Reinsurance Group of America (RGA) Global Headquarters

Spring 2014

Final Report

16600 Swingley Ridge Rd.
Chesterfield, MO

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Reinsurance Group of America (RGA) Global Headquarters

16600 Swingley Ridge Rd. Chesterfield, MO

Project Team

Owner: Reinsurance Group of America, Inc.
Owner Representative: Gateway Ridge LLC
General Contractor: Clayco
Architect: Gensler
Structural Engineer: Uzun & Case
Civil Engineer: Stock & Associates, Inc.
Landscape Architect: Forum Studio
Lighting Consultant: Randy Burkett Lighting Design, Inc.
MEP & Fire Protection: Environmental Systems Design, Inc.

Building Information

Occupancy: General office and training
Size: 405,000 gross square feet
Total Estimated Cost: $150 million
Project Delivery: Design-Build

Structural

- Two, 5 story steel office towers with composite floors with 3 1/2" semi-lightweight concrete topping
- Upper four levels cantilever 40' over the first level supported by a steel truss system
- Office towers have braced frame lateral system while parking garage utilizes reinforced concrete shear walls
- Parking garage is post-tensioned, reinforced concrete
- Drilled concrete piers 36" to 78" in diameter with allowable end bearing pressure of 80 ksf

Architecture

- Two skewed, 5 story office towers with curtain wall façades linked by an amenities level
- Open plan office towers with central core
- Amenities include kitchen, fitness center, café and landscaped terrace
- Two story underground parking garage with limestone façade where exposed
- Three landscaped bio-retention basins
- Designed to achieve LEED Silver

Mechanical

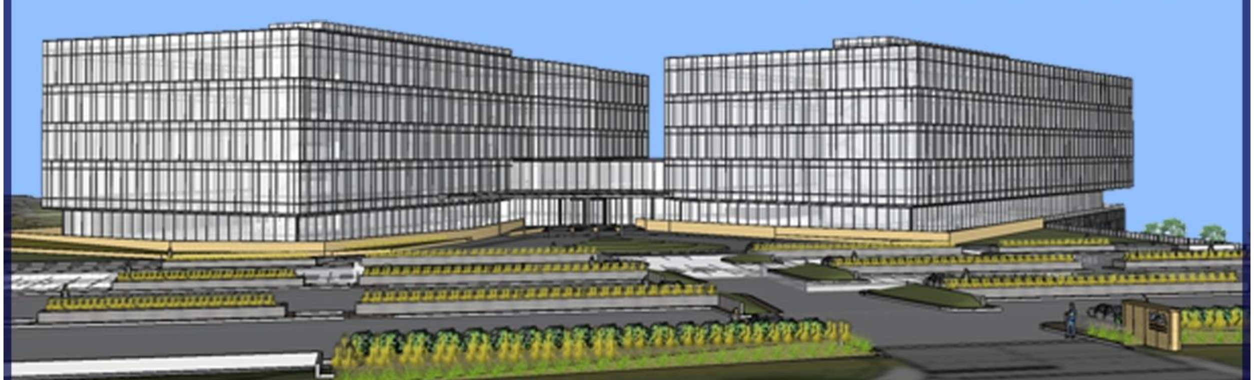
- Designed for year-round cooling
- Cooling towers serve three, 350 ton water cooled chillers
- Four 60,000 CFM air handling units serve the office towers
- A medium pressure loop on each floor for VAV branches
- Separate fan powered terminal units (FPTU) heat the floor cavity of the cantilever space counteracting the heat sink

Electrical

- Mechanical, lighting serviced by 480/277 volt system
- Office receptacles serviced by 208/120 volt system
- Both systems are fed by 3-phase, 4-wire buses
- Four main switchboards rated at 3000 amperes
- Diesel generator serves emergency equipment

Lighting & Controls

- Occupancy sensors in restrooms
- Exterior and restroom lighting fixtures on 277 volts
- Fluorescent lamps and LED lamps specified to date
- Interior lighting design is in the final design stages



Natasha Beck | Structural http://www.engr.psu.edu/ae/thesis/portfolios/2014/nmb5163/index.html

Acknowledgements

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StarSeismic, BRB Information

Samantha Bollinger, Horticulture Consultant

Deborah Dudenhoefer, Architecture Consultant

My family and friends for their continuous support

God for His grace and provision

Executive Summary

The Reinsurance Group of America's (RGA) Global Headquarters is located in Chesterfield, Missouri. The complex consists of two, five story office towers framed in steel with glass curtain wall façades and a two story, partially underground parking garage of post-tensioned reinforced concrete with a limestone panel façade. The lateral system consists of steel concentric braced frames in the office towers which change to reinforced concrete shear walls in the parking garage. Four of the five stories of the office towers are cantilevered over the first floor by five feet on three of the four sides and by forty feet on the fourth side. Housing a Fortune 500 company, the complex is meant to represent RGA's local and global presence and is designed for a LEED Silver Core and Shell Certification.

Purpose and Scope

The purpose of this report is to present in detail the analysis and design outcomes of the green roof garden amenity area addition on each steel office tower. This report contains an overview of the as built project's characteristics and structure and moves into detailed redesign calculations, considerations and comparisons for the green roof addition. Finally, supplemental material such as technical information and detailed calculations are provided in appendices. The investigation's scope is limited to the South Office Tower and parking garage structure below it due to time constraints.

First, the green roof garden breadth study is presented where the design outcome and considerations are discussed. Considerations included planting selection, code requirements, system selection, ASTM standards, public access, and aesthetics. Next, a structural depth study was performed on the gravity and lateral system using the structural considerations and revised weights of the green roof addition. The gravity cantilever truss system affected by this change was analyzed and redesigned for new loading and deflection limits. The roof system was redesigned as a composite steel system and the roof framing was redesigned considering composite action. After studying the gravity system, the lateral system was changed from conventional braced frames to buckling-restrained braced frames and designed. ETABS models were created for the roof system, the three gravity trusses, and the lateral system of the office structure to assist in the calculations. Finally, a construction breadth study was conducted in which a cost analysis and schedule analysis for each project option and their outcomes were compared to determine the additional cost and time the green roof garden will add to the project.

The results of this report show that adding a green roof garden is feasible for this project and the most critical factor in the decision for the owner is the additional project cost. Although adding a green roof will add almost two months to the schedule of each office tower, none of those activities lay on the critical path since construction on both towers overlap. The outcome of the lateral analysis showed that buckling-restrained brace frames can work for this project, but they are not the best choice over conventional braced frames. This is because the higher green roof mass required the highest yield strength available and almost the highest steel core area manufactured. Overall, the result of this investigation concludes that a green roof garden is feasible and that the lateral system should remain conventional braced frames.

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Building Project Background

The Reinsurance Group of America's Global Headquarters serves as an office and training facility for RGA- a Fortune 500 Company. This building complex features two office towers enclosed by curtain wall façades with a lobby and amenities space linking the two towers, see Figure 1. Inside, the office towers have an open floor plan with a centrally located core that maximizes tenant circulation through the building, flexibility, and functionality within the space. From the highway on the lower side of the site, the two parking garage levels are visible. On the opposing side, these levels are below grade, allowing for a third level of on-grade parking and fire truck access.

Figure 1: Rendering from Highway, Courtesy Gensler

Construction on this 405,000 square foot, $150 million project started in March 2013 and will continue until its expected completion in September, 2014. A Phase Two plan has been developed for the addition of a third office tower similar to the Phase One towers with additional parking to service the new tower. The site, seen below in Figure 2, features three bio-retention basins along the highway. This Design-Build project, at the request of the owner, utilized the LEED Silver Accreditation standards for the core and shell as a design basis. Finally, in Figure 3 the location and vicinity plans by Gensler give a broader context of the site location within Missouri.

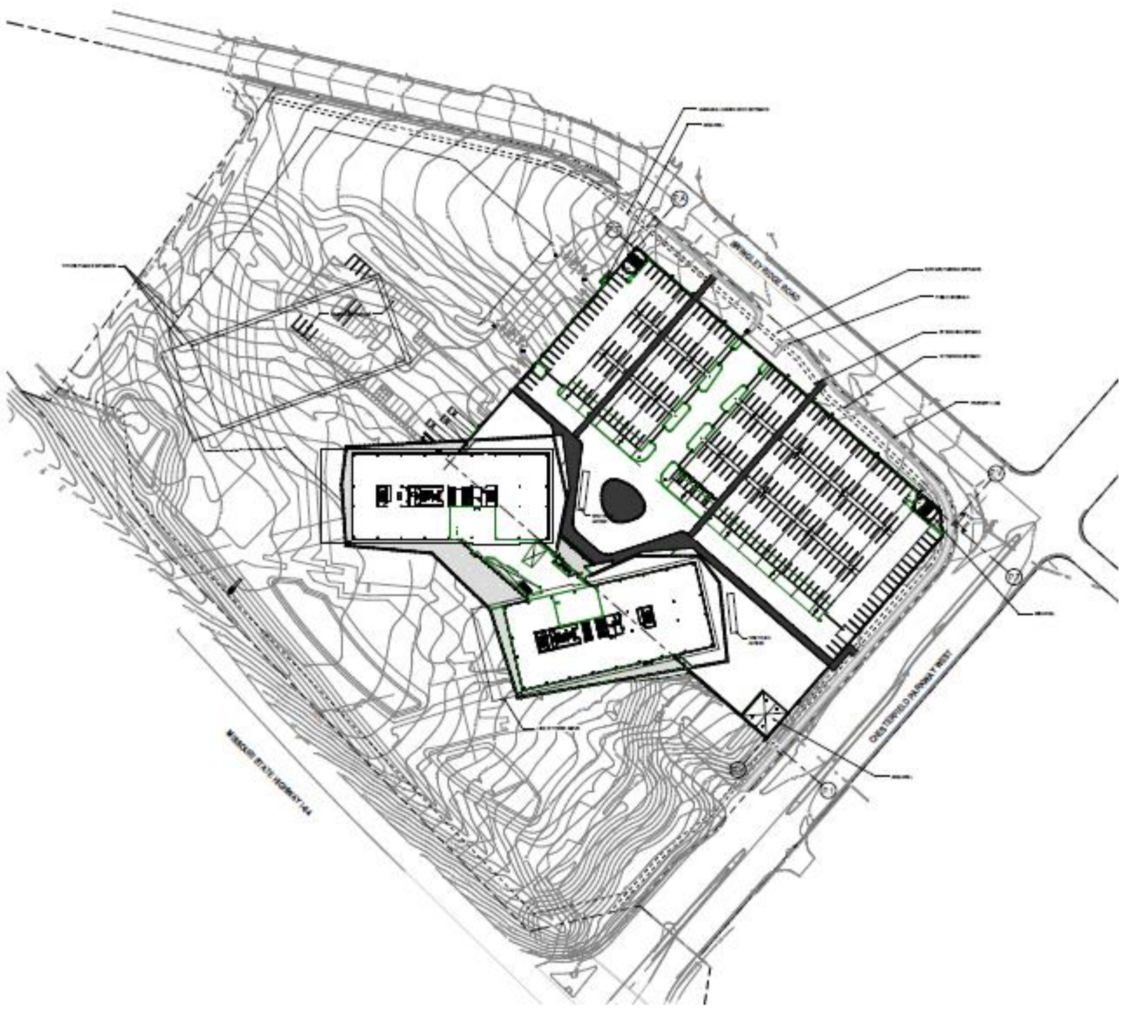
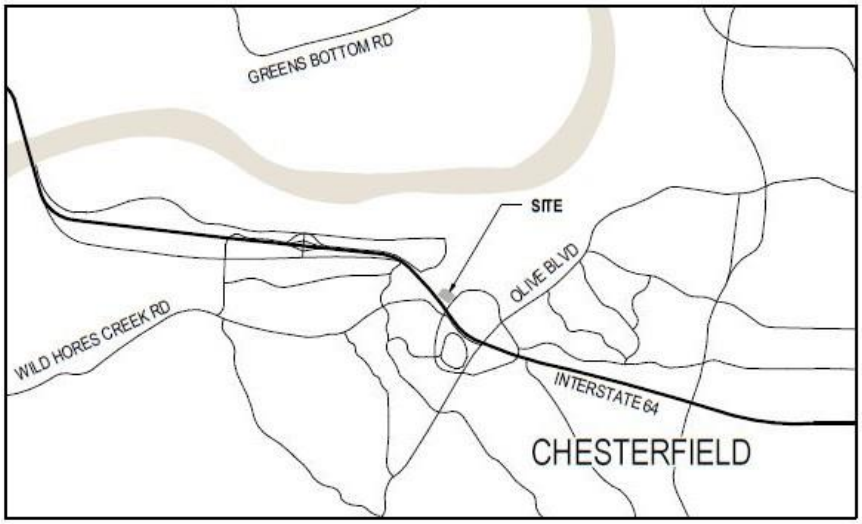


Figure 2: Site Plan Oriented to True North (Construction Documents)

Vicinity and Location Plans

VICINITY MAP



LOCATION PLAN

Figure 3: Vicinity and Location Plans by Gensler

Design Codes and Resources

Listed below are the codes and resources used in preparation of this report.

- *RGA Core and Shell Addendum A* Design Documents by the Project Team (See Abstract)
- Minimum Design Loads for Buildings and Other Structures, ASCE 7-05
- AISC Steel Construction Manual, AISC 360-10
- Seismic Provisions for Structural Steel Buildings, AISC 341-05 Section 16
- Vulcraft Composite Deck Tables
- Vulcraft Steel Roof and Floor Deck Tables
- *RSMeans Green Building Cost Data 2011*
- *RSMeans Building Construction Cost Data 2012*
- ANSI/GRHC SPRI/VR-1 Procedure for Investigating Resistance to Root Penetration on Vegetative Roofs, 2011
- ANSI SPRI/RP-14 Wind Design Standard for Vegetative Roofing Systems, 2010
- ANSI/SPRI VF-1 External Fire Design Standard for Vegetative Roofs, 2010
- ASTM E2396-11 Standard Test Method for Granular Drainage Media
- ASTM E2397-11 Determination of Dead and Live Loads of Green Roof Systems
- ASTM E2398-11 Standard Test Method for Media Retention of Water
- ASTM E2399-11 Media Dead Load Analysis of Green Roof Systems
- ASTM E2400-11 Standard Guide for Selection, Installation, and Maintenance of Plants for Green Roof Systems
- OSHA 1926.502 Fall Protection Systems Criteria and Practices
- Underwriter Laboratories Fire-Resistance Rated Assemblies
- Engineering News Record
- Bucking Restrained Braces Article by StarSeismic
- Bucking Restrained Braces Webcast
- StarSeismic, http://www.starseismic.net/
- "StarSeismic Buckling Restrained Braces in ETAS Integrated Building Design Software"
- *Unified Design of Steel Structures* by Louis Geschwindner
- United States Department of Agriculture Plant Hardiness Zone Maps
- Roofmeadow, http://www.roofmeadow.com/
- *Green Roof Plants*
- *The Green Roof Manual*
- *The Professional Design Guide to Green Roofs*
- *Managing the Construction Process: Estimating, Scheduling, and Project Control*
- *Award Winning Green Roof Designs: Green Roofs for Healthy Cities*
- "Challenges to Green Roof Construction"
- *Green Roofs* by Albert Jarrett
- "Green Roofs" in *Reducing Urban Heat Islands: Compendium of Strategies*

Design Codes

Listed below are the design codes and reference standards used for the design of RGA Global Headquarters. Structurally, the chosen design method is Load and Resistance Factor Design (LRFD).

Building: International Building Code, IBC 2009 amended by Ordinance 24, 444-2010

State/County: St. Louis County Ordinances

Structural: American Society of Civil Engineers, ASCE 7-05
American Concrete Institute, ACI 318-08
American Institute of Steel Construction, AISC 360-05
Masonry: ACI 530/ASCE 5/TMS 402-08

Mechanical: International Mechanical Code, IMC 2009

Electrical: National Electrical Code, NEC 2008

Plumbing: Uniform Plumbing Code, UPC 2009

Energy: International Energy Conservation Code, IECC 2009

Design codes listed below are those used in thesis study if they differ from above:

American Concrete Institute, ACI 318-11
American Institute of Steel Construction, AISC 360-10

Complete Citations

Bentley Structural Webcast Series: Design and Specification of Buckling-Restrained Braced Frame Structures Part 2. ZweigWhite, 7 Nov. 2013. Web. 7 Nov. 2013. <http://continuingeducation.zweigwhite.com/webcasts>.

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Gould, Frederick E. *Managing the Construction Process: Estimating, Scheduling, and Project Control*. 4th ed. Boston: Prentice Hall, 2012. Print.

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US Dept. of Agriculture. "USDA Plant Hardiness Zone Map." Map. *Agricultural Research Service*. N.p.: US Dept. of Agriculture, n.d. N. pag. Print.

Project Scope

Due to time constraints, it was necessary to narrow the scope of my studies. Figure 4 below shows the structural expansion joints in black for the parking garage levels which divide the parking structure into four separate structures. The area shaded in blue and the corresponding steel office tower above that were selected for in depth study. This portion was selected because the plans steel towers are mirrors of each other, so only one need be considered. Additionally, the parking structure portion shown in blue has more straight forward geometry than its counterpart and will allow for efficient structural study. In the interest of time, the post-tensioned parking structure was not studied in depth, but the shear walls and foundation walls were included as part of the lateral analysis of Technical Report 4. For the spring semester the steel office tower was the focus for my depth and breadth studies.

Figure 4: Project Area Considered Shown on Parking Garage Plan

Structural System Overview

RGA Global Headquarters has two five story, steel and curtain wall office buildings with mirrored, rectangular floor plans. Floors two through five are cantilevered 5' over the first floor on three sides and 40’ on the remaining side. A truss system bearing on a built up-plate girder supports the large cantilever. All exposed steel is finished as Architecturally Exposed Structural Steel (AESS) at the owner's request. The office buildings have a braced frame lateral system that transfers load into concrete shear walls in the below grade parking garage. Post-tensioned one-way slab systems supported by post-tensioned concrete beams comprise the parking garage's structure and support the loading above at the parking levels. The foundation consists of grade beams supported by concrete drilled piers, with the exception of a portion of the site where the bedrock rises to meet the parking garage; there the foundation is a rock bearing spread footing. This section of the report will provide more detail into these systems.

Foundation

A geotechnical report was conducted by SCI Engineering, Inc. in October, 2012, as a follow-up to their report done in January, 1999. Based on their findings, SCI Engineering recommended use of a combination of drilled pier foundations, rock bearing shallow foundations, aggregate piers, and shallow foundations as suitable. Predominant soils in the area were the topsoil, clays, shale, an area of unknown infill, and bedrock with groundwater appearing about 37' to 60' below the existing grade.

Drilled piers are the predominant foundation system selected, bearing on bedrock, with an allowable end bearing pressure of 80 ksf and a concrete compressive strength of 3,000 psi. Pier diameters range from 36" to 78" with Pier caps are typically 3' to 4' in depth. When tension piers are required, rock anchors with a 150 ksi minimum ultimate tensile strength are embedded a minimum of 10' into the limestone bedrock and lapped with vertical reinforcement. Tension piers most commonly support the lateral system and an overall detail is shown below in Figure 5. The rock bearing spread footings are designed for an 8,000 psf net allowable bearing pressure and soil beneath these footings is replaced with 2,000 psi lean concrete. In the case of a footing bearing on soil, a net allowable bearing capacity 2,500 psf is recommended.

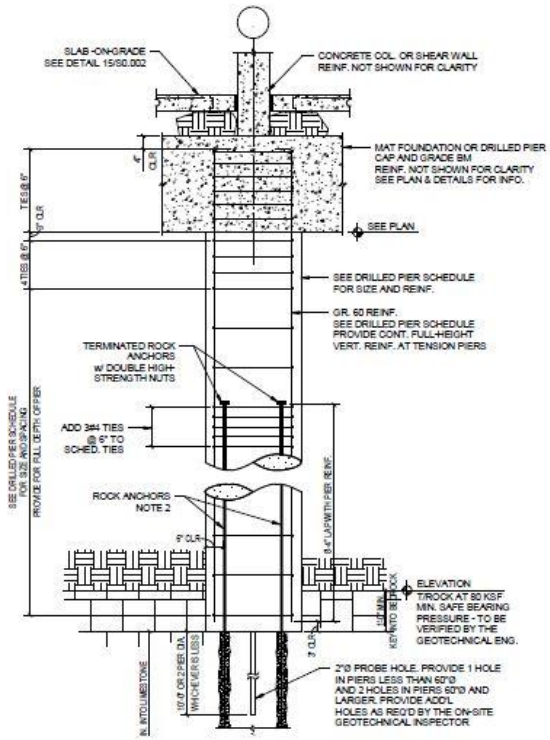


Figure 5: Typical Tension Pier Detail (Construction Documents)

The final component of the foundation system is the grade beams. They are typically 4,000 psi concrete ranging in size from 18"x18" to 42"x24" with several combinations in between. Reinforcement is Grade 60 and ranges from #8 bars through #11 bars with #4 stirrups. Figure 6 shows a typical detail.

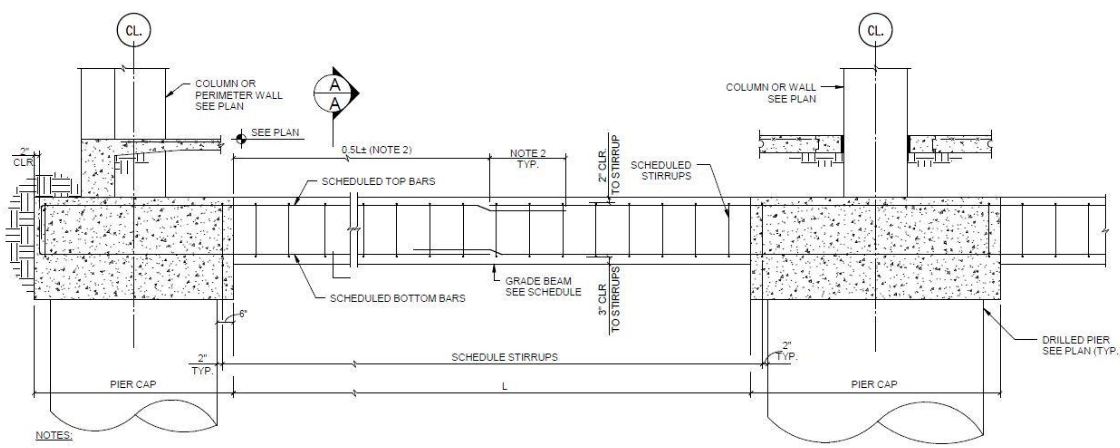


Figure 6: Typical Grade Beam Detail (Construction Documents)

Substructure

The lowest level of the parking garage is a slab on grade supported by grade beams. For the parking garage, the slab is 5" thick of 3,500 psi concrete placed on compacted subgrade. Mechanical rooms, loading docks and truck service area slabs on this level are 6" thick. Concrete exterior walls on this level are typically 16" thick.

The floor of the upper parking level increases in thickness to 7" and the floor system changes to a 5,000 psi concrete post-tensioned, one-way slab system supported by post-tensioned reinforced concrete beams. Exterior exposed concrete walls are 8" thick and increase to 12" when they are exposed to earth, below level 01 on the higher side of the site. The slab of the parking plaza, the on-grade level of parking, is also a post-tensioned one-way slab system supported by post-tensioned beams. The difference lies in the parking plaza's slab thickness. If there is no fire truck access, the slab is 8 1/2" thick and slabs with fire truck access areas are 9 1/2" thick.

Columns in the parking garage are typically 5,000 psi concrete. There is an exception of four columns of 7000 psi concrete that are continuations of the columns supporting the plate girder and compression members of the cantilever truss system. Square or rectangular column sizes range from 16"x16" up through 32"x32" with a common size of 24"x24" and circular columns range from a 24" diameter to a 36" diameter with the most common diameter being 28". Vertical reinforcement ranges from #8 to #11 bars in these columns.

Superstructure

This section discusses typical bay characteristics and area-specific characteristics that cause the bay configuration in that area to differ from the typical bay. A representative full structural framing plan for the superstructure can be found in Appendix A: Additional Plans

Typical Bay Characteristics

In a typical bay, gravity columns are A992 Grade 50 steel with typical sizes of W10x49, W12x65, W12x79, W12x87, W12x136 on lower levels and W12x65, W12x58, W12x53 on upper levels. When necessary, column splices occur 4' above Level 04. Beam sizes are discussed below. Bays are based on a 30' or 40' length and either a 25' width or a 40' width as shown in Figure 7.

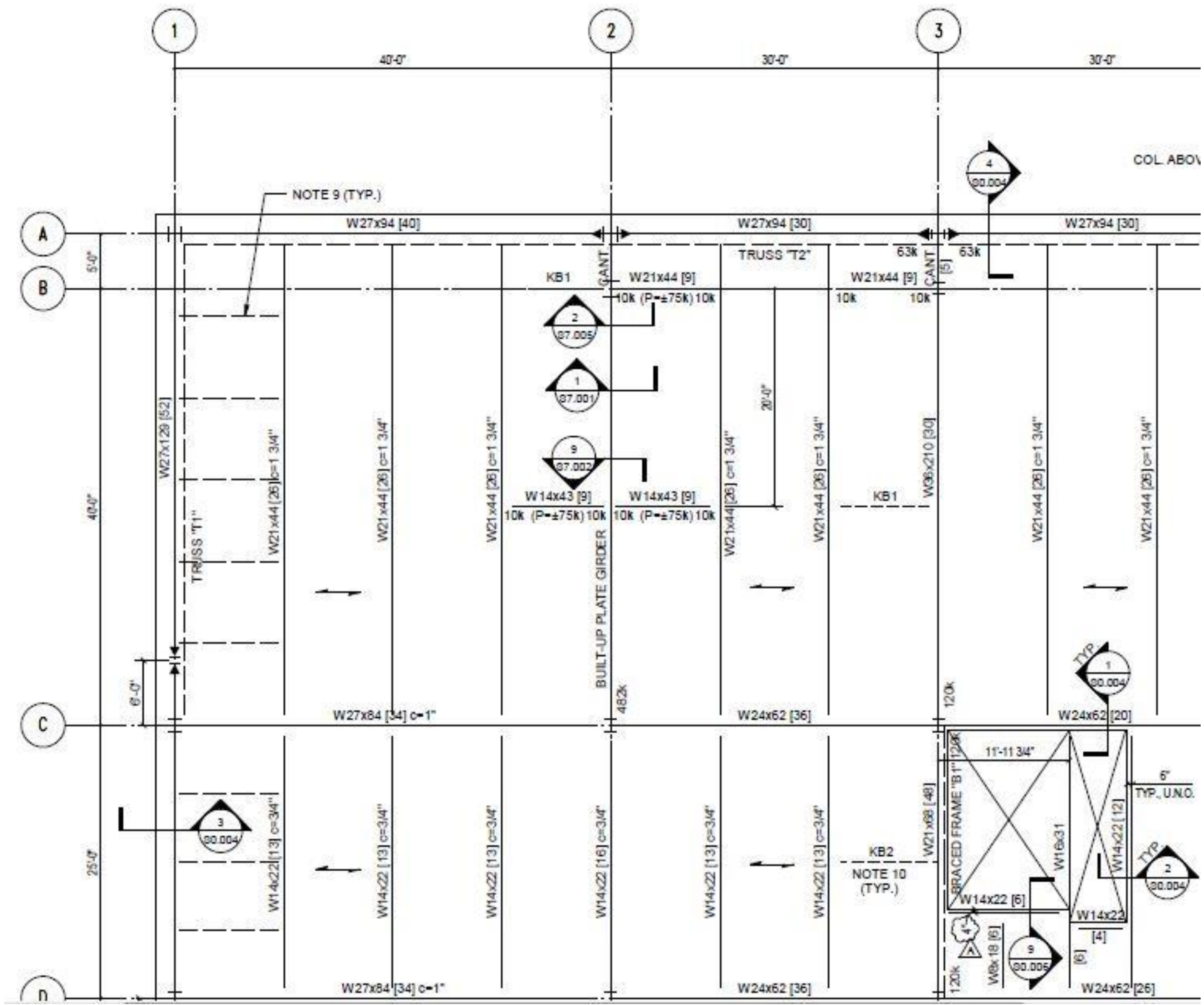


Figure 7: Partial Plan of Typical Bay Dimensions (Construction Documents)

Base plates are A36 steel and range in thickness from 1" to 2 3/4". Gravity column bases anchor into the foundation with four Grade 55 anchor rods with diameters of 3/4" to 1" and embedded a minimum of 1'. This connection type does not resist significant rotation, so the connection is a pinned base. Typical moment connections consist of a 3/8" minimum shear tab with 5/16" fillet weld to the beam flange and 3/4" diameter A325 slip critical bolts the full length of the shear tab. The flanges are field welded with a full penetration bevel weld with backing.

Area-Specific Characteristics

The floor system on Level 01 of the office structures has multiple sections. Where the office superstructure overlaps the parking structure, the floor is an overbuilt 4" thick, 3,000 psi semi-lightweight concrete slab reinforced with welded wire fabric 1" from the top of the slab. Where the superstructure does not overlap, the floor is a 25" deep pan joist system consisting of a 5" slab and 20" deep pans spaced a maximum of 6' center to center. Typical pan joists are 6" wide at the bottom and have bottom reinforcing ranging from #5 to #9 bars usually in a combination of sizes and top reinforcement sizes are #4 through #6 bars. Pan joists are supported on 25" deep post-tensioned or reinforced concrete beams. In the terrace area, the system changes to a one-way slab supported on concrete beams to support the extra dead load associated with the landscaping materials.

Levels 02 through 05 have a composite floor system consisting of 3" 20 gage galvanized type 3.0SB composite steel deck with 3 1/2" 3,000 psi semi-lightweight concrete topping for a 6 ½" total thickness. Shear studs in all composite floors are specified to be installed in the strong position. The slab is reinforced with welded wire fabric and is unshored during construction. The deck has a maximum span of 11'-9" for a three span condition. Typical beam sizes for these levels include typical interior girders of W24x62, typical perimeter girders of W21x50, and typical infill beams of W21x44 and W14x22 with cambers of 3/4" to 1 3/4". Beams are spaced evenly between columns where possible.

On Level 06, the roof deck is 3" 20 gage Type N composite deck. Typical framing sizes include typical interior girders of W21x50, typical exterior girders of W21x57, and typical infill beams of W21x44 and W12x19 cambered 3/4" were needed. Penthouse framing sizes are typicallyW16x26 girders and infill beams of W16x31 and W12x19 with the addition of C12x20.7 members that support roof davits.

Lateral System

In the steel superstructure, the lateral system is composed of ordinary concentric steel braced frames shown in Figure 9. A floor plan showing the locations of the braced frames is in Figure 8. Typical column sizes for the brace frames are W12x152, W12x136 and W12x120 for the first three stories and decreases to W12x87 for stories four and five with the column splices occurring 4'-0" above Level 04. Beams sizes in the braced frames are W24x84, W24x76, W24x68, W24x55, W21x68, W18x46, W18x35, W14x22 and W16x26. Larger beam sizes are in the lower levels of the braced frames and decrease in size moving upward. Bracing members range from HSS 6x6 to HSS 10x10 with thicknesses of ½" or 5/8" where, again, the larger braces are in the lower levels and decrease moving upward.

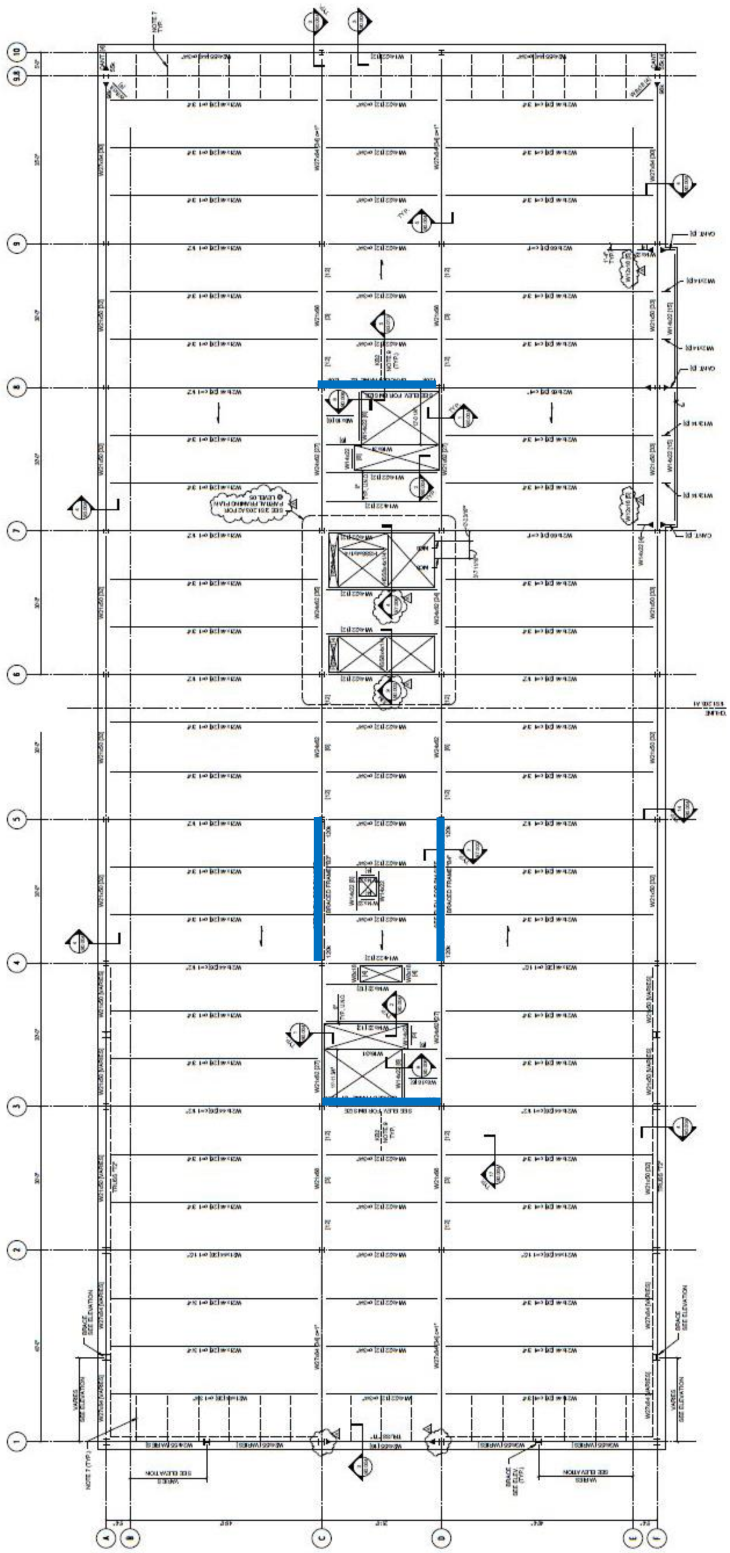


Figure 8: Braced Frame Locations

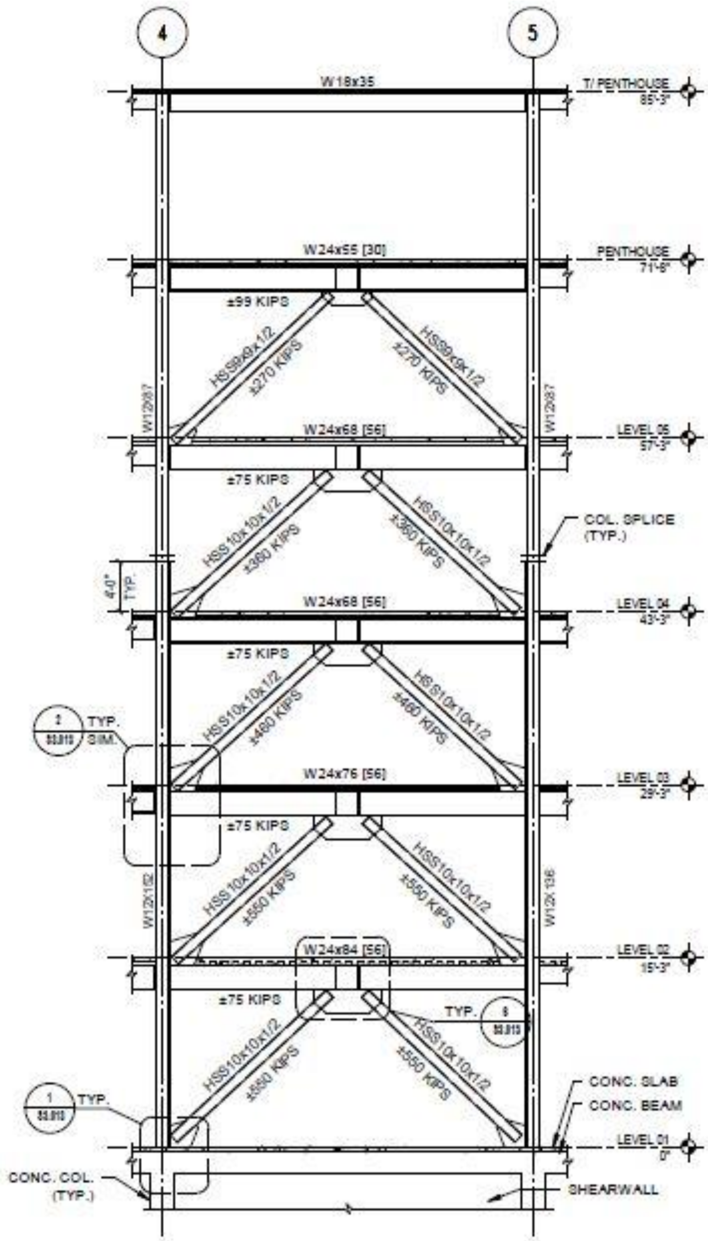


Figure 9: Typical Braced Frame Elevation with Penthouse Support Included (Construction Documents)

Additional floor diaphragm reinforcement is shown in Figure 10 below. The purpose for this additional reinforcement is to resist flexure the diaphragm, in plan, acts as a beam spanning between the supports of the braced frames. Reinforcement sizes for supplemental diaphragm reinforcement include #4, #5, and #6 bars.

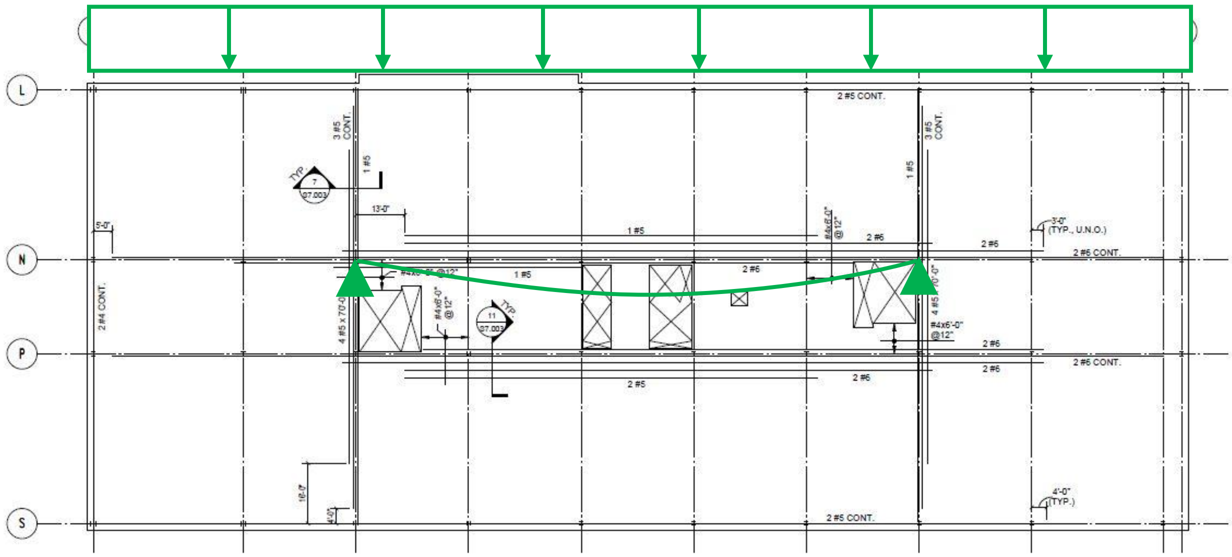


Figure 10: Floor Diaphragm Acting as a Beam Spanning Between Braced Frames

Moving down the building, the braced frames have a pinned base connection to the top of the shear walls. Brace members are welded to a gusset plate, which is welded to an embed plate. This plate, 3/4" thick, uses 3/4" diameter studs embedded into the concrete shear wall to transfer the horizontal forces from the braces into the shear wall. Column base plates are typically 3" thick made of A572 Grade 50 steel with 1 1/4" diameter, grade 105 anchor rods embedded 5' into the concrete column of the shear wall. The tensile and compressive loads are transferred into the shear wall through the base plate and anchor rods. Below in Figure 11 is a detail of this connection.

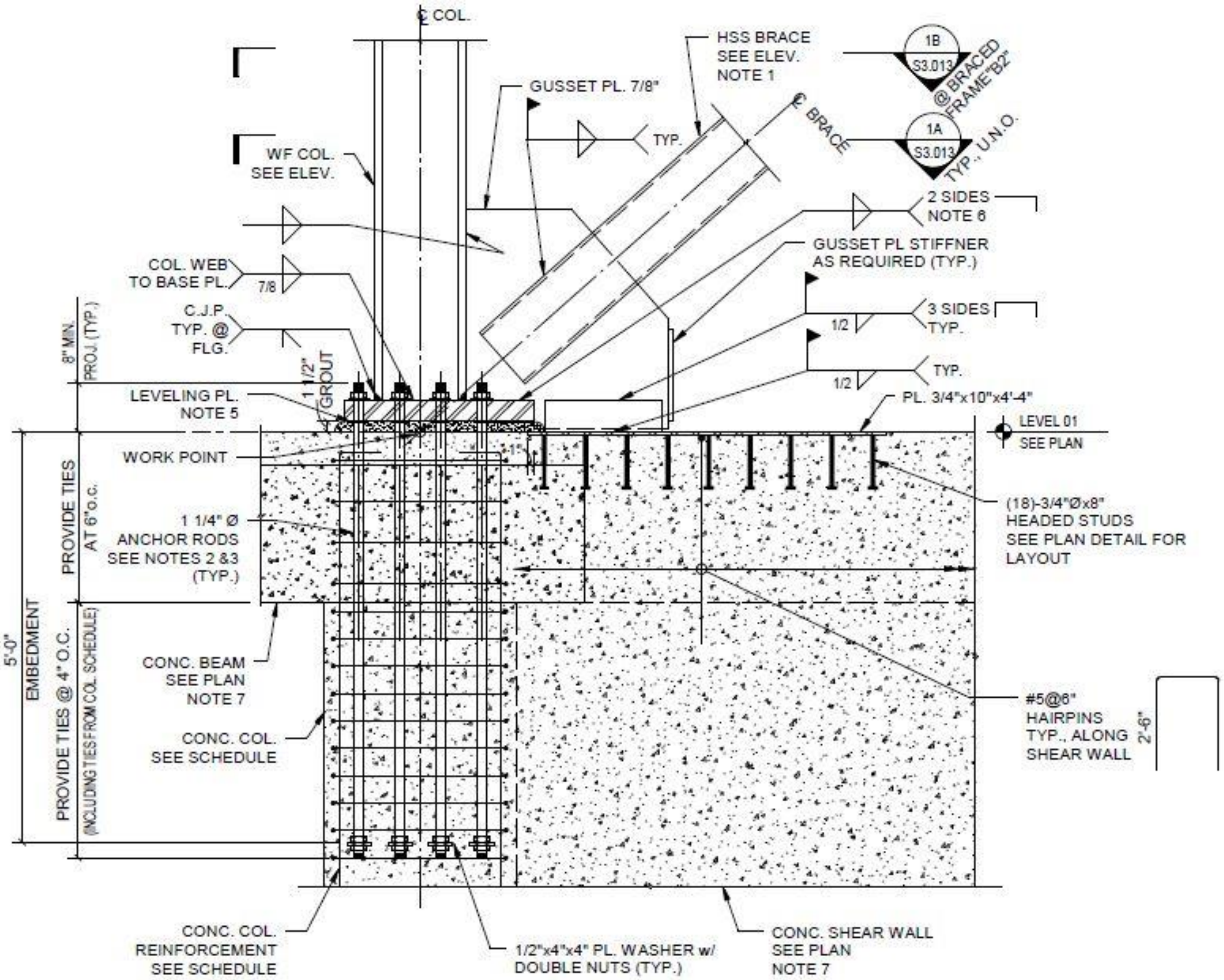


Figure 11: Typical Braced Frame to Shear Wall Connection (Construction Documents)

In the parking garage substructure, the braced frames are supported on 5,000 psi concrete shear walls. These shear walls are 16" thick with vertical reinforcement ranging from #6@12" o.c. to #10@9", 10", 12", or 13 o.c. bars and horizontal reinforcement of #5 bars at various spacing. Spacing varies based on floor levels and different walls. A sample plan of a shear wall is provided in Figure 12 below. These walls bear on grade beams which transfer the load to the foundation.

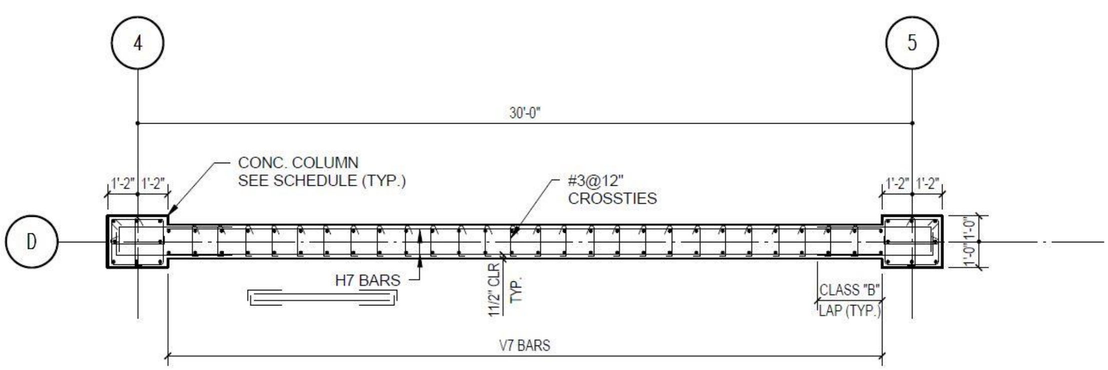


Figure 12: Shear Wall Sample Plan (Construction Documents)

Cantilever Truss System

Truss T2 is oriented along the longitudinal axis of the building. Two tension members in an inverted "V" and a vertical compression member are the main members of the system. T2 is supporting a 40' cantilever spanning from grid 1 to grid 2 in Figure 13 below. The most exterior tension member, running between grids 1 and 2, is designed for a tension load of 1544 kips and the back span diagonal, running from grids 2 to 4, is designed for a tension load of 1155 kips. Both tension diagonals are W14x176. The vertical compression member on grid 2, a W14x193, is designed for 2380 kips of compression load. These compression members on either side of the building bear on a built-up plate girder to be discussed later.

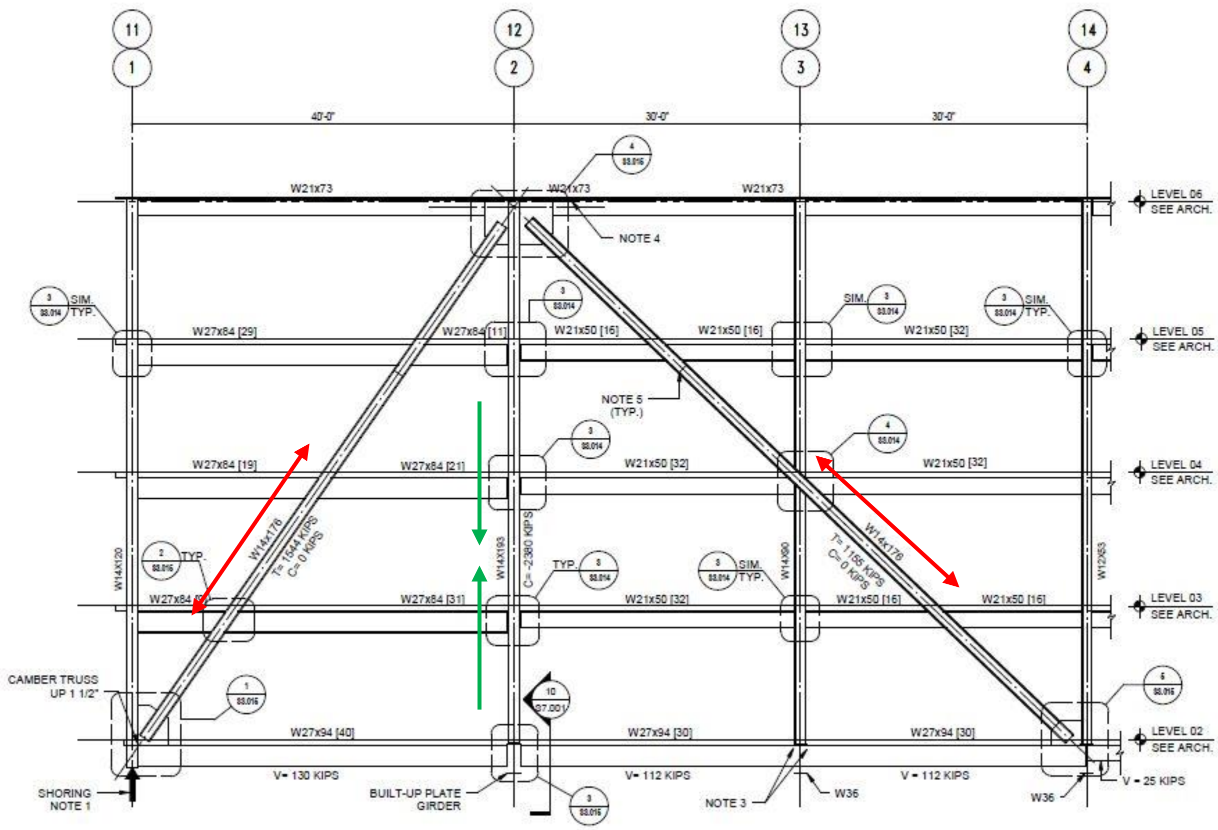


Figure 13: Truss T2 Elevation Highlighting Tensile and Compressive Forces

Truss T1, shown in elevation in Figure 14, is aligned in the transverse direction of the building consisting of W14x159 tension diagonals designed for a factored tension load of 891 kips. At the lower side of the tension members, the truss is cambered up 3/4" at Level 02 and grids N, P, C, and D. In terms of connections, the full moment splice has been offset from grid lines C and D to alleviate congestion at the column line and aid in constructability.

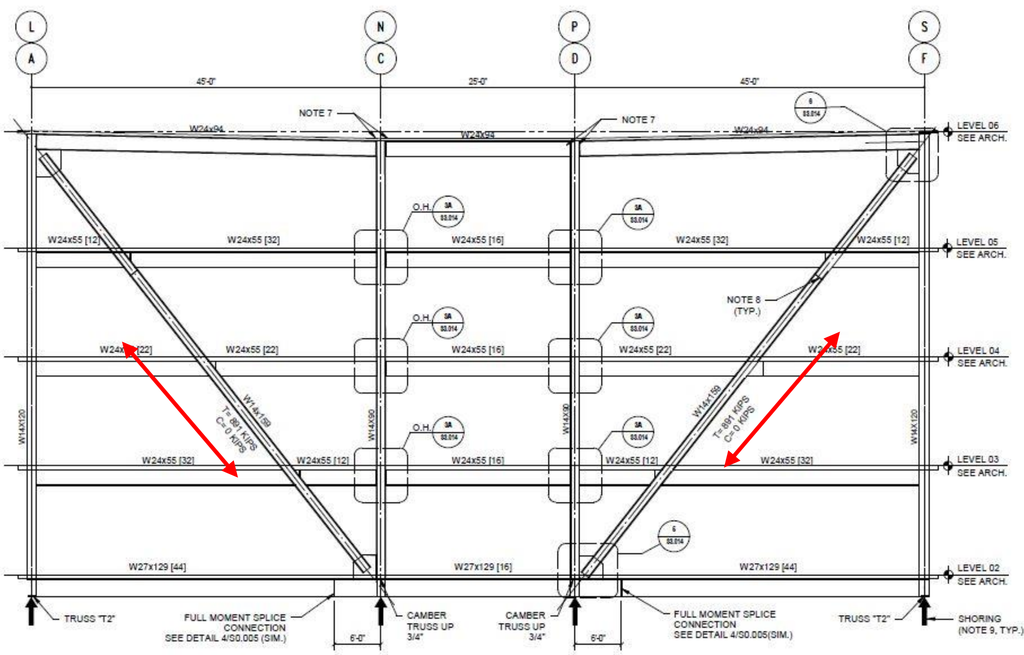


Figure 14: Truss T1 Elevation Highlighting Tensile Forces

To counteract the overturning of the cantilever, the beams on Level 06 are designed for axial tension starting where the exterior tension member of T2 meets the roof, circled in red in Figure 15 below. The truss overturning imposes axial tension loads on all beams going through the back span direction of the building, noted in red arrows in the diagram. The force decreases, or dissipates, as it moves away from the trusses. Under floor horizontal bracing, also designed for axial tension, starts where the exterior diagonal of truss T2 meets the roof which pulls the load toward the core and then follows the same horizontal path in plan through the building.

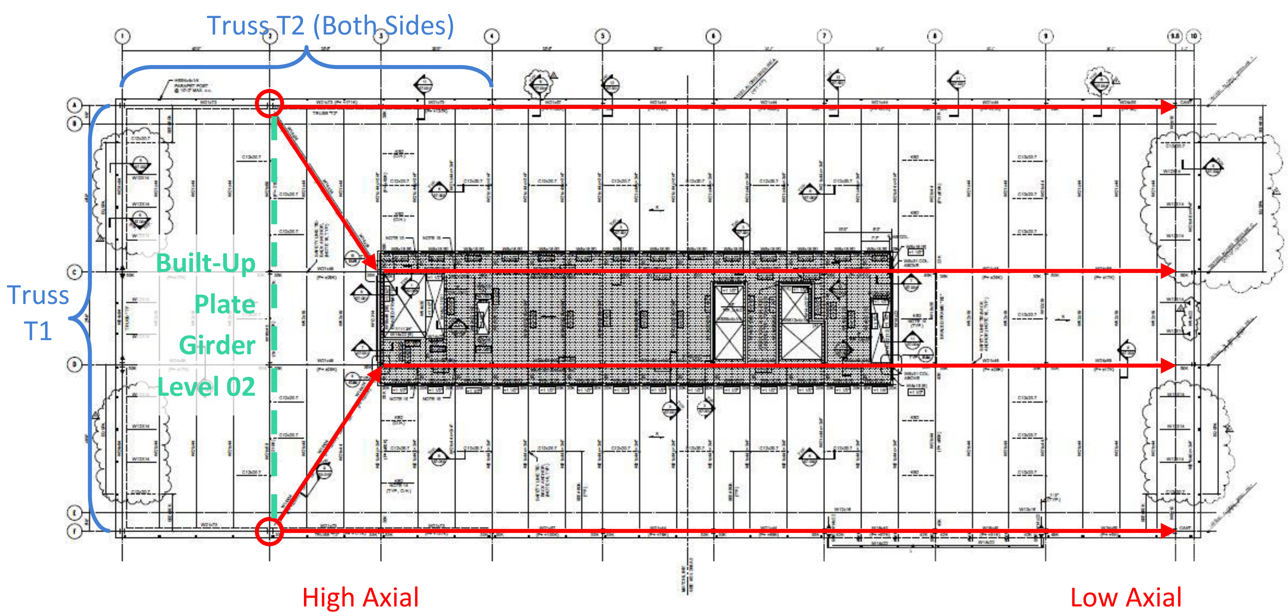


Figure 15: Roof Plan Showing Load Path of Truss System

At major connection points for both trusses, diagonal wide flange members are welded to 3/4" or 7/8" thick gusset plates. Where the truss diagonals intersect columns, the truss member stays continuous and the web is fitted with stiffeners that match the dimensions of the column it is splitting so that both members remain continuous through the connection. Columns and beams connect to girders stiffened with WT members cut to match the connecting column. Gravity beam connections inside these trusses consist of single angle, L4x4x3/8, shear tabs. At the outermost point of the cantilever, the truss system is cambered up 1 1/2" to counteract the deflection caused by dead load added after erection. An example is shown in Figure 16.

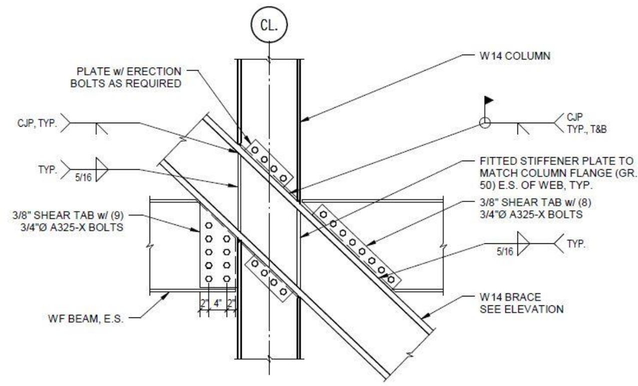


Figure 16: Truss Diagonal Joint Detail (Construction Documents)

As mentioned before, the compression members of truss T2 bear on the plate girder shown in Figure 17 below. The plate girder, A572 Grade 50 steel, is on Level 02 and spans between the columns of the outer bays in plan on Level 02 which bear on post-tensioned beams in the substructure. Dimensions of the girder are shown in Figure 17 with the exception of 3/8" stiffener plates. It ties into the floor system by studs, angles, and stiffeners. Simple connections made to plate girder are typically seated connections where the bottom flange of the connecting beam has a 3/8" A572 gusset plate welded to the bottom flange. Kicker angles, typically 2L3 1/2x3 1/2x5/16, are welded to the gusset plate and the stiffeners in the plate girder to brace the girder's bottom flange against lateral-torsional buckling.

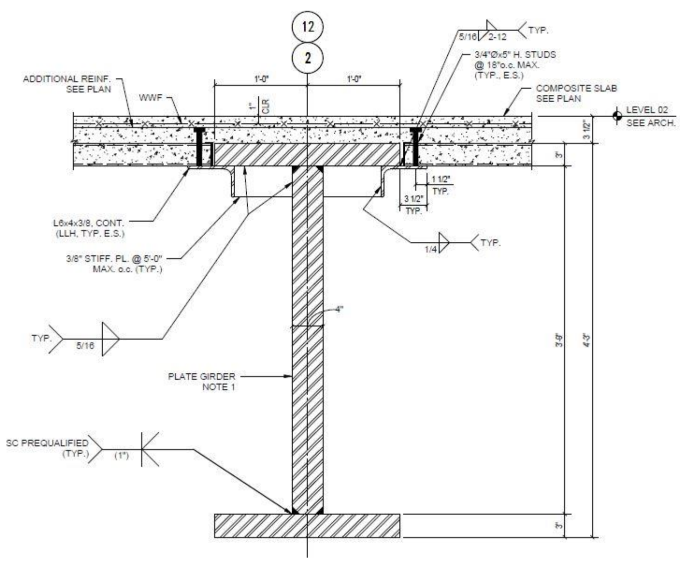


Figure 17: Plate Girder Detail (Construction Documents)

Loading

This section of the report will summarize the design loading for the as built project as determined from project documentation and previous technical reports.

Gravity Loading

The loads presented in the following Table 1 and Table 2 are given in the structural drawing notes and were found to be comparable to those listed in ASCE 7-05. Additionally, a framing allowance of 13 PSF was calculated for typical steel framing. Using Vulcraft's tables, a typical floor loading including the floor system weight and the framing allowance is 69 PSF. Similarly, the dead load of the roof system including the framing allowance was calculated to be 38 PSF. In terms of the curtain wall, industry standard weights of materials were used to calculate a line load on exterior beams of 211 PLF.

Table 1: Superimposed Design Loads

Superimposed Design Loads		
	Dead Load	Live Load
Office Floors*	20 PSF	50 PSF
Assembly Areas	10 PSF	100 PSF
Stairs	10 PSF	100 PSF
Roofs (UNO)	25 PSF	20 PSF
Office Lobby	40 PSF	100 PSF
Parking Garage	5 PSF	40 PSF
Landscaped Plaza	Per Dwgs	100 PSF
Balconies	50 PSF	100 PSF
Top Level Parking	5 PSF	100 PSF
Storage Rooms	10 PSF	125 PSF
Mechanical Rooms	10 PSF	125 PSF
Elevator Machine Rooms	10 PSF	150 PSF

*Live load includes 15 PSF allowance for partitions

Table 2: Snow Load

Snow Load	
Ground Snow Load	Pg = 20 PSF
Snow Exposure Factor	Ce = 0.9
Snow Importance Factor	I = 1.1
Thermal Factor	Ct = 1.0
Flat Roof Snow Load	Pf = 22 PSF

Lateral Loading

This section of the report summarizes the wind load and seismic load investigations for the as built project and whether the project is wind or seismic controlled.

Wind

Wind calculations were based on the Wind Loads chapter of ASCE 7-05. A summary of results of the wind calculations is presented here while complete calculations are available in Appendix B: As Built Wind Calculations. Table 3 summarizes the wind load factors used in analysis. On the following wind diagrams, the directions given are the directions of the building's longitudinal and transverse axes oriented to True North. Figure 18 and Figure 19 show the wind load calculations results of a maximum base shear of 733 kips and a maximum overturning moment of 39,615 kips. These figures show the raw wind pressures given by analysis, but the calculations were executed using the minimum wind pressure provision of 10 PSF of ASCE 7-05.

Table 3: Wind Load Factors

Wind Load Factors	
Basic Wind Speed	V=90 MPH
Importantance Factor	I=1.15
Exposure	B
Internal Pressure Coefficient	G
Topographic Factor	Kzt=1.0
Gust Factor NW-SE Direction	Gf=0.863
Gust Factor NE-SW Direction	Gf=1.00

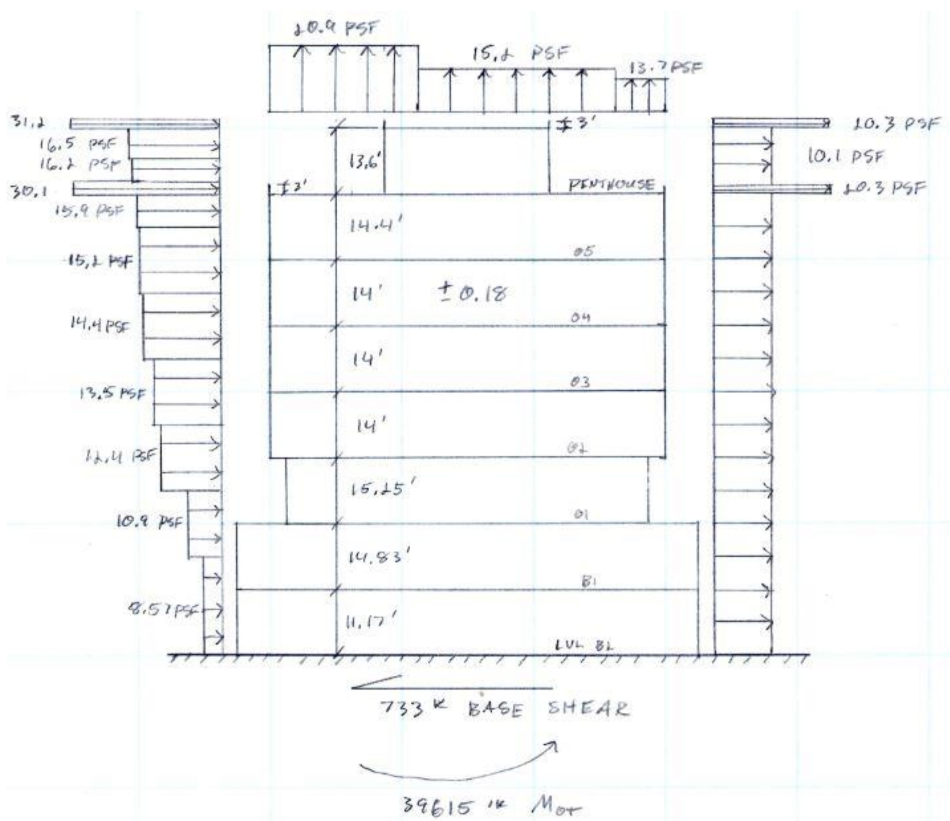


Figure 18: Wind Diagram for NE-SW Direction

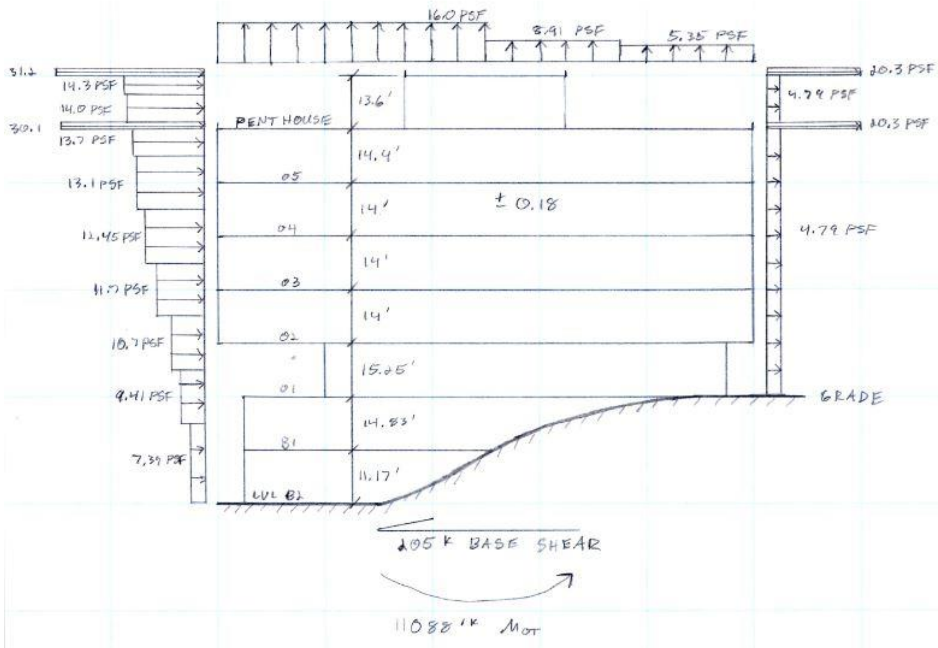


Figure 19: Wind Diagram for NE-SW Direction

Seismic

Seismic calculations are based on ASCE 7-05 using the Equivalent Lateral Force Procedure. The project team determined their design forces by the Modal Response Spectrum Analysis procedure; however I was able to replicate their design forces to 90% when comparing seismic base shears. Calculations are available in Appendix C: As Built Seismic Calculations. Seismic design parameters and Spectral Response Factors from the United States Geological Survey are shown in Table 4.

Table 4: Seismic Design Parameters and Spectral Response Factors

Seismic Parameters	
Site Class:	C
Occupancy:	III
Importance:	I=1.25
Seismic Design Cat.:	C

Spectral Response Factors	
Ss=	0.501
Sds=	0.400
S1=	0.153
Sd1=	0.168

The project's lateral system is composed of a concrete parking system of shear walls and a steel office system of braced frames directly above it. This two part system was analyzed using the Two Stage Equivalent Lateral Force Procedure outlined in ASCE 7-05 Section 12.2.3.1. In summary, this procedure states that the two buildings are first analyzed separately and their resulting base shears are combined using a ratio to transfer the upper structure base reactions base shear. This base shear then distributed into story shears as normal. The system factors of the separate structures are summarized in Table 5 and the ratio between the systems was determined to be 1.0, meaning that the overall base shear is directly additive. A total base shear of 4235 kips was found using this procedure which is larger than the maximum wind base shear of 733 kips, so the building is seismic controlled.

For analysis, the structures were separated at the Level 01 interface. To account for the seismic weight at this level, it was calculated and then lumped with the seismic weight at Level 02. This accounted for the weight in the lateral analysis and conservatively increased the moment arm of its contribution to the overturning moment. Resulting story forces and overturning moments are shown in Table 6 and a summary of the forces applied in ETABS modeling is shown in Table 7.

Table 5: Structural System Factors and Results

System Factors and Results		
	Office	Parking
R=	3	5
Ta=	0.561 sec.	0.018 sec
Cs=	0.125	0.100
Vbase=	3338 kip	897 kip

Table 6: As Built Seismic Story Forces

SEISMIC STORY FORCES						
Level	w_x(k)	h_x(ft)	$w_x h_x^k$(ft-k)	C_{vx}	F_x(k)	M_{OT}(ft.-k)
B1	8968	11.2	29972	0.017	**73**	812
1	13378	26.0	Weight Lumped to Level 2			0
2	15899	41.3	733262	0.420	**1779**	73367
3	2527	55.3	157491	0.090	**382**	21106
4	2527	69.3	198740	0.114	**482**	33383
5	2531	83.3	240549	0.138	**583**	48574
Penthouse	1680	97.7	188269	0.108	**457**	44593
PH Roof	1543	111.3	197690	0.113	**480**	53346
		$\Sigma w_x h_x^k$=	1745974	1	**4235**	**275180**

Table 7: Adjusted Forces for ETABS Modeling

MODELING ADJUSTED FORCES	
Level	F_x(k)
B1	73
1	-
2	1779
3	382
4	482
5	583
Penthouse	936
Sum=	4235 ok

In terms of ETABS modeling, the steel braced frame lateral system and the concrete shear wall lateral system were modeled as two separate models. The steel model, shown in Figure 20, entirely originated in ETABS while the concrete model grid was drawn and imported from AutoCAD and is shown in Figure 21. First, wind and seismic load cases were calculated for each system and applied in their separate models. Then, the controlling load cases from the steel office tower model were identified and those reactions at each of the braced frame bases were recorded. These reactions were applied to the corresponding connection point on the shear wall tops in the concrete parking garage model within their respective load cases. Results regarding the overall structure or the parking garage were determined with the brace frame reactions incorporated into the concrete model. For the steel member checks, the office model was used.

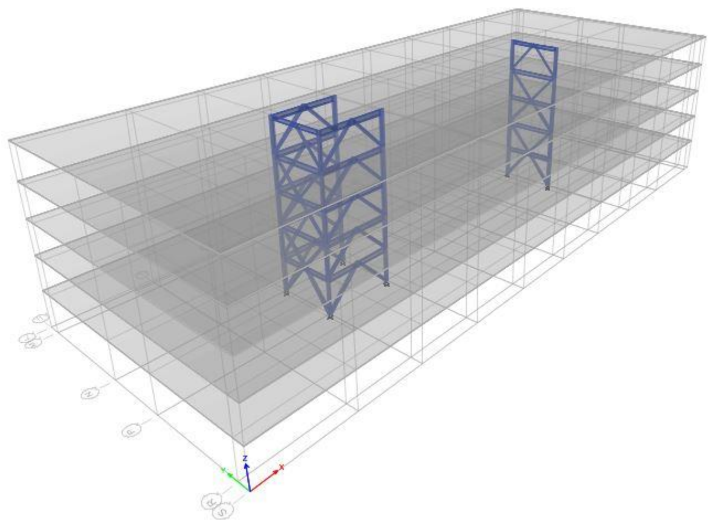


Figure 20: Office Structure Lateral ETABS Model

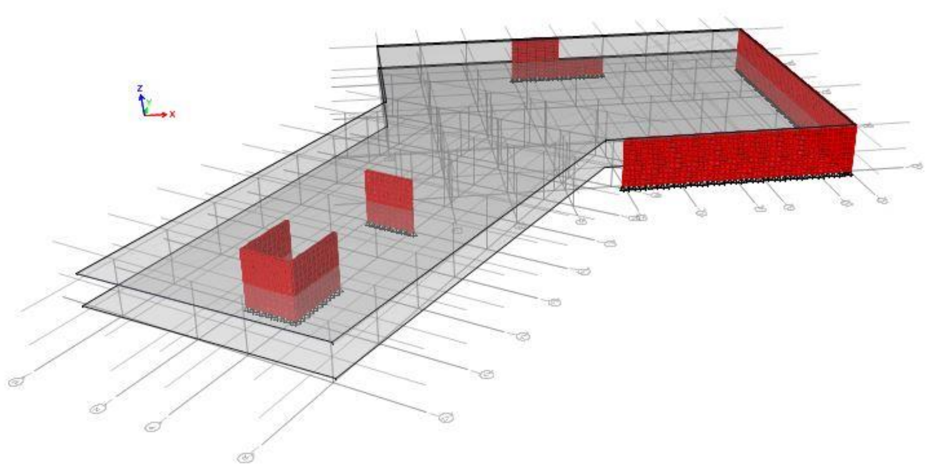


Figure 21: Parking Structure Lateral ETABS Model

Problem Statement

A scenario has been created in which the owner has decided to add an intensive green roof that also functions as a publicly accessible roof garden for the tenants of the office towers. In the current design, much of the exterior public spaces double as emergency vehicular access, limiting the material palette to decorative stone and concrete. While there are three bio-retention basins on site, little of the green space is designed for public access and enjoyment. LEED Silver Certification is a design basis for the headquarters complex per the owner's request and since has decided to embrace one of St. Louis County's Sustainability Initiatives. This particular initiative is to incorporate green roofs in new building projects. As a Fortune 500 company, Reinsurance Group of America, Inc. is proud of their employees and strives to provide a healthy work-life balance and therefore desires to use this new green roof as an amenity for the employees to enjoy.

Proposed Solution

This new request presents structural challenges. First, the gravity system will have much more weight to support, so the current gravity system will need resized for new loading. Specific attention will be given to the cantilever truss system and its supporting members. Adding extra weight to roof members also supporting the cantilever increases the flexure and axial tension loads and can make the force interaction more critical for design. Depending on the green roof's weight distribution on top of the cantilever and its back span may help mitigate the cantilever's overturning force couple or add to it, which could require the roof support to be redesigned completely. Secondly, adding a green roof garden adds significant seismic mass and, in turn the seismic force. This means that the lateral system will need to be stiffened to handle the new seismic load and operate within acceptable drift limits.

In addition to structural challenges, the green roof garden and its public spaces need to be designed with respect given to the current design. Also, the green roof garden will have significant cost, logistics, and schedule implications that must be considered going forward. These topics are elaborated upon in the following Breadth Study section.

Breadth Study

A green roof garden addition with public spaces impacts other non-structural aspects of the building project. In this study of the proposed solution, the green roof garden will be designed as an architecture breadth/system study. This project addition will have cost, construction logistics, and schedule implications which will be studied as a construction breadth.

Breadth Topic 1: Green Roof Garden System

The green roof garden will be designed considering appropriate plantings, maintenance concerns, code and safety requirements, and the relationship of the public spaces to the plantings. Plantings, if tall enough, may be seen from the surrounding roadways or buildings, impacting the architectural skyline of the site. The green spaces and public spaces of the green roof garden have their own design language that should complement the aesthetic of the project. For this reason, this breadth will begin researching and designing the green roof garden. Research of precedent projects, fundamentals of green roof design, roof garden design, and code requirements are critical to the design process. Design iterations and evaluations will be conducted until a successful design emerges.

Breadth Topic 2: Construction

A second breadth of study will evaluate on a comparison basis the cost, construction logistics, and schedule for the intensive green roof garden implementation. A detailed cost comparison will be completed for the green roof garden to determine the additional project cost along with a detailed cost comparison of the supporting structural changes. On the logistics side, a study will explore the material arrival on site, storage, and installation needs of the green roof garden and the structural redesign to determine any new or additional considerations needed. Finally, a project schedule comparison will be revised to include the green roof garden. Both the revised and the original project schedule activity durations will be compared to determine how the construction schedule is influenced by adding the green roof garden.

Green Roof Garden (Breadth 1)

The following comprehensive design narrative will present the design decisions, the factors effecting those decisions, and be reflective of the research involved in making those decisions.

Design Narrative

Design Decisions & Architectural Vision

Inspiration

The architectural vision for RGA's Global Headquarters was to embody the company's global and local market influences within the architecture of their headquarters. The global influence is represented by strong, clean lines in the office towers with a curtain wall of glass and aluminum panels. The local influence is evident in the connecting amenities and parking garage with materials of concrete and limestone paneling. A descriptive word study, shown below in Figure 22, using the Owner's Requirements project document was performed to find inspiration for the design concept of the green roof garden. First, the document was read in its entirety and descriptive words used by the client when discussing their desires for the project were pulled out and recorded. Words that were used more than once were written in bold. These words were pulled down into a list and a few synonyms were listed next to each one and relationships between concepts were noted with arrows. To the right of those word groups a fundamental descriptor was written in bold and underneath, noted by double arrows, are their envisioned applications to the design concept. Finally, at the bottom are two sketches of the design concept.

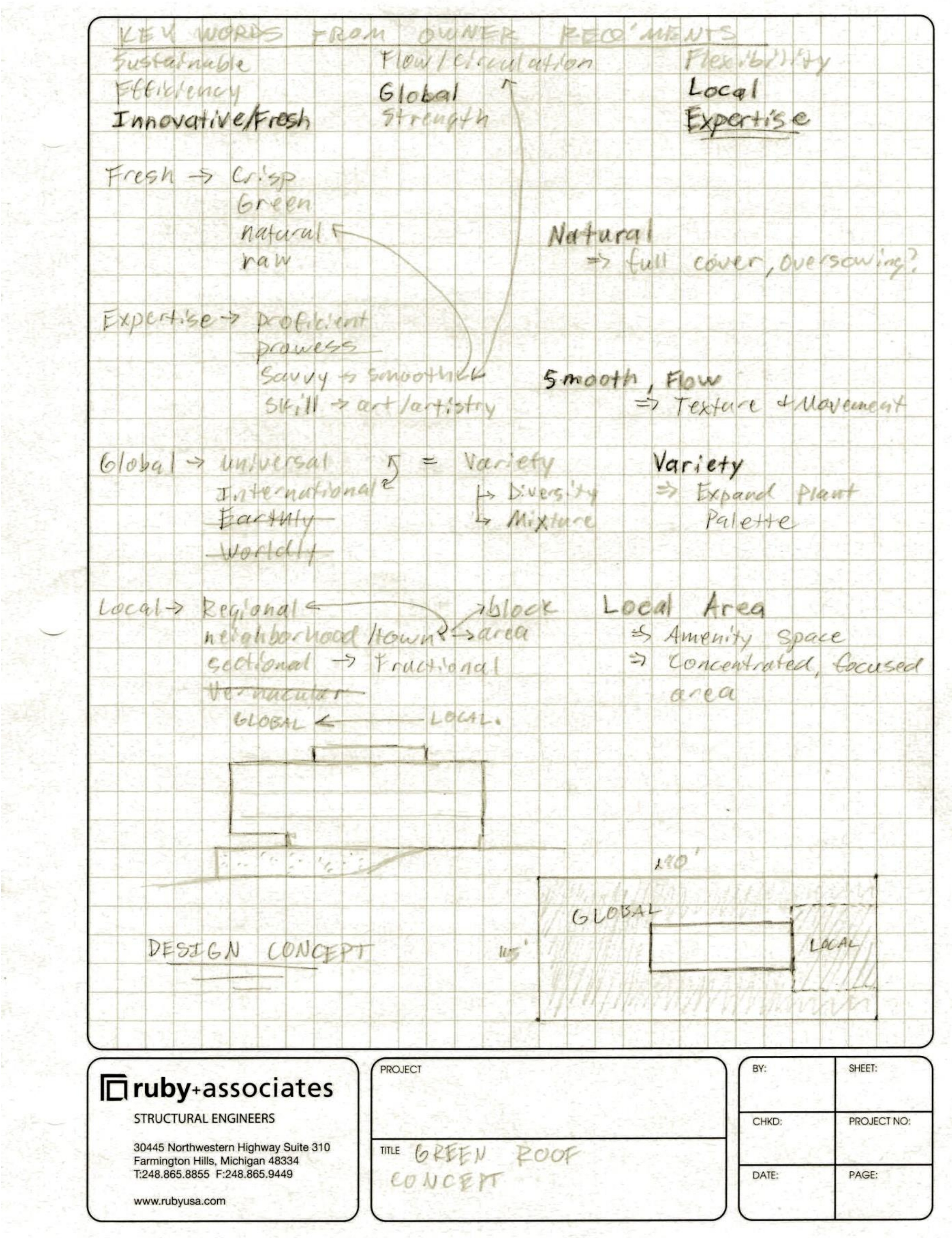


Figure 22: Word Study from Owner Recommendations

Decisions

When looking to add shading for the seating areas of the green roof, many factors were involved. For example, in order for the shading to also provide rain protection, the cover would have to be solid which would incur wind uplift being located on a roof. Additionally, any shading devices would need to be anchored directly into the structure, which would puncture the waterproofing membrane which must be avoided in green roof applications. From an architectural side, the shading structure would be seen from the ground and could disrupt the visually clean roofline. These factors led to a final decision to not cover the seating areas and proceed designing the space as a “fair weather” use.

To control the flow of people through the green roof space, a couple of different techniques were used. First, to restrict access to the sedum area over the cantilever, a simple plastic split rail fence and no walkways will deter tenants from walking on that side of the roof while a gate will allow maintenance access. In addition, seating nodes, or groups of seating, were placed to provide several seating arrangements to choose from as well as provide focal points for the tenants to spend their time in. Additionally, the orientation of the seating will lead the tenants to subsequent spaces focal spaces with the main space large enough to accommodate an organized event.

The material palette for the built aspects of the green roof is neutral colors and earth tones. The seating and tables are made of lightweight concrete so that the furniture is durable, neutral, and weighs enough that it will not be blown away by higher gusts at the roof level. A pedestal paver system from Hanover is made of pressed concrete and can be specified in neutral color tones. Neutral colors and earth tones will put visual focus on the colors of the plantings and not compete or clash.

Design Metrics

A list of design metrics was developed from the Owner Requirements project document. Similar to the architectural word study, repeated phrases and requirements from other building system discussions were extrapolated to the green roof garden application. These metrics are listed below:

- Reasonable initial cost
- Maintain or improve LEED Silver
- Amenity area for seating
- Open access for tenants
- As low maintenance as possible
- Plants are self-sustaining after establishment
- Architectural lines are uninterrupted

Layout

A schematic plan of the green roof garden is presented below in Figure 23. This plan will be referred to throughout this breadth discussion as the main graphic for a comprehensive view of the green roof garden information.

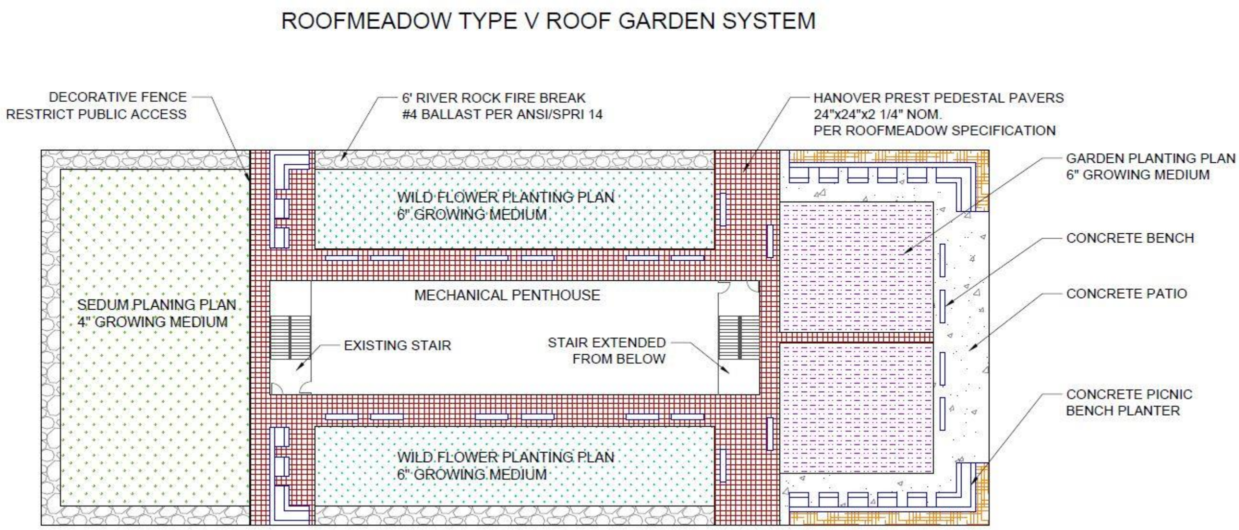


Figure 23: Schematic Plan of Green Roof

Access

In the current project, one stairwell extended to the roof level to serve mechanical space, but there was no door directly from the stairwell onto the roof. The only access to the roof required walking through the mechanical space first. A door open from the stairwell directly to the roof was added. In the layout of the roof however, this door would be far away, about 150 feet, from the main seating and gathering area. Another entrance closer to the main seating and gathering areas was needed. In the levels below, a stairwell similar to the one accessing the mechanical space stops on level five, the level below the roof. This stairwell was decided to be continued to the roof level and become the main access path to the green roof. This stairwell door opens right next to the main public space, making flow on and off of the roof easier for the tenants.

Structural Considerations

When designing the layout, the main concern was minimizing the load added over the 40 foot cantilever and heavier loads be added on the back span to help mitigate the overturning action of the cantilever. This resulted in the field over the cantilever being chosen to have the minimum growth media depth allowed by the planting of sedums. To further lighten the load, public access is restricted to maintenance only on this part of the roof to minimize the live load as well. Finally, a composite deck system was chosen to replace the as built roof deck because the membranes associated with green roofs bond better to a concrete surface than roof decking and the composite action will be needed to carry the increased loads.

Fire Protection

Fire protection for green roofs is covered in *ANSI/SPRI VF-1: External Fire Design Standard for Vegetative Green Roofs* within section 3.0. The provisions state that there must be a six foot wide continuous fire break spaced linearly no more than 125 feet and a six foot break continuous border around the rooftop perimeter, structures and equipment. A fire break means a break in all media of the green roof system. Also, the square footage of an area between fire breaks can be no more than 15,625 square feet. The first provision listed impacted the design in the longitudinal direction and led to the design of the seating nodes located on both sides of either end of the mechanical penthouse.

Wind Protection

Wind protection was determined by the provisions of ANSI/SPRI RP-14 to prevent uplift of the system and protect against wind scouring of the plants and media. From Table 2F, for a parapet height of 42 inches, basic wind speed of 90 mph, Exposure B, and a building height of 98 ft. above the lowest grade the maximum wind speed for System 1 is 90 mph. System one requires the membrane to be ballasted with a #4 ballast, which according to section 4.0 the growth media, if the dry weight is greater than 10 PSF, acts as a #4 ballast. The lightest growth media, the 4 inch sedum depth, weighs 28.3 PSF dry and is adequate for wind ballast, so no other provisions are needed.

Fall Protection

Fall protection requirements are outlined in OSHA 1926.502 which allows parapets meeting the requirements to function as fall protection. The height for fall protection is defined as the height from the highest working or walking level to the top of the parapet in this case. A minimum of 42 inches is required by OSHA 1926.502b1.The height of the green roof system and parapet for fall protection from the defined roof elevation is 4'-9" in comparison to the as built parapet height of 3 ft.

System Selection: Roofmeadows Type V

Roofmeadows was chosen for their availability of information and versatile system selection. Type V was chosen by process of elimination when measured against the project requirements. Type V meets the growth and drainage media depth requirements for both the low sedum and the deeper wild flower and garden beds and avoid interfacing two different systems. This system's conventional installation meets the insulation and paver installation requirements and is not actively equipped for active irrigation because the plantings chosen are self-sustaining after establishment. Finally, this system is compatible with the Hanover pavers that are specified in the as built project. Details of the Roofmeadows system edited to show the specific material depths specified previously are shown in Figure 24 and Figure 25. Additional technical information can be found in Appendix D: Roofmeadow System Information.

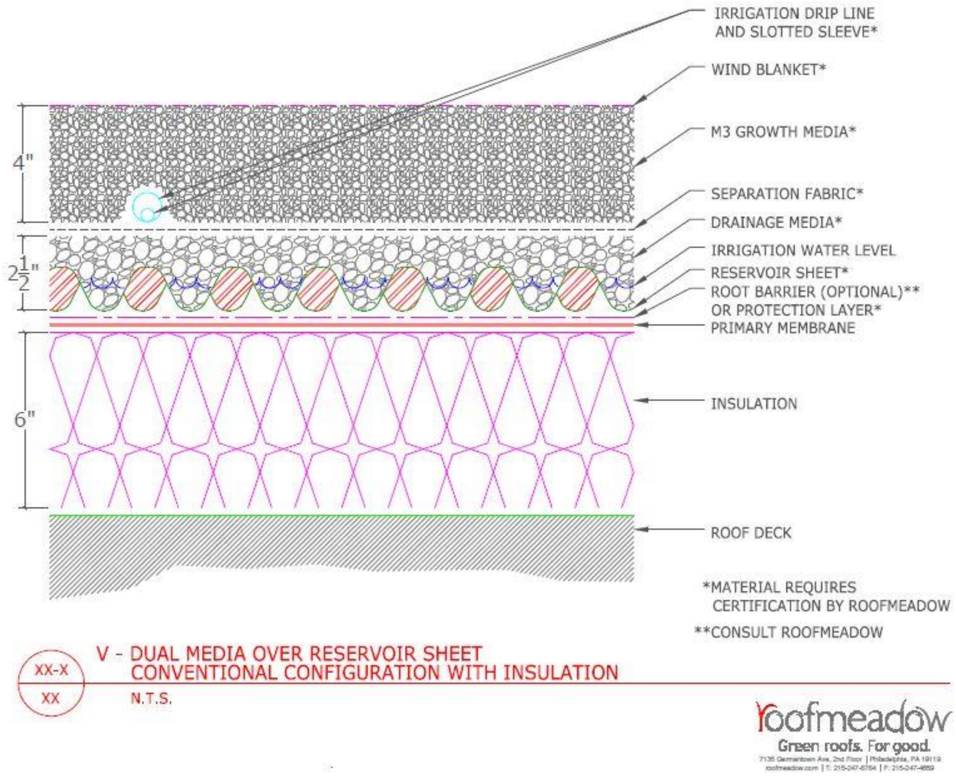


Figure 24: Edited Detail of 4" Growth Medium Roofmeadow Type V System

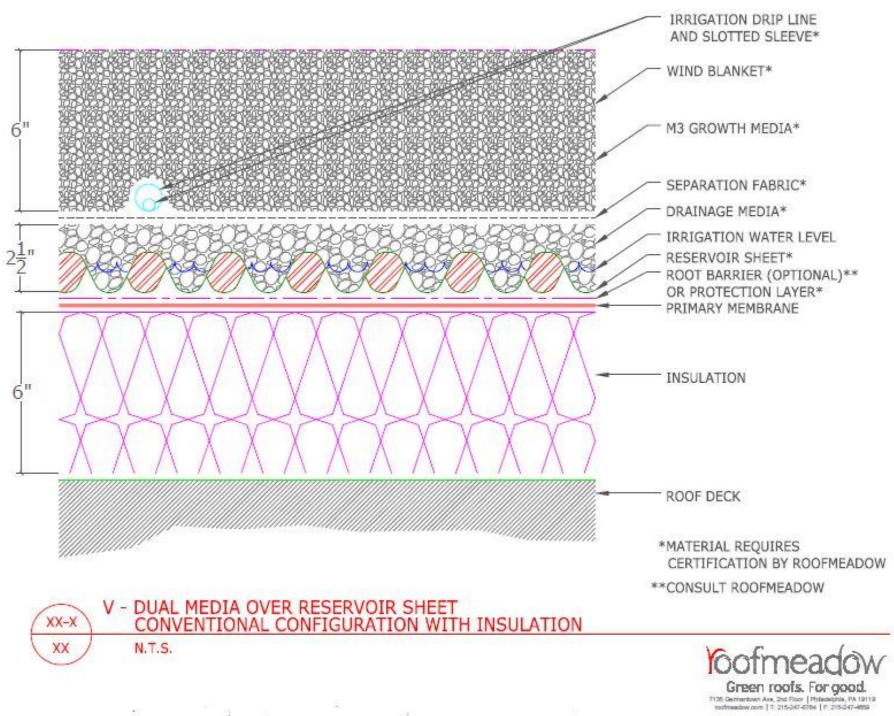


Figure 25: Edited Detail of 6" Growth Medium Roof Meadow Type V System

Plant Selection

The first requirement for plant selection was to narrow down plants that would survive in the harsh environment of the green roof. Using the United States Department of Agriculture's 2012 Plant Hardiness Zone Map for the state of Missouri, I determined my site to be in zone 6a as shown in Figure 26. This map is the standard horticulturalists and related use to select plants that will thrive in a location and is based on the annual average minimum winter temperature. Then, from the list of proven green roof plants from *Green Roof Plants* a list of plants meeting both requirements was drafted.

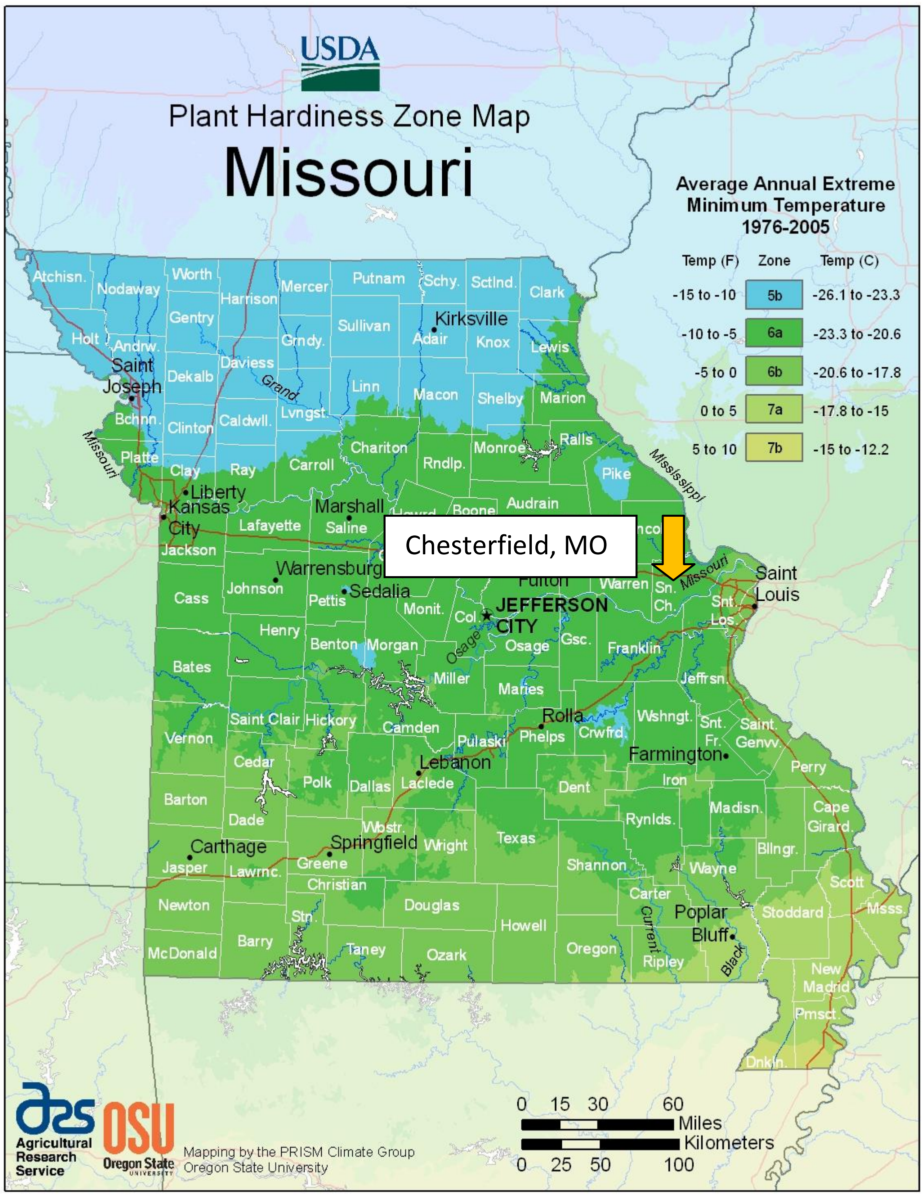


Figure 26: USDA Plant Hardiness Map for Chesterfield, MO

From the initial plant list, blooming time was a high priority so that there will be plants in bloom throughout the growing season and across the entire roof for the maximum use of the space and system advantages. In terms of planting, sedum cuttings will be used because installation in this way is easy and fast. Other plantings will use plugs for more control planting pattern. Aesthetically speaking, plants with a mounding growth habit are good border plants because they are dense and can enclose and emphasize space and pattern well. Plants that have mounding or shrub like growth habits tend to grow aggressively and will choke out delicate plants if they are planted next to each other. The advantage to a large planted area however, is that delicate plants can form a network and supports its own growth which is not always common for delicate plants on the ground. Also, shorter plants should be planted closer to the walking path so that they can be seen and taller plants either in the back or in the middle depending on the shape of the planting area. When planting in aesthetic layers it is also important to alternate blooming times when possible so that the planted area has visual interest throughout the growing season and not just for a fraction of it. Finally, contrast and visual dynamics can be implemented by placing plants of complementary colors and different textures next to each other. The following figures, Figure 27, Figure 28, Figure 29, show the plants selected by this criteria for the wild flower planting area, the garden planting area, and the sedum planting area, respectively.

Figure 27: Wild Flower Planting Selection

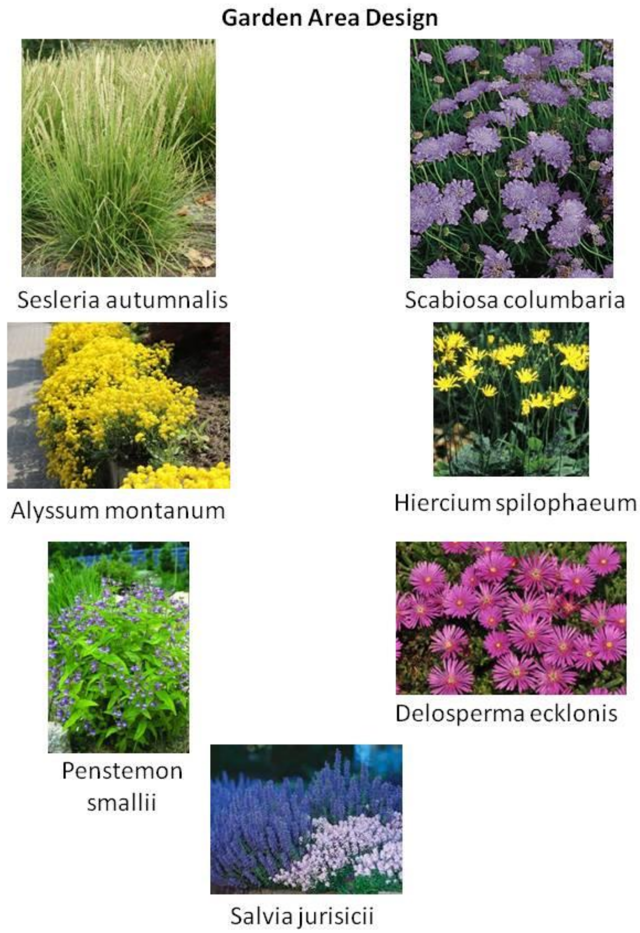


Figure 28: Garden Area Planting Selection

Figure 29: Sedum Planting Selection

Structural Depth

As mentioned before in the project scope, in the interest of time the structural study for this thesis report includes the steel office tower system only. The structural depth focuses on identifying and calculating building loading for gravity and seismic conditions because the project is seismic controlled. Gravity analysis and design focused on the gravity truss system for the 40 foot cantilever of four stories of the building and the roof framing. The lateral analysis focuses on replacing the as built HSS braces with buckling-restrained braces based on the information available from StarSeismic.

Green Roof Design Loads Summary

ASTMs E2397 and E2399 were used to determine structural material and water weights for green roof systems. These standards outline a procedure for taking off the weight of each component of the green roof. A summary of the design loads that are used in the structural depth is in Table 8. Additional green roof loading calculations can be found in Appendix E: Green Roof Calculations.

Table 8: Green Roof Design Loads

Green Roof Design Load Summary (PSF)		
	Sedum	Wild Flower & Garden
Dead	53	68
Water Live	26	34
Roof Live	20	20
People Live	0	100
Snow	22	22
Wind Uplift	-21	-21

Gravity System

Composite Roof System Design

Once the design loads were calculated, a new composite roof system had to be designed. The as built deck was a 3N20 roof deck supported by the steel roof framing. The controlling loading for the roof deck is in the area of 6 inch growing medium and public access shown in Figure 30. The superimposed load to use with the Vulcraft Composite Deck Tables is 68 PSF dead load plus 100 PSF live load for a total superimposed load of 168 PSF. The deck is designed for a 3 span condition and unshored construction with a span of 10 ft. between roof beams. From the Vulcraft tables, a 3VLI19 composite deck with a lightweight concrete topping of 3 1/2 inches meets all of the above structural requirements. Calculations for the roof system are provided in Appendix F: Roof Redesign ETABS Output.

Fire rating was determined using Vulcraft's Floor-Ceiling Assemblies with Composite Deck using a 2 hour, unprotected 3VLI deck the minimum required lightweight concrete topping is 3 1/4 inches and 3 1/2 inches has been provided. Finally, UL assemblies D826, D907 and D916 have unrestrained beam ratings of two hours with the addition of spray fireproofing to the beams, which is already in use in the as built project. The redesign has a two hour fire rating.

After the deck was designed, the total design loads were applied in ETABS and are summarized below in Figure 30. The gravity roof beams were analyzed and designed as their own model shown in Figure 31. The dead load accounts for the green roof and the weight of the roof deck and concrete topping. The loads from the unchanged mechanical room were accounted for by taking the loads of the mechanical floor perimeter beams and adding the loads as point loads onto the roof beams. Deflections for the roof structure were limited to L/240 for dead and live load deflection and L/360 for just live load deflection. In terms of loading, the roof saw an increase in both dead and live loading and is summarized below in Table 9.

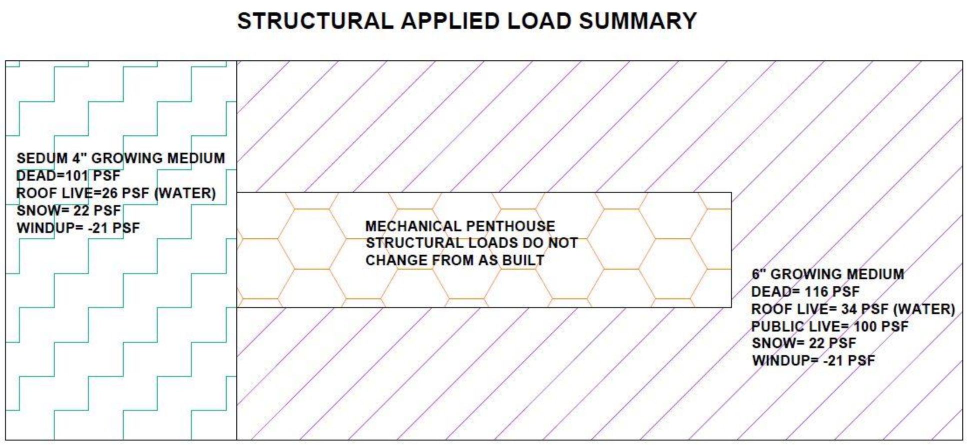


Figure 30: ETABS Applied Loading Summary

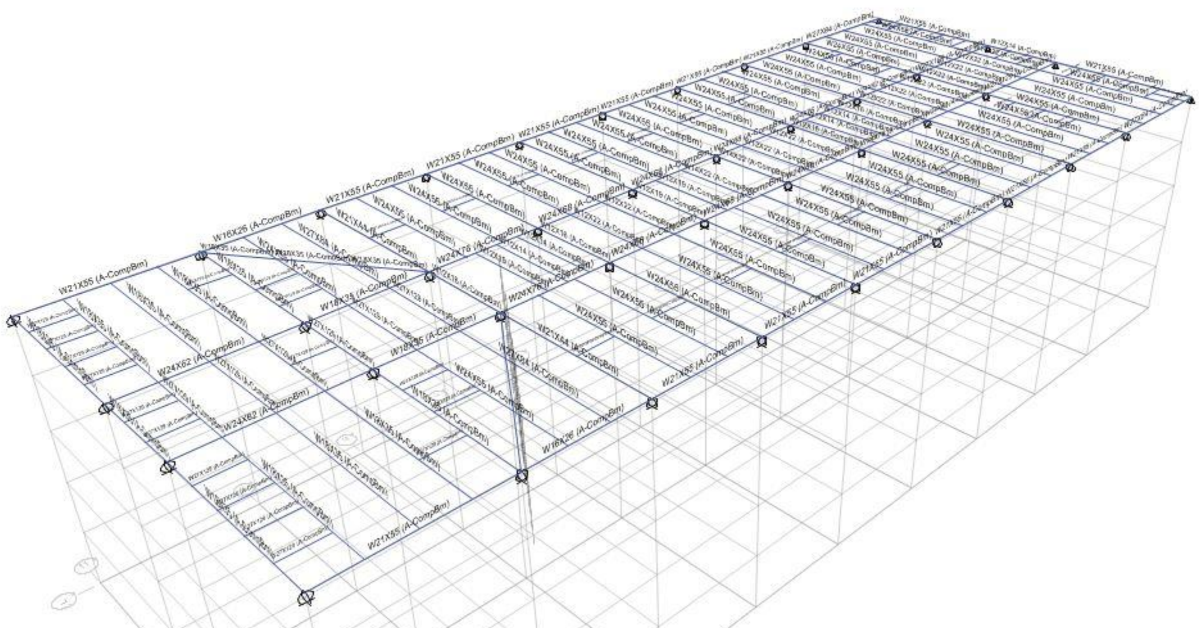


Figure 31: Gravity Roof Framing ETABS Model

Table 9: Percent Load Increase from Green Roof

Percent Increase of Load		
	4" GM	6" GM
Dead Load	112%	172%
Live Load	118%	355%

Building Trusses

Truss T1 is supported on each end by the two instances of Truss T2. Truss T2 at the roof level is supported by a roof truss that transfers the load to the braced frames and through the roof diaphragm. Overall deflection of the truss members supporting the curtain wall was limited to 3/4 inch to prevent loading of curtain wall elements.

For modeling in ETABS, separate models were created for Truss T1, Truss T2, and the embedded roof truss. This allowed each truss to be analyzed using 2D analysis and locking the appropriate degrees of freedom in ETABS. Roof point loads determined from reactions on roof gravity ETABS model and floor gravity loads were determined on the basis of floor loads and tributary area.

Truss T1

First, the Truss T1 was modeled, loaded, and designed. The bottom cord of Truss T1 has a 3/4 inch camber to counter the deflection due to dead load. In ETABS, this was modeled as a forced displacement at the interior vertical members. The allowable deflection under live loading was limited to 3/4 inch. Since the camber was accounted for in ETABS, the allowable net deflection was 1 1/2 inches. The end reaction for each load type was recorded and applied to the corresponding point on the model of Truss T2. Table 10 summarizes the loads acting on Truss T1 which are also shown graphically in Figure 32.

Table 10: Redesigned Truss T1 Loads and Reactions

Roof Reactions Applied to Truss T1	
Load Type	Reaction(k)
Live Roof	18.66
Snow	15.79
WindUp	-15.08
S. Dead	72.51
Live Public	0

Floor Gravity Loads on Truss T1		
	Dead(k)	Live(k)
P1=	12.5	5.63
P2=	46.5	29.3
P3=	12.5	5.63
P4=	37.7	23.7
P5=	12.5	5.63
P6=	25.2	18.1
P7=	55.7	35

Truss T1 Reactions to Truss T2		
	FZ (k)	FY (k)
Live Roof	19	3.28
Snow	16	2.76
WindUp	-15	2.59
S. Dead	275	44.21
Live Public	123	19.76
*FY is toward truss interior, at Level 02 Ext. Verticals		

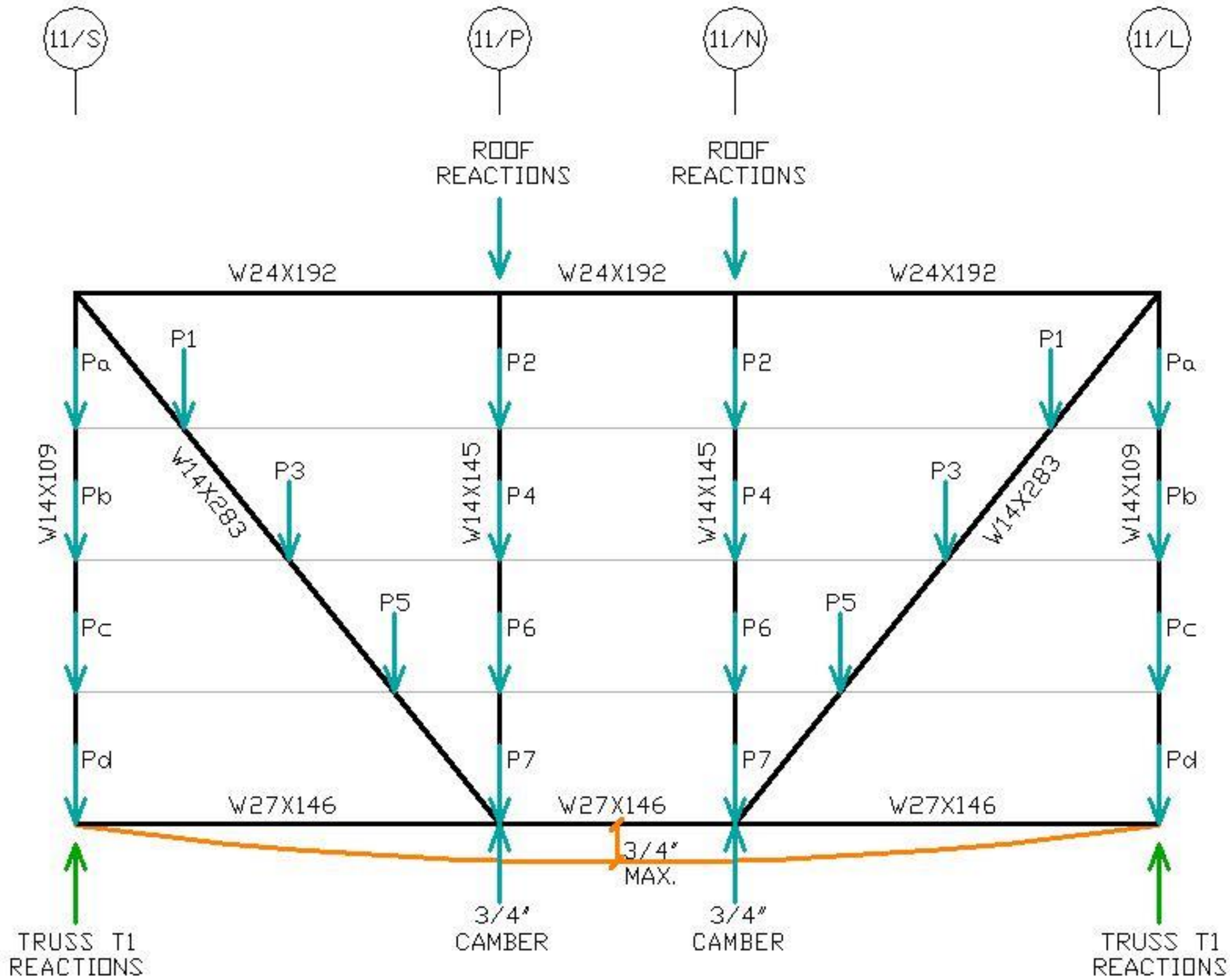


Figure 32: Line Drawing of Redesigned Truss T1 Showing Loads, Reactions, and Deflections

Truss T2

Truss T2 was then loaded and designed while deflection was again limited to 3/4 inch for attachments. Reactions of Truss T2 at the roof level were recorded and applied to the corresponding point on the roof truss. Again, a summary of the loads acting on Truss T2 is provided in Table 11 and represented graphically in Figure 33.

Table 11: Redesigned Truss T2 Loads, Reactions, and Deflections

Roof Reactions Applied to Truss T2		
Load Type	Reaction(k) J11	Reaction(k) J19
Live Roof	11.23	21.02
Snow	9.51	17.79
WindUp	-9.07	-16.98
S. Dead	43.64	81.67
Live Public	0	0

Floor Gravity Loads on Truss T2		
Shared Column Loads with T1		
	Dead(k)	Live(k)
Pa=	9.15	5.75
Pb=	18	11.3
Pc=	26.9	16.9
Pd=	35.8	22.5
Floor Gravity Loads		
Pe=	22.48	14.34
Pf=	44.26	28.24
Pg=	66.31	42.3
Ph=	61.72	39.38
P8=	35.27	22.5
P9=	26.45	16.88
P10=	35.27	22.5
P11=	53.08	33.86
P12=	35.27	22.5
P13=	52.91	33.75

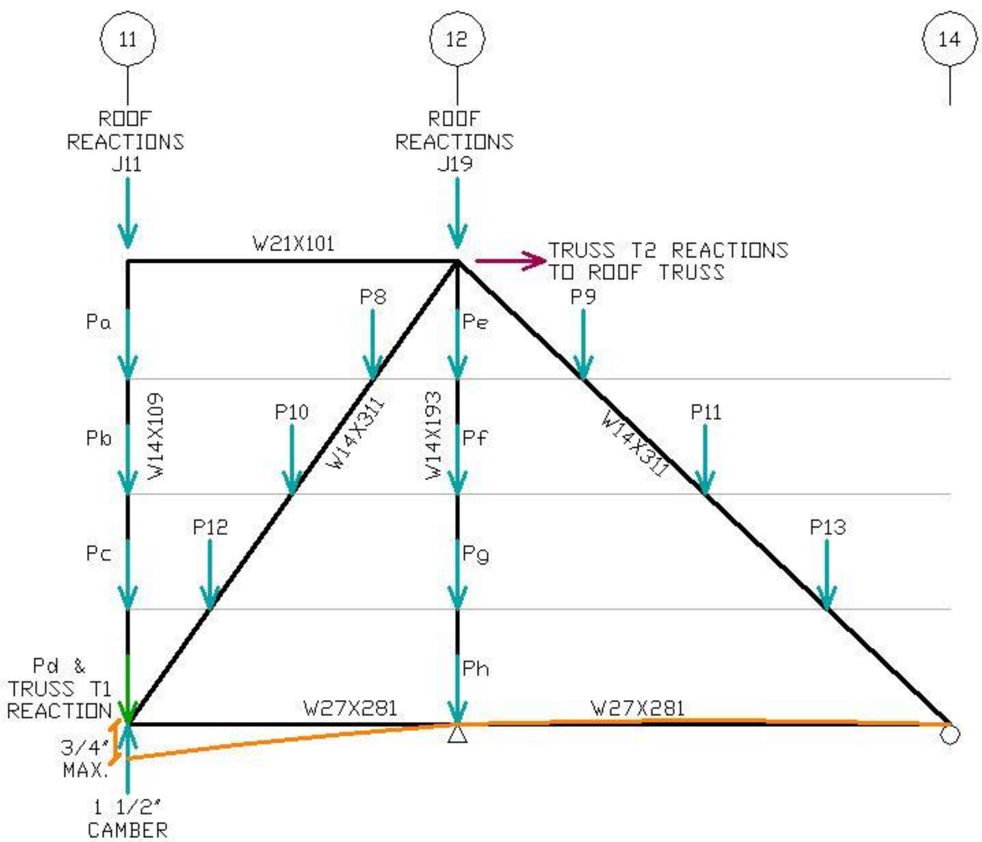


Figure 33: Line Drawing of Redesigned Truss T2 Showing Loads, Reactions, and Deflections

Roof Truss

The roof truss was then loaded, analyzed and designed with deflection at the ends limited to 3/4 inch. Table 12 summarizes the loads on the Roof Truss which are summarized in Figure 34.

Table 12: Redesigned Roof Truss Loads and Reactions

Truss T2 Reactions to Roof Truss	
	FX (k)
Live Roof	20
Snow	16.9
WindUp	-16
S. Dead	298
Live Public	137

Roof Truss Reactions to Braced Frames	
	FX (k)
Live Roof	0.989
Snow	0.836
WindUp	-0.791
S. Dead	14.76
Live Public	6.777

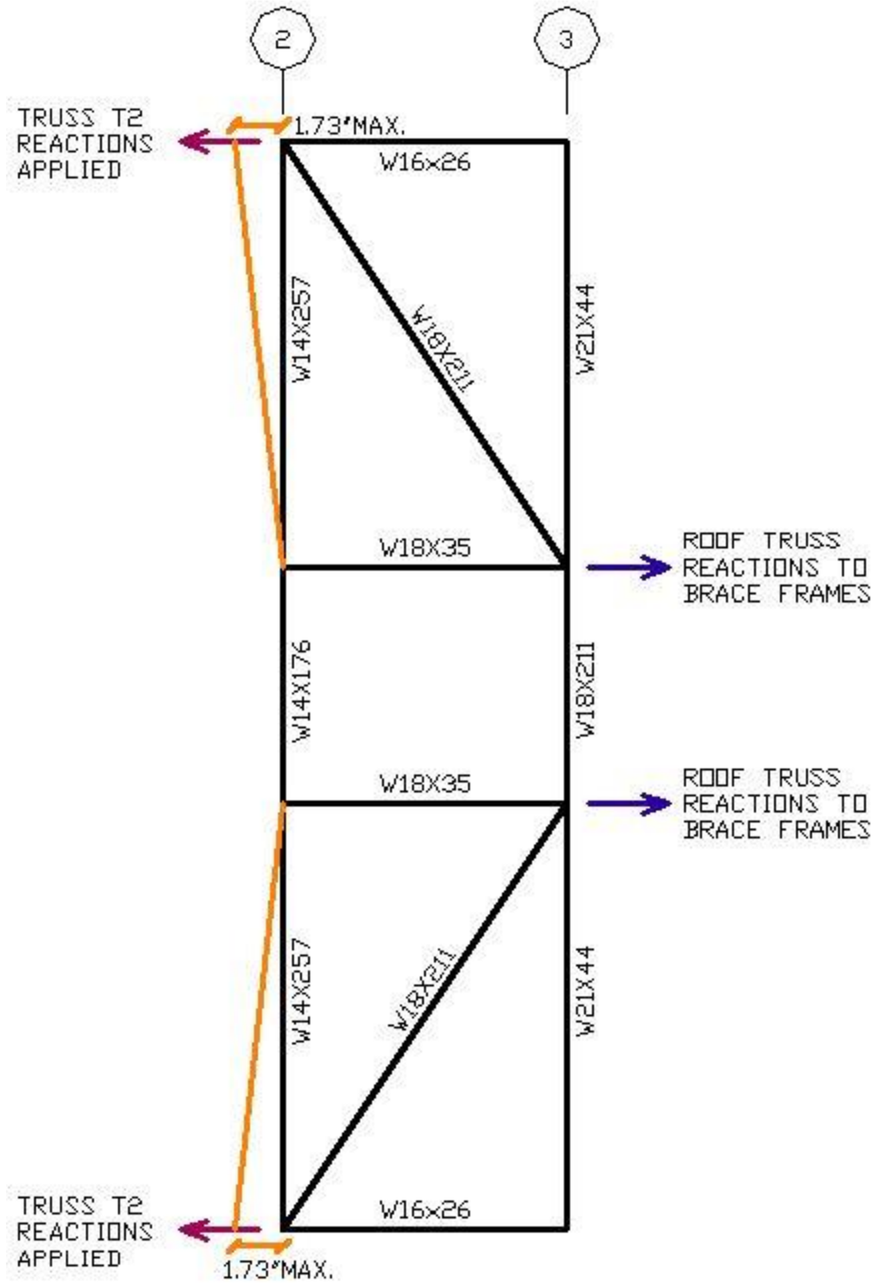


Figure 34: Line Drawing of Redesigned Roof Truss Showing Loads, Reactions, and Deflections

Deflections

Finally, actual deflections are summarized in Table 13. These deflections are taken from ETABS and the net deflection calculations take camber into account and test it against the 3/4 inch deflection limit for curtain wall attachments. Deflections were calculated using service dead and live loads.

Table 13: Maximum Deflections and Deflection Limit Checks

Truss T1 Deflections (in.)			
Camber=	0.75		
DL=		-0.5414	
LL=		-0.242	
DL+LL=	-0.7835	-0.7834	OK
Net Deflect.=	**-0.0335**	**< 3/4" OK**	

Truss T2 Deflections	
Camber=	1.5
DL+LL=	-2.1119
Net Deflect.=	**-0.6119 < 3/4" OK**
DL=	-1.4566
In X Direction:	
Max. Deflect.=	-1.7288
Limit to impose on Roof Truss	

Roof Truss Deflection (in.)		
Deflection=	**-1.663**	**< 1.7288 OK**
Controlling Case: 1.2D+1.6L+0.5Lr		

Note: Deflection checks are shown as a comparison of magnitude for clarity

While there were no strict section limits, truss web members in tension and compression were kept W14 when possible because geometrically W14's are box-like and efficient for tension and compression. Also the top and bottom cords of trusses T1 and T2 were kept at the same nominal depth as the as built project sections so that extra depth was not unnecessarily added. Truss calculations and ETABS output is provided in Appendix G: Truss Loading Calculations & ETABS Output. Finally, the plate girder was checked for strength and serviceability under the new loading and it was found to be adequate. An increase of only 12% dead load and 18% live load are present in this area of the roof. Calculations and RISA Output are provided in Appendix H: Plate Girder Calculations and RISA Output.

Lateral System

Seismic Load Revisions

Seismic loads under the redesign changed in two main ways. First, the extra mass at the roof level due to the green roof system had to be accounted for and incorporated into the seismic load calculations. The adjusted modeling weights similar to those presented for the as built system are shown below in Table 14 and the full calculations can be found in Appendix I: Seismic Loading Recalculations.

Table 14: Story Weights Adjusted for ETABS Model

MODELING ADJUSTMENTS		
Level	**Weight(k)**	**Total(k)**
B1	8968	8968
1	To 2	
2	15899	
3	2527	
4	2527	
5	2531	
6	5421	
Penthouse	1543	30448

Second, determination of the seismic forces is dependent on the lateral system resisting it. The new Buckling-Restrained Braced Frames presented a new value for the Response Modification Coefficient, R, specifically R=7 which will be discussed in the next section. Using S_{DS}=0.400, S_{D1}=0.501, and I=1.25 determined previously:

Determine new Cs using Buckling-Restrained Braces article to determine new Ta

$$Cs = \frac{0.400}{7/_{1.25}} = 0.0714$$

$$Ta = 0.3h^{0.75} = 0.3 * 85.25^{0.75} = 8.42\ seconds$$

$$Cs, max = \frac{0.501}{8.42 * \left(7/_{1.25}\right)} = 0.0106\ which\ controls$$

$V_{Office} = C_{s,max}w = 0.0106 * 30448\ kip = 323\ kip$ is the new office base shear

Determine the new conversion ratio between the steel and concrete systems

ρ=1.0 because SDC C

$$\frac{R_{Office}/_{\rho}}{R_{Parking}/_{\rho}} = \frac{7/_{1}}{4/_{1}} = 1.75\ versus\ 1.0\ of\ as\ built\ system$$

This ratio is used to determine the new base shear shown in Table 15 which still controls over the wind base shear found earlier. This new base shear is distributed into the story forces shown in Table 16.

Table 15: Determination of New Base Shear

New Base Shear for Distribution to Stories			
Cs=	0.0106		
Ratio=	1.25		
$V_{Base,Office}$=	323	kip	$V=C_s w$
$V_{Base,Parking}$=	987	kip	(Previous)
$V_{Base,Total}=1.25*V_{Base,Office}+V_{Base,Parking}$			
$V_{Base,Total}$=	**1552**	**kip**	

Table 16: Story Forces Adjusted for ETABS Load Cases

Modeling Adjusted Forces	
Level	FX (k)
B1	21
1	-
2	526
3	113
4	142
5	172
6	577
Sum=	1552

For comparison purposes, the story forces for the braced frames and the story forces for the buckling-restrained brace frames are shown in Table 17. The base shear for the BRBF is 25.6% of the conventional brace frame design.

Table 17: Comparison of Seismic Story Forces for Brace Frames and Buckling-Restrained Brace Frames

Seismic Story Force Comparison			
	BF	BRBF	
6	2258	577	
5	675	172	
4	557	142	
3	442	113	
2	2056	526	BRB as % of
B1	84	21	BF Force
Sum=	6072	1552	25.6

Next, the seismic load cases were recalculated and the ETABS lateral model updated to reflect the changes. The model was analyzed to find the controlling load combination for the braces the design axial loads which are summarized in Figure 35. The controlling load case 1.28D+L+0.2S+1.0E is a result of the seismic load combinations in section 12.4.2.3 of ASCE 7-05.

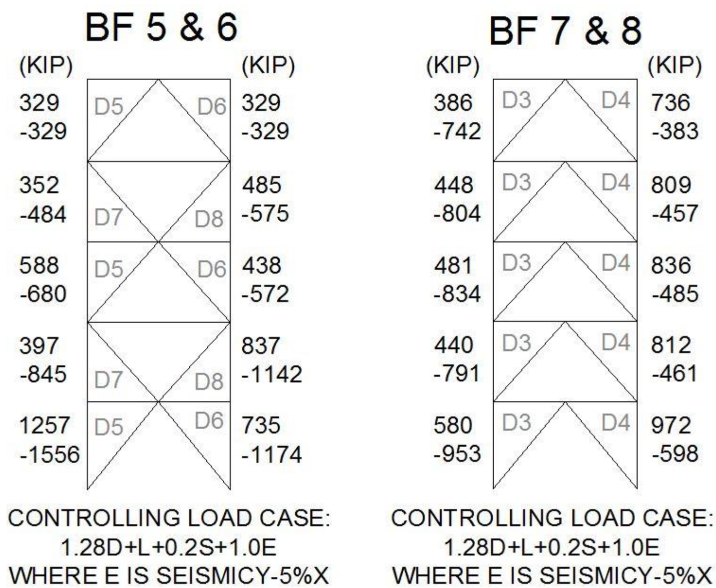


Figure 35: Design Axial Loads for Braces and Controlling Load Case

Buckling Restrained Braced Frames (BRBF)

After viewing a webinar on buckling restrained braced frames, I had an interest in studying them in my thesis project. The best layout for buckling restrained brace frames is a concentrically braced frame, which is already the configuration of the as built brace frames. The yielding core is encased in concrete and a steel HSS covering. The as built braces are structural HSS braces. The architectural look will not dramatically change because the layout of the braces can remain the same and the difference in profile of a BRB compared to a HSS brace is minimal. Additional calculations and ETABS output is provided in Appendix J: BRBF.

Code Considerations

In order to study their impact in comparison to the as built HSS braces, I have assumed that in my redesign the braced frames will be seismically detailed according to AISC 341-05 so that I can advantage of the higher "R" value allowed by ASCE 7-05. In the as built project, the beam-column connection of the braced frames is not a moment-resisting connection shown in Figure 36. For comparison purposes, and a conservative design approach the beam-column connections will also be non-moment-resisting connections. In ASCE 7-05 Table 12.2-1, the R value for buckling-restrained braced frames with non-moment-resisting beam-column connections is 7 and for Seismic Design Category C there is no height limit for this system type. Finally, Cd=5.5 and Ω_0=2. The over-strength factors were assumed to be the values suggest by the Buckling Restrained Braces article to be Bω=1.5 and ω=1.1 for the purposes of AISC 341-05 Section 16.2.

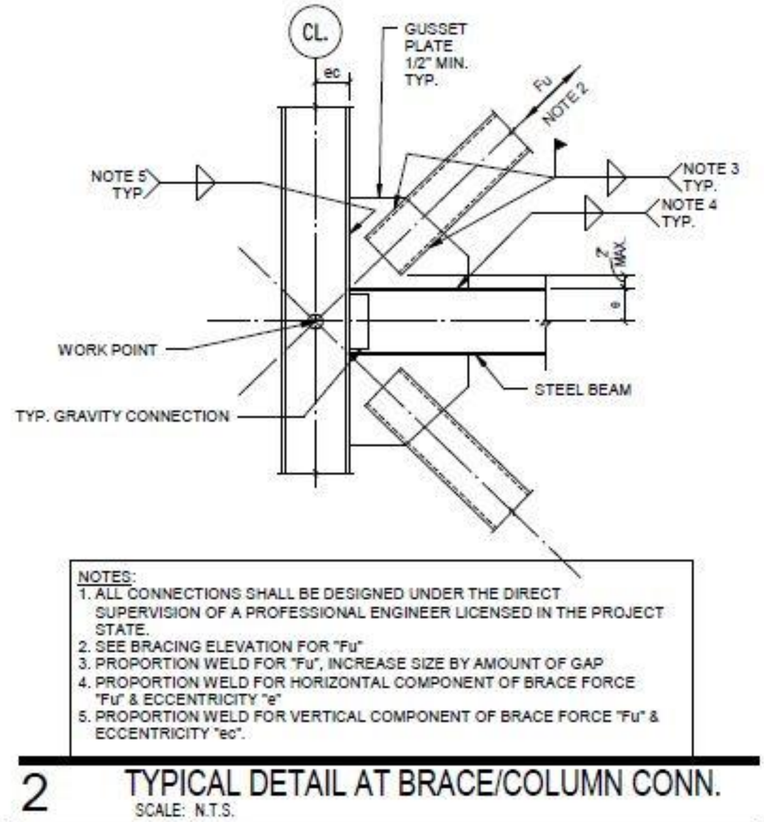


Figure 36: As Built Braced Frame Beam-Column Connection (Construction Documents)

ETABS Modeling Process

The modeling process used to analyze and design the buckling-restrained brace frames is suggested by brace manufacturer StarSeismic in the "StarSeismic Buckling Restrained Braces in ETABS Integrated Building Design Software" guide. StarSeismic provides a package of ETABS BRB sections available to download and use for analysis and design. After downloading this package and importing the sections into the ETABS section library, a new steel core material was defined. This material was initially assigned the minimum yield stress of 38 ksi and the minimum tensile strength to be 58 ksi and this material was assigned to the BRB sections. Preliminary sections were determined by the following equation:

$$Asc = \frac{Pu}{0.9Fyscmin}$$

The design axial loads were determined and presented previously under the seismic load revisions section. Upon calculating the minimum steel core area using Fy,sc=38 ksi, it was found that some of the braces on the first two stories failed capacity for the highest steel area that StarSeismic specifies. Next, an Fy,sc=46 ksi was used and the trial sizes shown in Table 18. The largest trial size was 38 square inches which would meet capacity and leave room for the sizes to increase to control drift. For comparison, the final design sizes and the difference are shown in the last two columns. The steel core material in ETABS was then edited to reflect the change in Fy,sc. Additionally, a cross-sectional area modification factor of 1.5 was applied to each of the StarSeismic BRB sections to reflect the effective axial stiffness of the braces when accounting for core plate transitions and end connections.

Table 18: Preliminary BRB Steel Core Area Sizes and Design Comparison

Preliminary BRB Sizes Compared with the Design Sizes					
	Fy,sc=	46	ksi		
	Story	Pu,reqd kip	Trial Asc Sq. in.	Design Asc Sq. in.	Difference Sq. in.
B5, B6	1	1556	38	48	10
	2	1142	28	36	8
	3	680	16	24.5	8
	4	575	14	21.5	8
	5	329	8	10	2
B7, B8	1	972	23	30	7
	2	812	20	22.5	3
	3	836	20	25.5	5
	4	809	20	23.5	4
	5	742	18	23.5	6

Next, the brace frame model was updated with the calculated trial sections above. In order to allow ETABS to properly design the buckling-restrained braces, the article suggests using the code defaults for special concentrically braced frames, SCBF, with some modifications and project specific values. These are summarized below in Table 19.

Table 19: BRB Design Factors

BRB Factors	
Frame:	SCBF
SDC:	C
I:	1.25
ρ:	1
S_{DS}:	0.4
R:	7
Ω_0:	2
Cd:	5.5

In the following Figure 37 and Figure 38 show the final beam, column and brace sizes for the BRBFs along with their code strength check ratio. In the final design, the column splices above Level 04 are reflected in the column sizing. The geometry of the brace frame layout meant that all columns were the same sizes for both directions of frames to maintain stiffness and represent the shared columns of braced frames 5, 7 and 8.

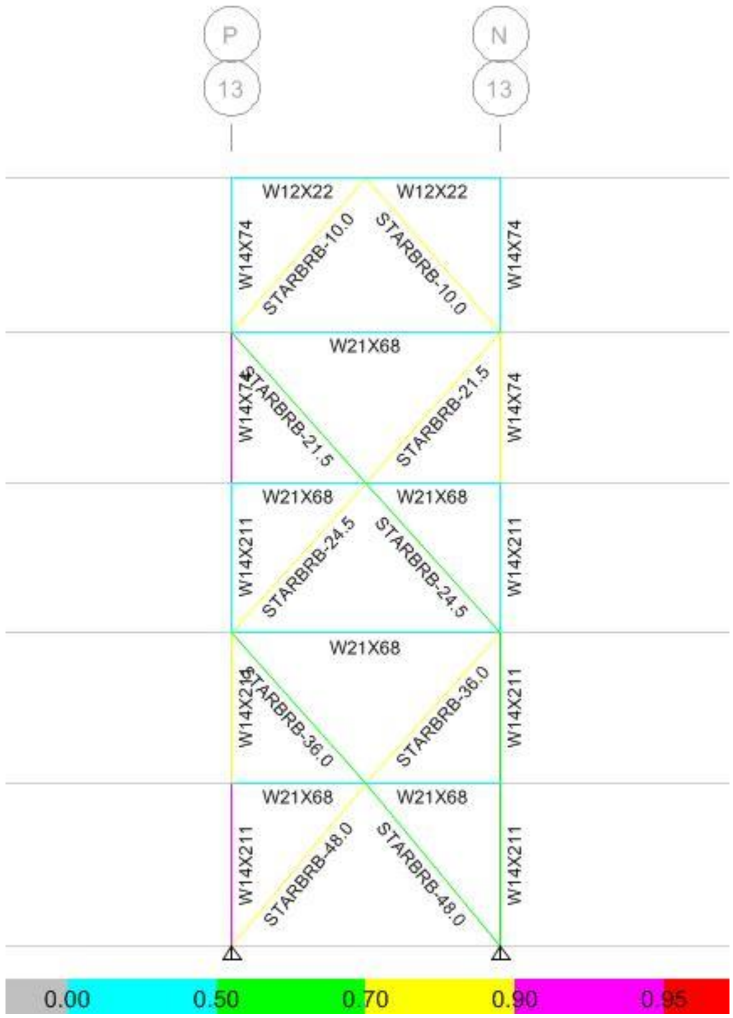


Figure 37: Buckling-Restrained Braced Frames 5 and 6 Sizes and Code Check

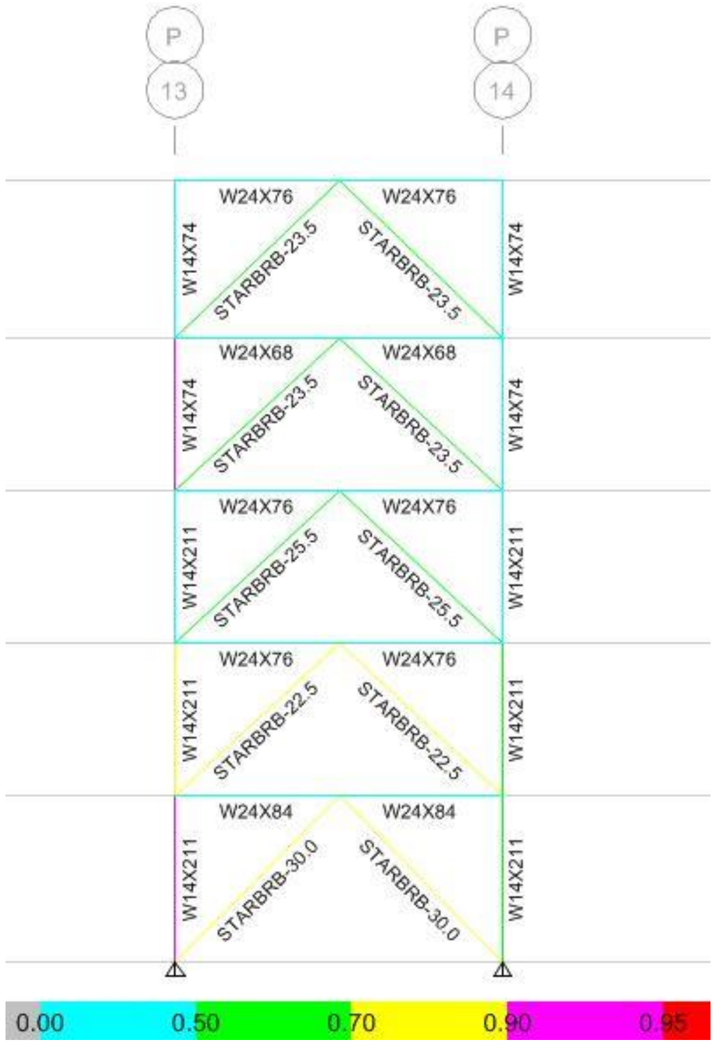


Figure 38: Buckling-Restrained Braced Frames 7 and 8 Sizes and Code Check

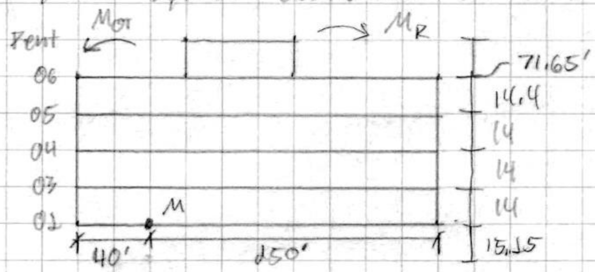
In summary, the gravity system revisions were acceptable for strength and deflections. The BRBFs were shown to work through analysis, but at the critical brace the available yield strength and steel core area are at maximum or very close to it. Buckling-restrained braces are not the most efficient system to carry the added load of the green roof garden causing high seismic loads.

System Comparison

The controlling tensile force on the foundation piers from lateral analysis is 4133 kip and according to the project drawings, the ultimate load capacity for the tension piers is 226 kips. Although foundation redesign is outside the scope of this report, the foundation would need to be redesigned as well to meet the new seismic forces incurred by the green roof. In Figure 39 below, the ratios of cantilever overturning moment versus the back span resisting moment for the as built system and green roof garden system are calculated. The overall trend is that by adding the green roof and controlling the loading did bring the ratio down meaning that the resisting moment became larger with respect to the overturning moment.

SYSTEM COMPARISON: CANTILEVER

• Original system evaluated at level 2 pivot



Neglect penthouse & Floors 2-5 b/c they don't change

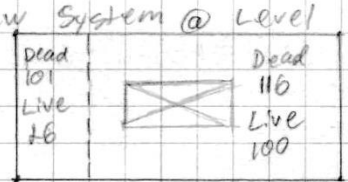
M_{OT}

$(38\text{ PSF})(115)(40)(56.4) = 9859$ ft k $\qquad \frac{9859}{51115} = 19.3\%$

M_R (23850 SF)

$(38)(56.4)(250\cdot115 - 35\cdot140) = 51{,}115$ 1k

• New System @ Level 2 pivot



M_{OT} Dead

$(101)(115)(40)(56.4) = 26203 \qquad \frac{26203}{156036} = 16.8\%$

M_R Dead

$(116)(23850)(56.4)/1000 = 156036$

M_{OT} Live

$(26)(115)(40)(56.4)/1000 = 6745 \qquad \frac{6745}{134514} = 5.0\%$

M_R Live

$(100)(23850)(56.4)/1000 = 134514$

M_{OT} D+L

$(101+26)(115)(40)(56.4)/1000 = 32949 \qquad \frac{32949}{290550} = 11.3\%$

M_R D+L

$(116+100)(23850)(56.4)/1000 = 290550$

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Figure 39: Cantilever Overturning and Resisting Moments for the As Built Project and the Green Roof Garden Project

Construction (Breadth 2)

The as built roof system and structure was compared to the green roof garden roof system and structure through a cost comparison using RSMeans Building Construction Data. Then, a schedule comparison using the same systems and quantities as the cost comparison was carried out and the duration of the tasks of both project options compared. Due to the sensitive nature of detailed cost, schedule, and site information the as built system and green roof garden systems were both taken off and compared to each other after being calibrated to the activity durations supplied by Clayco.

First, a cost analysis of the as built system for the roof system, roof framing, and gravity trusses was taken off. The values presented here are for the South Tower specifically, as stated in the project scope, however the values will be duplicated for the North Tower. The results are summarized below in Table 20. This project total includes allowances for waste and accessories; Missouri state sales tax, general conditions and contingency as suggest by RSMeans. The adjusted cost was then modified for time and location. The complete calculations are available in Appendix K: Construction Breadth Calculations. For framing sizes not specifically shown in RSMeans, the cost and schedule information was interpolated using the next higher and lower entries as appropriate. Additionally, RSMeans suggests a 10% increase in material quantity of steel framing to account for connections, bolts and other accessories. Similarly, it also suggests a 10% material quantity increase for concrete waste. Finally, a 10% waste allowance for the plants of the green roof was assumed because of the delicate nature and care needed to maintain plant health prior to planting on the roof.

Table 20: As Built Project Cost Summary

As Built Cost Summary		
		Notes
Sum=	$1,260,168.14	10% Overhead and Profit
Sales Tax:	1.04	State of Missouri, 4%
Adjusted Cost=	$1,310,574.87	
General Conditions:	1.1	Assume 10%
Adjusted Cost=	$1,441,632.35	
Cotingency:	1.05	Assume 5%
Adjusted Cost=	$1,513,713.97	
Location:	1.026	St. Louis, Missouri
Adjusted Cost=	$1,553,070.53	
Time:	1.106	
Adjusted Cost=	$1,717,696.01	
Total Cost=	**$1,717,700**	
Cost Per Sq. Ft.=	**$60.38**	

The next step was to perform a cost analysis of equivalent scope for the green roof garden system after the system was analyzed and designed. The results are shown below in Table 21 which follows the same process and adjustments. The cost analysis of the two systems showed a 236% percent increase in cost per square foot for the green roof garden addition.

Table 21: Green Roof Garden System Cost Summary

Green Roof Garden Cost Summary		
		Notes
Sum=	$4,236,440.18	10% Overhead and Profit
Sales Tax:	1.04	State of Missouri, 4%
Adjusted Cost=	$4,405,897.79	
General Conditions:	1.1	Assume 10%
Adjusted Cost=	$4,846,487.57	
Cotingency:	1.05	Assume 5%
Adjusted Cost=	$5,088,811.94	
Location:	1.026	St. Louis, Missouri
Adjusted Cost=	$5,221,121.05	
Time:	1.106	
Adjusted Cost=	$5,774,559.89	
Total Cost=	**$5,774,600**	
Cost Per Sq. Ft.=	**$202.97**	

Next, a schedule analysis and comparison for both project options was carried out using the same scope and items from the cost analysis. In order to calibrate my as-built project duration to the to the actual activity duration of ten days given by Clayco, the number of crews was modified until my duration was similar to the duration provided. For comparison purposes, similar decisions for numbers of crews were made in the determination of green roof garden system duration. The results are shown below in Table 22 and complete calculations are available in Appendix J: BRBF Calculations and ETABS Output.

Table 22: Summary of Assembly Durations per Project Option

Summary of Schedule Durations			
	Trusses	Roof Framing	Roof System
As Built	1.77	5.57	10.18
Green Roof	1.54	5.63	68.55

Overall, adding the proposed green roof garden would add an estimated $4,056,900, or $142.59 per square foot to the project for each office tower. The schedule comparison shows an additional 58 days to add the green roof garden per office tower. Although this is added time to the project, according to the schedule overview provided by Clayco none of the roofing system elements lay on the critical path of work because of building both towers at overlapping times. Project cost is the more critical concern for the green roof garden addition.

Construction and Logistics Concerns

Research into the components and construction of a green roof system from the references listed at the beginning of this report stressed some specific concerns in terms of the construction process and site logistics.

In terms of construction, all structural and protective work done to and performed on the roof must be completed prior to planting because the plants will not survive under foot and equipment traffic. It is vital that the waterproofing membrane is protected at all times by boards or sheets to prevent damage. The most common green roof failures are leaks, small and large, and plant loss which both can be prevented in part by paying special care to the waterproofing membrane. Drainage on a green roof has a first stage where the green roof system retains rainwater until a second stage where the system is full and the roof drains the same as a conventional roof.

Site logistics are vital to the successful establishment of the green roof system. The project site must be kept clean and materials must be protected from contamination that could alter the medium and seeds that could produce weeds. Plant plugs arrive on site in stacked palettes which should be unpacked and spread out as soon as possible after arrival to prevent plant damage. If plants will not be directly installed on the roof immediately after arrival, then special storage will be required to preserve plants and may take up a large amount of space. Storing materials and plants on site saves time and cost, but they should not be stockpiled on the structure to avoid overburdening the structure.

Conclusion

A scenario was created where the owner of RGA Global Headquarters wanted to investigate adding a green roof system to the office towers that also acted as an amenity space for the employees. Expanding on the structural analysis conducted in the fall semester, a green roof garden was researched and designed. Then, the loading was analyzed and a structural analysis under the new loading was performed. Additionally, the braced frames in the as built project were converted to buckling-restrained brace frames for the purpose of studying their analysis and design processes. Finally, a cost and schedule comparison study on both the as built project and the green roof garden addition was performed to discover the implications of adding the green roof garden.

The green roof garden design study involved a large amount of research into green roof systems and how to adapt general concepts to project specifics. A workable design was derived that uses systems and components available in industry as well as meets code requirements for wind, fire, and fall protection. Plantings were selected on the basis of their hardiness, aesthetics, and growth habits. Revisiting the design metrics, the green roof garden system design was a success. All of the metrics were met, with the final say of if the system has a reasonable initial cost is left to the owner. The cost falls into reasonable range of cost per square foot values for semi-intensive roof systems, but the owner will ultimately decide if the extra cost is worth the outcome.

The analysis of the gravity system under the new loading focused on the roof framing and the gravity trusses. The roof decking system of the as built project was converted to a composite deck system to support the increase in loading and to provide a more suitable surface for the construction and support of the green roof system. It was found that the as built truss load path and configurations were adequate for the redesign. In addition to resizing the truss members for capacity, deflection limitations for curtain wall attachments were imposed. These deflections were tracked throughout the load path of the trusses to ensure that the deflection criteria in each of the computer models were met, but also that the deflections were compatible between models.

The lateral system study included the conversion of braced frames to buckling-restrained brace frames. It was assumed that these frames in the green roof garden project option are seismically detailed in order to take full advantage of the buckling-restrained braces. This study found that although the geometry is well suited to conversion to BRBFs, the high additional loading of the green roof and high seismic forces do not make BRBFs an effective choice for this project. To control drift, the yielding of the steel core had to be at the maximum yield stress value and the controlling brace steel core area was just two inches below the threshold of steel core area that is normally manufactured.

A cost and schedule comparison was conducted for the as built project and the green roof garden project. This analysis revealed that although adding the green roof garden added about two months to the project schedule for each tower, cost is the critical factor. The green roof garden showed a cost increase of 236% per square foot.

In conclusion, based on the research, analysis and design outcomes of this study and considering the thesis project scope, adding a green roof garden to this project is feasible if the owner wishes to pursue this scenario. I would suggest that the lateral system remain conventional braced frames over buckling-restrained braced frames because of the higher seismic loads caused by the green roof garden addition. Overall, this thesis investigation was a success in exploring the design of a green roof system, revisions and understanding of the complexities of a large cantilever, the behavior of buckling restrained braces, and finally the cost, schedule and logistics associated with a green roof addition.