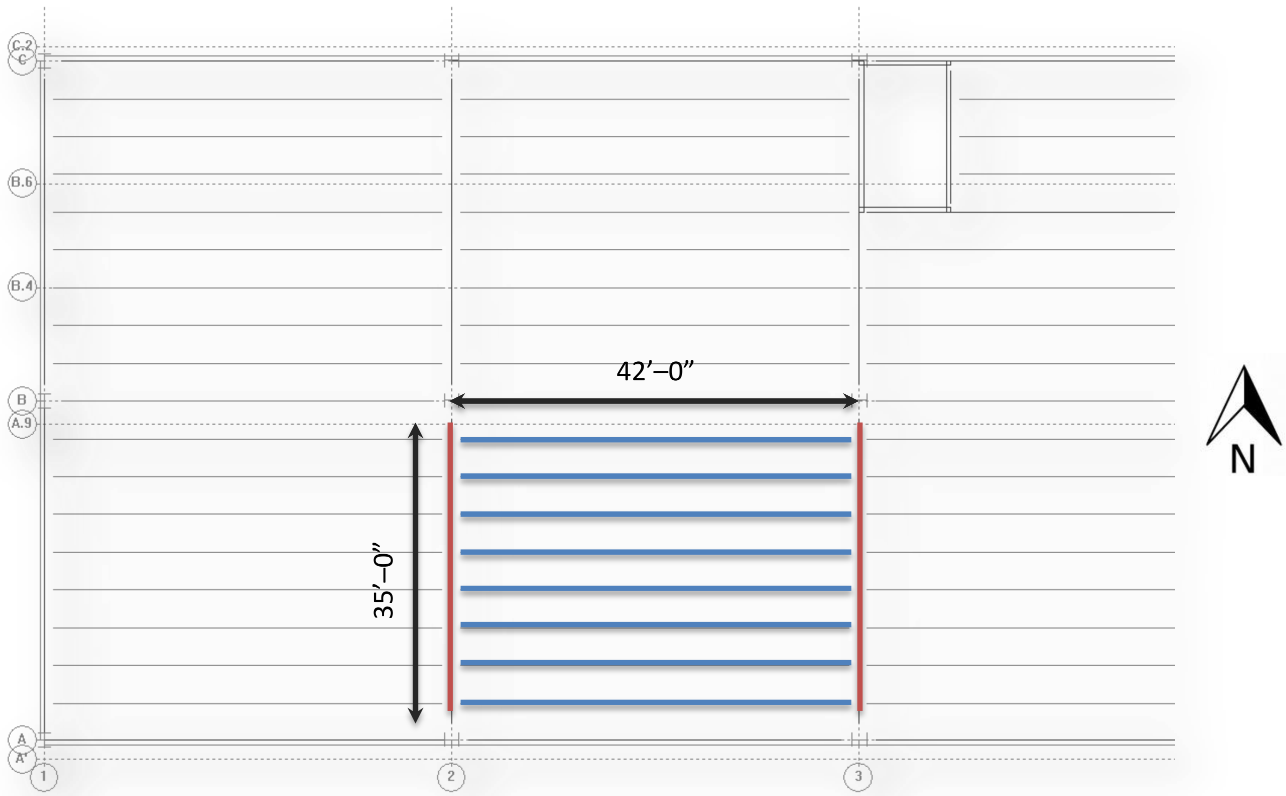


Figure 18: http://theconstructor.org/structural-engg/composite-steel-joists/5895/

The existing column layout was planned to remain unchanged so long as joists meeting deflection and vibration criteria while simultaneously meeting the goal of elevating the acoustical tile ceiling. Joists and gravity columns were modeled using RAM and program sizes were verified by hand calculations found in Appendix B.

The area of concern for using joists is the 42′ by 35′ typical bay shown below in Figure 19, where the joists must span 42 feet. In keeping with the goal of raising the acoustic tile ceiling, the depth of the joists was restricted to 28″. Floor 1, with the higher live load of 100psf, was used for analysis as upper floors with lighter loads are assumed to meet the criteria determined from Floor 1.



28LH11 Joist

W30x108 c=1/2″

Figure 19: Typical 42′ x 35′ bay – Floor 1

Nine open web joists per bay are supported by wide flange girders that mimic the original girder plan. Under an 80psf dead load, 100psf live load, and 28″ depth restriction, RAM determined that a 28LH11 joist and W30x108 girders with a ½″ camber was acceptable by code. Identical sizing and camber was found to be acceptable through hand calculations in Appendix C.

VIBRATIONS

Once an acceptable joist and girder design was found, vibration was the next criteria of concern. Since Orchard Plaza serves only as an office building with no sensitive equipment, vibrations due to walking was the only vibration criteria considered. Design Guide 11 published by the American Institute of Steel Construction was cited for verifying that the new floor system was acceptable for vibration considerations. Using the criteria listed below, the new floor system passes for vibration control. Full calculations can be found in Appendix D.

Given:

- 28LH11 Joist spaced at 3'-11"on center with 2 ½" joist seat
- 2" non-composite deck with 4 ½" normal weight slab
- W30x108 girder with 35'-0" clear span
- 80psf dead load + 100psf live load

$$\frac{a_o}{g} = \frac{65e^{(-0.35)(0.005)}}{0.003\ (603348)} = 0.0035 < 0.005 \quad ✓$$

HVAC INTEGRATION

The original floor system of composite beams and girders required the acoustic tile ceiling be hung 36" below the finished floor surface of the floor above. Auxiliary ducts running perpendicular to the central duct must pass underneath the beams, while the joists allow for 8" round ducts to pass directly through the depth of the structure. Shown below in Figure 20 are identical 30" x 18" main and 8" wide auxiliary ducts that service the office's open floor plan. While not a drastic change, it is proven that the goal of integrating the MEP systems with the open web steel joist construction was achieved with and the ceiling can be raised.

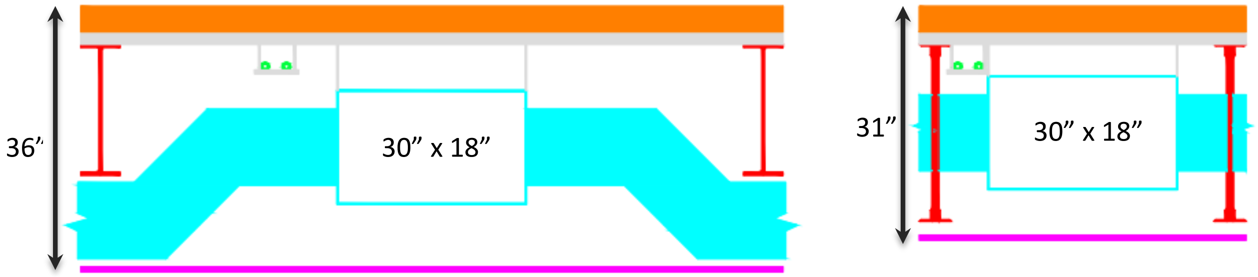


Figure 20: HVAC Interaction Comparison

LATERAL SYSTEM REDESIGN

When considering how the lateral force-resisting force system of Orchard Plaza could be improved to be a more efficient use of materials, a change from eccentric braced frames, which perform almost as moment frames, to concentrically braced frames was deemed most appropriate. Changing to concentrically braced frames, which are fundamentally much stiffer, would also allow for the removal of several frame bays while maintaining the necessary drift control. In keeping with the architect's intent of providing an open floor space with an uninterrupted array of windows on the facades, concrete shear walls were not seen as a practical solution. Changing to concentrically braced frames, which are much stiffer, would also allow for the removal of several frame bays.

RAM ANALYSIS

Both the existing and proposed lateral force-resisting system was modeled using RAM. To eliminate variables between the results of each model, the new steel joist and girder gravity system was held constant between the models. When modeling the new lateral system, frame bays were eliminated one at a time and the new model was reanalyzed for story drifts. Interior frames were eliminated first in order to maintain a center of rigidity as similar as possible to the original structure. The table below shows the slight change in the center of rigidity with the reference point taken from the southwest corner of the structure. Each story COR was averaged to find the average overall COR. RAM output can be found in Appendix E

Center of Rigidity		
	X (ft)	**Y (ft)**
Eccentric	100.8	59.3
Concentric	98	56.5

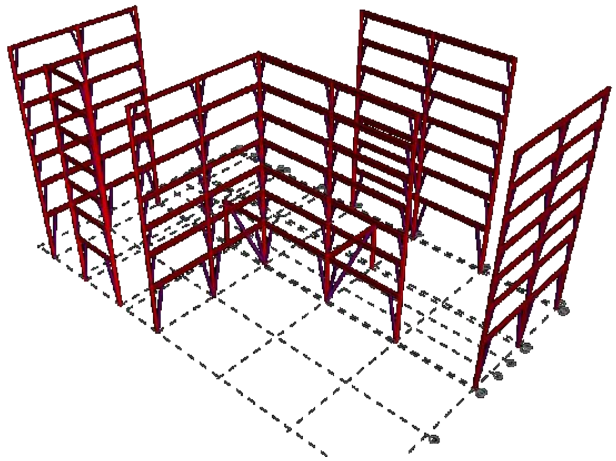


Figure 21: Eccentric Lateral Frames

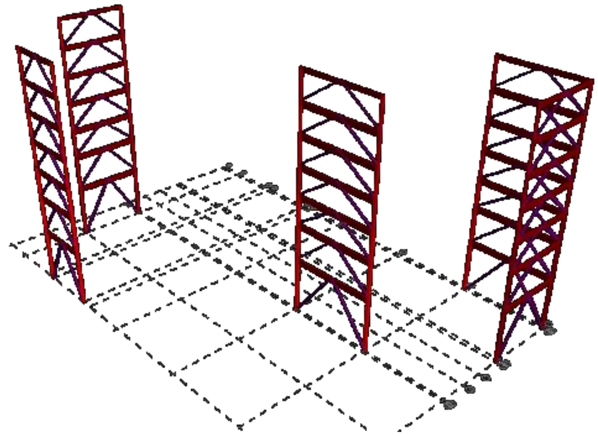


Figure 22: Concentric Lateral Frames

Figure 21 shows the original eleven full-height eccentric frame bays as compared to the final five full-height concentric frame bays in Figure 22. Figure 23 below shows the new concentric frames in orange. The resulting center of rigidity is also indicated in purple.

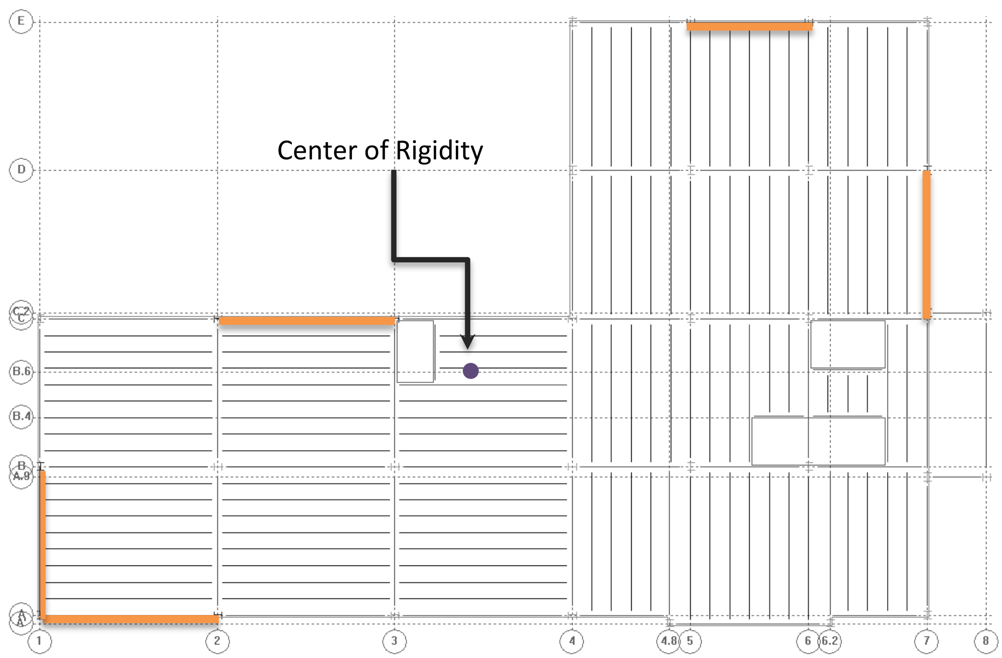


Figure 23: Concentric Lateral Frames – Plan View

RAM MODEL OVERVIEW

Figures 24 and 25 below show an overview of the RAM model used to model the new concentric framing system. Lateral elements are shown in red and gravity elements in blue. The deck and slab was hidden to show the joists in detail. A larger model including decking and slabs is located in Appendix E.

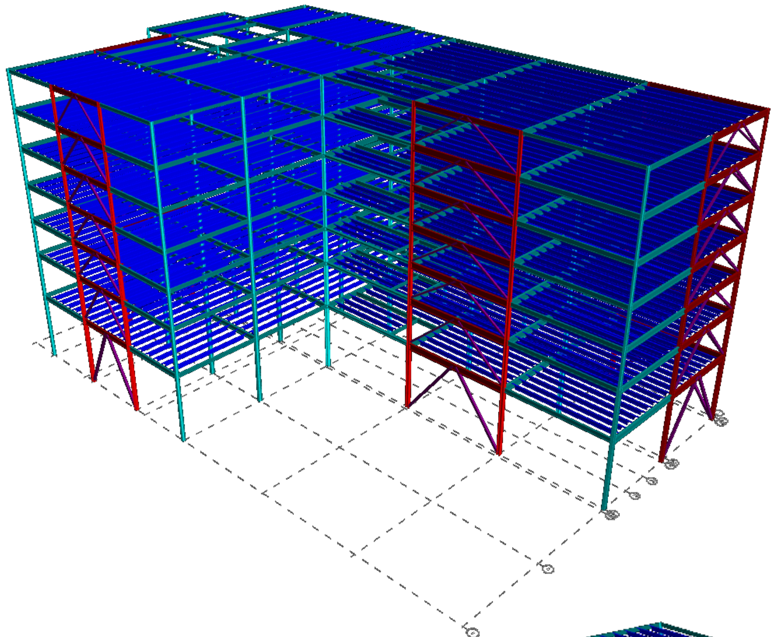


Figure 24:
RAM Model – Northwest Corner

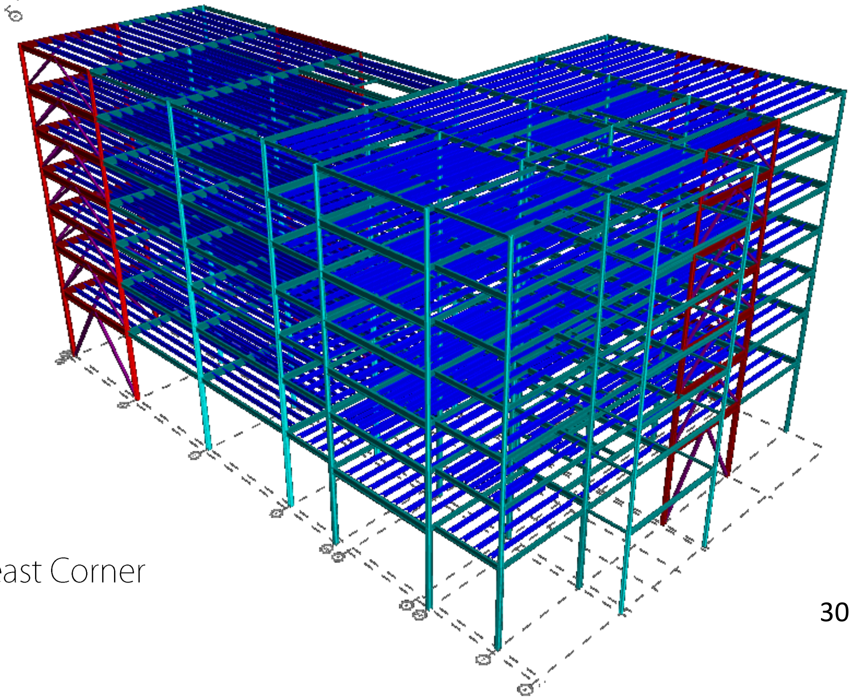


Figure 25:
RAM Model – Southeast Corner

SYSTEM UTILIZATION

In order to effectively compare the material efficiency between the eccentric and concentric lateral systems, member sizes remain identical across the two systems. Exact sizes and properties from the original structure were used for this analysis. The west-most frame for both systems was analyzed individually to prove that a single width frame utilizes the bracing capacities similarly to the two-bay width eccentric frame. While more braces in the eccentric of Figure 25 frame utilize the members better, an economic gain is made in both material and labor costs through only having to construct the single width concentric frame in Figure 26.

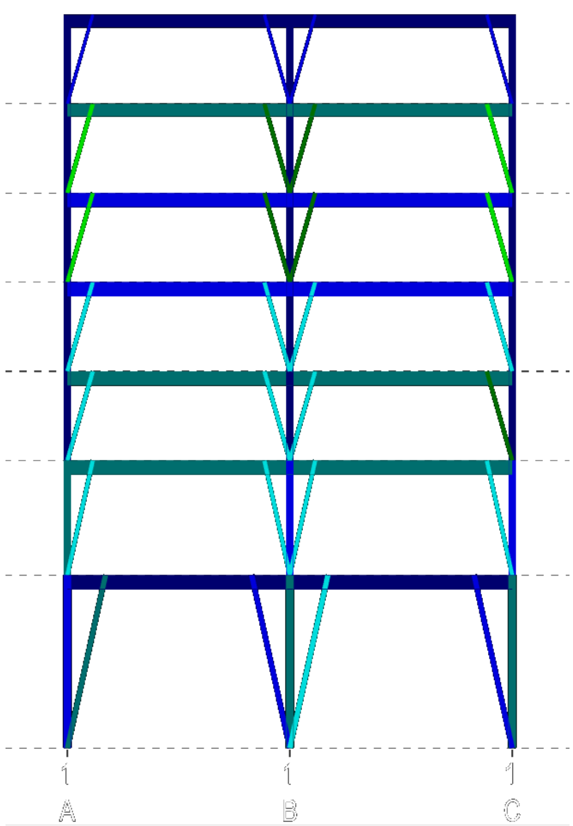
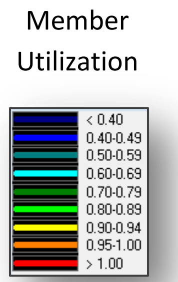


Figure 25: Eccentric Frame Utilization



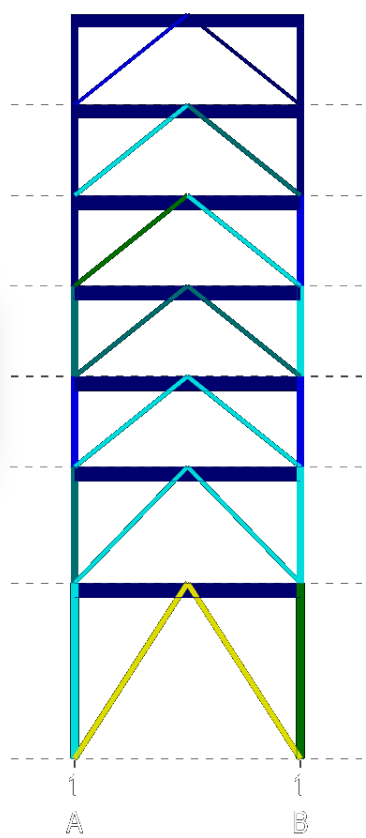


Figure 26: Concentric Frame Utilization

STORY DRIFT

The primary criteria used to prove the efficiency and effectiveness of the lateral system redesign was story drift. Using the loading combinations presented earlier, it was found that even with less than half of the frame bays, eliminating six total, the structure still gained significant stiffness as seen in the comparison below. RAM output of drift values is located in Appendix E

Maximum Story Drift	
	Worst Load Case Drift
Eccentric	3.28 in.
Concentric	1.52 in.

ARCHITECTURAL IMPACT

Architecture was the driving force behind the initial design to use highly eccentric frames. Without shear walls and the desire for uninterrupted glazing, an eccentrically braced lateral system allows for windows to be placed within each bay of every frame. The decision to place the west-most frames at a corner stemmed from the existing curtain found on the southeast corner of Orchard Plaza as seen in Figure 27. A matching curtain wall could wrap around the concentric frames while maintaining the uninterrupted glazing. A solution to other frames centered on its respective wall section could implement a single curtain wall strip the full height of the structure. The eastern most

Figure 27: Southeast Curtain Wall

frame, not being centered, could use a row of faux windows to not interrupt the façade continuity.

STRUCTURAL REDESIGN SUMMARY

The alternative gravity system of non-composite open web steel joists with wide flange girders has been proven as a successful alternative to the existing composite steel beam and girder design. A 2" non-composite deck with 4 ½" slab substantially satisfies strength requirements but also achieves the necessary two-hour fire rating required by code. Additionally, all joists require spray-on fireproofing to maintain a two-hour fire rating. Integration of the HVAC system with the joists, while only minimally allowing for a higher ceiling, does achieve this goal. RAM proved to be a very effective tool in determining the size and depth of my joists, while all output was checked by hand only to result in identical sizing for verification.

Switching from highly eccentric braced frames to concentrically braced frames proved to be a highly efficient alternative that not only allowed from a total frame bay reduction from eleven to five, but reduced cumulative story drift from over two inches to less than one inch. By requiring fewer bracing members, both raw material costs and labor installation costs are directly reduced. Conversely, a concentric lateral framing system creates an architectural challenge in which solutions to best satisfy the architect's goal of both an open floor space and uniform glazing elements were explored and presented.

DAYLIGHTING BREADTH

Orchard Plaza currently has very few elements that take advantage or help redirect sunlight entering the office space. In order to better understand the interaction of Orchard Plaza's façade during various times of the day in both January and June, a model was produced to aid in visualizing the shadows produced.

JANUARY

During the winter solstice, when sunlight angles are lowest on the horizon and most direct into the office space, the south facing wall experiences several hours of penetrating sunlight while the North and West facing walls are continually shadowed. Figure 28 below verifies these statements.

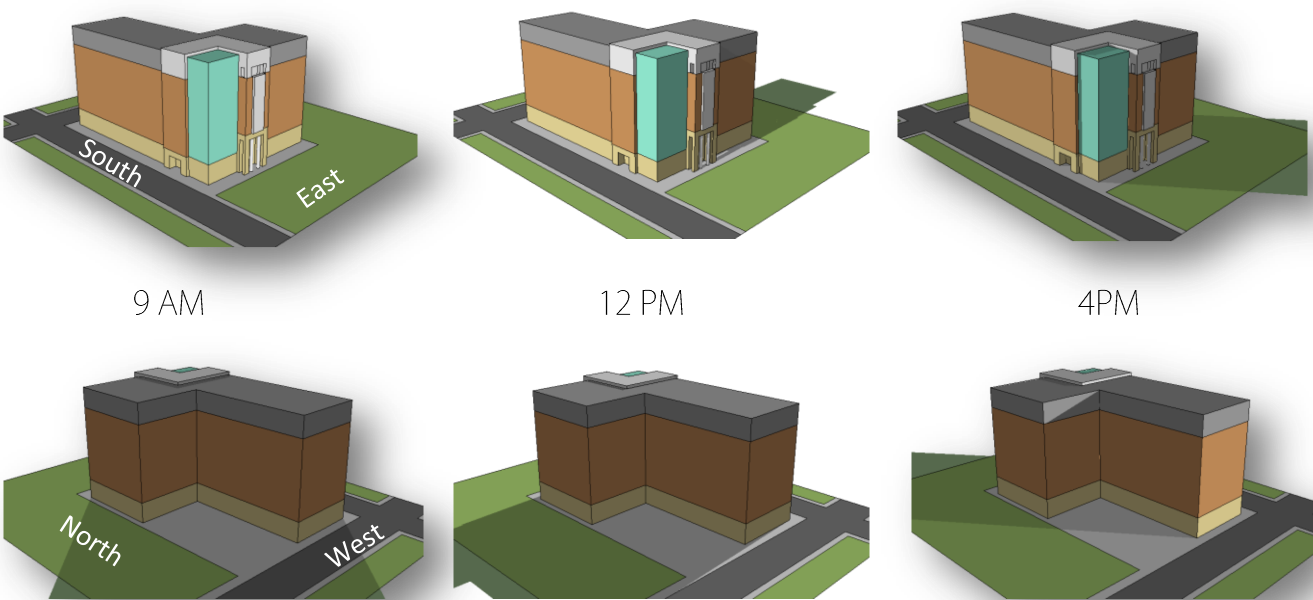


Figure 28: Winter Solstice Shadows

JUNE

In June, the building experiences its most direct sunlight in the morning from the East. All other walls receive high angle sunlight but this light does not deeply penetrate the space as seen in January. The North and West facing walls continue to see no direct sunlight. Figure 29 below expresses these findings.

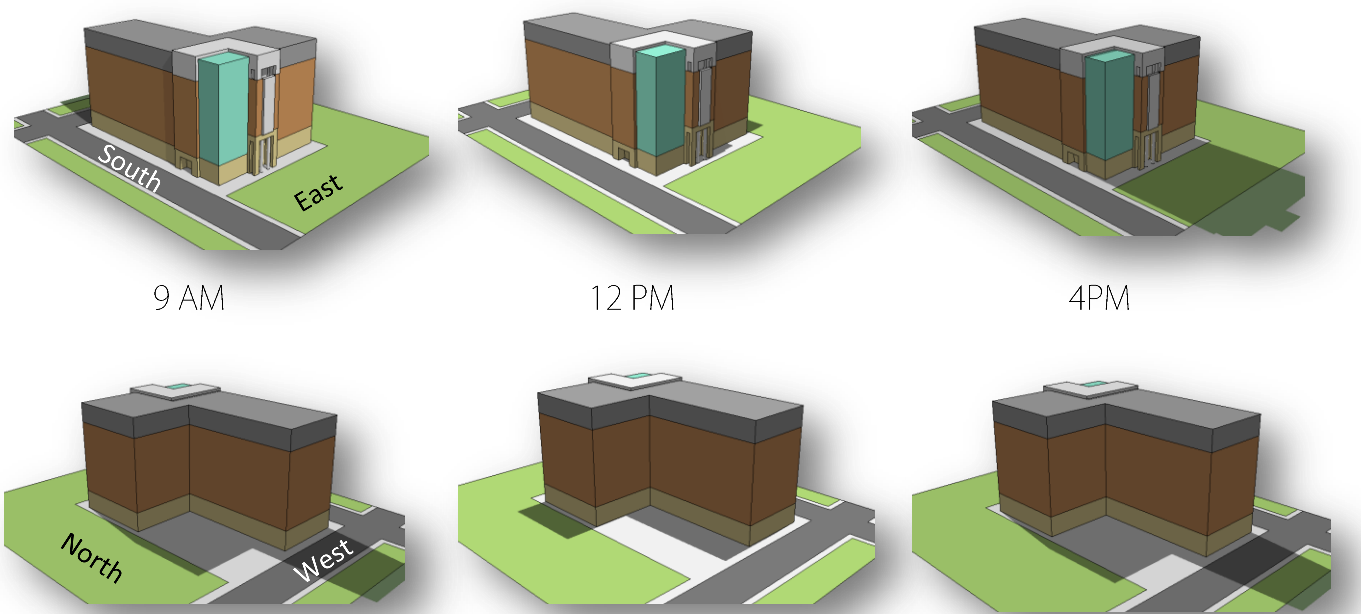


Figure 29: Summer Solstice Shadows

PROPOSED IMPROVEMENTS & SOLUTIONS

During the summer, the office space is most efficiently shaded from high-angle sunlight using horizontal shading shelves cantilevered off the structure at the top of each window. Shown in Figure 30, these shelves would be most appropriately placed on the South facing wall, and additionally on the East wall for architectural continuity. Direct sunlight entering the east wall is not effectively mitigated through permanent architectural elements, therefore permanent blinds or fritted glass would be required.

Figure 30: Horizontal Light Shelves
http://levolux.blogspot.com/2012_02

In winter months, similar to eastern morning sun in the summer, the low sunlight angles make shading very difficult using permanent architectural features. Operative blinds would be the best solution when sunlight is penetrating the South wall. In the morning and evening, when sunlight angles are low but not perpendicular to the South wall, Vertical shading fins similar to those in Figure 31would shade glazing from low angle light in the morning and evening.

Figure 31: Vertical Shading Fins
http://www.angarch.com/products/brise-soleil_01_archive.html

GREEN ROOF BREADTH

For a second breadth study, a green roof was explored for the 84' by 70' green highlighted space shown in Figure 32. It was determined that an extensive green roof, that is, a low profile and self-sustaining would be most suitable for this structure. With the roof remaining an unoccupied space, and Orchard Plaza being one of the tallest structures in the town it resides in, there is no need for a garden-like intensive system for visual appeal.

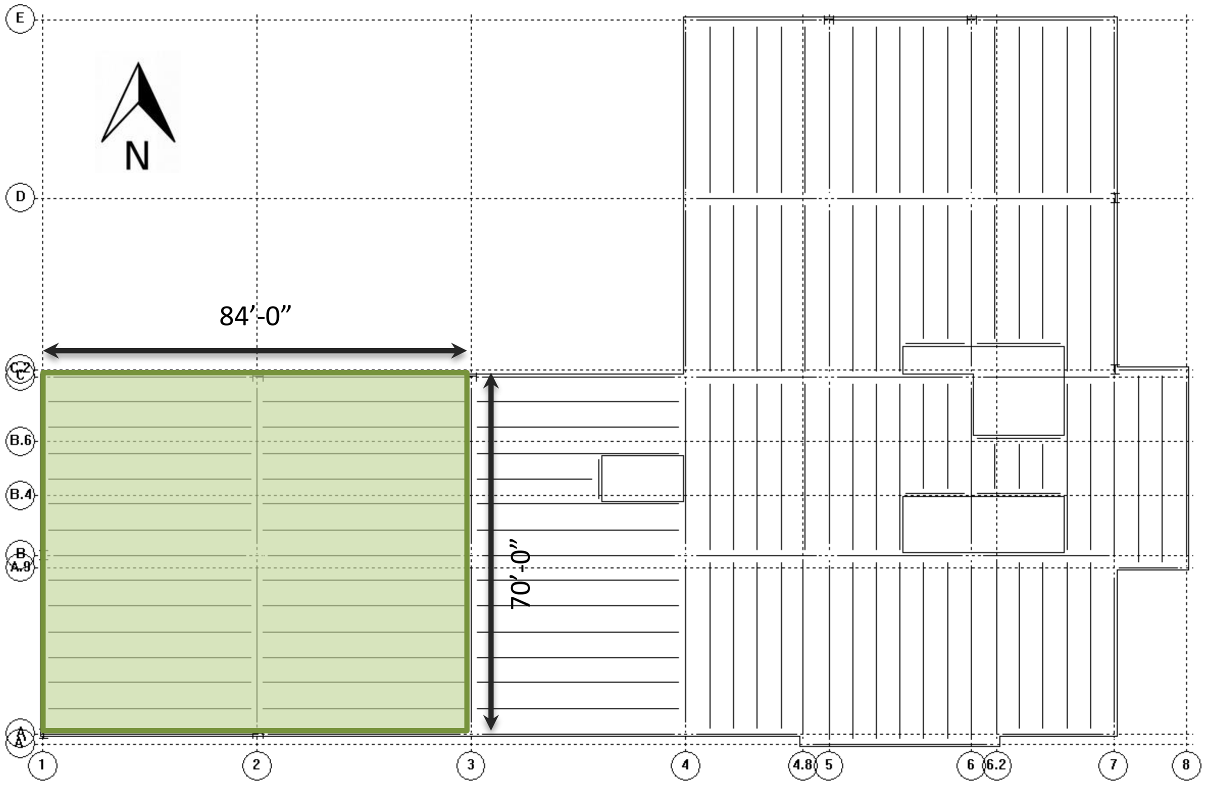


Figure 32: Roof Plan & Green Rood Location

LOADING IMPLICATIONS

The addition of a green roof substantially increases not only the roof dead load of the structure, but OSHA required safety anchor posts for roofs with low or no edge wall for fall protection could potentially induce an impact load to the roof structure. Given a green roof system the averages 30psf, an additional resulting axial load increase of 176.4 kips is applied to column B-2 in Figure 32. The additional dead load would also amplify seismic effects of the roof given its 115' height from ground level.

A typical roof anchor post suitable for direct attachment to roof decking, seen in Figure 33, requires an activation impact load of 1000lb and can support a person weighing up to 310lb. While required by code, it is critical that these posts, which permeate the weatherproofing of the green roof system, must be sealed to ensure no moisture can enter through to the roof deck. Product details are located in Appendix F.

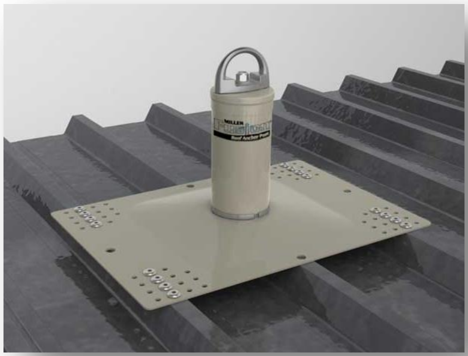


Figure 33: Anchor Post

COST & MAINTENANCE

Citing *Reducing Urban Heat Islands: Compendium of Strategies*, an extensive green roof installation is estimated at approximately \$20 - \$25 per square foot. Therefore, an 84' by 70' plot would cost roughly\$117,600 to \$147,000. Maintenance for extensive systems is very low, so rooftop checks could be done simultaneously with other roof-top mechanical checks and maintenance. Overall, a small green roofing system, while requiring an initial investment, would insulate the building over the life of the structure to reduce heating and air conditioning costs while requiring very little additional labor to maintain.

CONCLUSION

To evaluate the success of the structural design goals proposed and researched in this report, each goal is listed below along with a respective evaluation of their success.

Goal 1: Design an open-web steel joist gravity system that allows for the integration of HVAC systems and thus allows for the acoustic tile ceiling to be raised.

- Through the use of RAM structural design backed up by hand calculations, an acceptable 28" steel joist that meets vibration criteria was found to span a clear span of 42'. While minimal, it was proven that an integrated gravity and HVAC system would allow the ceiling in the office to be raised 5".

Goal 2: Propose a concentric lateral system that simultaneously decreases the number of frame bays needed, thus reducing material and construction costs, while maintaining story drift limitations.

- This goal was met with great success as the number of frame bays was reduced from eleven to five. Even with less than half of the frame bays remaining, as expected, the concentric frames increased the building stiffness by over 200%.

Goal 3: Provide architectural solutions for integrating the new concentric frames as they will disrupt the façade's glazing configuration.

- Placing two frames perpendicular at the southwest corner to encourage a matching corner curtain wall was the best solution to this structural integration concern. Curtain wall strips where frames are centered on the wall would maintain symmetry and be a fair architectural compromise. The east-most frame that is not centered still remains of concern and merits an improved solution.

APPENDIX A

WIND PRESSURES

Wind Pressure (North-South)										
Level	**z**	**kz**	**qh**	**qz (psf)**	**Windward (psf)**	**Leeward (psf)**	**Trib. Area (sf)**	**Force (k)**	**Story Shear (k)**	**Overturning Moment (ft-k)**
1	27.17	0.57	16.74	10.04	9.54	-9.81	3840	74	469	2011
2	45.17	0.61	16.74	10.75	10	-9.81	2987	59	395	2673
3	59.17	0.71	16.74	12.51	11.15	-9.81	2987	63	335	3704
4	73.17	0.79	16.74	13.92	12.06	-9.81	2987	65	273	4778
5	87.17	0.85	16.74	14.98	12.75	-9.81	2987	67	208	5878
6	101.17	0.91	16.74	16.03	13.43	-9.81	2987	69	140	7021
Roof	115.17	0.95	16.74	16.74	13.85	-9.81	2987	71	71	8139
Base Shear (k) = 469										
Total Overturning Moment (ft-k) = 34204										

Wind Pressure (East - West)										
Level	**z**	**kz**	**qh**	**qz (psf)**	**Windward (psf)**	**Leeward (psf)**	**Trib. Area (sf)**	**Force (k)**	**Story Shear (k)**	**Overturning Moment (ft-k)**
1	27.17	0.57	16.74	10.04	9.64	-9.92	2592	51	320	1386
2	45.17	0.61	16.74	10.75	10.11	-9.92	2016	40	270	1825
3	59.17	0.71	16.74	12.51	11.27	-9.92	2016	43	229	2527
4	73.17	0.79	16.74	13.92	12.20	-9.92	2016	45	186	3263
5	87.17	0.85	16.74	14.98	12.90	-9.92	2016	46	142	4010
6	101.17	0.91	16.74	16.03	13.60	-9.92	2016	47	96	4795
Roof	115.17	0.95	16.74	16.74	14.07	-9.92	2016	48	48	5574
Base Shear (k) = 320										
Total Overturning Moment (ft-k) = 23380										

GUST FACTOR CALCULATIONS

Wind Loads ASCE 7-02

Basic Wind Speed = 90 mph

Importance Factor = $I_W = 1.0$

Building Category II

Exposure Category B

Internal Pressure $GC_{pi} = \pm 0.18$

$K_d = 0.85$

$K_{zt} = 1.0$

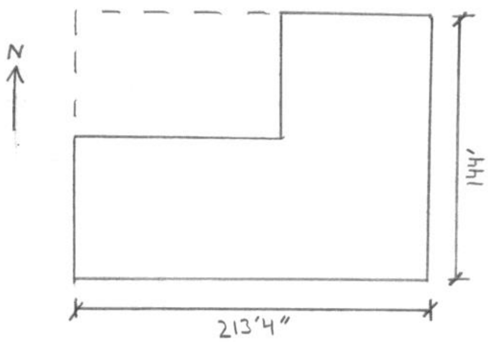
$h = 88'$

$B = 213.33$ ft

$L = 144$ ft

$g_Q = g_v = 3.4$ §6.5.8.1

Rigid Structure



Consider building as if it were perfecty rectangular

Values from Table 6-2

$z_g = 1200$ $\quad \alpha = 7$ $\quad \ell = 320$

$z_{min} = 30$ ft $\quad c = 0.3$ $\quad \bar{\epsilon} = 0.33$

$\bar{z} = 0.6(88\text{ft}) = 52.8 > 30$ ✓ok

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{52.8}\right)^{1/6} = 0.277$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{52.8}{33}\right)^{0.33} = 374.3$$

Wind Loads Cont.

North-South

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{213.33+88}{374.3}\right)}} = 0.803$$

East-West

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{144+88}{374.3}\right)}} = 0.825$$

Gust Factor

North-South

$$G = 0.925\left(\frac{1+1.7g_Q I_{\bar{z}} Q}{1+1.7g_v I_{\bar{z}}}\right) = 0.925\left(\frac{1+1.7(3.4)(0.277)(0.803)}{1+1.7(3.4)(0.277)}\right)$$

$$G = 0.8128$$

East-West

$$G = 0.925\left(\frac{1+1.7(3.4)(0.277)(0.825)}{1+1.7(3.4)(0.277)}\right)$$

$$G = 0.8254$$

$$K_z = \begin{cases} 2.01\left(\frac{z}{z_g}\right)^{2/\alpha} & 15' < z < z_g \\ 2.01\left(\frac{15}{z_g}\right)^{2/\alpha} & z < 15' \end{cases}$$

$$q_h = 0.00256\, K_z K_{zt} K_d V^2 I_w$$

$$q_h = 0.00256(0.95)(1)(0.85)(90^2)(1)$$

$$q_h = 16.74 \text{ psf}$$

Wind Loads Cont.

$$p = qGC_p - q_i(GC_{pi})$$

WW $C_p = 0.8$ Table 6-6

LW $C_p = -0.5$ Table 6-6

North - South

Windward

$$P_{WW} = q_z(0.8128)(0.8) - q_i(-0.18)$$

Leeward

$$P_{LW} = q_z(0.8128)(-0.5) - q_i(0.18)$$

East - West

Windward

$$P_{WW} = q_z(0.8254)(0.8) - q_i(-0.18)$$

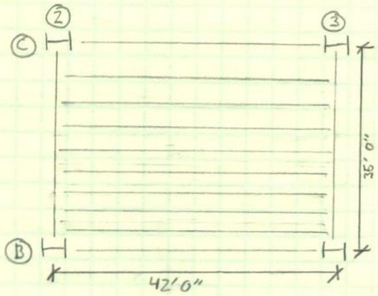
Leeward

$$P_{LW} = q_z(0.8254)(-0.5) - q_i(0.18)$$

APPENDIX B

GRAVITY CHECK – JOISTS – FLOOR 1

Gravity Check - Joists (Floor 1)



Trial - use 8 joists per bay

- Restrict depth to 28" for raising ceiling goal

Dead Load = 68 psf + 12 psf misc = 80 psf

Live Load = 100 psf

Tributary width per joist = $35/9 = 3.89'$

Factor Loads

$W_{Ultimate} = [1.2(80) + 1.6(100)](3.89') = 996\ plf$

$W_{TL} = (80 + 100)(3.89) = 700\ plf + 1.2(\text{joist self wt})$

$\Delta_{TL\ max} = \frac{l}{240} = \frac{42(12)}{240} = 2.1''$

Try 28LH11
- depth = 28"
- 25 plf self wt
- 1143 plf capacity at 42' span

$996 + 25(1.2) = 1026 < 1143$ ✓ ok

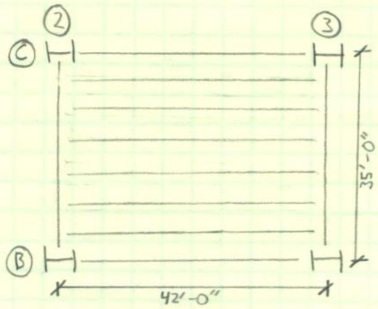
W for $l/360 = 475\ plf$

W for $l/240 = 475(1.5) = 713\ plf > 700\ plf$ ✓ ok

28LH11 ✓ ok

GRAVITY CHECK – JOISTS - ROOF

Gravity Check - Joists (Roof)



- 25 psf snow load
- 30 psf dead load

Joist spacing – Try 6 joists

Spacing = 5'

Restrict depth to 28"

Dead Load = 30 psf

Live Load = 30 psf

Snow Load = 25 psf

Factor Loads

Dead = 30(5) = 150 plf

Live = 30(5) = 150 plf

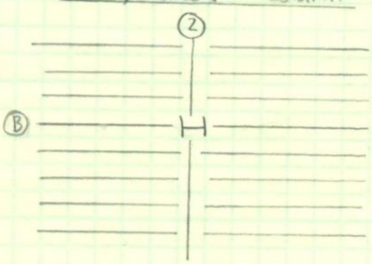
Try 28K9

Live allowable = 208 plf > 150 plf √ ok

Total allowable = 208 plf (1.5) = 312 plf > 150 + 150 = 300 plf √ ok

GRAVITY CHECK – COLUMN

Gravity Check – Column (Floor 1)



- Use W14 size

$A_T = 42' \times 35' = 1470\ sf$

Roof load $= (30\ psf\ \text{dead} + 30\ psf\ \text{live})(1470) = 88.2\ k$

Floor 2-6

Live Load = 80 psf

$K_{LL} = 4 \qquad K_{LL} A_T = 4(1470) = 5880\ sf > 400\ sf$ ✓ ok

$L = 80\left(0.25 + \frac{15}{\sqrt{5880}}\right) = 35.6\ psf$

Dead load = 80 psf

Total load per floor $= (80 + 35.6)(1470) = 170\ k$

5 floors $= 170 \times 5 = 850\ k$

Floor 1

Live load = 100 psf

$L = 100\left(0.25 + \frac{15}{\sqrt{5880}}\right) = 44.6\ psf$

Dead load = 80 psf

Total load $= (80 + 44.6)(1470) = 183.2\ k$

Total load on base column

$88.2k + 850k + 183.2k = 1121.4k$

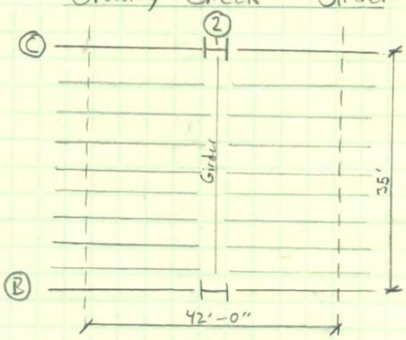
Effective length = 28'

Table 4.1 → W14 × 211 ✓ ok

APPENDIX C

GRAVITY CHECK – GIRDERS

Gravity Check - Girder (Floor 1)



Dead Load - 80psf

Live Load - 100 psf

Factor Loads

Tributary Area = 42'(35') = 1470 SF

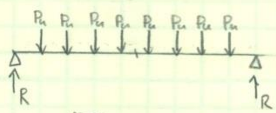
$K_{LL} = 2$

$K_{LL} A_T = 2(1470) = 2940$ SF > 400 ✓

Joist Self weight = 25plf (42') = 1050 lbs = 1.05 k

$L = 100\left(0.25 + \frac{15}{\sqrt{2940}}\right) = 53$ psf

$P_u = 1.05 + (53 + 80)(3.89')(42') = 21.7$ k



Total $P_u = 21.7 \times 8 = 174$k

$R = \frac{174}{2} = 87$k

$M_A = 3.89(87 + 65.3 + 43.6 + 21.7) = 846$ ft-k

Try W30 x 108

$M_u = 863$ ft-k > 846 ft-k ✓ satisfies strength

Check Deflection

$W_{LL} = (53)(42') = 2.23$ klf $\quad I$ for W30x108 = 4470 in^4

$$\Delta_{LL} = \frac{5(2.23)(35)^4(1728)}{384(29000)(4470)} = 0.58''$$

$$\Delta_{LL\,max} = \frac{l}{360} = \frac{35(12)}{360} = 1.17''$$

$W_T = (53 + 80)(42') = 5.59$ klf

$$\Delta_{TL} = \frac{5(5.59)(35)^4(1728)}{384(29000)(4470)} = 1.46'' \qquad \Delta_{TL\,max} = \frac{l}{240} = \frac{35(12)}{240} = 1.75'' > 1.46'' \checkmark$$

Camber

$W_{DL} = (80)(42') = 3.36$ klf $\qquad I$ for W30×108 $= 4470\ in^4$

$$\Delta_{DL} = \frac{5(3.36)(35)^4(1728)}{384(29000)(4470)} = 0.875''$$

Camber 80% of dead load

$0.875(0.8) = 0.7'' \longrightarrow$ round down with 0.25" increments

$\therefore$ camber 0.5"

W30×108 with 0.5" ✓ OK

APPENDIX D

VIBRATION CRITERIA

$$\frac{L}{d} = \frac{42(12)}{28} = 18$$

$$C_r = 0.9\left(1 - e^{-0.28(18)}\right)^{2.8} = 0.88$$

$$\gamma = \frac{1}{C_r} - 1 = \frac{1}{0.88} - 1 = 0.136$$

$$I_j = \frac{1}{\frac{0.136}{850.5} + \frac{1}{1909}} = 1463 \text{ in}^4$$

Find Δ_j

28LH11 self wt

$$w_j = 3.89(70 + 100 + 10) + 25 = 725.2 \text{ plf}$$

$$\Delta_j = \frac{5(.725)(42)^4(1728)}{384(29000)(1463)} = 1.19''$$

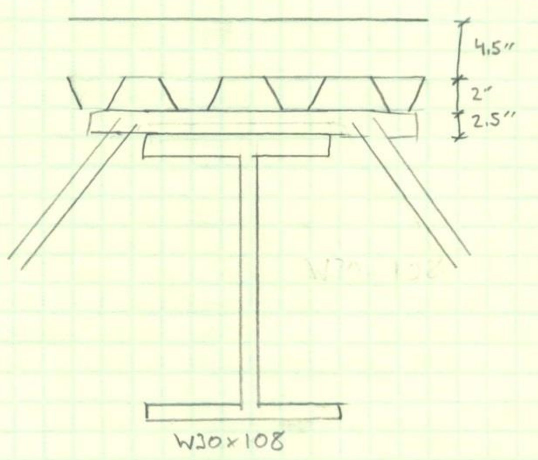
Find W_j

$$w_j = \frac{725}{3.89} = 186 \text{ psf}$$

$$D_s = \frac{12(4.5+1)^3}{12(6.8)} = 24.5 \qquad D_j = \frac{I_j}{S} = \frac{1463}{5} = 292.6$$

$$B_j = 3.89\left(\frac{24.5}{292.6}\right)^{1/4}(42) = 87.9$$

$$W_j = 186(87.9)(42) = 686675 \text{ lb}$$



$$\frac{b}{h} = 0.4 L_g = 0.4(35)(12)$$

$$= 168 \rightarrow \text{controls}$$

$$= 42(12) = 504$$

$$\bar{y} = \frac{\Sigma A_y}{\Sigma A} = \frac{\left(168/6.8\right)(4.5)(2) + \left(168/6.8\right)(2)(0.5)(5.5) + 31.7(9)\left(29.8/2\right)}{\left(168/6.8\right)(4.5) + \left[\left(168/(6.8/2)\right)\right](2) + 31.7}$$

$$\bar{y} = \frac{222.4 + 135.9 + 4251}{111 + 98.8 + 31.7} = 19.09''$$

$$I_{comp} = \Sigma I + \Sigma A d^2$$

I_x for W30×108

$$= \frac{\left(168/6.8\right)(4.5)^3}{12} + \frac{\left[\frac{168}{(6.8/2)}\right](2)^3}{12} + 4470 + \left(168/6.8\right)(4.5)(19.09 - 2)^2$$

$$+ \left[\frac{168}{(6.8/2)}\right](2)(19.09 - 2.5)^2 + 31.7\left[\left(\frac{29.8}{2}\right) + 9 - 19.09\right]^2$$

$$= 187.6 + 32.9 + 4470 + 32471.1 + 27199 + 733.4$$

$$I_{comp} = 65094 \text{ in}^4$$

$$I_g = 4470 + (65094 - 4470)(1/4) = 19626 \text{ in}^4$$

$$W_g = \frac{W_j}{S} L_j + \text{self weight}$$

$$= \left(\frac{725}{3.89}\right)(42) + 108 = 7936 \text{ plf}$$

$$\Delta_g = \frac{5(7.936)(35)^4(1728)}{384(29000)(19626)} = 0.47$$

$L_g = 35$ $\quad B_j = 45.2$ $\quad L_g < B_j$

$$\Delta_g' = \frac{35}{45.2}\Delta_g \quad 0.77(0.47) = 0.362$$

$$w_g = \frac{7936}{42'} = 189 \text{ psf}$$

$$D_j = 292.6$$

$$D_g = \frac{19626}{42'} = 467.3$$

$$B_g = 1.6\left(292.6/467.3\right)^{0.25}(35) = 49.8$$

$$W_g = (189)(49.8)(35) = 329427 \text{ lb}$$

4

$$W = \left(\frac{\Delta_j}{\Delta_j + \Delta'_g}\right) W_j + \left(\frac{\Delta'_g}{\Delta_j + \Delta'_g}\right) W_g$$

$$W = \left(\frac{1.19}{1.19 + 0.362}\right) 686675 + \left(\frac{0.362}{1.19 + 0.362}\right) 329427$$

$$W = 526510 + 76838 = 603348 \text{ lb}$$

$$f_n = 0.18\sqrt{\frac{386.4}{603348}} = 0.005 \text{ Hz}$$

$$P_o = 65 \text{ lb} \qquad \beta = 0.03$$

$$\frac{a_o}{g} = 0.5\% \, (386.4) = 0.005g$$

$$\frac{a_p}{g} = \frac{65\, e^{(-0.35)(.005)}}{0.03(603348)} = 0.0035 < \frac{a_o}{g} = 0.005 \quad \checkmark \text{OK}$$

∴ 28LH11 with 42' span spaced at 3.89' OC + 2.5" joist seats
2" deck with 4.5" topping - NWC $f'_c = 3000$ psi
W30 × 108 girder with 35' span
80 psf dead load + 100 psf live load

Passes for walking excitation vibration

APPENDIX E

ECCENTRIC CENTER OF RIGIDITY

Center of Rigidity

RAM Frame v14.05.03.00
DataBase: Eccentric 04/06/14 22:16:18

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.

			Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Type**	**Xr**	**Yr**	**Xm**	**Ym**
			ft	**ft**	**ft**	**ft**
ROOF	1	Rigid	100.66	54.76	119.89	53.80
SIXTH	1	Rigid	103.96	56.62	120.99	53.84
FIFTH	1	Rigid	103.65	57.64	120.72	54.47
FOURTH	1	Rigid	103.94	58.65	120.89	54.65
THIRD	1	Rigid	103.42	59.96	121.34	55.62
SECOND	1	Rigid	99.13	61.81	120.66	56.20
FIRST	1	Rigid	90.85	65.32	119.81	55.49

			Story Lateral Stiffness	
Level	**Diaph. #**	**Type**	**KX**	**KY**
			kips/ ft	**kips/ ft**
ROOF	1	Rigid	3384.03	2375.12
SIXTH	1	Rigid	4281.74	4088.42
FIFTH	1	Rigid	4544.74	4659.64
FOURTH	1	Rigid	5127.91	5504.14
THIRD	1	Rigid	5216.14	6481.93
SECOND	1	Rigid	4603.29	5425.89
FIRST	1	Rigid	7760.13	9514.47

CONCENTRIC CENTER OF RIGIDITY

Center of Rigidity

RAM Frame v14.05.03.00
DataBase: Concentric3 04/06/14 23:06:24

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.

Level	Diaph. #	Type	Centers of Rigidity Xr ft	Yr ft	Centers of Mass Xm ft	Ym ft
ROOF	1	Rigid	92.29	50.00	125.69	54.32
SIXTH	1	Rigid	93.88	52.12	119.38	52.88
FIFTH	1	Rigid	96.03	54.85	119.28	53.08
FOURTH	1	Rigid	98.26	57.28	119.36	53.20
THIRD	1	Rigid	100.72	59.70	119.58	53.68
SECOND	1	Rigid	102.37	60.75	118.70	53.90
FIRST	1	Rigid	102.73	60.82	118.87	54.26

Level	Diaph. #	Type	Story Lateral Stiffness KX kips/ ft	KY kips/ ft
ROOF	1	Rigid	11391.61	6291.37
SIXTH	1	Rigid	16239.30	9404.00
FIFTH	1	Rigid	19122.90	11494.65
FOURTH	1	Rigid	27700.74	16871.45
THIRD	1	Rigid	32360.50	20657.14
SECOND	1	Rigid	35050.37	22413.24
FIRST	1	Rigid	20382.65	12736.32

RAM ECCENTRIC DISPACEMENT

Story Displacements

RAM Frame v14.05.03.00
DataBase: Eccentric 04/06/14 22:16:18
Building Code: IBC

CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Rfp	PosRoofLiveLoad	RAMUSER
W1	WIND	Wind_ASCE710_1_X
W2	WIND	Wind_ASCE710_1_Y
W3	WIND	Wind_ASCE710_2_X+E
W4	WIND	Wind_ASCE710_2_X-E
W5	WIND	Wind_ASCE710_2_Y+E
W6	WIND	Wind_ASCE710_2_Y-E
W7	WIND	Wind_ASCE710_3_X+Y
W8	WIND	Wind_ASCE710_3_X-Y
W9	WIND	Wind_ASCE710_4_X+Y_CW
W10	WIND	Wind_ASCE710_4_X+Y_CCW
W11	WIND	Wind_ASCE710_4_X-Y_CW
W12	WIND	Wind_ASCE710_4_X-Y_CCW
E1	SEISMIC	EQ_ASCE710_X_+E_F
E2	SEISMIC	EQ_ASCE710_X_-E_F
E3	SEISMIC	EQ_ASCE710_Y_+E_F
E4	SEISMIC	EQ_ASCE710_Y_-E_F

Level: ROOF, Diaph: 1

Center of Mass (ft): (119.89, 53.80)

LdC	Disp X in	Disp Y in	Theta Z rad
D	0.11820	-0.08342	0.00001
Lp	0.11016	-0.09004	0.00001
Rfp	0.00674	-0.00275	0.00000
W1	2.22230	-0.04378	-0.00024
W2	0.02240	3.28226	0.00041
W3	1.64713	-0.10994	-0.00054
W4	1.68632	0.04428	0.00018
W5	0.06176	2.65518	0.00121
W6	-0.02815	2.26822	-0.00059

RAM CONCENTRIC DISPLACEMENT

Story Displacements

RAM Frame v14.05.03.00
DataBase: Concentric3 04/06/14 23:34:16
Building Code: IBC
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CRITERIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Mesh Criteria :
Max. Distance Between Nodes on Mesh Line (ft) : 4.00
Merge Node Tolerance (in) : 0.0100
Geometry Tolerance (in) : 0.0050
Walls Out-of-plane Stiffness Not Included in Analysis.
Sign considered for Dynamic Load Case Results.

LOAD CASE DEFINITIONS:

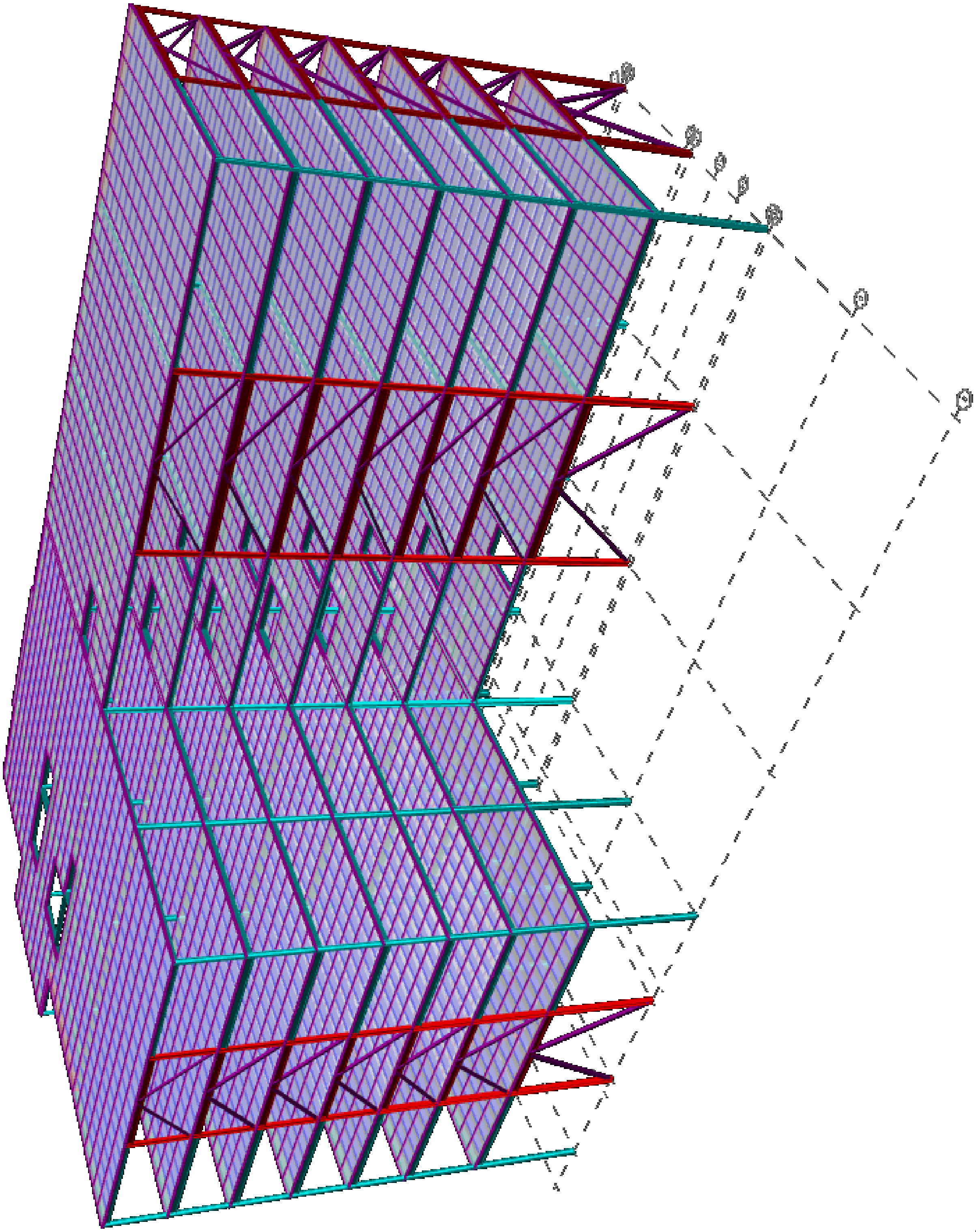
D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Rfp	PosRoofLiveLoad	RAMUSER
W1	WIND	Wind_ASCE710_1_X
W2	WIND	Wind_ASCE710_1_Y
W3	WIND	Wind_ASCE710_2_X+E
W4	WIND	Wind_ASCE710_2_X-E
W5	WIND	Wind_ASCE710_2_Y+E
W6	WIND	Wind_ASCE710_2_Y-E
W7	WIND	Wind_ASCE710_3_X+Y
W8	WIND	Wind_ASCE710_3_X-Y
W9	WIND	Wind_ASCE710_4_X+Y_CW
W10	WIND	Wind_ASCE710_4_X+Y_CCW
W11	WIND	Wind_ASCE710_4_X-Y_CW
W12	WIND	Wind_ASCE710_4_X-Y_CCW
E1	SEISMIC	EQ_ASCE710_X_+E_F
E2	SEISMIC	EQ_ASCE710_X_-E_F
E3	SEISMIC	EQ_ASCE710_Y_+E_F
E4	SEISMIC	EQ_ASCE710_Y_-E_F

Level: ROOF, Diaph: 1

Center of Mass (ft): (125.69, 54.32)

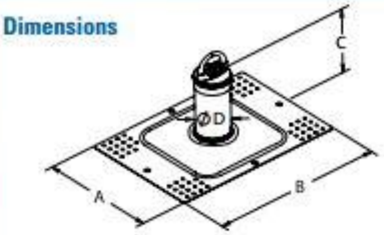
LdC	Disp X in	Disp Y in	Theta Z rad
D	0.02521	0.18049	0.00002
Lp	0.02858	0.13737	-0.00001
Rfp	0.00278	0.00799	-0.00000
W1	0.52800	0.02869	-0.00007
W2	0.09243	1.52773	0.00014
W3	0.39798	-0.00453	-0.00012
W4	0.39402	0.04758	0.00001
W5	0.06379	1.21241	0.00028
W6	0.07487	1.07918	-0.00007

RAM MODEL OVERVIEW



APPENDIX F

Dimensions



<table>
<tr><th>SKU</th><th>WIDTH
A</th><th>LENGTH
B</th><th>HEIGHT
C</th><th>POST DIA.
D</th></tr>
<tr><td>X10000
X10010</td><td rowspan="3">15.25 in.
(387 mm)</td><td>18.0 in.
(457 mm)</td><td rowspan="2">8.6 in.
(218 mm)</td><td rowspan="4">4.0 in.
(102 mm)</td></tr>
<tr><td>X10001
X10011
X10020
X10040
X10050</td><td rowspan="2">22.0 in.
(559 mm)</td></tr>
<tr><td>X10030
X10031</td><td>9.0 in.
(229 mm)</td></tr>
<tr><td>X10002</td><td>15.6 in.
(396 mm)</td><td>26.0 in.
(660 mm)</td><td>9.56 in.
(243 mm)</td></tr>
</table>

Specifications

Roof Anchor Post Materials

Energy Absorber:	Stainless Steel
Internal Connecting Components:	Stainless Steel
Top and Bottom Post Plates:	Anodized Cast Aluminum
Standing Seam/Wood/Metal Base Plate:	Two-layer Zinc/Powder-Coated Steel
Post Tube:	Zinc/Powder-Coated Steel
Post/Base Plate Seat:	HDPE
Post Cap:	Vinyl w/UV Inhibitor

Connection Components Materials

Standing Seam Clamps:	Anodized Aluminum/Stainless Steel
Extender Bar for Standing Seams:	Anodized Aluminum/Stainless Steel
Hardware for Metal Sheathing:	Hot Dip Galvanized/Neoprene
Hardware for Membrane:	Zinc-Plated Steel/PVC/Neoprene
Hardware for Wood:	Zinc-Plated Steel
Hardware for Concrete:	Stainless Steel

Performance

Activation Force:	1000 lbs. *(4.4 kN)*
Maximum Capacity:	310 lbs. *(140.6 kg)*

Fusion Roof Anchor Post

SKU	Description	Designed to Accomodate
STANDING SEAM ROOFING – Includes post with base and standing seam clamping assembly kit		
X10000	Small base	Standing seam spacing from 11.75 in. *(298 mm)* to 17 in. *(432 mm)*
X10001	Large base	Standing seam spacing from 11.75 in. *(298 mm)* to 21.25 in. *(540 mm)*
X10002	Large base & extension bars	Standing seam spacing from 11.75 in. *(298 mm)* to 24 in. *(610 mm)*
METAL SHEATHING ROOFING – Includes post with base and rivet kit with sealing washers and mastic tape		
X10010	Small base	Metal sheathing w/minimum thickness of 24 gauge *(0.024 in. [0.61 mm])*
X10011	Large base	Metal sheathing w/minimum thickness of 24 gauge *(0.024 in. [0.61 mm])*. Trapezoidal spacing of 8 in. *(203 mm)* to 20 in. *(508 mm)* in one-inch *(25.4 mm)* increments.
MEMBRANE/BUILT-UP ROOFING – Includes post with base and toggle bolt kit		
X10030	Up to 5.5 in. *(140 mm)* thickness	Fastens through membrane, insulation & into metal sheathing, wood sheathing or concrete with a combined thickness of up to 5.5 in. *(140 mm)*
X10031	> 5.5 in. *(140 mm)* & up to 10.5 in. *(267 mm)* thickness	Fastens through membrane, insulation & into metal sheathing, wood sheathing or concrete with a combined thickness of > 5.5 in. *(140 mm)* up to 10.5 in. *(267 mm)*
WOOD SHEATHING (TEMPORARY INSTALLATIONS ONLY) – Includes post with base and lag screw kit		
X10040	Wood sheathing	Plywood with minimum thickness of 5/8-in. *(15.9 mm)* CDX
CONCRETE ROOFING – Includes post with base and concrete expansion bolt anchor kit		
X10050	Concrete	Concrete decking with minimum thickness of 6.5 in. *(165 mm)* & minimum concrete compressive strength of 3000 PSI *(20.7 MPa)*
MULTI-PURPOSE METAL SHEATHING, WOOD AND CONCRETE ROOFING (NO HARDWARE INCLUDED) – Includes post with base. Hardware selection is based on the application. See instruction manual for hardware specifications.		
X10020	Metal sheathing, wood or concrete	• Metal sheathing w/minimum thickness of 24 gauge *(0.024 in. [0.61 mm])* • Trapezoidal spacing of 8 in. *(203 mm)* to 20 in. *(508 mm)* in one-inch *(25.4 mm)* increments. • Plywood with minimum thickness of 5/8-in. *(15.9 mm)* CDX • Concrete decking with minimum thickness of 6.5 in. *(165 mm)* & minimum concrete compressive strength of 3000 PSI *(20.7 MPa)*

Meets or exceeds all applicable industry standards including OSHA, ANSI A10.32 and Z359.1-2007.

⚠ *This equipment should only be used after reading the manufacturer's instructions. Failure to follow instructions could result in serious injury or fatality.*

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