Reinsurance Group of America (RGA) Global Headquarters

Spring 2014

Final Report

16600 Swingley Ridge Rd.
Chesterfield, MO

Natasha Beck, Structural
Heather Sustersic
9 April 2014

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

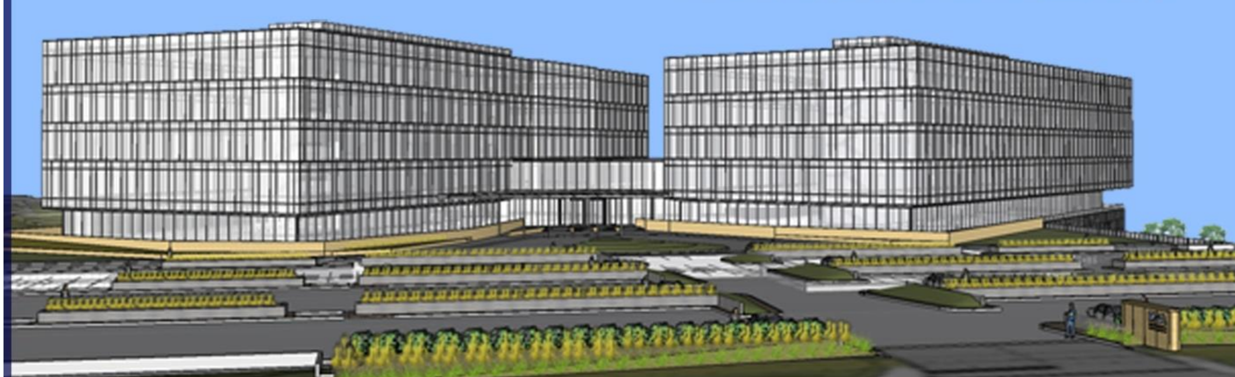
- Designed for year-round cooling
- Cooling towers serve three, 350 ton water cooled chillers
- Four 60,000 CFM air handling units serve the office towers
- A medium pressure loop on each floor for VAV branches
- Separate fan powered terminal units (FPTU) heat the floor cavity of the cantilever space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

I want to extend a special thank you to the following people for their assistance and support in making this year possible:

Heather Sustersic, Faculty Advisor

Ruby+Associates, Sponsor

Perry Esslinger and Tom Rolfes with Clayco, Information and Permissions

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.

Table of Contents

List of Figures

List of Tables

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.

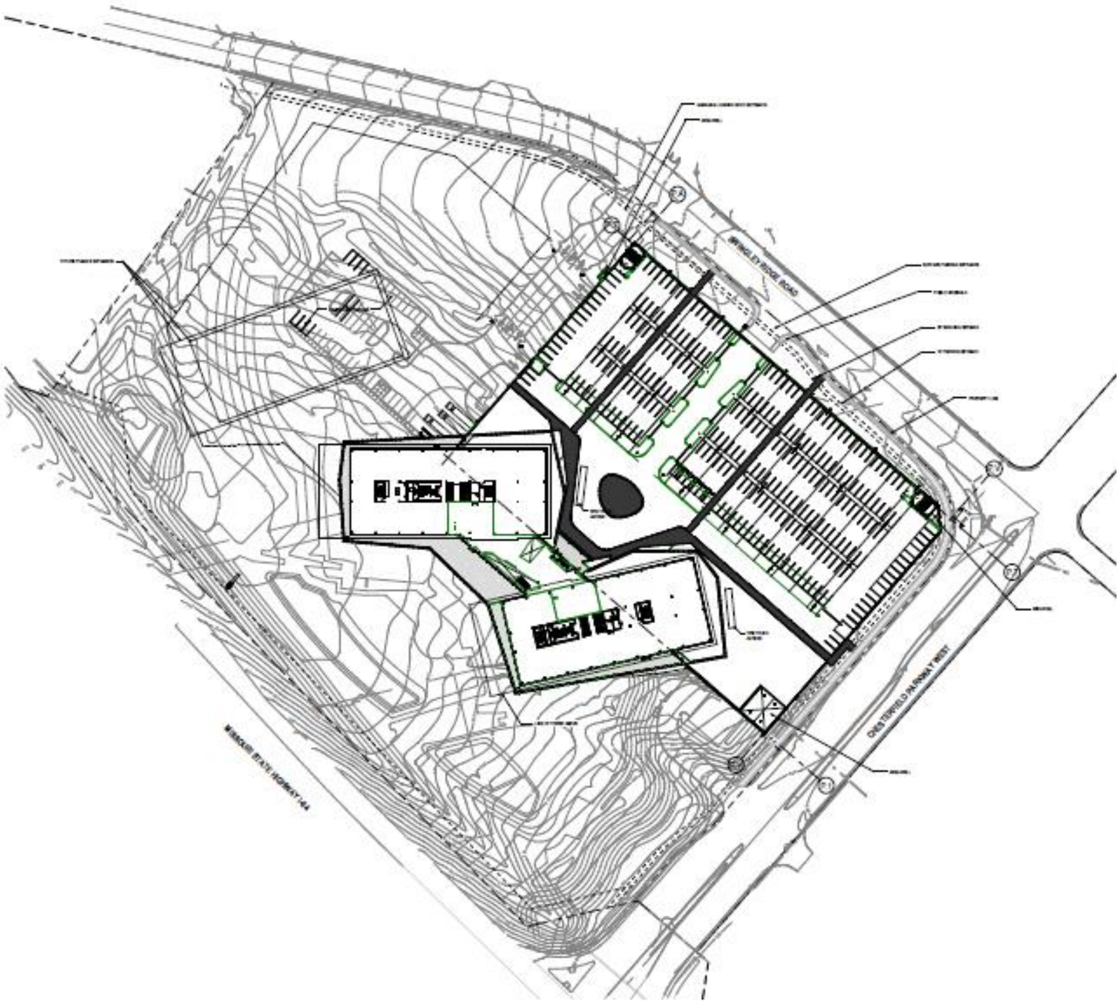


Figure 2: Site Plan Oriented to True North (Construction Documents)

Vicinity and Location Plans

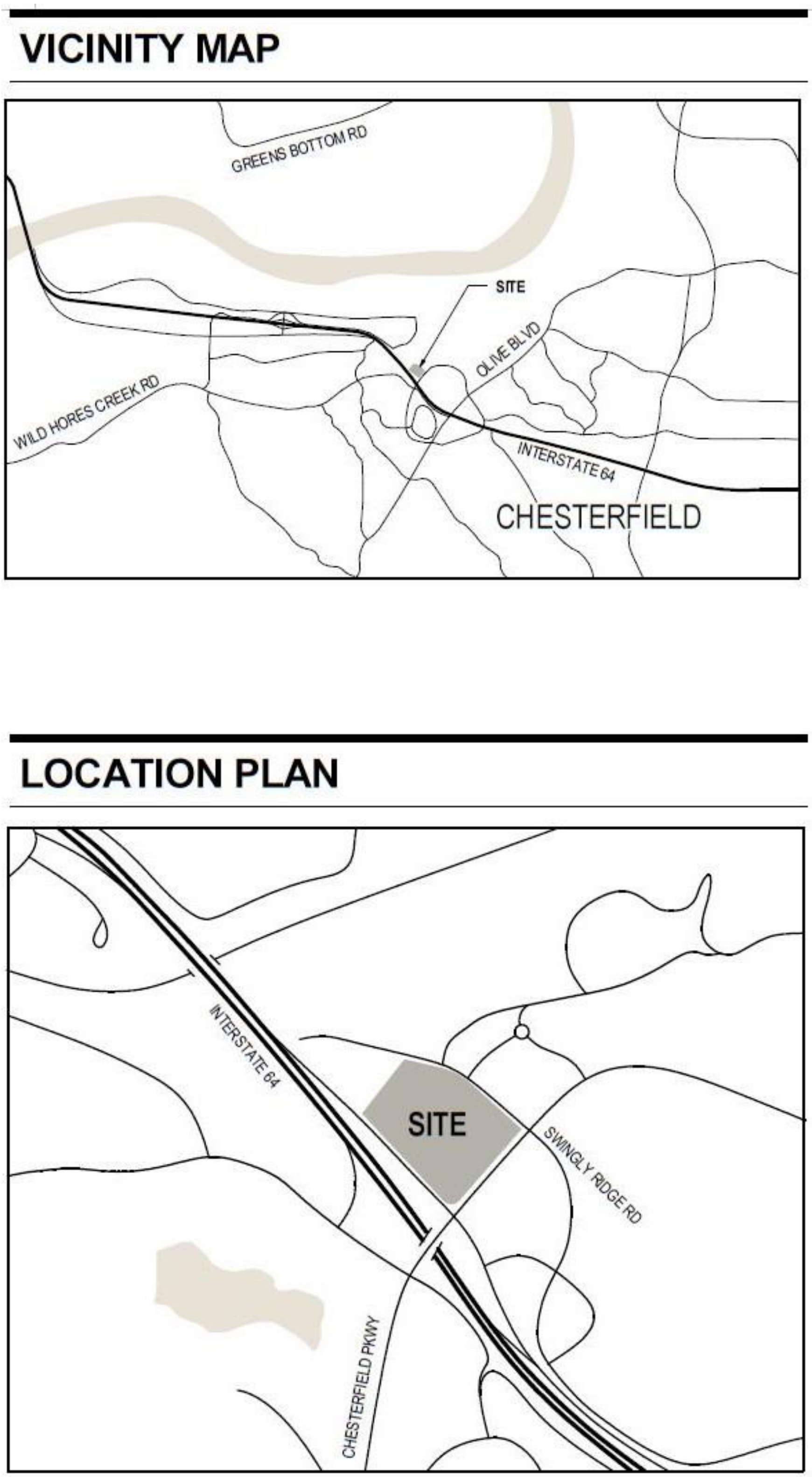


Figure 3: Vicinity and Location Plans by Gensler

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>.

"Challenges to Green Roof Construction." *GSA Green Roof Benefits and Challenges*: 77-88. Print.

Dakin, Karla, Lisa Lee Benjamin, and Mindy Pantiel. *The Professional Design Guide to Green Roofs*. Portland: Timber, 2013. Print.

Geschwindner, Louis F. *Unified Design of Steel Structures*. 2nd ed. N.p.: John Wiley & Sons, 2012. Print.

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

Green Roofs. N.p.: n.p., n.d. Print. Reducing Urban Heat Islands: Compendium of Strategies.

Jarrett, Albert R., and Robert D. Berghage. *Green Roofs*. Research rept. no. F262. University Park: The Pennsylvania State U, n.d. Print.

Peck, Steven W. *Award Winning Green Roof Designs: Green Roofs for Healthy Cities*. Atglen: Schiffer, 2008. Print.

Robinson, Kimberley, S.E., and Angus W. Stocking, L.S. "Buckling Restrained Braces-An Overview." *Continuing Education*. ZweigWhite, n.d. Web. 10 Dec. 2013. <http://continuingeducation.zweigwhite.com/>.

Snodgrass, Edmund C., and Linda McIntyre. *The Green Roof Manual: A Professional Guide to Design, Installation, and Maintenance*. Portland: Timber, 2010. Print.

Snodgrass, Edmund C., and Lucie L. Snodgrass. *Green Roof Plants: A Resource and Planting Guide*. Portland: Timber, 2006. Print.

US Dept. of Agriculture. "Missouri." Map. *Plant Hardiness Zone Map*. N.p.: US Dept. of Agriculture, n.d. N. pag. Print.

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.

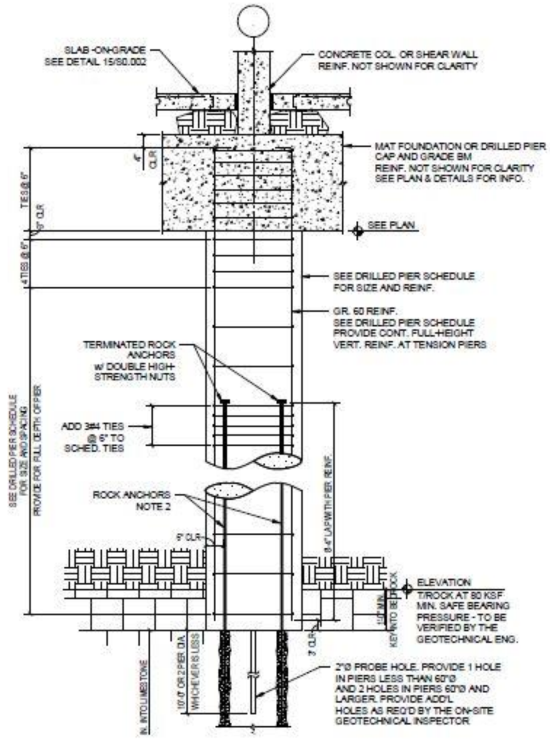


Figure 5: Typical Tension Pier Detail (Construction Documents)

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.

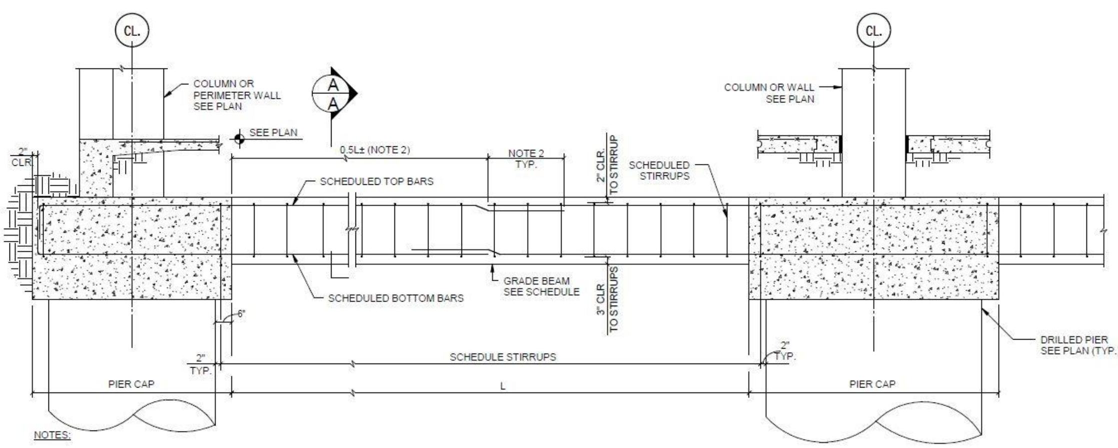


Figure 6: Typical Grade Beam Detail (Construction Documents)

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.

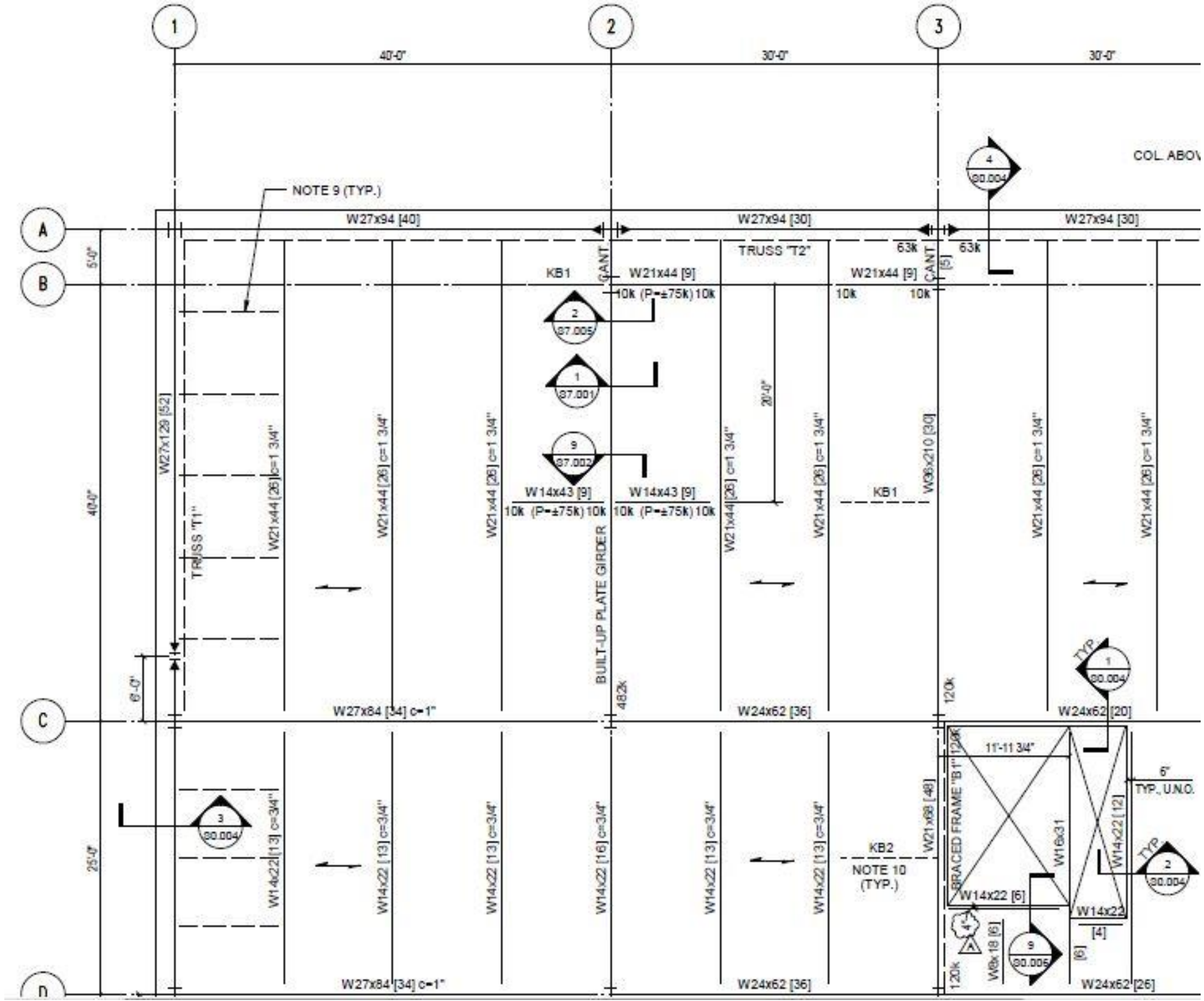


Figure 7: Partial Plan of Typical Bay Dimensions (Construction Documents)

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.

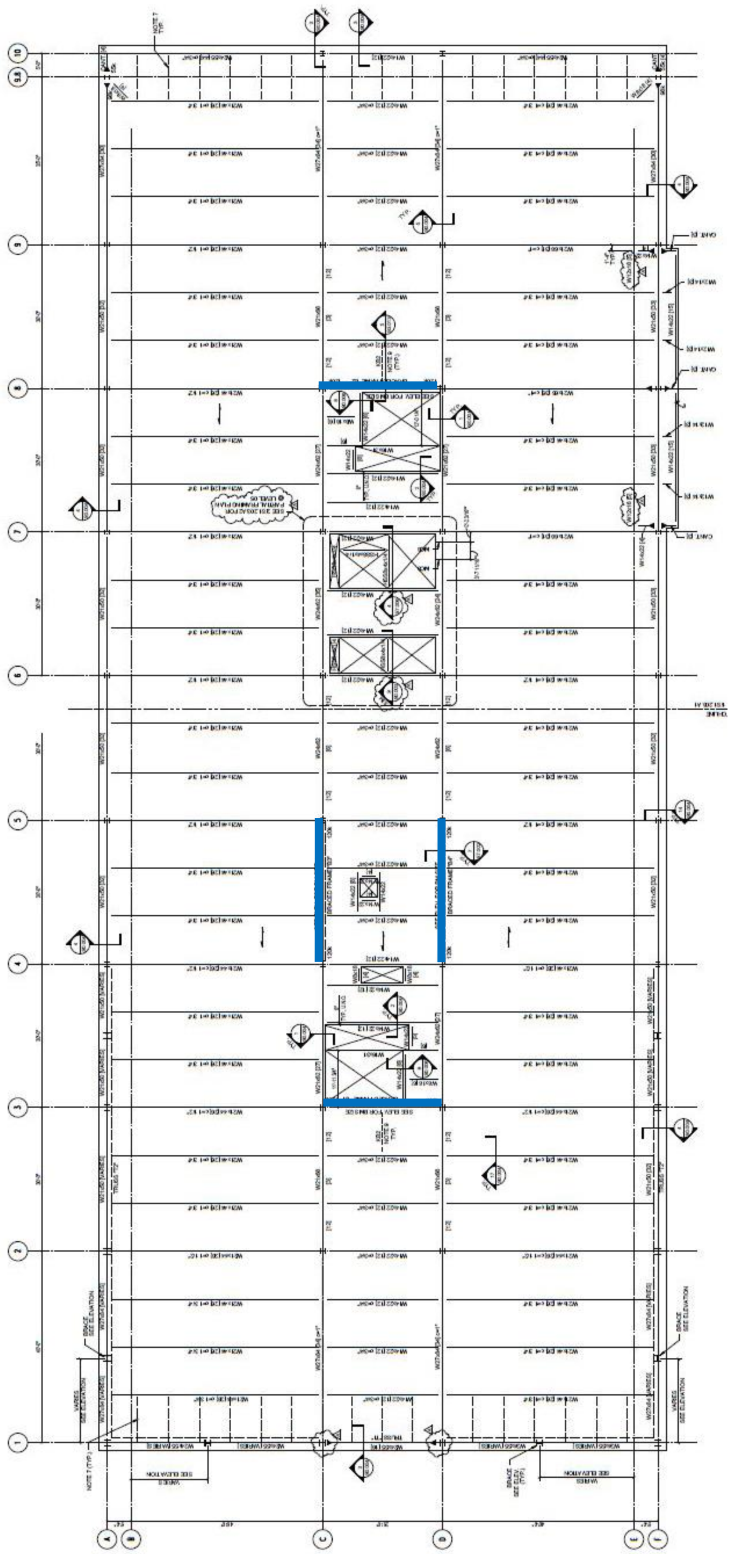


Figure 8: Braced Frame Locations

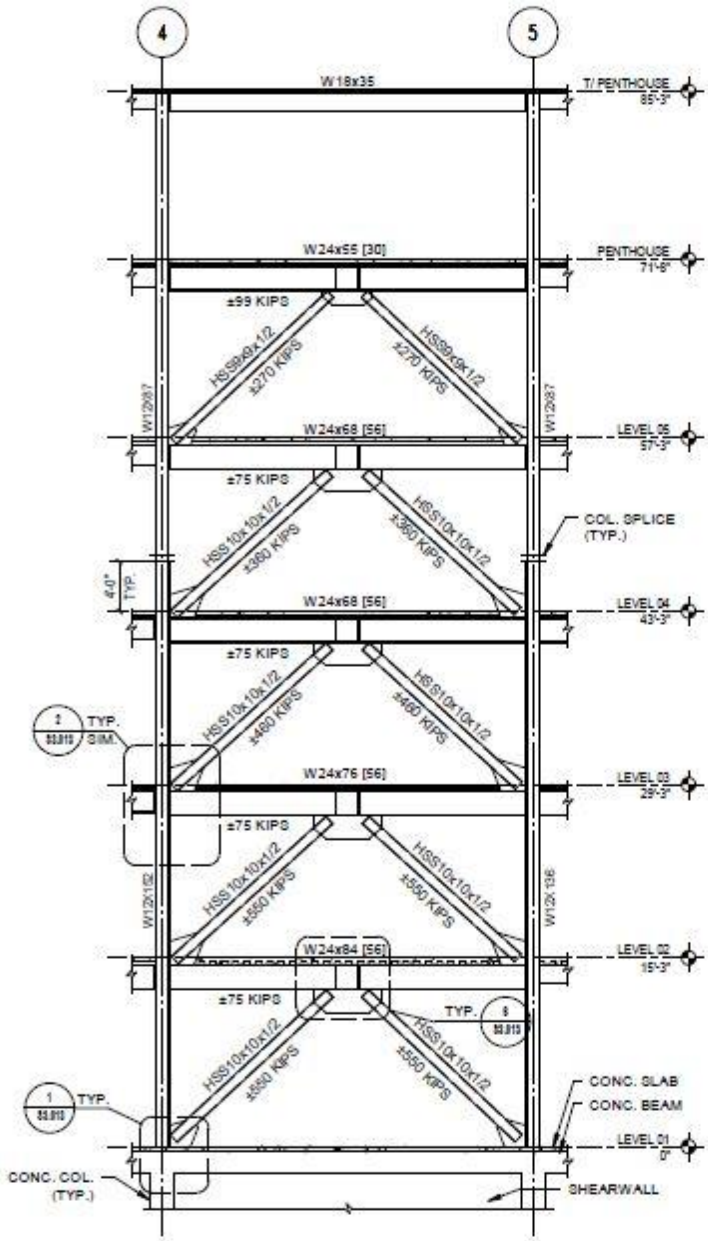


Figure 9: Typical Braced Frame Elevation with Penthouse Support Included (Construction Documents)

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.

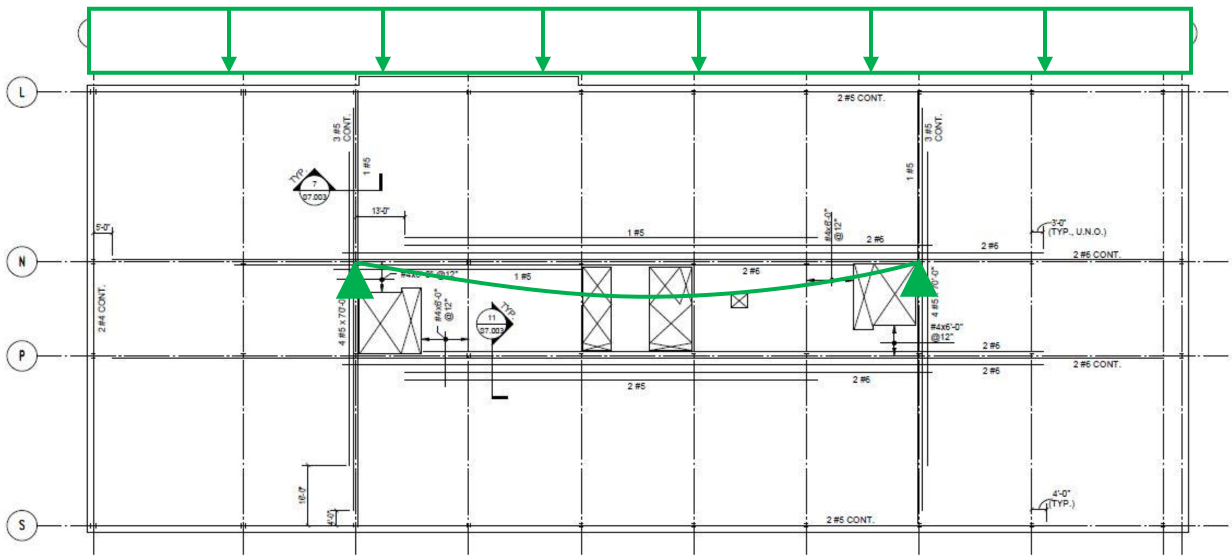


Figure 10: Floor Diaphragm Acting as a Beam Spanning Between Braced Frames

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.

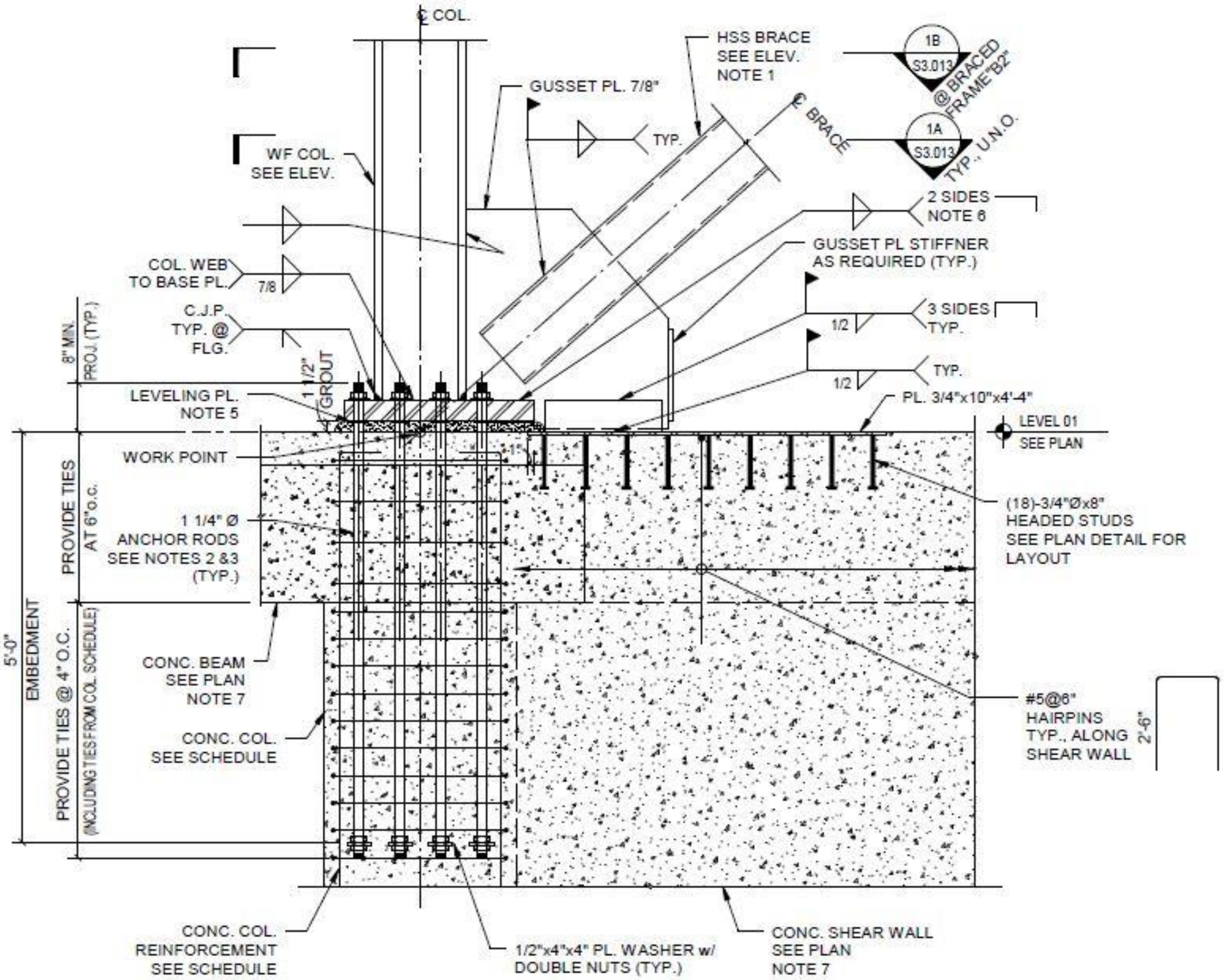


Figure 11: Typical Braced Frame to Shear Wall Connection (Construction Documents)

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.

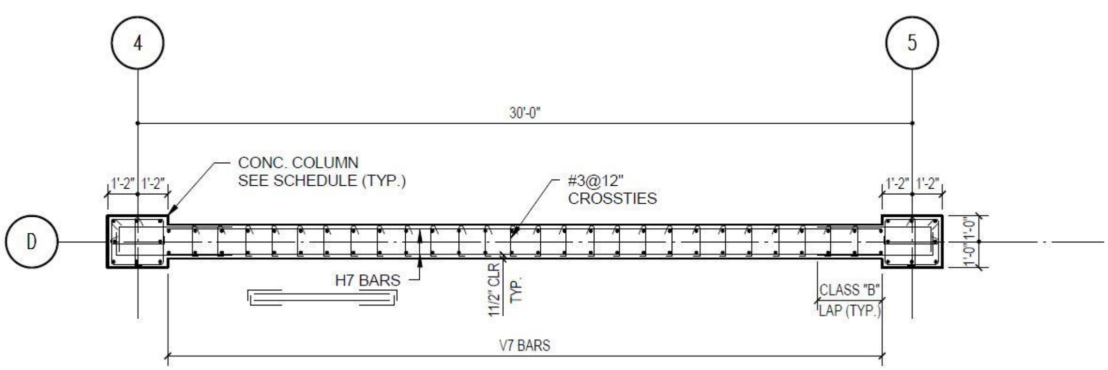


Figure 12: Shear Wall Sample Plan (Construction Documents)

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.

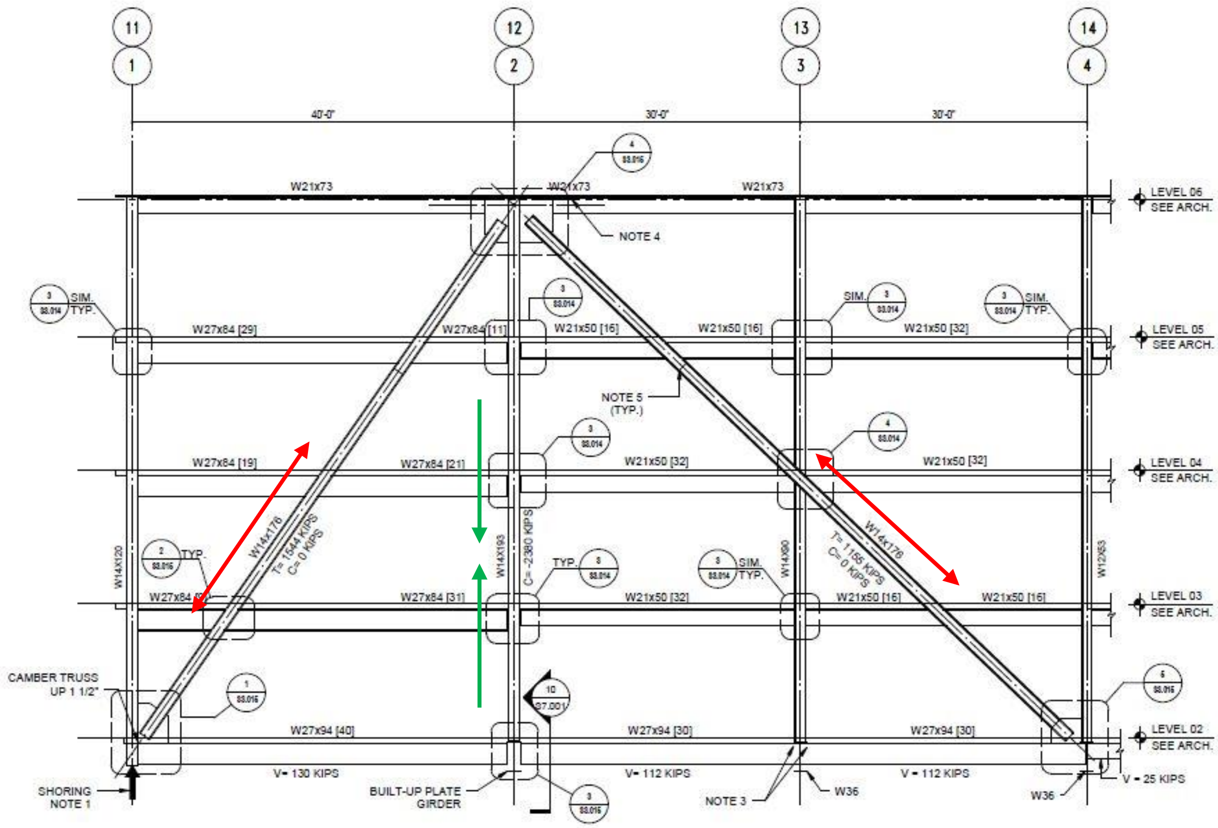


Figure 13: Truss T2 Elevation Highlighting Tensile and Compressive Forces

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.

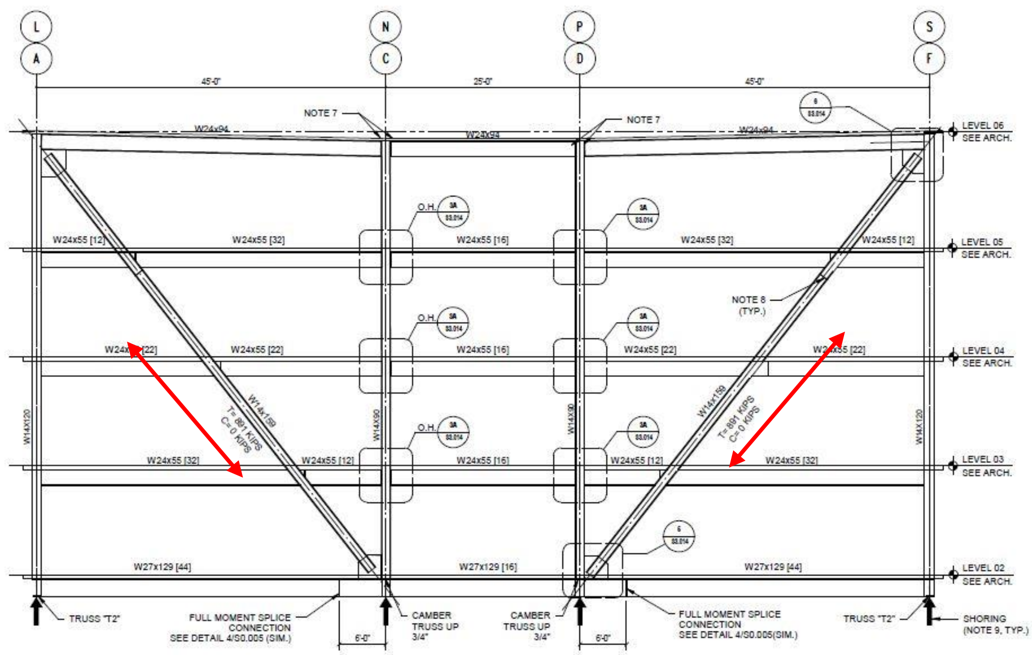


Figure 14: Truss T1 Elevation Highlighting Tensile Forces

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.

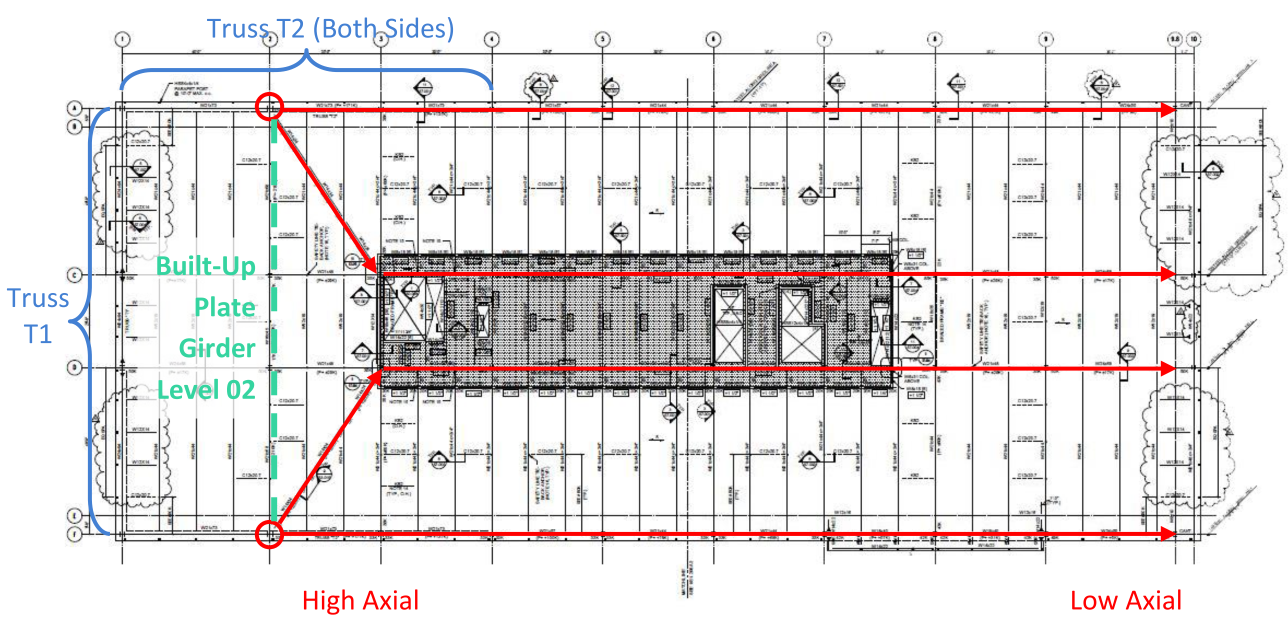


Figure 15: Roof Plan Showing Load Path of Truss System

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.

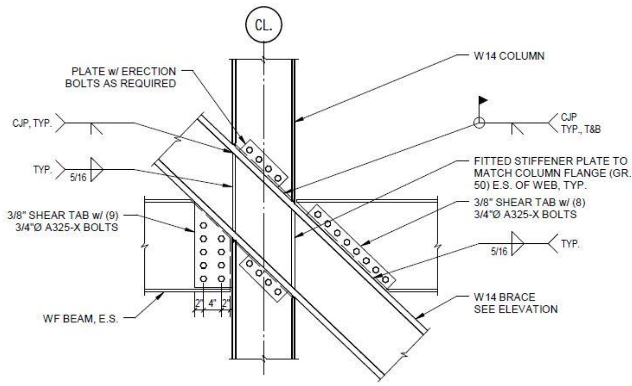


Figure 16: Truss Diagonal Joint Detail (Construction Documents)

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.

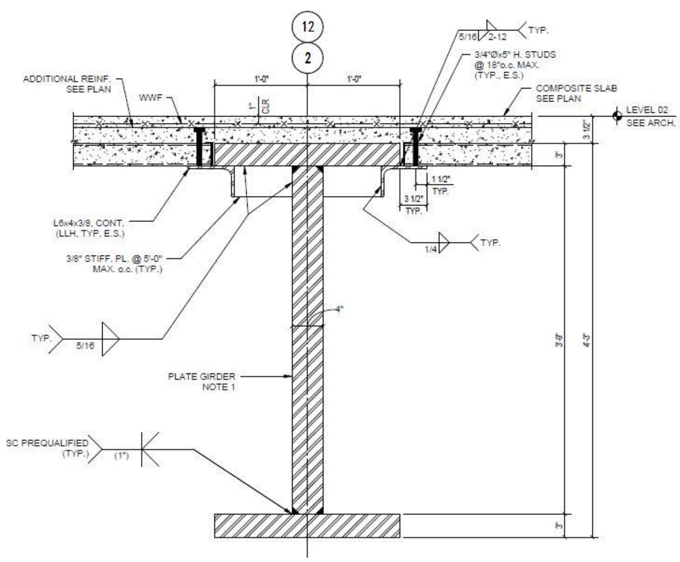


Figure 17: Plate Girder Detail (Construction Documents)

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

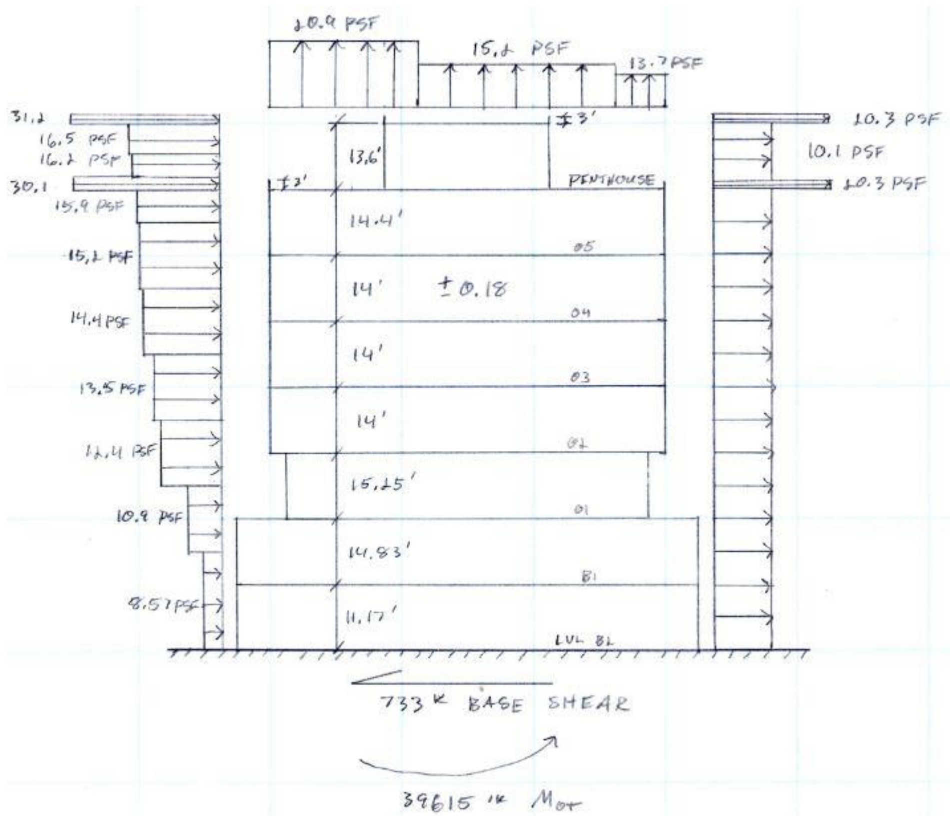


Figure 18: Wind Diagram for NE-SW Direction

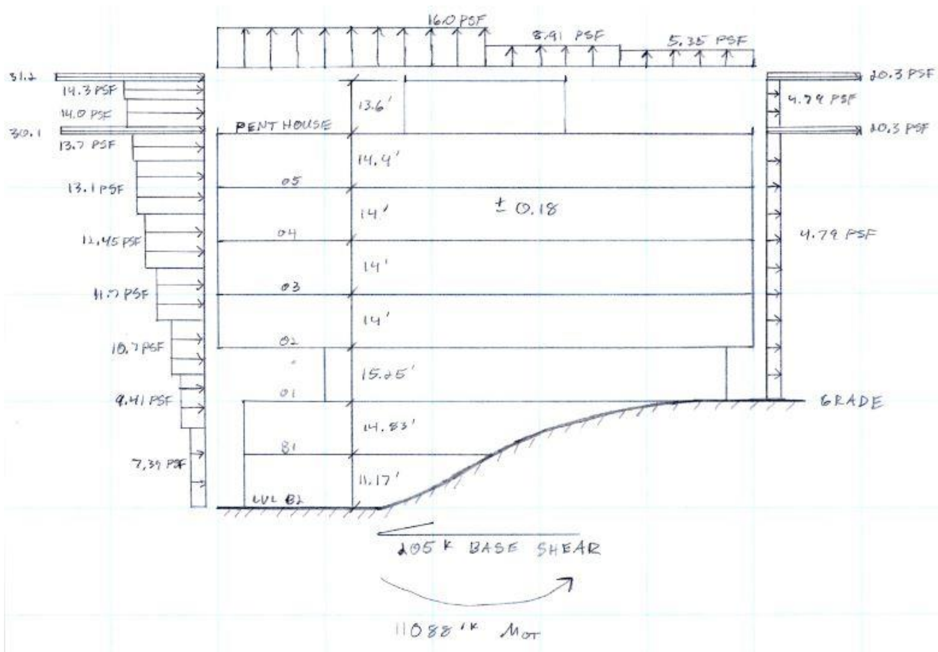


Figure 19: Wind Diagram for NE-SW Direction

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)	$w_xh_x^k$(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
		$\Sigma w_xh_x^k$=	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.

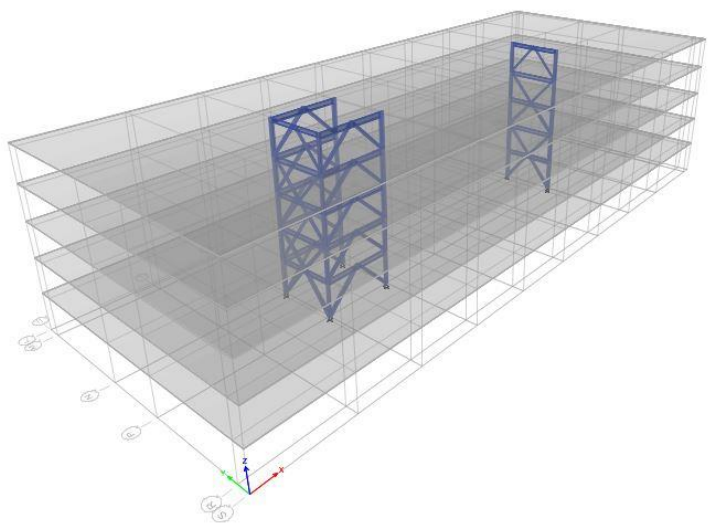


Figure 20: Office Structure Lateral ETABS Model

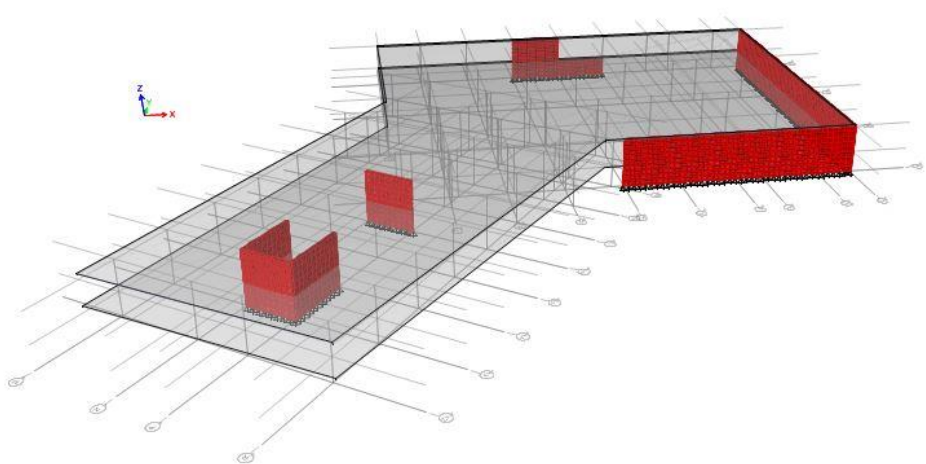


Figure 21: Parking Structure Lateral ETABS Model

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.

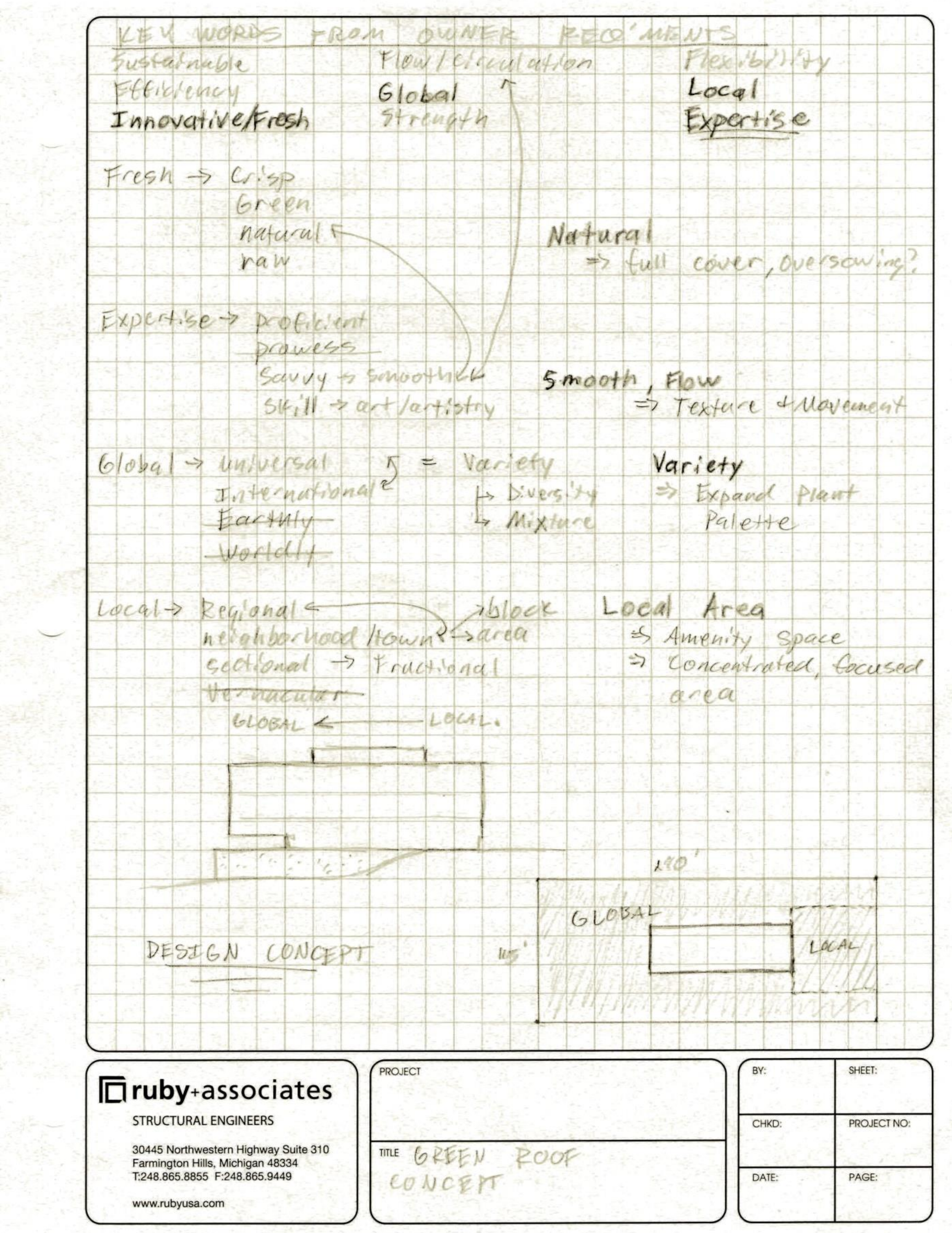


Figure 22: Word Study from Owner Recommendations

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.

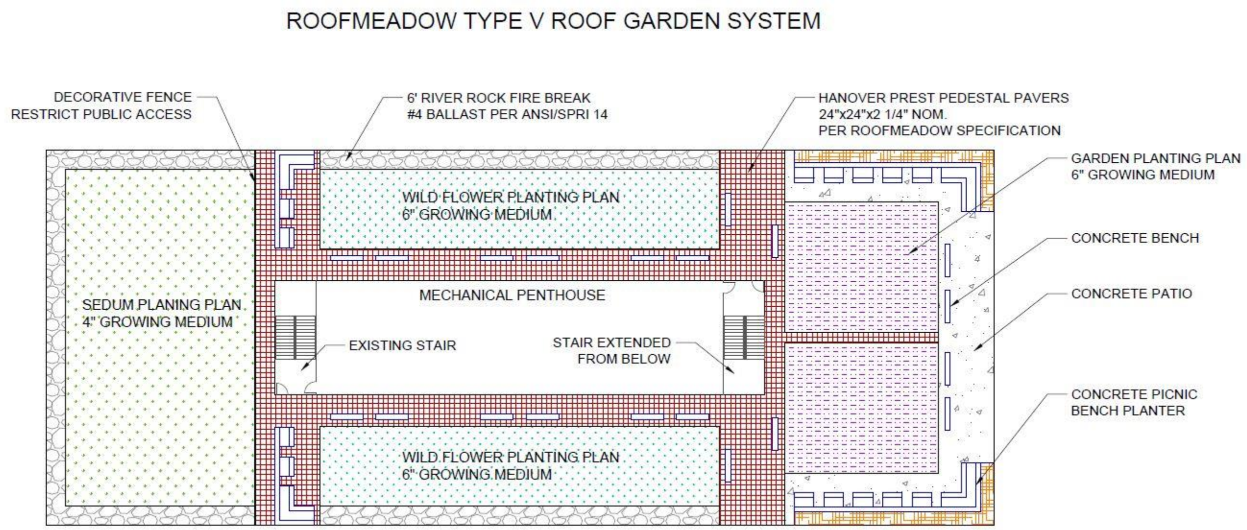


Figure 23: Schematic Plan of Green Roof

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.

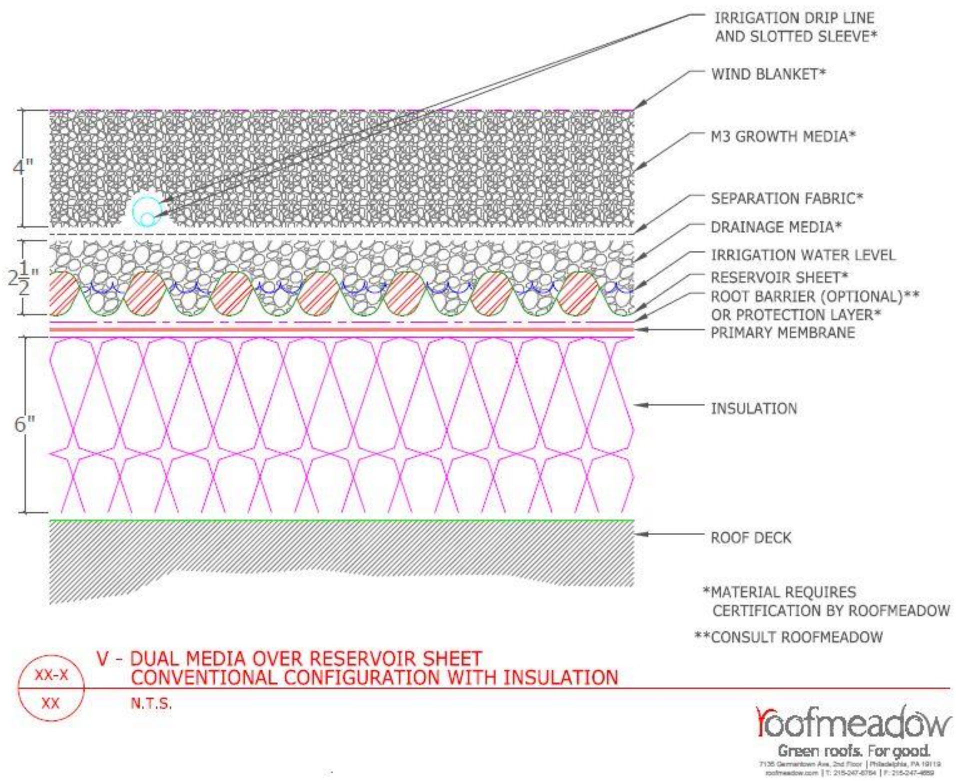


Figure 24: Edited Detail of 4" Growth Medium Roofmeadow Type V System

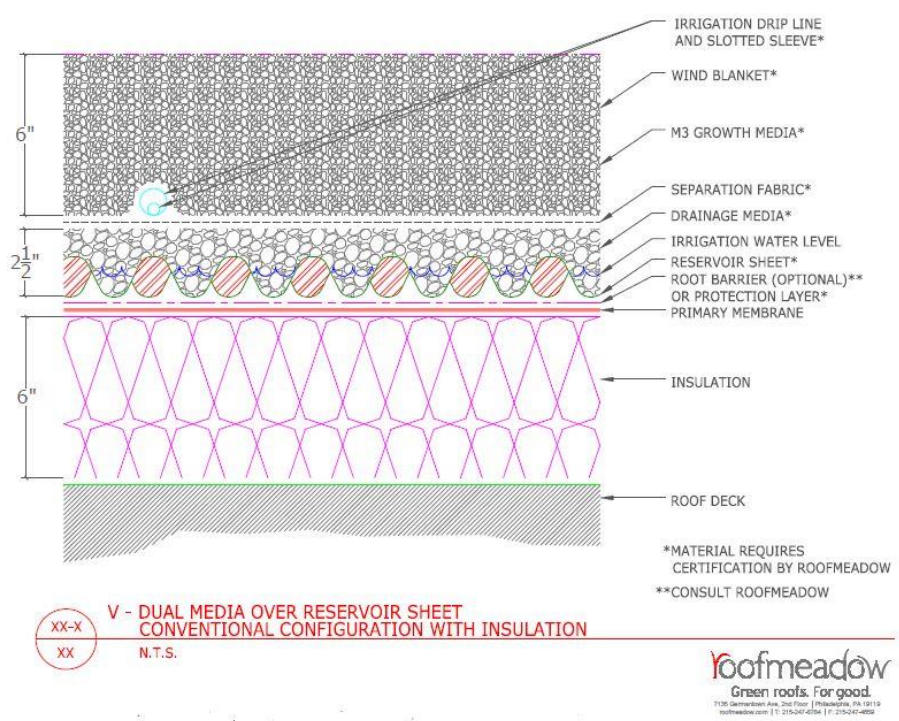


Figure 25: Edited Detail of 6" Growth Medium Roof Meadow Type V System

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.

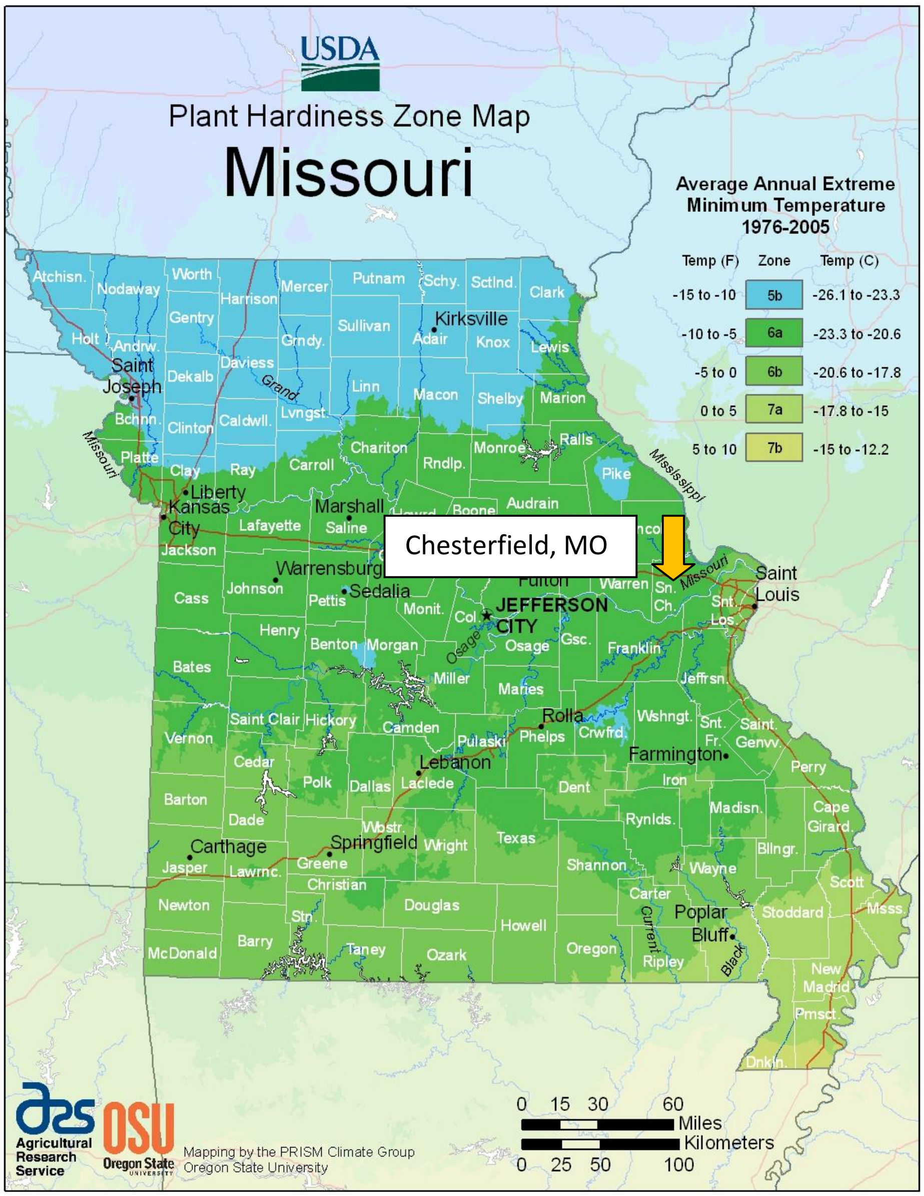


Figure 26: USDA Plant Hardiness Map for Chesterfield, MO

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

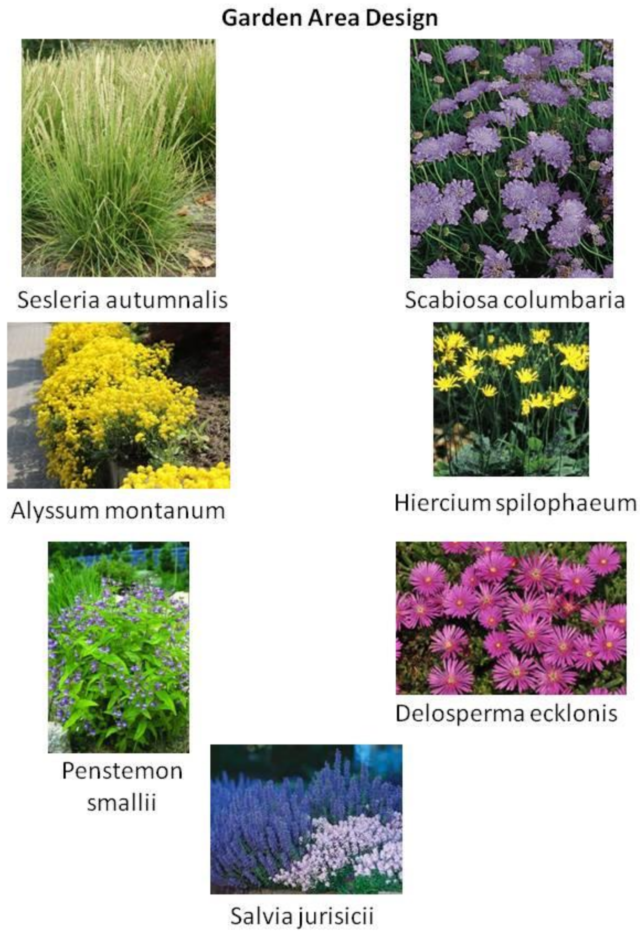


Figure 28: Garden Area Planting Selection

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.

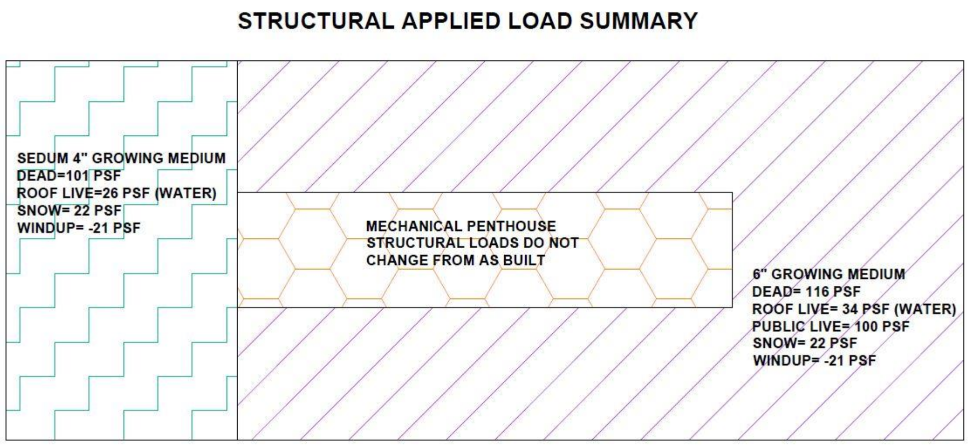


Figure 30: ETABS Applied Loading Summary

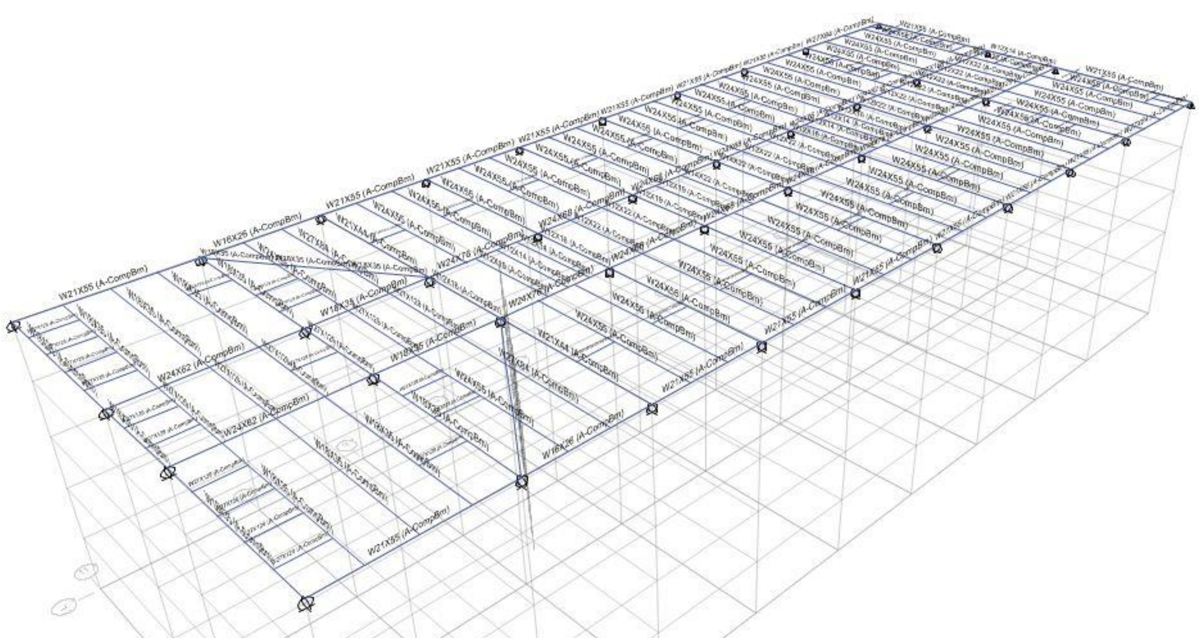


Figure 31: Gravity Roof Framing ETABS Model

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		

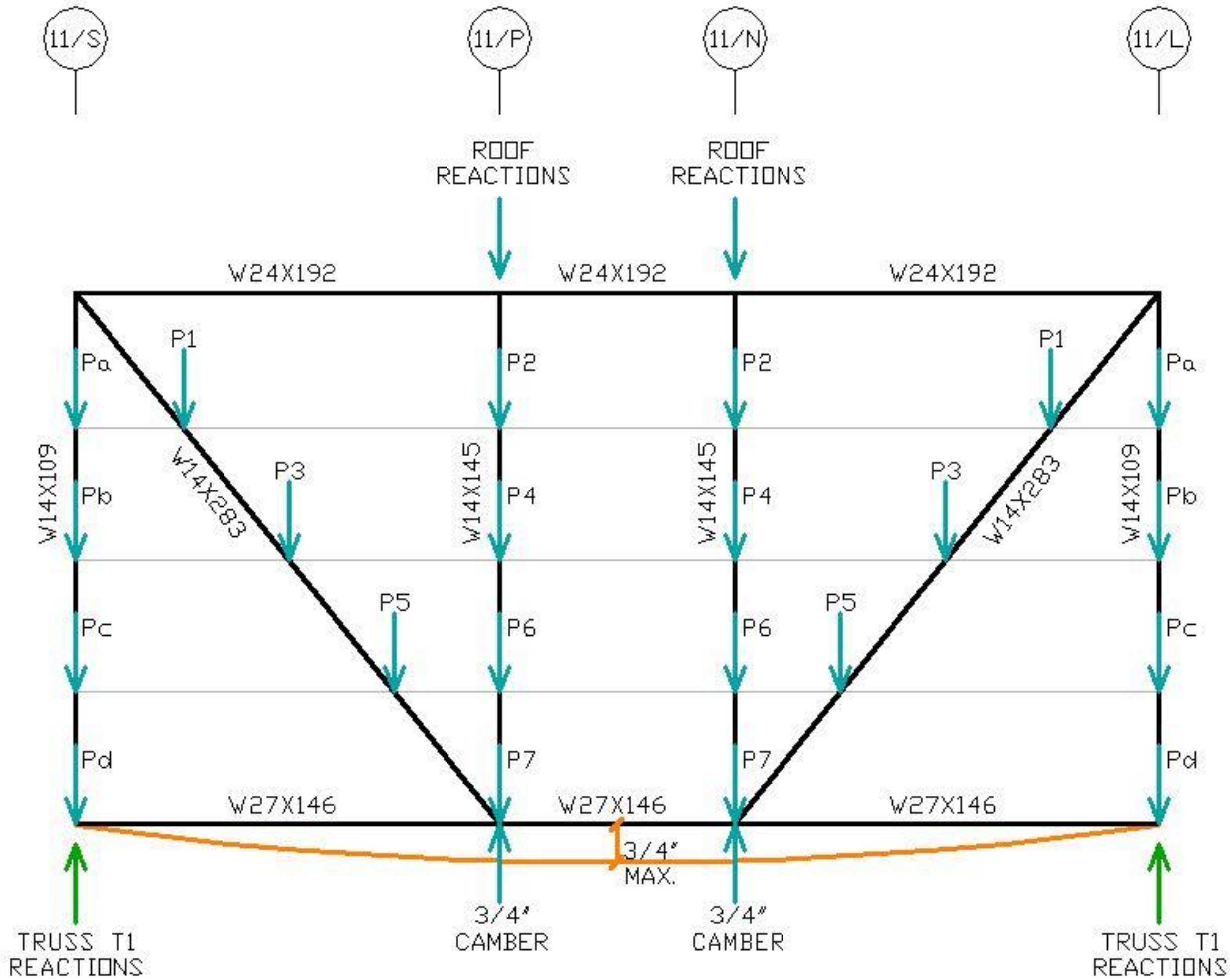


Figure 32: Line Drawing of Redesigned Truss T1 Showing Loads, Reactions, and Deflections

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

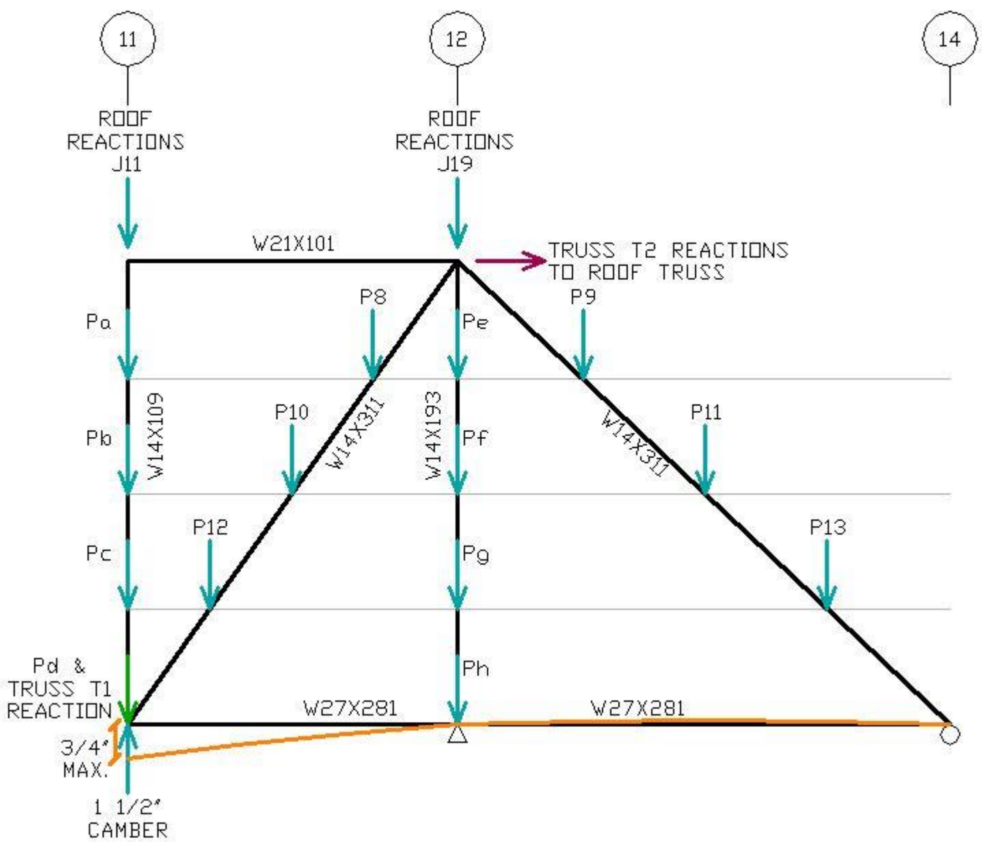


Figure 33: Line Drawing of Redesigned Truss T2 Showing Loads, Reactions, and Deflections

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

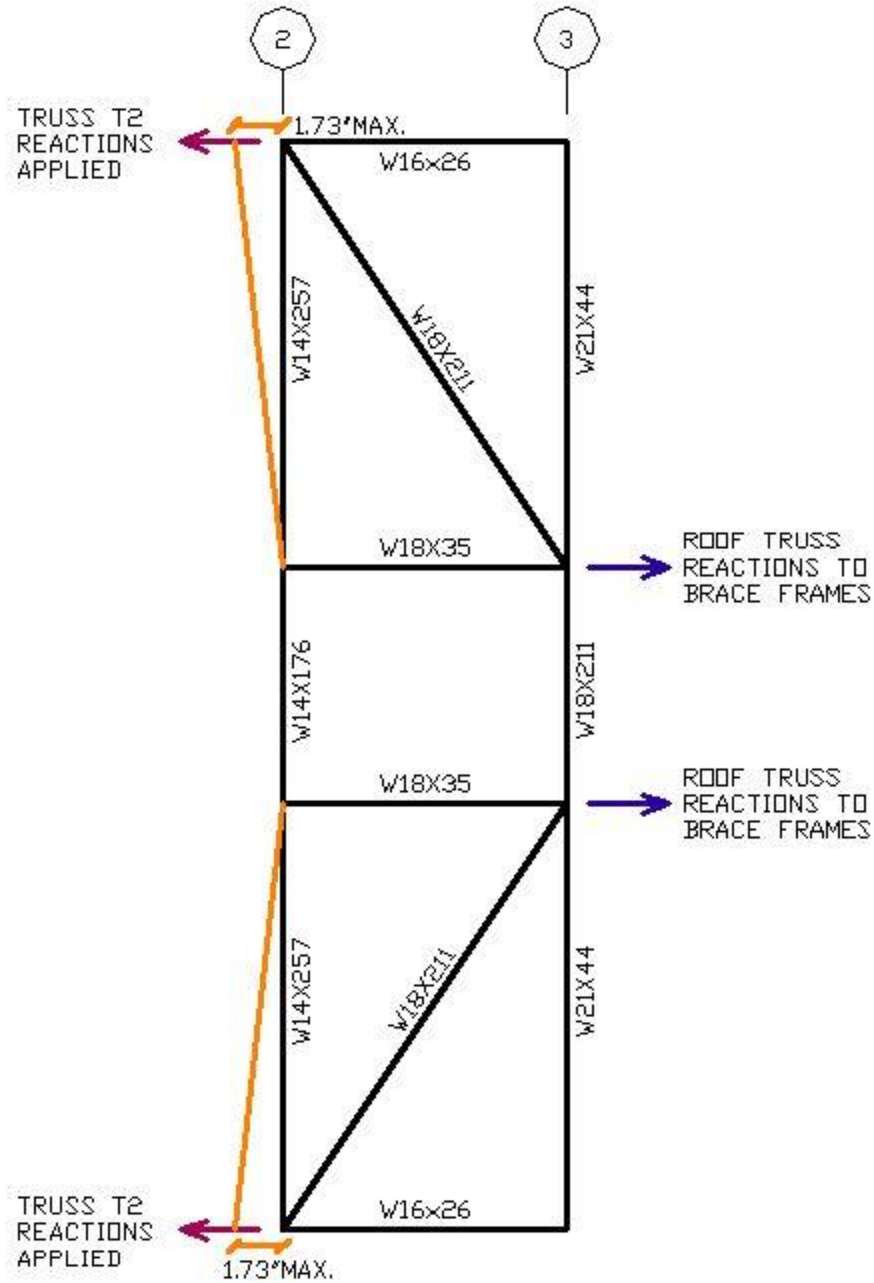


Figure 34: Line Drawing of Redesigned Roof Truss Showing Loads, Reactions, and Deflections

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.

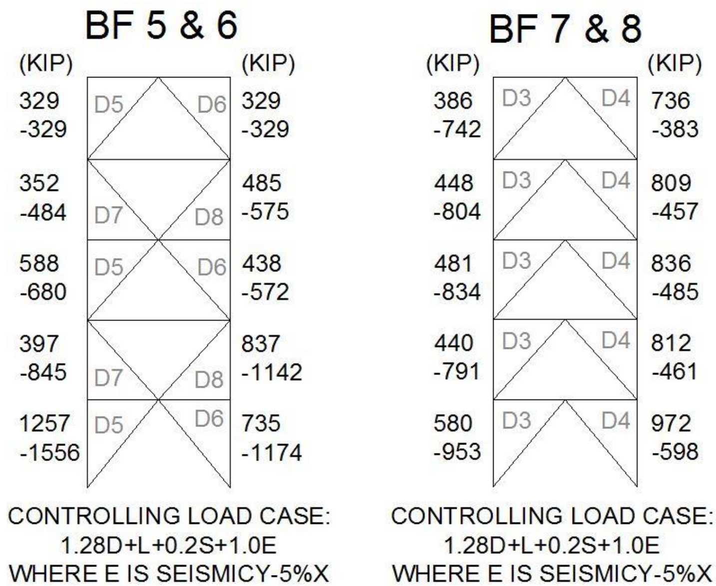


Figure 35: Design Axial Loads for Braces and Controlling Load Case

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.

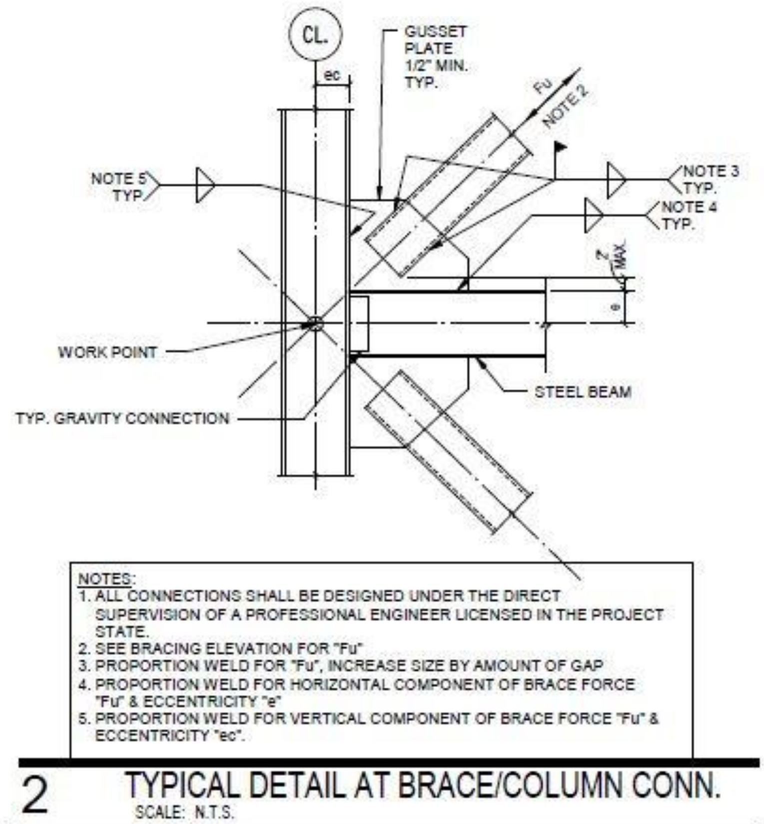


Figure 36: As Built Braced Frame Beam-Column Connection (Construction Documents)

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.

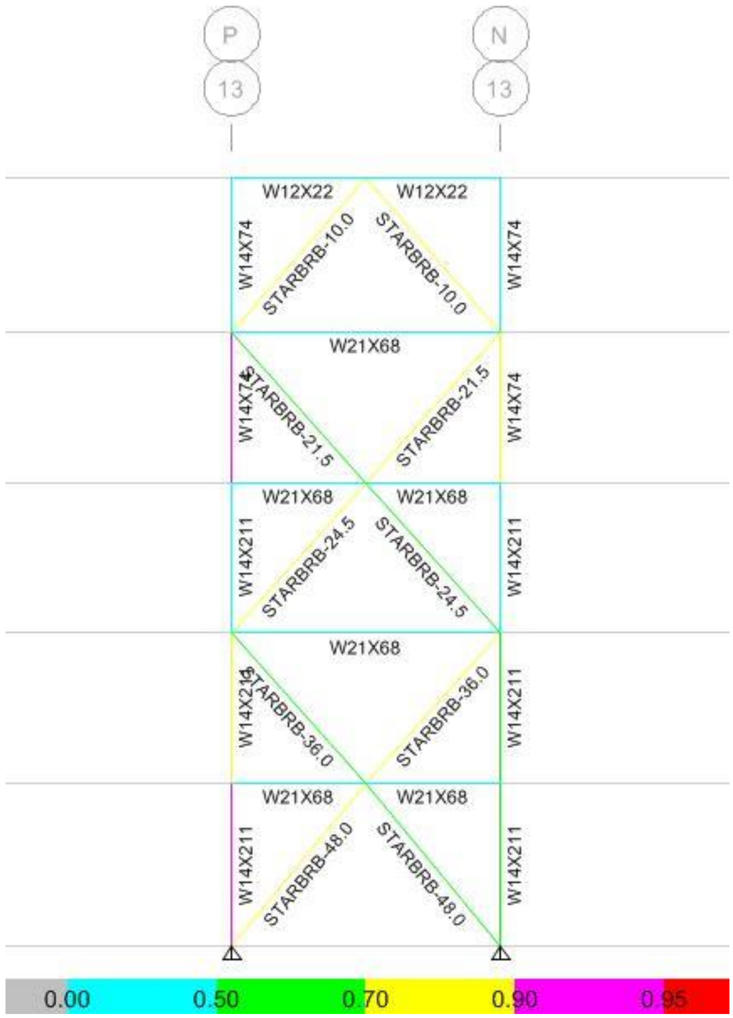


Figure 37: Buckling-Restrained Braced Frames 5 and 6 Sizes and Code Check

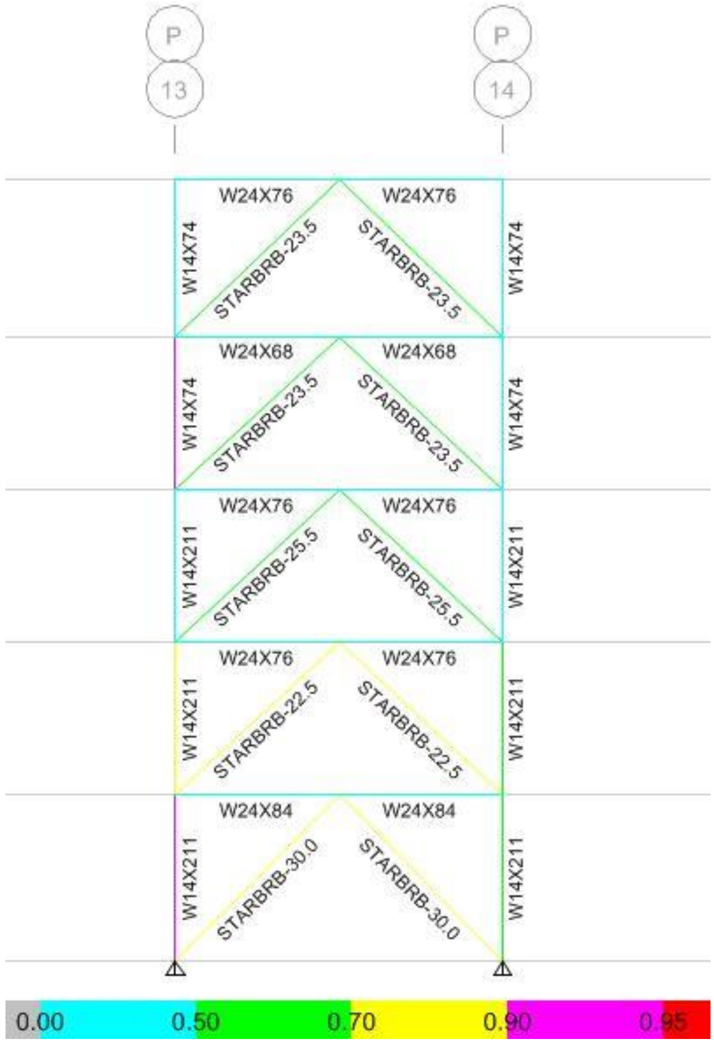


Figure 38: Buckling-Restrained Braced Frames 7 and 8 Sizes and Code Check

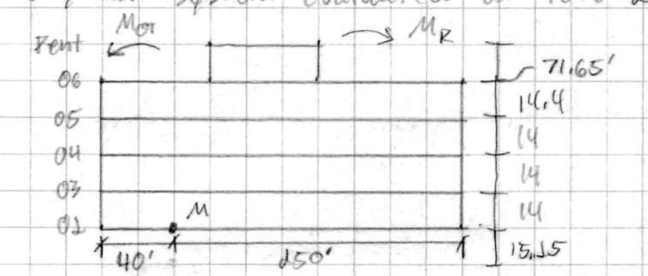
In summary, the gravity system revisions were acceptable for strength and deflections. The BRBFs were shown to work through analysis, but at the critical brace the available yield strength and steel core area are at maximum or very close to it. Buckling-restrained braces are not the most efficient system 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}

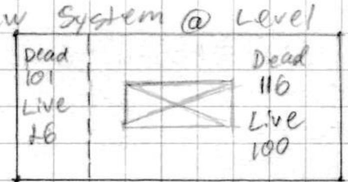
(38 PSF)(115)(40)(56.4) = 9859 ft k

$\frac{9859}{51115} = 19.3\%$

M_R (23850 SF)

(38)(56.4)(250·115 − 35·140) = 51,115 1k

• New System @ Level 2 pivot



M_{OT} Dead

(101)(115)(40)(56.4) = 26203

$\frac{26203}{156036} = 16.8\%$

M_R Dead

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

M_{OT} Live

(26)(115)(40)(56.4)/1000 = 6745

$\frac{6745}{134514} = 5.0\%$

M_R Live

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

M_{OT} D+L

(101+26)(115)(40)(56.4)/1000 = 32949

$\frac{32949}{290550} = 11.3\%$

M_R D+L

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

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT

TITLE

BY: SHEET:
CHKD: PROJECT NO:
DATE: PAGE:

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.

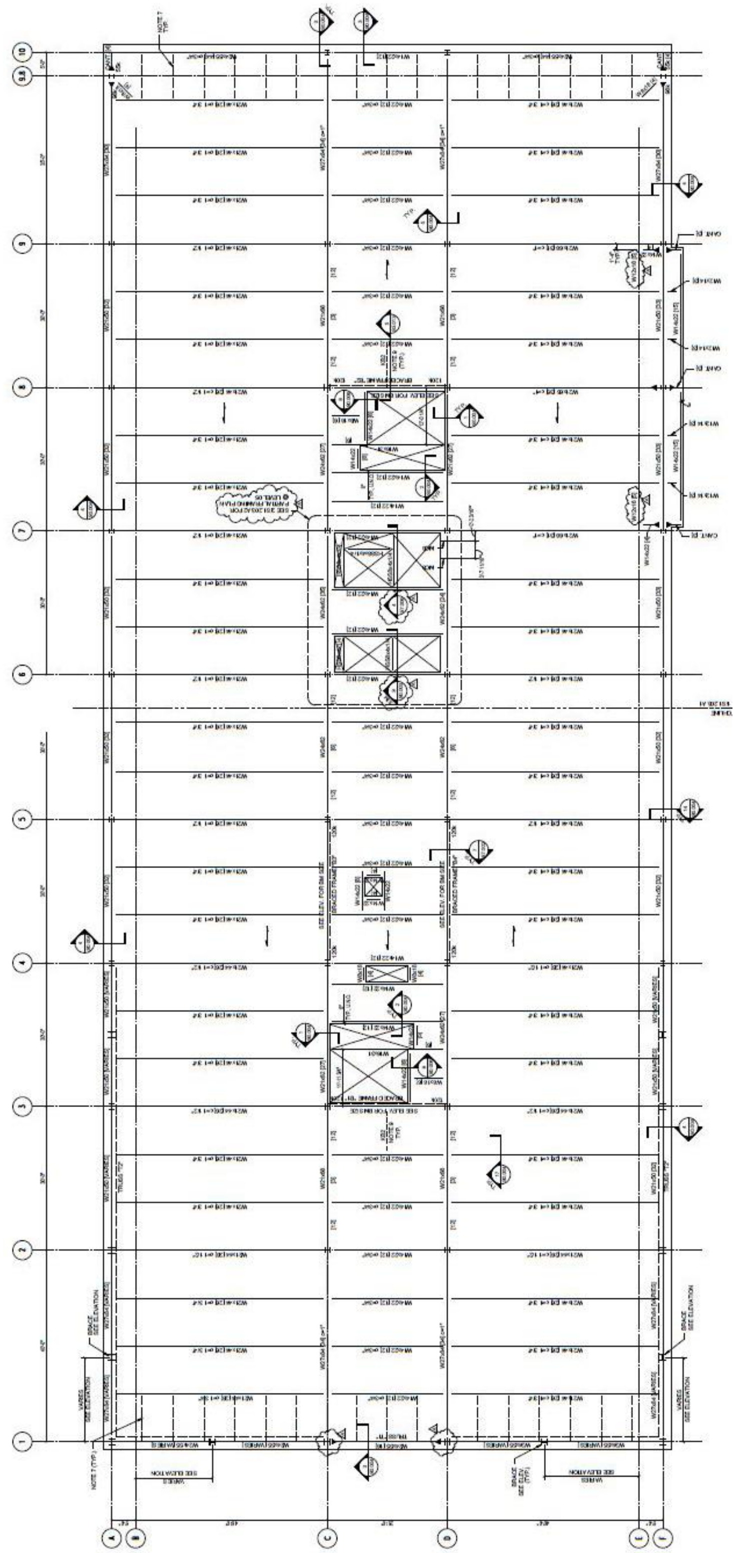
The lateral system study included the conversion of braced frames to buckling-restrained brace frames. It was assumed that these frames in the green roof garden project option are seismically detailed in order to take full advantage of the buckling-restrained braces. This study found that although the 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.

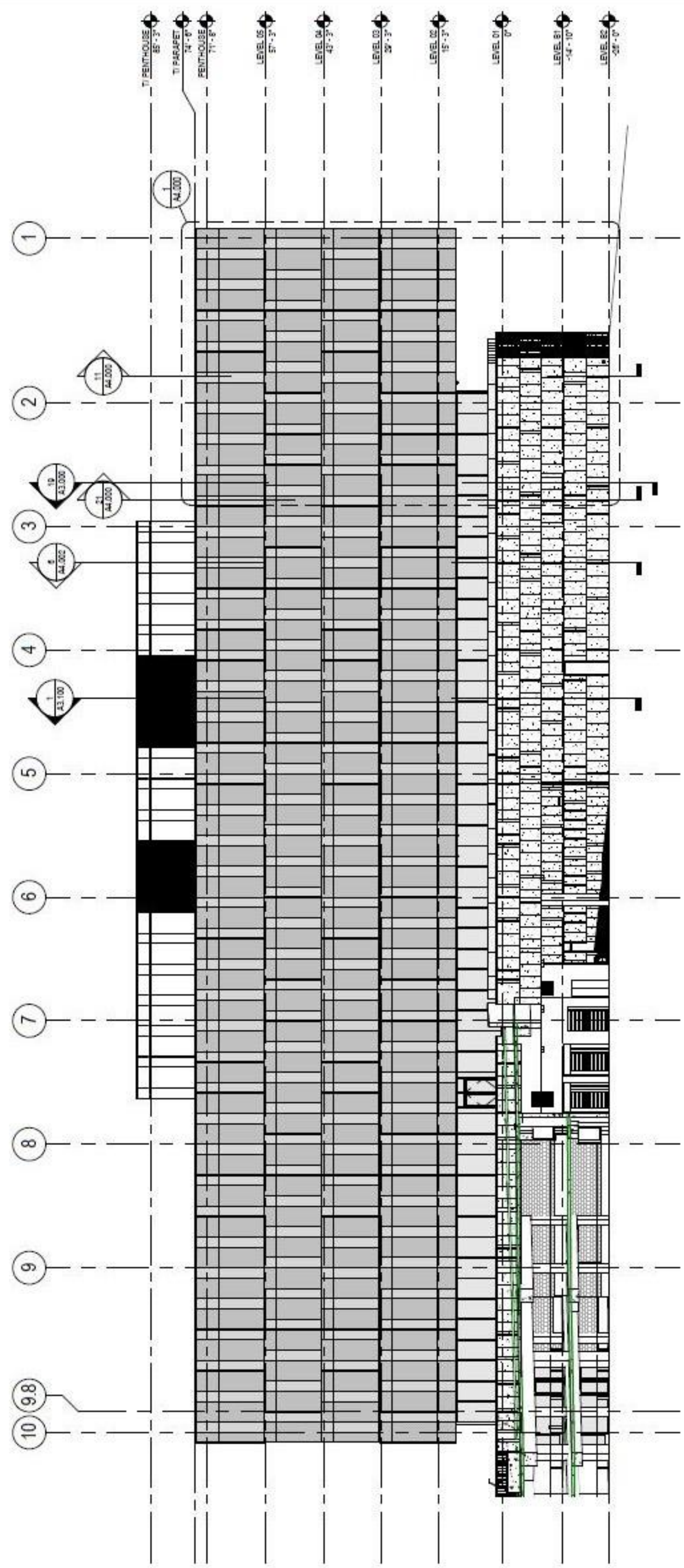
In conclusion, based on the research, analysis and design outcomes of this study and considering the thesis project scope, adding a green roof garden to this project is feasible if the owner wishes to pursue this scenario. I would suggest that the lateral system remain conventional braced frames over 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.

Appendices

Appendix A: Additional Plans



(Construction Documents)



(Construction Documents)

Appendix B: As Built Wind Calculations

Begins next page.

WIND LOADING

ASCE 7-05 ANALYTICAL PROCEDURE

- DETERMINE BASIC WIND SPEED, V
  $V = 90$ mph [FIG. 6-1]
- WIND DIRECTIONALITY FACTOR, K_d
  FOR BUILDINGS, $K_d = 0.85$ [TABLE 6-4]
- IMPORTANCE FACTOR, I
  OCCUPANCY CATEGORY III [TABLE 1-1]
  $I = 1.15$ [TABLE 6-1]
- EXPOSURE CATEGORY
  - HILLY TERRAIN WITH SURROUNDING DEVELOPMENTS AND SOME TREES ∴ EXPOSURE B [§ 6.5.6]
- TOPOGRAPHIC FACTOR, K_{zt}
  - BUILDING BUILT INTO LOW HILL NOT ON TOP ∴ $K_{zt} = 1.0$
- GUST FACTOR
  - ESTIMATE NATURAL FREQUENCY
    $n_1 = 75/H$ [EQN. C6-18]
    $H < 300$ ft, $< 4 L_{eff}$ OK
    $n_1 = 75/111.25 = 0.674 < 1$ ∴ FLEXIBLE
  - DETERMINE G_f IN NW-SE DIRECTION
    $g_Q = g_V = 3.4$
    $g_r = \sqrt{2 \ln(3600 \cdot 0.674)} + \frac{0.577}{\sqrt{2 \ln(3600 \cdot 0.674)}} = 4.09$
    - DETERMINE RESPONSE FACTOR, R
      $\bar{b} = 0.45$
      $\bar{\alpha} = 1/4.0 = 0.25$ [TABLE 6-2]
      $\bar{z} = \max \begin{cases} 0.6h = 0.6(104) = 62.4 \\ z_{min} = 30' \end{cases}$

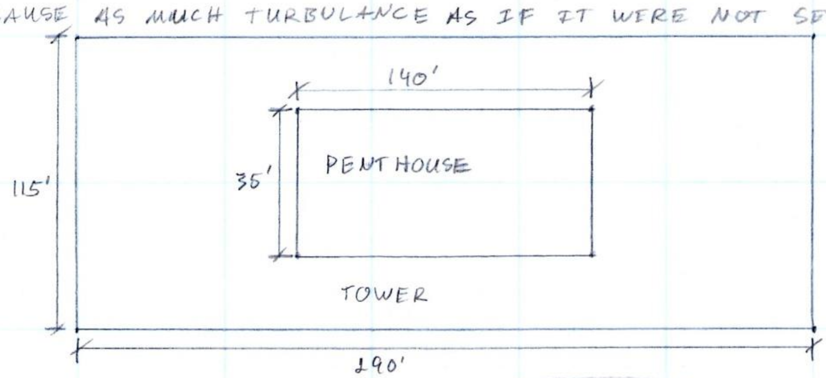
JUSTIFICATION NEXT PAGE $\longrightarrow h = \frac{97.65' + 111.25'}{2} = 104'$

$\therefore \bar{z} = 62.4'$

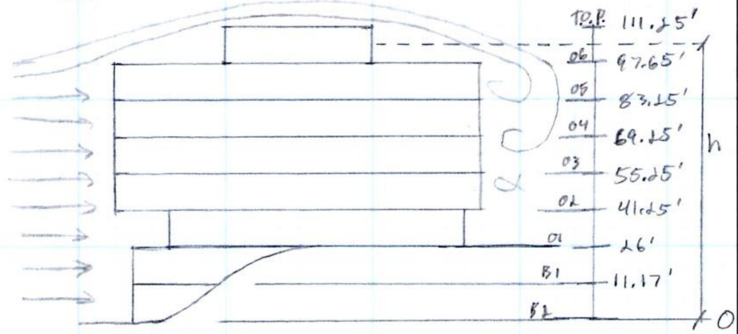
ruby+associates STRUCTURAL ENGINEERS 30445 Northwestern Highway Suite 310 Farmington Hills, Michigan 48334 T:248.865.8855 F:248.865.9449 www.rubyusa.com	PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 2	BY: NMB	SHEET: 2
	TITLE: WIND LOAD WIND FACTOR CALCS	CHKD:	PROJECT NO: TECH 2
		DATE:	PAGE: 22

MEAN ROOF HEIGHT JUSTIFICATION

- BECAUSE PENTHOUSE IS SET BACK, IT WILL NOT SEE FULL WIND/ CAUSE AS MUCH TURBULANCE AS IF IT WERE NOT SET BACK.

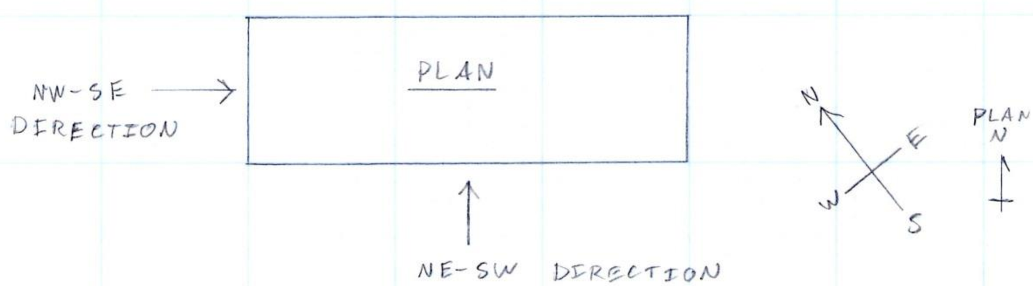


- AREA RATIO: $\frac{(140)(35)}{(115)(290)} = 0.15 < 0.5$
- LENGTH RATIO: $140/290 = 0.48 < 0.5$
- WIDTH RATIO: $35/115 = 0.30 < 0.5$



∴ USE MEAN ROOF HEIGHT TO THE MIDHEIGHT OF THE PENTHOUSE.

BUILDING AND WIND DIRECTIONS W.R.T. TRUE NORTH



ruby+associates STRUCTURAL ENGINEERS 30445 Northwestern Highway Suite 310 Farmington Hills, Michigan 48334 T:248.865.8855 F:248.865.9449 www.rubyusa.com	PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 2	BY: NMB	SHEET: 3
		CHKD:	PROJECT NO: TECH 2
	TITLE: WIND LOAD M.R.H JUSTIFICATION	DATE:	PAGE: 23

$$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{62.4}{33}\right)^{0.25}(90)\left(\frac{88}{60}\right) = 69.7$$

$\ell = 320'$

$\bar{\epsilon} = 1/3.0 = 0.33$ [TABLE 6-2]

$$L_z = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{62.4}{33}\right)^{0.33} = 395$$

$$N_1 = \frac{n_1 L_z}{\bar{V}} = \frac{0.674(395)}{69.7} = 3.82$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(3.82)}{(1+10.3 \cdot 3.82)^{5/3}} = 0.060$$

DAMPING RATIO, β FROM COMMENTARY §C6.5.8

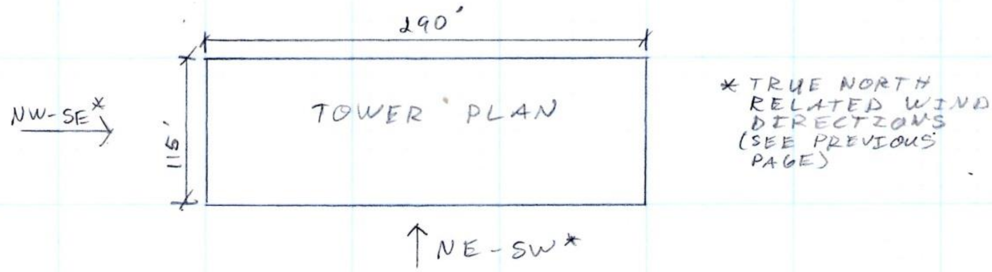
ASSUME $\beta = 0.015$

- FOR R_h

$$\eta = \frac{4.6 n_1 h}{\bar{V}_z} = \frac{4.6(0.674)(104)}{69.7} = 4.626$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$$

$$= \frac{1}{4.626} - \frac{1}{2 \cdot 4.626^2}(1 - e^{-2 \cdot 4.626}) = R_h = 0.192$$



	NW-SE	NE-SW
B	115'	290'
L	290'	115'

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 2

TITLE: WIND LOAD GUST FACTOR CALCS

BY: NMB
SHEET: 4
CHKD:
PROJECT NO: TECH 2
DATE:
PAGE: 24

- FOR R_B

$$\eta = \frac{4.6 n_1 B}{\bar{V}_z} = \frac{4.6(0.672)(115')}{62.7} = 5.670$$

$$R_B = \frac{1}{5.67} - \frac{1}{2 \cdot 5.67^2}\left(1 - e^{-2 \cdot 5.67}\right) = 0.161$$

- FOR R_L

$$\eta = \frac{15.4 n_1 L}{\bar{V}_z} = \frac{15.4(0.674)(290')}{62.7} = 48.0$$

$$R_L = \frac{1}{48.0} - \frac{1}{2 \cdot 48.0^2}\left(1 - e^{-2 \cdot 48.0}\right) = 0.021$$

$$R = \sqrt{\tfrac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$
$$= \sqrt{(1/0.015)(0.060)(0.192)(0.161)(0.53 + 0.47 \cdot 0.021)}$$

$R = 0.258$

- FIND INTENSITY OF TURBULENCE, I_z

$c = 0.30$ [TABLE 6-2]

$$I_z = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{62.4}\right)^{1/6} = 0.270$$

- BACKGROUND RESPONSE, Q

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{115 + 104}{395}\right)^{0.63}}}$$

$Q = 0.835$

$$G_f = 0.925\left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z}\right)$$
$$= 0.925\left(\frac{1 + 1.7(0.270)\sqrt{3.4^2 \cdot 0.835^2 + 4.09^2 \cdot 0.258^2}}{1 + 1.7(3.4)(0.270)}\right)$$

$G_{f\ NW-SE} = 0.863$

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 2	BY: NMB	SHEET: 5	
TITLE: WIND LOAD GUST FACTOR CALCS	CHKD:	PROJECT NO: TECH2	
	DATE:	PAGE: 25	

- DETERMINE G_f IN NE-SW DIRECTION

$g_Q = g_v = 3.4$ $\quad \bar{V}_{\bar{z}} = 69.7$ $\quad R_n = 0.060$

$g_r = 4.09$ $\quad L_{\bar{z}} = 395$ $\quad \beta = 0.015$

$\bar{z} = 62.4'$ $\quad N_1 = 3.82$ $\quad I_{\bar{z}} = 0.270$

$R_h = 0.192$ $\quad h = 104'$

- FOR R_B

$$n = \frac{4.6(0.674)(240)}{69.7} = 12.9$$

$$R_B = \frac{1}{12.9} - \frac{1}{2 \cdot 12.9^2}\left(1 - e^{-2 \cdot 12.9}\right) = \mathbf{0.745}$$

- FOR R_L

$$n = \frac{15.4(0.674)(115')}{69.7} = 17.1$$

$$R_L = \frac{1}{17.1} - \frac{1}{2 \cdot 17.1^2}\left(1 - e^{-2 \cdot 17.1}\right) = 0.0568$$

- R

$$R = \sqrt{(1/0.015)(0.060)(0.192)(0.745)(0.53 + 0.47 \cdot 0.0568)} = 0.564$$

- Q

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{240 + 104}{345}\right)^{0.63}}} = \mathbf{0.738}$$

- G_f

$$= 0.925\left(\frac{1 + 1.7(0.270)\sqrt{3.4^2(0.738^2) + (4.09^2)(0.564^2)}}{1 + 1.7(3.4)(0.270)}\right)$$

$$\underline{G_{f\,NE-SW} = 1.00}$$

- ENCLOSURE CLASSIFICATION [§6.5.9]
  - NO OPENINGS ∴ **ENCLOSED**

- INTERNAL PRESSURE COEFFICIENT, GC_{pi}

$GC_{pi} = \pm 0.18$ [FIG. 6-5]

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT	RGA GLOBAL HQ TECHNICAL REPORT 2
TITLE	WIND LOAD GUST FACTOR CALCS
BY:	NMB
SHEET:	6
CHKD:	
PROJECT NO:	TECH 2
DATE:	
PAGE:	26

• EXTERNAL PRESSURE COEFFICIENTS, C_p

- WALLS NW-SE DIRECTION [FIG. 6-6]

WINDWARD WALL: $C_p = 0.8$ (USE w/ q_z)

SIDE WALL: $C_p = -0.7$ (USE w/ q_h)

LEEWARD WALL:

$L/B = 290/115 = 2.52$

INTERPOLATE:

L/B	C_p
2	-0.3
2.52	-0.274
4	-0.2

$\frac{(-0.2 - -0.3)}{4-2}(2.52-2) + -0.3 =$

$C_p = -0.274$ (USE w/ q_h)

- WALLS NE-SW DIRECTION

WINDWARD WALL: $C_p = 0.8$ (USE w/ q_z)

SIDEWALL: $C_p = -0.7$ (USE w/ q_h)

LEEWARD WALL:

$L/B = 115/290 = 0.397 \therefore C_p = -0.5$ (USE w/ q_h)

- ROOF NW-SE DIRECTION

$h/L = 104/290 = 0.36 < 0.5$

HORIZ DIST FROM WINDWARD EDGE	C_p
0 TO 52'	-0.9, -0.18
52' TO 104'	-0.9, -0.18
104' TO 208'	-0.5, -0.18
> 208	-0.3, -0.18

- ROOF NE-SW DIRECTION

$h/L = 104/115' = 0.904$

• 0 TO 52':

0.5	-0.9
0.904	$\boxed{-1.01}$
1.0	$-1.3(0.8) = -1.04$

$\frac{-1.04 - -0.9}{1-0.5}(0.904-0.5) + -0.9$

REDUCE? ⇒ (39.25')(290') ~ 11000 ∴ REDUCE BY 0.8

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT	BY:	SHEET:
RGA GLOBAL HQ TECHNICAL REPORT 2	NMB	7
	CHKD:	PROJECT NO: TECH 2
TITLE: WIND LOAD C_p CALCULATIONS	DATE:	PAGE: 27

• 52' TO 104'

0.5	−0.9	
0.904	$\boxed{-0.738}$	$\frac{-0.7 - -0.9}{1 - 0.5}(0.904 - 0.5) + -0.9$
1.0	−0.7	

• 104' TO 208'

0.5	−0.5	
0.904	$\boxed{-0.662}$	$\frac{-0.7 - -0.5}{1 - 0.5}(0.904 - 0.5) + -0.5$
1.0	−0.7	

• > 208'

0.5	−0.3	
0.904	$\boxed{-0.623}$	$\frac{-0.7 - -0.3}{1 - 0.5}(0.904 - 0.5) + -0.3$
1.0	−0.7	

* SEE FOLLOWING EXCEL SHEETS FOR PRESSURE CALCULATIONS

• DESIGN WIND PRESSURES

- FOR FLEXIBLE BUILDINGS

$$p = q G_f C_p - q_i (GC_{pi}) \quad [\text{EQN. } 6\text{-}19]$$

- FOR PARAPETS

$$P_p = q_p GC_{pn} \quad [\text{EQN. } 6\text{-}20]$$

$GC_{pn} = +1.5$ WINDWARD PARAPET
$GC_{pn} = -1.0$ LEEWARD PARAPET

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 2	BY: NMB — SHEET: 8
TITLE: WIND LOAD C_p CALCULATIONS	CHKD: — PROJECT NO: TECH2
	DATE: — PAGE: 28

$K_z=2.01(z/z_g)^{2/\alpha}$

$q_z=0.00256K_zK_{zt}K_dV^2I$

Kd=	0.85	
Kzt=	1	
V=	90	mph
I=	1.15	
z_g=	1200	ft

Determine K_z and q_z

Floor	z	z_g (ft)	α	K_z	q_z	OR:
B1	11.17	1200	7	0.528	10.7	
1	26	1200	7	0.673	13.6	
2	41.25	1200	7	0.767	15.6	
3	55.25	1200	7	0.834	16.9	
4	69.25	1200	7	0.890	18.0	
5	83.25	1200	7	0.938	19.0	
6	97.65	1200	7	0.982	19.9	
Tower Parapet	100.65	1200	7	0.990	20.1	q_p
Mean Roof Height	104	1200	7	0.999	20.3	q_h
T.O. Penthouse	111.25	1200	7	1.019	20.7	
Penthouse Parapet	114.25	1200	7	1.027	20.8	q_p

SHT: 9
PAGE: 29

MWFRS ANALYSIS: NE-SW Walls

Floor	z	q	Windward (PSF)	Leeward (PSF)	Tribuary Height(ft.)	Tributary Area(SF)	Story Shear(k)	Story M_{OT}(ft.-k)
B1	11.17	10.7	8.57	-10.1	18.585	5390	100.76	189.2
1	26	13.6	10.91	-10.1	15.04	3685	77.50	1432.3
2	41.25	15.6	12.44	-10.1	14.625	4241	95.73	3248.8
3	55.25	16.9	13.53	-10.1	14	4060	96.04	4633.8
4	69.25	18.0	14.43	-10.1	14	4060	99.70	6206.2
5	83.25	19.0	15.21	-10.1	14.2	4118	104.33	7944.8
6	97.65	19.9	15.92	-10.1	7.2	2088	54.38	5114.5
Tower Parapet	100.65	20.1	30.10	-20.3	3	870	43.81	4344.0
Mean Roof Height	104	20.3	16.21	-10.1	6.975	977	25.71	2584.7
T.O. Penthouse	111.25	20.7	16.52	-10.1	3.625	508	13.52	1480.1
Penthouse Parapet	114.25	20.8	31.21	-20.3	3	420	21.62	2437.3
						Base Shear and M_{OT}=	**733**	**39615**

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

G_f=	1.0	
C_p=	0.8	Windward
	-0.5	Leeward
GC_{pn}=	1.5	Windward
	-1.0	Leeward
q_h=	20.7	ft.

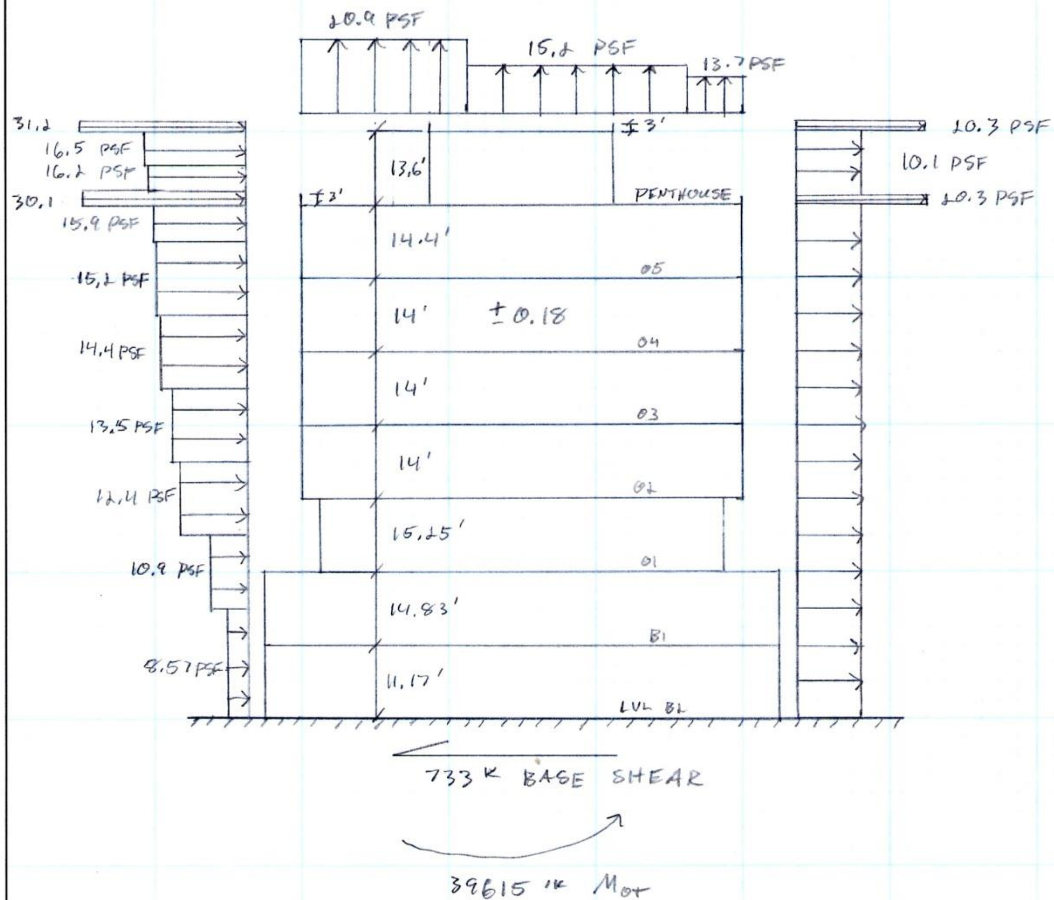
MWFRS ANALYSIS: NE-SW ROOF

Dist. H	0' to 52'	52' to 104'	104' to 208'	>208'
C_p	-1.01	-0.738	-0.662	-0.623
Pressure (PSF)	**-20.9**	**-15.2**	**-13.7**	**-12.9**

SHT:10

PAGE:30

WIND DIAGRAM: NE-SW DIRECTION



ruby+associates
STRUCTURAL ENGINEERS

30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449

www.rubyusa.com

PROJECT: RGA GLOBAL HQ TECHNICAL REPORT 2

TITLE: WIND LOAD PRESSURE DIAGRAM

BY: NMB
SHEET: 11
CHKD:
PROJECT NO: TECH 2
DATE:
PAGE: 31

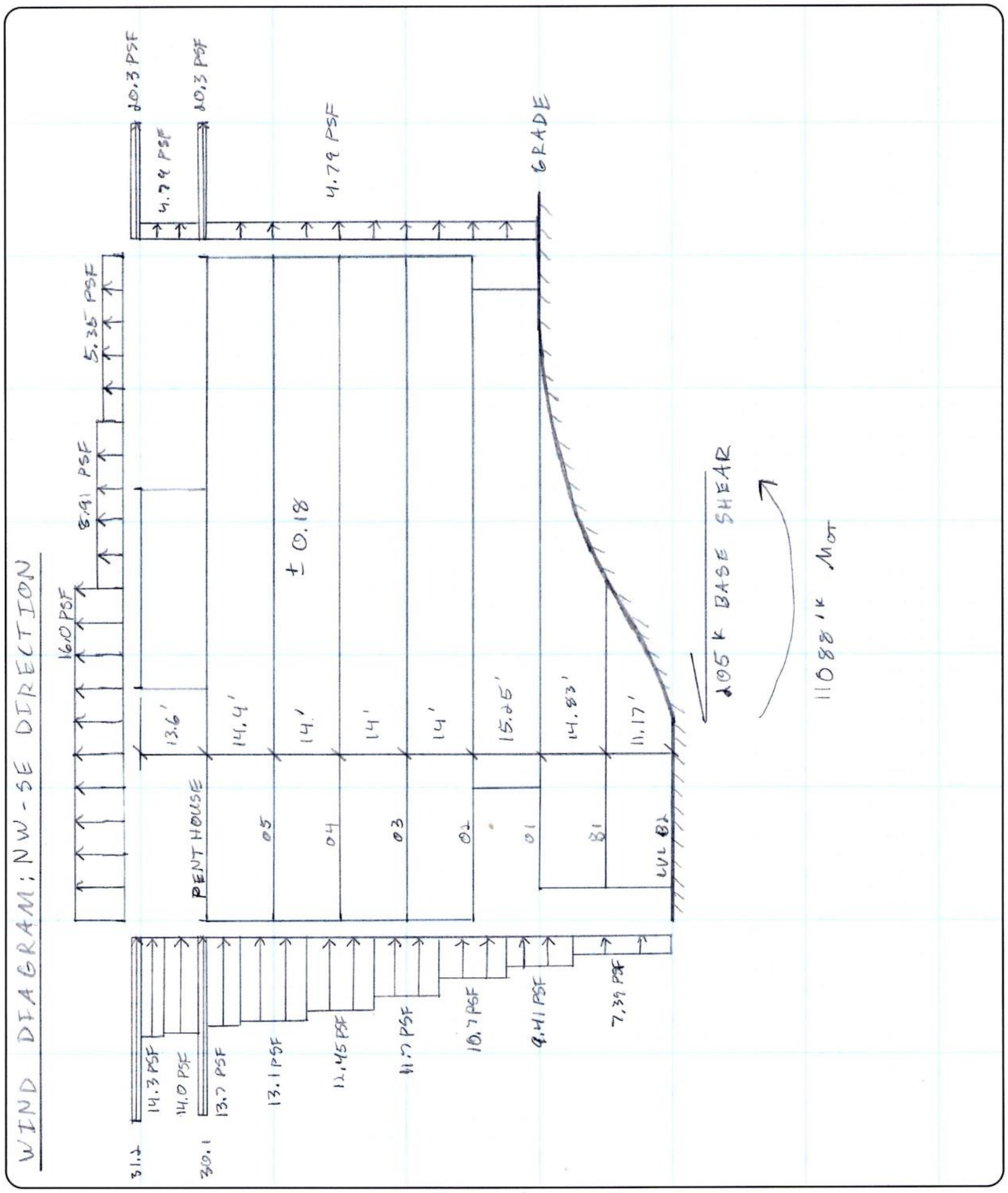
MWFRS ANALYSIS: NW-SE Walls								
Floor	**z**	**q**	**Windward (PSF)**	**Leeward (PSF)**	**Tribuary Height(ft.)**	**Tributary Area(SF)**	**Story Shear(k)**	**Story M_{OT}(ft.-k)**
B1	11.17	10.7	7.39	-4.79	18.585	2323	28.30	53.1
1	26	13.6	9.41	-4.79	15.04	1579	22.43	414.4
2	41.25	15.6	10.74	-4.79	14.625	1682	26.12	886.3
3	55.25	16.9	11.67	-4.79	14	1610	26.51	1278.9
4	69.25	18.0	12.45	-4.79	14	1610	27.76	1727.9
5	83.25	19.0	13.12	-4.79	14.2	1633	29.25	2227.6
6	97.65	19.9	13.74	-4.79	7.2	828	15.34	1442.7
Tower Parapet	100.65	20.1	30.10	-20.26	3	345	17.37	1722.6
Mean Roof Height	104	20.3	13.99	-4.79	6.975	244	4.58	460.7
T.O. Penthouse	111.25	20.7	14.26	-4.79	3.625	127	2.42	264.5
Penthouse Parapet	114.25	20.8	31.21	-20.26	3	105	5.40	609.3
						Base Shear and M_{OT}=	**205**	**11088**

$p=qG_fC_p-q_i(GC_{pi})$	**G_f=**	0.863	
	C_p=	0.8	Windward
		-0.274	Leeward
	GC_{pn}=	1.5	Windward
		-1.0	Leeward
	q_h=	20.7	ft.

MWFRS ANALYSIS: NW-SE ROOF				
Dist. H	0' to 52'	52' to 104'	104' to 208'	>208'
C_p	-0.9	-0.9	-0.5	-0.3
Pressure (PSF)	**-16.04**	**-16.04**	**-8.91**	**-5.35**

SHT: 12

PAGE: 32



ruby+associates
STRUCTURAL ENGINEERS

30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449

www.rubyusa.com

PROJECT	RGA GLOBAL HQ TECHNICAL REPORT 2
TITLE	WIND LOAD PRESSURE DIAGRAM

BY: NMB	SHEET: 13
CHKD:	PROJECT NO: TECH 2
DATE:	PAGE: 33

Appendix C: As Built Seismic Calculations

COMBINED STORY WEIGHTS (k)						
	Parking Structure				Office	
Level	Walls	Columns	Slabs	Beams	Total	Total
B1	1286	431	5839	1412	0	8968
1	702	246	7201	3348	1881	13378
2	0	0	0	0	2521	2521
3	0	0	0	0	2527	2527
4	0	0	0	0	2527	2527
5	0	0	0	0	2531	2531
6	0	0	0	0	1680	1680
Penthouse	0	0	0	0	1543	1543

MODELING ADJUSTMENTS		
Level	Weight(k)	Total(k)
B1	8968	8968
1	To 2	
2	15899	
3	2527	
4	2527	
5	2531	
6	1680	
Penthouse	1543	26707

A_b=	66733	SF
h_n=	26	ft.
h_i=	26	ft.

APPROX. FUNDAMENTAL PERIOD: PARKING				
SW #	D_i	A_i	NW-SE Dir.	NE-SW Dir.
5	55.7	1447	1225.45	-
	22.7	589	-	281.70
6	23.0	598	-	290.20
10	60.0	1560	-	1349.65
Wall PV	180	4680	4600.33	-
Wall P1	134	3484	-	3378.43
		Sum=	5826	5300
		C_w=	8.730	7.942
		T_a=	**0.017**	**0.018**

k_{office}=	1.03	
$k_{parking}$=	0.5	
V_{total}=	4235	k

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**

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

SEISMIC LOAD CASE ECCENTRICITIES									
Level	Force(k)	XCM(ft.)	XCR(ft.)	ex (ft.)	5%Bx(ft.)	YCM(ft.)	YCR(ft.)	ey(ft.)	5%By(ft.)
Penthouse	936	130	134.6	4.59	5.750	57.5	57.5	0	14.5
5	583	127.5	137.1	9.61	5.750	57.5	57.5	0	14.5
4	482	130	140.3	10.34	5.750	57.5	57.5	0	14.5
3	382	127.5	142.9	15.45	5.750	57.5	57.5	0	14.5
2	1779	130	144.9	14.93	5.750	57.5	57.5	0	14.5
B1	73	213.9734	330.1601	116.19	17.500	-108.5944	-103.6872	-4.9072	18.55
Sum=	4235								

Mtax=Fx(ey+5%Bx)
Mtay=Fx(ex+5%By)

100+30			
EX	30%EY	30%EX	EY
73	21.81	21.81	73

SEISMIC LOAD CASES														
	Case 1			Case 2			Case 3				Case 4			
Level	EX	Mtax(ft-k)-	Mtax(ft-k)-	EY	Mtay+	Mtay-	EX	Mtax+	Mtax-	EY	EY	Mtay+	Mtay-	EX
Penthouse	936	5383	-5383	936	17875	-9274	936	9683	-1083	936	936	9274	-17875	936
5	583	3355	-3355	583	14070	-2851	583	8964	2254	583	583	2851	-14070	583
4	482	2772	-2772	482	11974	-2006	482	7756	2212	482	482	2006	-11974	482
3	382	2197	-2197	382	11441	363	382	8098	3705	382	382	-363	-11441	382
2	1779	10227	-10227	1779	52351	772	1779	36789	16335	1779	1779	-772	-52351	1779
B1	73	916	-1629	73	9795	7098	73	10076	7531	73	73	-7455	-10152	73

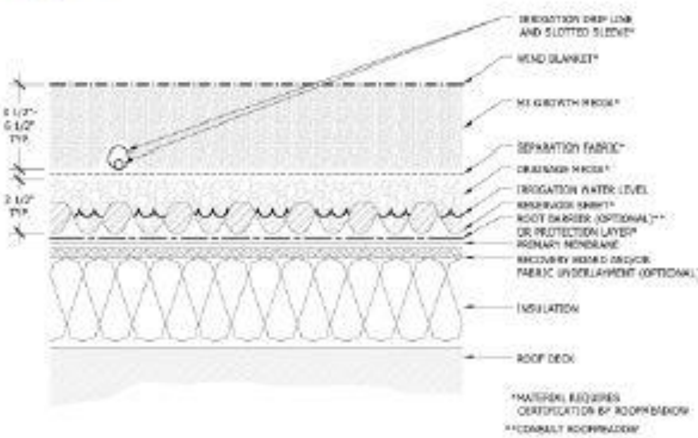
Appendix D: Roofmeadow System Information

Roofmeadow
Green roofs. For good.

Roofmeadow® Type V Data Sheet

Our experience demonstrates that the most efficient designs for the vast majority of American green roofs can be derived from five basic green roof types (Type I, II, III, IV, V). Roofmeadow® has developed assemblies for each of these types.

The selected assembly depends in part on project conditions including climate, desired plant community, performance requirements, and load bearing capacity of the building. All assemblies will include the following elements: 1) protection of the waterproofing membrane from root and biological attack, 2) protection of the waterproofing membrane from physical abuse and accident, 3) a base drainage layer, 4) a separation layer to prevent fine-grained engineered soils from fouling or clogging the drainage layer system, and 5) an engineered soil to support the vegetation.



Type V: Dual Media With Reservoir Sheet

A synthetic reservoir sheet over a protection fabric forms the base of the Type V assembly, which offers one solution to installing a three-course green roof over a PMR roofing system. A deep reservoir sheet is required; typical reservoir sheets are 2.6 to 2.4 inches (4 to 6 cm) thick and usually retain between 0.2 and 0.4 inches (0.5 to 1.0 cm) of water when filled with granular media. The coarse, large-grained granular media in the reservoir sheet cups 1) stabilizes the sheet, 2) facilitates drainage, and 3) reduces the potential for drought stress. A root-permeable separation fabric separates the fine-grained growth media from the granular media and prevents the fines from mixing with the granular media. The reservoir sheet stores captured rain or irrigation water for the root mass, and irrigation is provided by surface or sub-surface (just above the reservoir sheet) drip lines. Typical assembly thicknesses range from 6 to 10 inches (15 to 26 cm).

The profile of a Type V assembly is as follows:

Wind Erosion Stabilization System
Growth Medium
Root-permeable Separation Fabric
Light-weight Granular Drainage Media
Synthetic Reservoir Sheet (water storage layer)
Protection Fabric
Root Barrier Membrane (when required)
Waterproofing System

Roofmeadow ♦ 7135 Germantown Ave., 2nd Floor ♦ Philadelphia, PA, 19119 ♦ o. 215.247.8784 ♦ f. 215.247.4659 ♦ info@roofmeadow.com

HANOVER®
Architectural Products

PRODUCT DATA SHEET

Hanover® Prest® Pavers for Roofs & Waterproofed Decks

Hanover® Prest® Pavers, high density pressed concrete units, are manufactured to 1/8" tolerances and produced by subjecting the concrete mix to a minimum pressure of 1,000 pounds per square inch over the entire surface area. This results in a product with the density and strength of natural stone.

Hanover® Prest® Pavers provide durability and protection for the roof or waterproofed deck system from harsh weather conditions. Hanover® Pavers make roofs and decks safer for pedestrians and simplify repairs. Hanover® Support Pedestals, together with Hanover® Pavers, provide effective drainage between the pavers and the system below. Hanover® Support Pedestals make roof and deck plazas serviceable, functional and attractive.

Metric Size	Actual Size	1 1/4"	1 1/2"	2"	2 1/4"	2 1/2"	3"	4"
297mm x 297mm	11 3/4" x 11 3/4"		X	X	X	X	X	
303mm x 303mm	11 15/16" x 11 15/16"		X	X	X	X	X	
378mm x 378mm	14 7/8" x 14 7/8"		X	X	X	X	X	
297mm x 447mm	11 3/4" x 17 5/8"		X	X	X	X	X	
297mm x 597mm	11 3/4" x 23 1/2"	X	X	X	X	X	X	
447mm x 447mm	17 5/8" x 17 5/8"		X	X	X	X	X	X
447mm x 597mm	17 5/8" x 23 1/2"		X	X	X	X	X	
447mm x 899mm	17 5/8" x 35 3/8"		X	X	X	X	X	
597mm x 597mm	23 1/2" x 23 1/2"	X	X	X	X	X	X	X
597mm x 747mm	23 1/2" x 29 1/2"			X	X	X	X	
597mm x 897mm	23 1/2" x 35 3/8"			X	X	X	X	
756mm x 756mm	29 3/4" x 29 3/4"			X	X	X	X	
908mm x 908mm	*35 3/4" x 35 3/4" x 2 1/2"					X	X	

= Standard Thickness Weight (2" thickness): 25 lbs/sf *NOTE INCREASED THICKNESS & WEIGHT FOR THIS SIZE PAVER

RELATIVE STRENGTHS: (at 2" thickness)
Compressive: 8,500 psi at 28 days
Density: 155 lbs/cu. ft.
Flexural: 1,100 psi
Finish: Tudor®
Absorption: less than 5%
Weight: 25 lbs/sf

The test results displayed are taken from samples of Hanover's Prest® Pavers with a standard mix design. Hanover® Prest® Pavers, high density, hydraulically pressed concrete units, are manufactured to 1/8" tolerances and produced by subjecting the concrete mix to a minimum pressure of 1,000 pounds per square inch over the entire surface area. This results in a product with the density and strength of natural stone.

Pedestal® Paver

This patented paver incorporates the idea of an elevated paver drainage system with the use of an integral footed, concrete paver of the highest quality. An elevated clearance of 1/2" allows effective drainage.

Actual Size: 23 1/2" x 23 1/2" x 2 1/4"
Color: Natural
Metric Size: 597mm x 597mm x 57mm
Finish: Tudor®
Weight: 22 lbs/sf

Standard Colors:
Limestone Gray, Quarry Red, Cream, Tan, Brown, Red 15, Charcoal, and Natural
Custom color and aggregate blending is available on special order and when quantity ordered permits.

5000 Hanover Road, Hanover, PA 17331 • 717.637.0500 • Fax 717.637.7145 • www.hanoverpavers.com

Appendix E: Green Roof Calculations and Supplemental Information

Plant Image Sources

http://www.sedumphotos.net/v/sedum-pqr/Sedum+pluricaule+ezawe+compact+form+5.jpg.html

http://www.greenroofplants.com/catalog/plant-catalog/viewplant/?plantid=766

http://florafind.mainegardens.org/weboi/oecgi2.exe/INET_ECM_DispPl?NAMENUM=14716&DETAIL=1#images

http://www.qscaping.com/Content/Images/Photos/F593-18.jpg

http://www.thebattery.org/images/plants/autumn-m83.jpg

http://www.mrugala.net/Nature/Plantes/Photos/index.php?page=34

http://navigate.botanicgardens.org/weboi/oecgi2.exe/INET_ECM_DispPl?NAMENUM=47129&DETAIL=1#images

http://store.theodorepayne.org/product/SI_FESID.html

http://www.contracosta.watersavingplants.com/eplant.php?plantnum=24352&return=l5

http://www.heritageflowerfarm.com/buyPerennialPlantsDetail.asp?cat=4&ID=4&pID=405

http://flora.nhm-wien.ac.at/Seiten-Arten/Petrorhagia-saxifraga.htm

http://plants.usda.gov/core/profile?symbol=PESA9

http://jeansgarden.wordpress.com/2010/05/26/wildflower-wednesday-may-2010/

http://tinea.narod.ru/e/gallery/plantae/hieracium_pilosella002.html

https://gobotany.newenglandwild.org/species/hieracium/pilosella/

http://www.phytoimages.siu.edu/imgs/Cusman1/r/Boraginaceae_Echium_russicum_40442.html

http://www.solovivaces.com/echium-russicum/

http://www.anpc.ab.ca/wiki/index.php/Anthemis_tinctoria

http://commons.wikimedia.org/wiki/File:Anthemis_April_2009-1.jpg

http://www.finegardening.com/plantguide/salvia-jurisicii-yugoslavian-cutleaf-sage.aspx

http://www.thebattery.org/plants/plantview.php?id=238

National Weather Service Climatological Report

Select Other Date ▾

These data are preliminary and have not undergone final quality control by the National Climatic Data Center (NCDC). Therefore, these data are subject to revision. Final and certified climate data can be accessed at the NCDC - http://www.ncdc.noaa.gov.

Climatological Report (Annual)

```
000
CXUS53 KLSX 061651
CLAUIN

CLIMATE REPORT
NATIONAL WEATHER SERVICE ST LOUIS MO
1021 AM CST MON JAN 6 2014

...................................

...THE QUINCY IL CLIMATE SUMMARY FOR THE YEAR OF 2013...

CLIMATE NORMAL PERIOD 1981 TO 2010
CLIMATE RECORD PERIOD 1901 TO 2014

WEATHER          OBSERVED          NORMAL  DEPART
                  VALUE   DATE(S)  VALUE   FROM
                                           NORMAL
................................................
TEMPERATURE (F)
RECORD
 HIGH              114   07/15/1936
 LOW               -29   02/13/1905
HIGHEST            100   09/09
                         08/30
LOWEST                   12/24
AVG. MAXIMUM      61.7              62.1    -0.4
AVG. MINIMUM      42.1              43.3    -1.2
MEAN              51.9              52.7    -0.8
```

```
DAYS MAX >= 90      29
DAYS MAX <= 32      42
DAYS MIN <= 32     127
DAYS MIN <= 0        1

PRECIPITATION (INCHES)
RECORD
 MAXIMUM        66.60   1973
 MINIMUM        20.00   1953
TOTALS          35.67             37.33   -1.66
DAYS >= .01        93
DAYS >= .10        56
DAYS >= .50        21
DAYS >= 1.00       12
GREATEST
 24 HR. TOTAL    2.69  04/18 TO 04/18

DEGREE_DAYS
HEATING TOTAL    5906               5582     324
 SINCE 7/1       2316               2191     125
COOLING TOTAL    1242               1095     147
 SINCE 1/1       1242               1094     148
.................................................

WIND (MPH)
HIGHEST WIND SPEED/DIRECTION    46/320    DATE  06/23
HIGHEST GUST SPEED/DIRECTION    77/260    DATE  04/17


WEATHER CONDITIONS. NUMBER OF DAYS WITH
THUNDERSTORM             47      FOG W/VIS <= 1/4 MILE     23

-  INDICATES NEGATIVE NUMBERS.
R  INDICATES RECORD WAS SET OR TIED.
MM INDICATES DATA IS MISSING.
T  INDICATES TRACE AMOUNT.
```

SEE STLPNSLSX 1030 AM CST MON JAN 6 2014 FOR SUPPLEMENTAL ANNUAL CLIMATE DATA

Plant Selection List for Hardiness Zone 6a

Begins on following page.

Plant	Page	Hardiness Zone	Flower/Foliage	Blooming Time	Groundcover or Accent?	Self Sowing?	Native Area	Height (Up to)	Spread	Medium Depth	Light Requirements	Notes
Agastache Rupestris	91	6	Orange flowers, blue-green foliage	Midsummer to midautumn	Accent	No	SW US	25"	10"	6"	Full sun, mixed sun/shade	Flowers attract hummingbirds, becomes shrublike
Alyssum montanum 'Berggold'	94	6	Yellow flowers, green foliage	Early summer	Groundcover	No	Europe	6"	10"	6"	Full sun	Can grow to 10,000 ft in altitude
Anacyclus pyrethrum var. depressus	95	6	White flowers w/ yellow centers, gray green foliage	Early summer	Accent	No	Spain, Morocco	4"	8"	6"	Full sun, mixed sun/shade	Red accents on flower petals, use in cooler summer locations
Anthemis tinctoria	96	6	Yellow flowers, green foliage	Midsummer	Accent	Yes	Southern Europe	19"	10"	6"	Full sun	Can be weedy, won't survive long dry periods
Delosperma basuticum 'Gold Nugget'	107	6	Yellow flowers, green foliage	Late Spring	Accent	No	South Africa	2"	4"	4"	Full sun	Flowers obscure foliage, may rebloom later in some years
Delosperma basuticum 'White Nugget'	108	6	White flowers w/ yellow centers, green foliage	Late Spring	Accent	No	South Africa	2"	4"	4"	Full sun	
Delosperma cooperi	108	6	Pink flowers, green foliage	Midsummer to midautumn	Groundcover	No	South Africa	4"	12"	4"	Full sun	Most common Delosperma, rapid growth, large flowers
Delosperma dyeri	108	6	Red flowers w/ light center, green foliage	Midsummer to midautumn	Groundcover	No	South Africa	3"	6"	4"	Full sun	Color fades with sun producting multiple shades of red
Delosperma ecklonis var. latifolia	109	6	Pink-purple flowers, green foliage	Midsummer to midautumn	Groundcover	No	South Africa	4"	10"	4"	Full sun	Reliably hardy, rapid coverage, can hang over an edge
Delosperma 'Kelaidis'	110	6	Salmon flowers, green foliage	Midsummer to midautumn	Groundcover	No	South Africa	4"	12"	4"	Full sun	Unusual flower color, rapid growth
Dianthus spiculifolius	114	6	White flowers with red eye, green foliage	Late Spring	Accent	No	Eastern Carpathians	6"	8"	6"	Full sun	Nice fragrance, dense foliage
Echium russicum	115	6	Dark red flowers, green foliage	Early to Late Summer	Accent	No	Europe, Africa, W. Asia	23"	8"	6"	Full sun	Tall red spikes, long bloom time, good plant for border
Euphorbia myrsinites	118	6	Yellow Flowers, blue green foliage	Late Spring	Accent	Yes	Mediterranean	10"	10"	6"	Full sun	Nice foliage and structure, can spread and may need controlled
Festuca idahoensis	118	6	Silver-blue flowers, blue green foliage	Late Spring	Accent	Yes	Western US	12"	8"	6"	Mixed sun/shade	Can be used in mass planting but may need divided or replanted over time
Fragaria chiloensis	118	6	White flowers, green foliage	Late Spring	Accent	No	Western US, South America	8"	10"	6"	Full sun, mixed sun/shade	Wild strawberry with edible fruit, can attract birds in habitat creation
Hieracium pilosella	121	6	Pale yellow flowers, hairy green foliage	Early to Late Summer	Groundcover	Yes	Europe, NW Siberia, Asia Minor	12"	8"	6"	Full sun	Forms tight mat and spreads by seed and stolons
Hieracium spilophaeum 'Leopard'	122	6	Yellow flowers, green foliage with purple brown	Early to Late Summer	Groundcover	Yes	Western and Central Europe	10"	8"	6"	Full sun	Colorful variegation provides more visual interest than other Hieracium outside bloom period

Hieracium villosum	122	6	Yellow flowers, hairy green foliage	Early to Late Summer	Groundcover	Yes	Alps, Carpathians, Apennines, others	12"	8"	6"	Full sun	Very hairy leaves, attractive when covered with morning due
Orostachys aggregeatum	131	6	White flowers, apple green foliage	Early autumn to midautumn	Groundcover	No	Northern Asia	6"	6"	4"	Full sun	All Orostachys send out plantlets on stolons in spring and summer creating mat of rosettes
Orostachys boehmeri	131	6	White flowers, gray foliage	Early autumn to midautumn	Groundcover	No	Northern Asia	6"	6"	4"	Full sun	Unusual gray foliage and dunce cap shaped flower stalks in the fall
Orostachys fimbriata	132	6	White flowers, gray brownish red foliage	Early autumn to midautumn	Accent	No	Northern Asia	6"	6"	4"	Full sun	More likely to die from winter wet than cold, needs sharp drainage, flowers bloom to sun
Penstemon smallii	133	6	Purple flowers, green foliage	Early to Late Summer	Accent	No	Southeastern US	22"	10"	6"	Full sun, mixed sun/shade	Native for dry shade, may need irrigation during dry periods
Petrorhagia saxifraga	134	6	Light pink flowers, green foliage	Early summer to early autumn	Groundcover	Yes	Southern Europe, Asia Minor	7"	12"	6"	Full sun	Lots of small pink flowers throughout summer, may need cut back before winter
Rosularia chrysantha	137	6	Creamy white flowers, yellow green foliage	Midsummer	Accent	No	Asia Minor, Central Asia	4"	5"	4"	Full sun	Foliage turns redish in winter, not fast growing, small mounds of rosettes provide interest in border
Rosularia muratdaghensis	138	6	White flowers, gray green foliage	Midsummer	Accent	No	Asia Minor, Central Asia	3"	4"	4"	Full sun	Foliage not as hairy as R. Chrysantha but same mounding habit
Salvia jurisicii	139	6	Pink lilac flowers, green foliage	Midsummer to late summer	Accent	No	Balkans	10"	12"	6"	Full sun	Good choice for Mediterranean conditions, fine textured foliage
Scabiosa columbaria 'Misty Butterflies'	141	6	Pink purple flowers, green foliage	Early summer to early autumn	Accent	No	Europe, Africa, Asia	10"	10"	6"	Full sun, mixed sun/shade	Colorful, another optioin is S. columbaria 'Pincushion Pink'
Sedum hispanicum	150	6	White flowers, blue green foliage	Midsummer	Groundcover	No	Sicily to Turkey	3"	8"	4"	Full sun	Rapid growing low sedum. Blues, pinks, and purples in foliage
Sedum 'Matrona'	154	6	Pink flowers, green gray foliage	Early autumn	Accent	No	Japan	12"	10"	4"	Full sun	Provides some height in plant in thin medium
Sedum pluricaule var. ezawe	157	6	Pink flowers, green to purple foliage	Midsummer to late summer	Groundcover	No	Eastern Siberia	3"	6"	4"	Full sun	Attractive tightly clustered leaves in shades of purple. Use for foliage more than flower.
Sedum sieboldii	161	6	Pink flowers, blue green foliage with pink tinges	Midautumn	Accent	No	Japan	8"	8"	4"	Full sun	Mounding. Needs more care to establish than groundcover sedums
Sedum spathulifolium	162	6	Yellow flowers, gray foliage	Midsummer to late summer	Groundcover	No	US Northwest	4"	6"	4"	Full sun, mixed sun/shade	Does not do well in the midwestern or eastern US
Sedum telephium 'Emperor's Waves'	166	6	Purple red flowers, blue green foliage	Late Spring	Accent	No	Japan	16"	8"	4"	Full sun	Exciting dark foliage. Good for contrast in both height and color.
Sedum urvillei	168	6	Yellow Flowers, blue green foliage	Late Summer	Groundcover	No	Eastern Europe, Middle East	2"	6"	4"	Full sun	Tight growth, can take the summer heat, foliage red in winter
Sesleria autumnalis	170	6	Golden brown flowers, green to golden foliage	Early autumn	Accent	Yes	Italy to Albania	16"	12"	6"	Full sun, mixed sun/shade	Not attractive as an accent, effective in mass, with golden fall color

Sporobolus heterolepis	173	6	Brown flowers, green foliage	Midsummer to early autumn	Accent	Yes	US Great Plains	30"	12"	6"	Full sun, mixed sun/shade	Graceful native grass, May need to mound medium before planting in drier locations
Talinum calycinum	173	6	Neon pink flowers, green flowers	Midsummer to midautumn	Accent	Yes	North America	4"	2"	4"	Full sun	Showiest of talinums, a favorite of honeybees, foliage disappears at the first sign of frost
Talinum parviflorum	174	6	Light pink flowers, green foliage	Midsummer to midautumn	Accent	Yes	North America	8"	4"	4"	Full sun	Suited for very dry and windy locations
Talinum teretifolium	175	6	Rose pink flowers, green foliage	Midsummer to midautumn	Accent	Yes	Appalachians, PA to GA	12"	6"	4"	Full sun	Threatened throughout the Mid-Atlantic region due to its shrinking habitat, opportunity for conservation

Yellow	Low Sedum
Green	Wild Flower
Red	Garden Area

Green Roof Load Calculations

GREEN ROOF LOADS

- 4" Sedum: Non public
  - Plants: 2 PSF
  - Allow: 1 PSF = 53 PSF Dead
  - Insul: 1.5 PSF/in (6") = 9 PSF
  - Growing: 85 PCF (4/12) = 28.3 PSF
  - Draining: 60 PCF (2.5/12) = 12.5 PSF
  - Retained water = = 26 PSF Live water
    - (85 PCF)(4/12)(0.624) = 17.7 PSF
    - (60 PCF)(2.5/12)(0.624) = 7.8 PSF
  - L_r = 20 PSF
  - Snow = 22 PSF

- 6" "Wild flower" Public
  - Plants: 3 PSF
  - Growing: 85 PCF (6/12) = 42.5 PSF
  - Allow: 1 PSF = 68 PSF Dead
  - Draining: 60 PCF (2.5/12) = 12.5 PCF
  - Insul: 1.5 PSF/in (6") = 9 PSF
  - Retained water = = 34 PSF Live Water
    - (85)(6/12)(0.624) = 26.5 PSF
    - (60)(2.5/12)(0.624) = 7.8 PSF
  - L_r = 20 PSF L_o = 100 PSF
  - Snow = 22 PSF [Roof used for roof gardens or assembly purposes, ASCE]

- Special coverage
  - Structural glass: 15 PSF/in
  - Hanover pedestal Paver (2'x2' nom.) = 22 PSF
  - Structural glass span/depth
    - $S/d = 30 \quad S/30 = d \Rightarrow 42''/30 = 1.4'' \Rightarrow$ use 2"
    - $\rightarrow$ (15 PSF/in)(2") = 30 PSF (42/12) = **105 PLF**

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT
TITLE
BY: SHEET:
CHKD: PROJECT NO:
DATE: PAGE:

- Mechanical point loads
  Dead load: W8X18 = 18 PLF
  Conc: = 33.8 PSF
  Deck: = 2.14 PSF
  SDL = 10 PSF
  trib: 2.5' × 5'
  Cladding = 226 PLF
  Dead = (2.5)(5)(33.8+2.14+10) + 5(18+226) = 1.79 K, 3.58 K
  Live = (2.5)(5)(125 PSF) = 1.56 K, 3.12 K

- New composite Deck (Public)
  Live load = 100 PSF } SL = 168 PSF
  SDL = 68 PSF
  - Use composite instead of roof deck b/c a concrete slab is more compatible w/ green roof + loads
  - 2 hr fire rating: unprotected deck, 3 1/4" Ltwt conc.
  - 3VLI19 Ltwt conc t = 3.25"
    Max Unshored 3 span 14'-6" > 10' OK
    Strength @ 10' = 168 = 168 FCfy
  - ☆ 3VLI19 Ltwt conc t = 3.50" → use this
    Max unshored 3 span 14'-4" > 10' OK
    Strength @ 10' = 176 PSF > 168 OK
    System wt = 48 PSF
- New composite Deck (non public)
  Live Load = 26 PSF
  SDL = 53 PSF Same as previous (fire)

- ETabs Loads:
  - 4" Sedum
    Dead: 53 + 48 = 101 PSF Live = 26 PSF
  - 6" W.bl
    Dead: 68 + 48 = 116 PSF Live = 100 PSF
  - Also: Glass, pavers, furniture, mechanical

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT
TITLE
BY: SHEET:
CHKD: PROJECT NO:
DATE: PAGE:

Appendix F: Roof Redesign ETABS Output

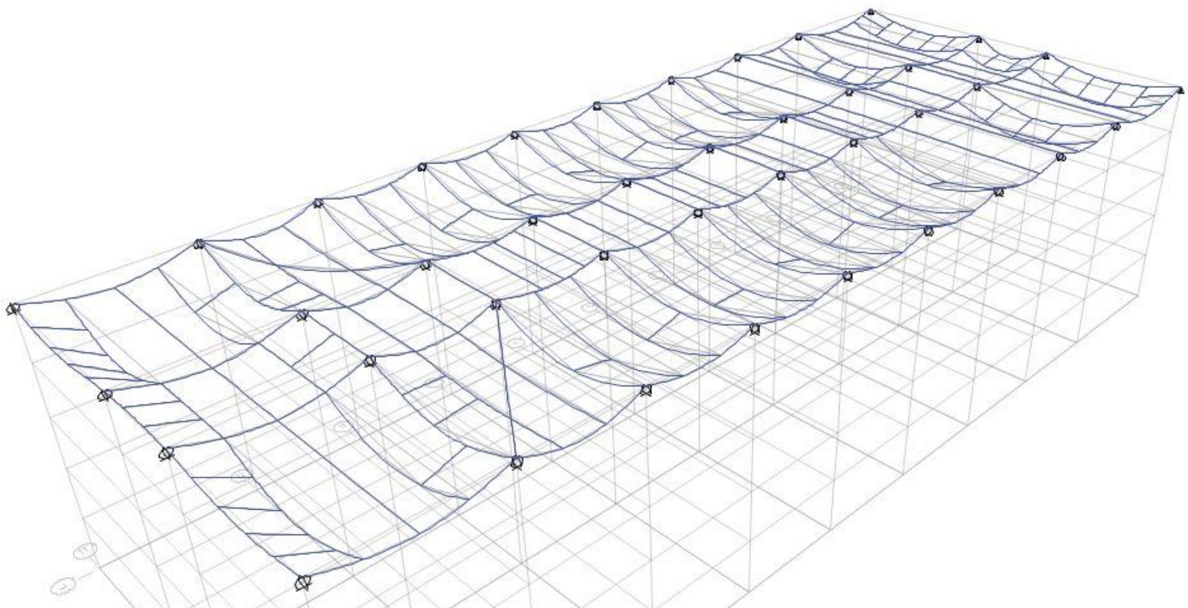


Figure 40: Gravity Roof Framing Deflected Shape

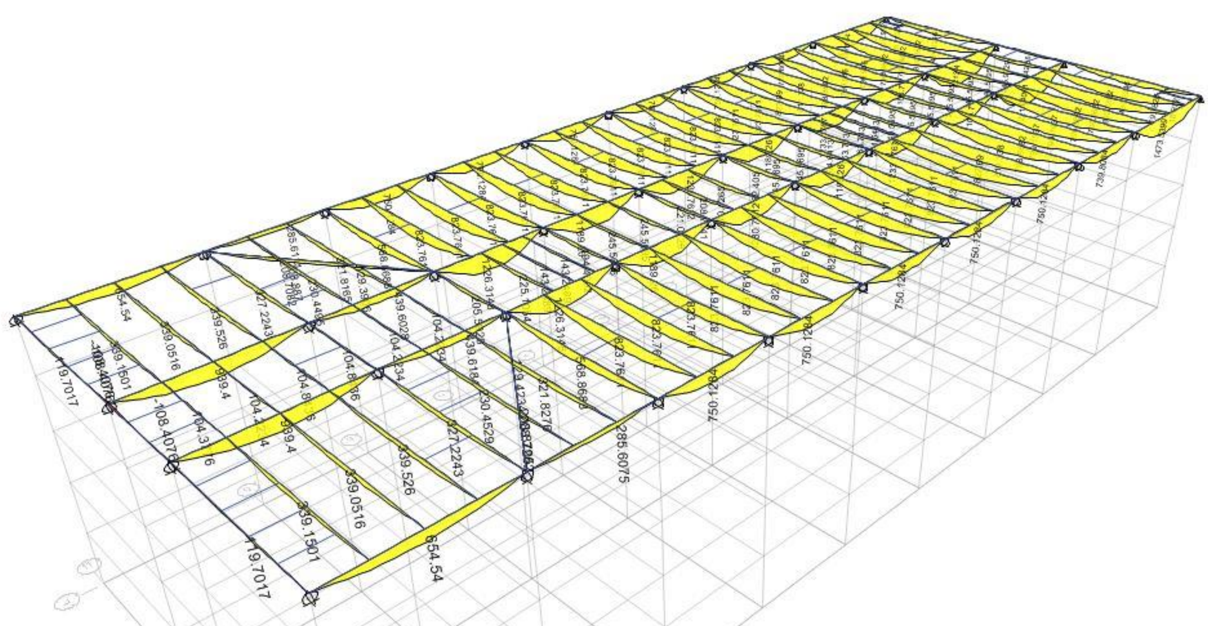


Figure 41: Gravity Roof Framing Moment Diagram

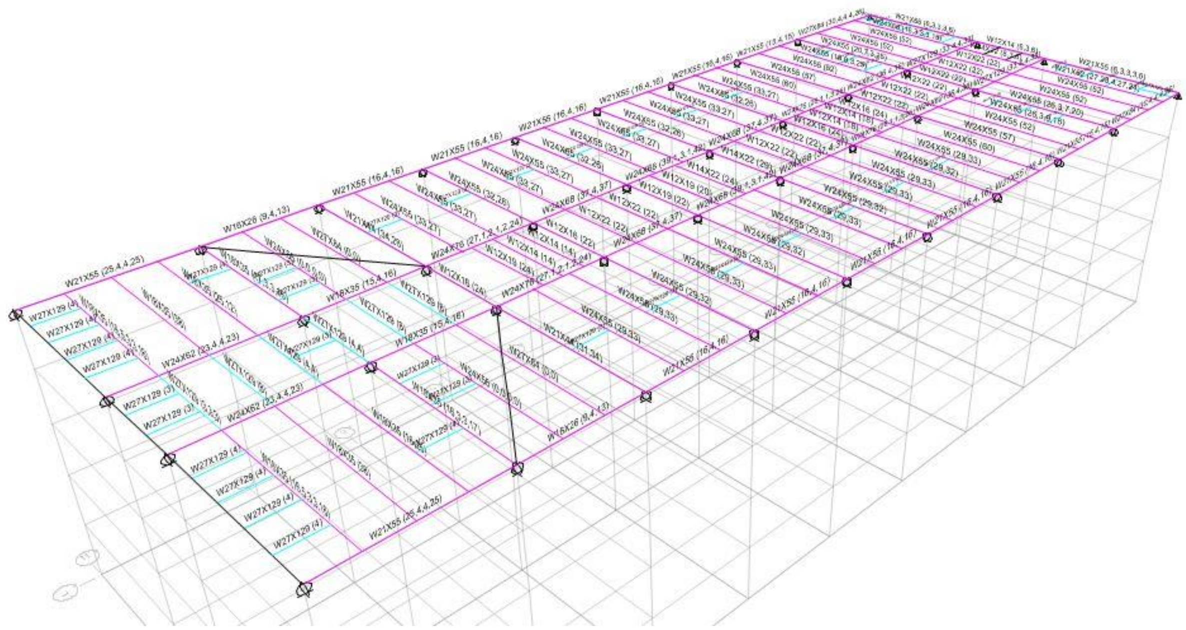


Figure 42: Gravity Roof Framing Code Check

GRAVITY ROOF FRAMING COMPOSITE BEAM SUMMARIES								
Design Section	Fy lb/in²	Stud Diameter in	Stud Layout	Pass/Fail	Left Reaction kip	Right Reaction kip	Max +Moment kip-ft	Overall Ratio
W21X55	50000	0.75	25; 4; 4; 25	Passed	53.63	53.63	695.38	0.993
W21X55	50000	0.75	25; 4; 4; 25	Passed	53.63	53.63	695.38	0.993
W24X62	50000	0.75	23; 4; 4; 23	Passed	74.235	74.235	989.8	0.998
W24X62	50000	0.75	23; 4; 4; 23	Passed	74.235	74.235	989.8	0.998
W18X35	50000	0.75	16; 3; 3; 17	Passed	27.102	31.226	344.7803	0.994
W18X35	50000	0.75	17; 3; 3; 16	Passed	31.226	27.102	344.7803	0.994
W27X129	50000	0.75	4; 4	Passed	17.675	17.675	110.4686	0.111
W18X35	50000	0.75	16; 5; 3; 3; 16	Passed	31.815	31.815	357.3459	1
W18X35	50000	0.75	38	Passed	31.815	31.815	357.2422	0.979
W18X35	50000	0.75	16; 25	Passed	31.815	31.815	357.742	0.981
W18X35	50000	0.75	16; 5; 3; 3; 16	Passed	31.815	31.815	357.3459	1
W18X35	50000	0.75	38	Passed	31.815	31.815	357.2422	0.979
W18X35	50000	0.75	25; 12	Passed	31.815	31.815	357.742	0.981
W27X129	50000	0.75	3; 3; 3	Passed	17.675	17.675	109.9775	0.125
W27X129	50000	0.75	8	Passed	17.675	17.675	109.8151	0.111
W27X129	50000	0.75	4; 4	Passed	17.675	17.675	110.4686	0.111
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	4	Passed	0	0	0	0.08
W27X129	50000	0.75	4	Passed	0	0	0	0.08

GRAVITY ROOF FRAMING COMPOSITE BEAM SUMMARIES								
Design Section	Fy lb/in²	Stud Diameter in	Stud Layout	Pass/Fail	Left Reaction kip	Right Reaction kip	Max +Moment kip-ft	Overall Ratio
W16X26	50000	0.75	9; 4; 13	Passed	28.391	31.84	303.6345	0.999
W16X26	50000	0.75	9; 4; 13	Passed	28.393	31.839	303.6372	0.999
W18X35	50000	0.75	15; 4; 16	Passed	45.084	46.322	463.2691	0.996
W18X35	50000	0.75	15; 4; 16	Passed	45.083	46.325	463.2532	0.996
W21X44	50000	0.75	31; 34	Passed	47.306	47.383	532.1524	0.995
W21X44	50000	0.75	34; 28	Passed	47.383	47.306	532.1524	0.995
W12X16	50000	0.75	24	Passed	30.95	30.95	192.2929	0.969
W24X55	50000	0	0; 0; 0; 0	Passed	20.51	26.166	242.134	0.926
W27X84	50000	0	0; 0	Passed	30.859	29.89	339.4096	0.953
W24X55	50000	0	0; 0; 0; 0	Passed	26.166	20.512	242.1305	0.926
W27X84	50000	0	0; 0	Passed	29.893	30.859	339.3972	0.953
W27X129	50000	0.75	8	Passed	17.675	17.675	109.8151	0.111
W27X129	50000	0.75	8	Passed	17.675	17.675	109.8151	0.111
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W24X76	50000	0.75	27; 1; 2; 1; 2; 24	Passed	108.301	120.768	1164.8226	0.993
W24X76	50000	0.75	27; 1; 2; 1; 2; 24	Passed	108.301	120.768	1164.8226	0.993
W24X55	50000	0.75	29; 32	Passed	68.352	75.576	780.7434	0.998
W12X16	50000	0.75	22	Passed	29.608	29.608	183.9571	0.955
W24X55	50000	0.75	32; 26	Passed	75.576	68.352	780.7434	0.998
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993

GRAVITY ROOF FRAMING COMPOSITE BEAM SUMMARIES								
Design Section	Fy lb/in²	Stud Diameter in	Stud Layout	Pass/Fail	Left Reaction kip	Right Reaction kip	Max +Moment kip-ft	Overall Ratio
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W24X68	50000	0.75	37; 4; 37	Passed	112.976	112.976	1129.76	0.995
W24X68	50000	0.75	37; 4; 37	Passed	112.976	112.976	1129.76	0.995
W24X55	50000	0.75	29; 32	Passed	68.352	75.576	780.7434	0.998
W24X55	50000	0.75	32; 26	Passed	75.576	68.352	780.7434	0.998
W12X19	50000	0.75	22	Passed	33.66	33.66	209.1302	0.983
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W24X55	50000	0.75	29; 32	Passed	68.352	75.576	780.7434	0.998
W24X55	50000	0.75	32; 26	Passed	75.576	68.352	780.7434	0.998
W24X68	50000	0.75	39; 1; 3; 1; 42	Passed	116.716	107.366	1169.03	0.999
W24X68	50000	0.75	39; 1; 3; 1; 42	Passed	116.716	107.366	1169.03	0.999
W14X22	50000	0.75	29	Passed	43.01	43.01	267.2219	0.989
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W24X55	50000	0.75	32; 26	Passed	75.576	68.352	780.7434	0.998
W24X55	50000	0.75	29; 32	Passed	68.352	75.576	780.7434	0.998
W24X68	50000	0.75	37; 4; 37	Passed	112.976	112.976	1129.76	0.995
W24X68	50000	0.75	37; 4; 37	Passed	112.976	112.976	1129.76	0.995
W12X16	50000	0.75	24	Passed	31.167	31.167	193.6391	0.964
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993

GRAVITY ROOF FRAMING COMPOSITE BEAM SUMMARIES								
Design Section	Fy lb/in²	Stud Diameter in	Stud Layout	Pass/Fail	Left Reaction kip	Right Reaction kip	Max +Moment kip-ft	Overall Ratio
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W24X55	50000	0.75	57	Passed	67.836	71.448	768.328	0.993
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W21X55	50000	0.75	16; 4; 16	Passed	72.15	72.15	711.8784	0.993
W24X55	50000	0.75	57	Passed	71.448	67.836	768.328	0.993
W24X76	50000	0.75	26; 1; 1; 3; 26	Passed	119.209	112.976	1171.3155	0.999
W24X76	50000	0.75	26; 1; 1; 3; 26	Passed	119.209	112.976	1171.3155	0.999
W24X55	50000	0.75	33; 27	Passed	75.576	68.352	780.7434	0.993
W24X55	50000	0.75	60	Passed	75.576	68.352	779.68	0.992
W24X55	50000	0.75	29; 33	Passed	68.352	75.576	780.7434	0.993
W24X55	50000	0.75	60	Passed	68.352	75.576	779.68	0.992
W21X55	50000	0.75	15; 4; 15	Passed	71.118	71.118	701.5584	0.995
W24X55	50000	0.75	26; 3; 7; 20	Passed	67.32	67.32	756.2263	1
W21X55	50000	0.75	15; 4; 15	Passed	71.118	71.118	701.5584	0.995
W24X55	50000	0.75	20; 7; 3; 25	Passed	67.32	67.32	756.976	1
W24X62	50000	0.75	36; 4; 36	Passed	104.72	104.72	1047.2	0.997
W24X62	50000	0.75	36; 4; 36	Passed	104.72	104.72	1047.2	0.997
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W24X55	50000	0.75	52	Passed	67.32	67.32	755.9183	0.993
W24X55	50000	0.75	26; 3; 9; 18	Passed	67.32	67.32	756.2263	1
W24X55	50000	0.75	52	Passed	67.32	67.32	755.9183	0.993
W24X55	50000	0.75	18; 9; 3; 25	Passed	67.32	67.32	756.976	1
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063

GRAVITY ROOF FRAMING COMPOSITE BEAM SUMMARIES								
Design Section	Fy lb/in²	Stud Diameter in	Stud Layout	Pass/Fail	Left Reaction kip	Right Reaction kip	Max +Moment kip-ft	Overall Ratio
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X84	50000	0.75	30; 4; 4; 4; 26	Passed	106.044	109.784	1397.0399	1
W27X84	50000	0.75	30; 4; 4; 4; 26	Passed	106.044	109.784	1397.04	1
W27X129	50000	0.75	33; 4; 4; 33	Passed	157.08	157.08	2094.4001	0.994
W27X129	50000	0.75	33; 4; 4; 33	Passed	157.08	157.08	2094.4	0.994
W21X55	50000	0.75	6; 3; 3; 3; 6	Passed	35.617	39.357	442.7116	0.995
W21X55	50000	0.75	6; 3; 3; 3; 6	Passed	39.357	35.617	442.7116	0.995
W12X14	50000	0.75	6; 3; 6	Passed	21.865	21.865	136.0489	0.971
W24X55	50000	0.75	52	Passed	67.32	67.32	755.9183	0.993
W24X55	50000	0.75	52	Passed	67.32	67.32	755.9183	0.993
W21X62	50000	0.75	27; 23; 4; 27; 24	Passed	63.58	67.32	757.2562	1
W24X55	50000	0.75	52	Passed	67.32	67.32	755.9183	0.993
W24X55	50000	0.75	52	Passed	67.32	67.32	755.9183	0.993
W24X68	50000	0.75	16; 3; 3; 3; 15	Passed	67.32	63.58	757.2563	0.997
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W14X22	50000	0.75	8; 3; 8	Passed	37.4	37.4	232.7107	0.969
W27X129	50000	0.75	2; 2	Passed	3.74	3.74	18.7001	0.083
W27X129	50000	0.75	2; 2	Passed	3.74	3.74	18.7	0.083
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	7.48	7.48	17.952	0.111

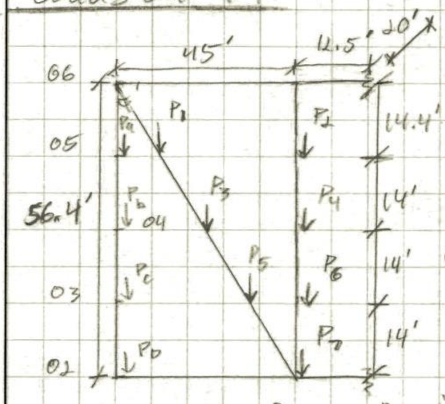
GRAVITY ROOF FRAMING COMPOSITE BEAM SUMMARIES								
Design Section	Fy lb/in²	Stud Diameter in	Stud Layout	Pass/Fail	Left Reaction kip	Right Reaction kip	Max +Moment kip-ft	Overall Ratio
W27X129	50000	0.75	3	Passed	7.48	7.48	17.952	0.111
W12X16	50000	0.75	24	Passed	31.167	31.167	193.6391	1
W12X14	50000	0.75	18	Passed	24.933	24.933	154.9112	0.952
W12X14	50000	0.75	18	Passed	24.933	24.933	154.9112	0.952
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W12X22	50000	0.75	22	Passed	37.4	37.4	232.3669	0.998
W12X19	50000	0.75	24	Passed	34.283	34.283	213.003	1
W12X14	50000	0.75	14	Passed	21.817	21.817	135.5473	0.941
W12X14	50000	0.75	14	Passed	21.817	21.817	135.5473	0.941
W12X19	50000	0.75	20	Passed	31.79	31.79	197.5118	0.969
W14X22	50000	0.75	24	Passed	41.14	41.14	255.6036	0.99
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063
W27X129	50000	0.75	3	Passed	0	0	0	0.063

Appendix G: Truss Loading Calculations & ETABS Output

Note: All ETABS truss diagrams are shown for the controlling load case 1.2D+1.6L+.05Lr. Appendix begins on the next page.

Gravity Load Calculations

Loads on T1



From Fall:
Live Public (floor) = 50 psf
Dead Load = 69 psf
Floor = 56 psf
Frame = 13 psf
Curtain = 211 plf

$\theta = \tan^{-1}(45/56.4) = 38.6°$

• Level 05: P_a 5.75 | 5.75 P_1 16.75 | 16.75 P_2 12.5'
11.5' | 33.5'

$14.4 \tan 38.6 = 11.5'$
$45 - 11.5 = 33.5'$
$14.4/56.4 = 0.255$

P_a Dead = 9.15^k
(5.75')(20')(69) + (5.75)(211)
P_a live = (5.75)(20)(50) = 5.75^k
P_1 Dead = 12.5^k
(5.75 + 16.75)(5)(69) + (5.75 + 16.75)(211)
P_1 Live = (5.75 + 16.75)(5)(50) = 5.63^k
P_2 Dead = 46.5^k
(16.75 + 12.5)(20)(69) + (16.75 + 12.5)(211)
P_2 Live = (16.75 + 12.5)(20)(50) = 29.3^k

• Level 04: P_b 11.3 | 11.3 P_3 11.2 | 11.2 P_4 12.5'
22.67 | 22.33'

$(14.4 + 14) \tan 38.6 = 22.67'$
$45 - 22.67 = 22.33'$

P_b Dead = (11.3')(20)(69) + (11.3)(211) = 18.0^k
P_b live = (11.3)(20)(50) = 11.3^k

$\frac{14.4 + 14}{56.4} = 0.504$

P_3 Dead = 12.5^k
(11.3 + 11.2)(5)(69) + (22.5)(211)
P_3 live = (22.5)(5)(50) = 5.63^k
P_4 Dead = 37.7^k
(11.2 + 12.5)(20)(69) + (23.7)(211)
P_4 live = (23.7)(20)(50) = 23.7^k

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT
TITLE
BY: SHEET:
CHKD: PROJECT NO:
DATE: PAGE:

• Level 03

P_c P_5 P_6

16.9' | 16.9' | 5.58 | 5.58 | 12.5'

33.85' 11.15'

$(14.4+2 \cdot 14)\tan 38.6 = 33.85$

$45 - 33.85 = 11.15'$

P_c Dead: $(16.9)(20)(69) + (16.9)(211) = 26.9^k$

P_c Live: $(16.9)(20)(50) = 16.9^k$

P_5 Dead $= 12.5^k$

$(16.9+5.58)(5)(69) + 22.48(211)$

$\frac{14.4 + 2 \cdot 14}{56.4} = 0.752$

P_5 Live $= (16.9+5.58)(5)(50) = 5.63^k$

P_6 Dead $= 25.2^k$

$(5.58+12.5)(20)(69) + 18.08(211)$

P_6 Live $= (18.08)(20)(50) = 18.1^k$

• Level 02

P_D P_7

12.5' | 22.5' | 12.5'

P_D Dead $= 35.8^k$

$(22.5)(20)(69) + 22.5(211)$

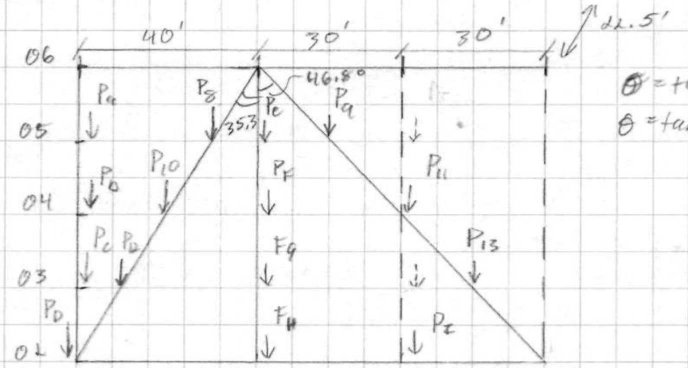
P_D Live $= (22.5)(20)(50) = 22.5^k$

P_7 Dead $= 55.7$

$(22.5+12.5)(20)(69) + (35)(211)$

P_7 Live $= (35)(20)(50) = 35.0^k$

LOADS ON T2



$\theta = \tan^{-1}(40/56.4) = 35.3°$

$\theta = \tan^{-1}(60/56.4) = 46.8°$

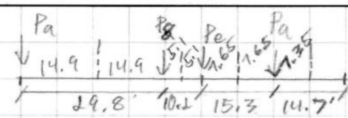
P_I Dead $= [(30)(22.5)(69) + (30)(211)](2 \text{ FLRS}) = 105.8^k$

P_I Live $= (30)(22.5)(50)(2) = 67.50^k$

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT
TITLE
BY: SHEET:
CHKD: PROJECT NO:
DATE: PAGE:

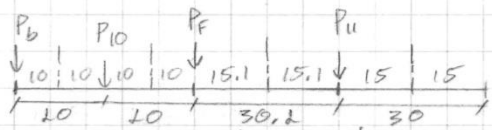
• Level 05



14.4 tan 35.3 = 10.2'
14.4 tan 46.8 = 15.3'

P8 Dead = 35.27^K
(14.9 + 5.1)(22.5)(69) + (20)(211)
P8 Live = (20)(22.5)(50) = 22.5^K
Pe Dead = 22.48^K
(5.1 + 7.65)(22.5)(69) + (12.75)(211)
Pe Live = (12.75)(22.5)(50) = 14.34^K
P9 Dead = 26.45^K
(7.65 + 7.35)(22.5)(69) + (15)(211)
P9 Live = (15)(22.5)(50) = 16.88^K

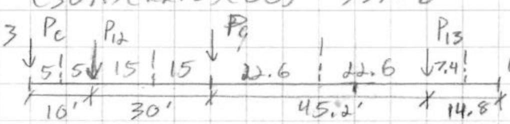
• Level 04



28.4 tan 35.3 = 20.1'
28.4 tan 46.8 = 30.2'

P10 Dead = (20)(22.5)(69) + (20)(211) = 35.27^K
P10 Live = (20)(22.5)(50) = 22.5^K
PF Dead = (25.1)(22.5)(69) + (25.1)(211) = 44.26^K
PF Live = (25.1)(22.5)(50) = 28.24^K
P11 Dead = (30.1)(22.5)(69) + (30.1)(211) = 53.08^K
P11 Live = (30.1)(22.5)(50) = 33.86^K

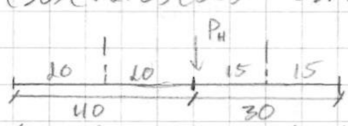
• Level 03



42.4 tan 35.3 = 30.0'
42.4 tan 46.8 = 45.2'

P9 Dead = (15 + 22.6)(22.5)(69) + (37.6)(211) = 66.31^K
P9 Live = (37.6)(22.5)(50) = 42.3^K
P13 Dead = (22.6 + 7.4)(22.5)(69) + (30)(211) = 52.91^K
P13 Live = (30)(22.5)(50) = 33.75^K

• Level 02



PH Dead = (35)(22.5)(69) + (35)(211) = 61.72^K
PH Live = (35)(22.5)(50) = 39.38^K

ruby+associates
STRUCTURAL ENGINEERS
30445 Northwestern Highway Suite 310
Farmington Hills, Michigan 48334
T:248.865.8855 F:248.865.9449
www.rubyusa.com

PROJECT	BY:	SHEET:
TITLE	CHKD:	PROJECT NO:
	DATE:	PAGE:

Maximum Truss T2 Deflection Limit for Roof Truss		
Combo	UX	UZ
Dead	-0.88097	-0.50585
Live	-0.4008	-0.22712
Live Roof	-0.0608	-0.03642
Snow	-0.05132	-0.03075
WindUp	0.04853	0.02912
DStlS1	-1.23336	-0.7082
DStlS2	-1.7241	-0.98579
DStlS3	**-1.72884**	-0.98863
DStlS4	-1.54007	-0.88335
DStlS5	-1.55524	-0.89242
DStlS6	-1.40598	-0.80292
DStlS7	-1.56126	-0.89612
DStlS8	-1.41072	-0.80576
DStlS9	-1.566	-0.89895
DStlS10	-1.10045	-0.63293
DStlS11	-1.17809	-0.67953
DStlS12	-1.11562	-0.642
DStlS13	-1.19326	-0.6886
DStlS14	-0.71523	-0.40867
DStlS15	-0.87051	-0.50187
DStlD1	-0.88097	-0.50585
DStlD2	-1.28177	-0.73297
Max. Roof Deflection= 1.723		
LC=1.2D+1.6L+0.5Lr		

Roof Truss Virtual Work Member Contribution for Preliminary Member Sizes				
P=	10	k		
E=	29000	ksi		
Member	Section	L	A	L/AE
3	W14x22	30	6.49	0.0001594
4	W14x22	30	6.49	0.0001594
5	W14x22	30	6.49	0.0001594
13	W14x22	30	6.49	0.0001594
14	W14x257	45	75.6	0.0000205
15	W14x257	45	75.6	0.0000205
37	W14x90	45	26.5	0.0000586
38	W14x90	45	26.5	0.0000586
16	W14x120	25	35.3	0.0000244
39	W18x71	25	20.9	0.0000412
64	W18x86	54	25.3	0.0000736
56	W18x86	54	25.3	0.0000736

Truss T1 ETABS

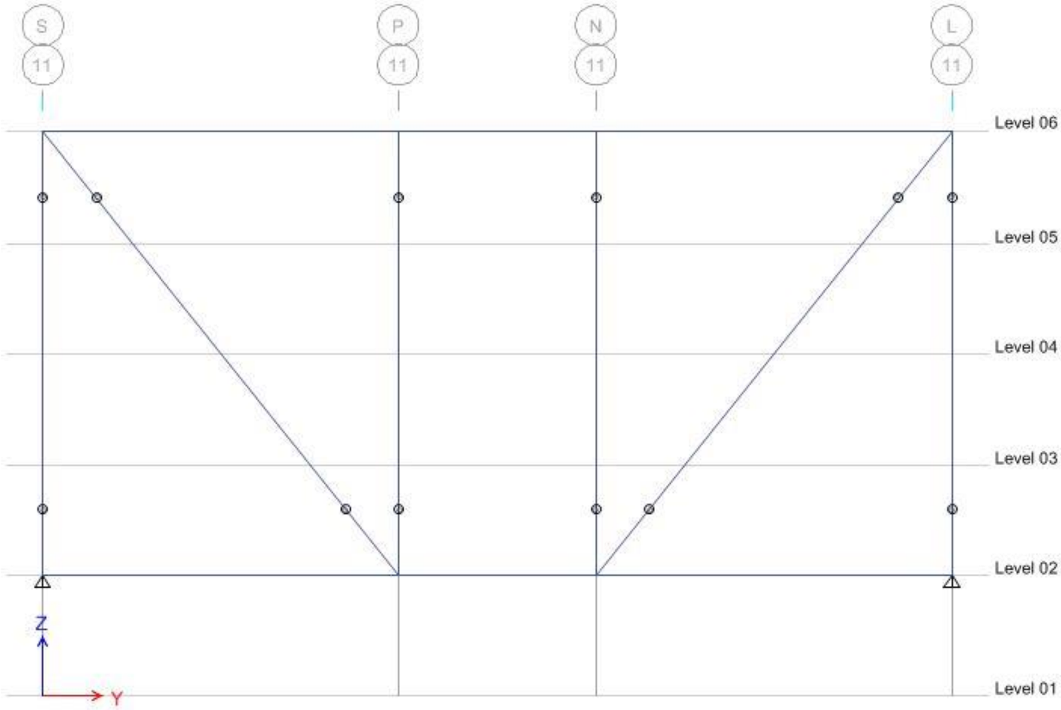


Figure 43: Truss T1 Model

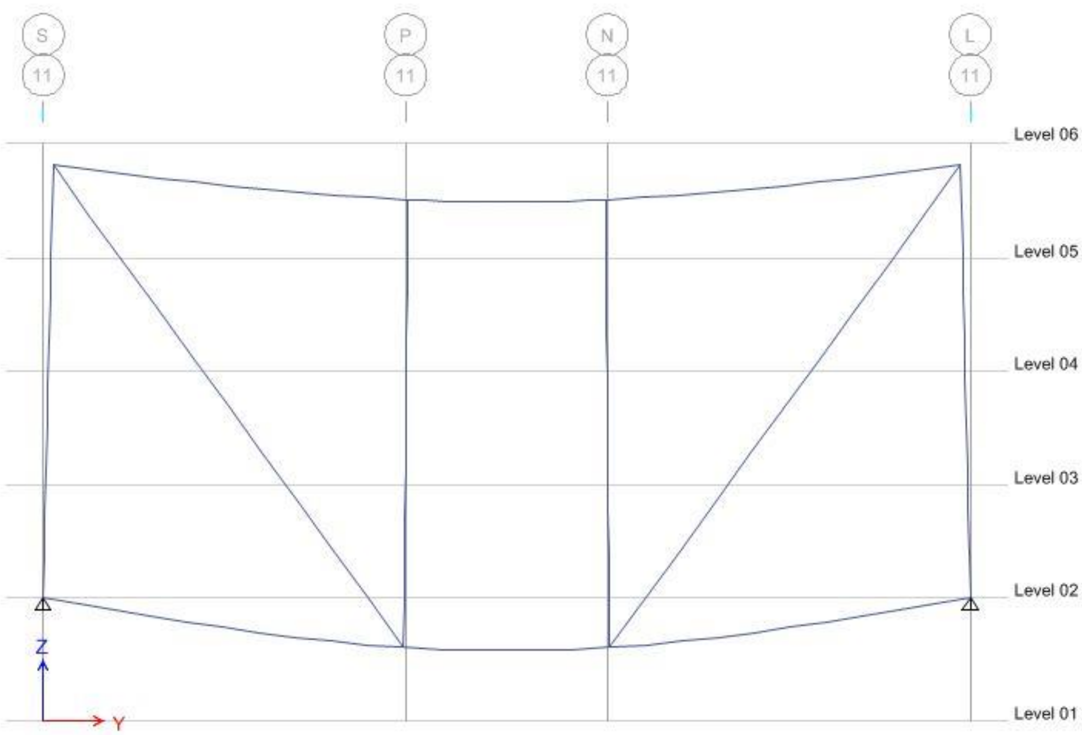


Figure 44: Truss T1 Deflected Shape

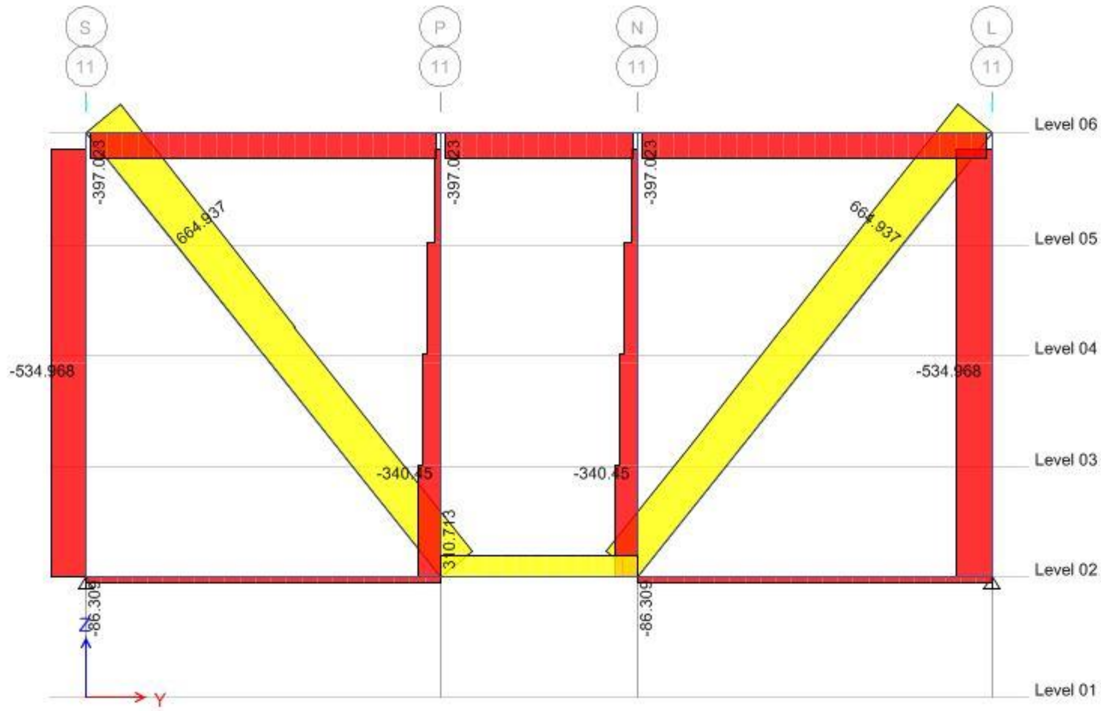


Figure 45: Truss T1 Axial Diagram

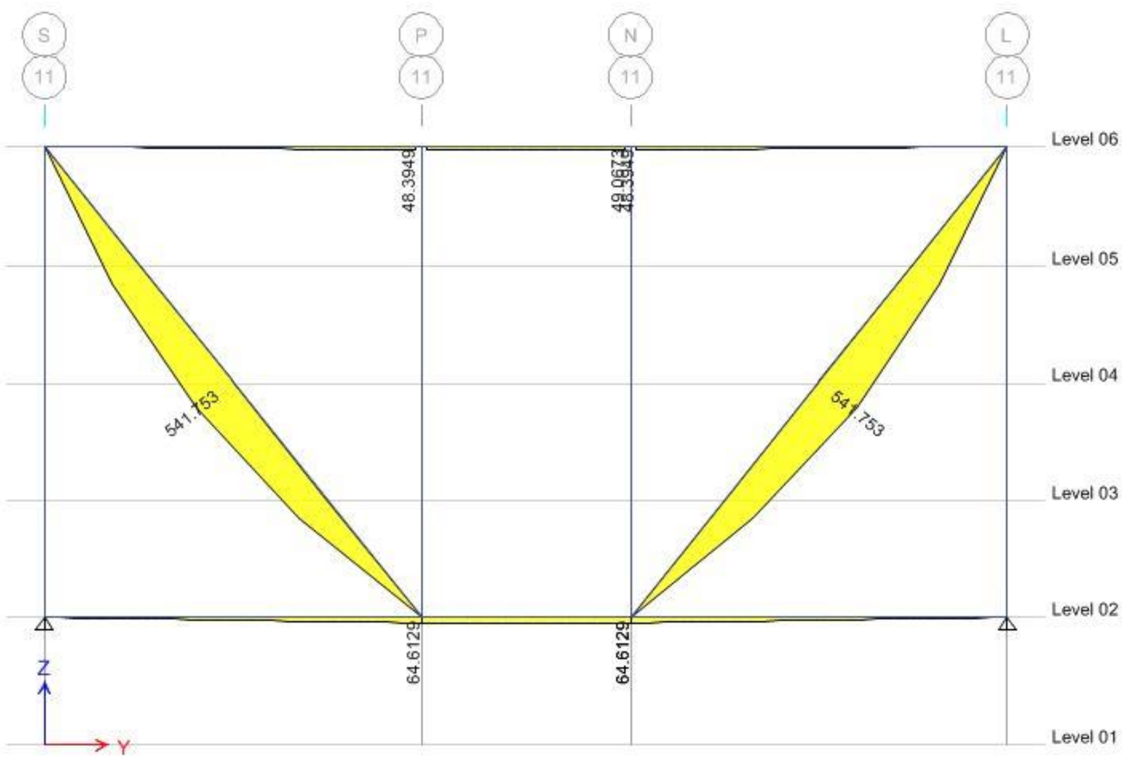


Figure 46: Truss T1 Moment Diagram

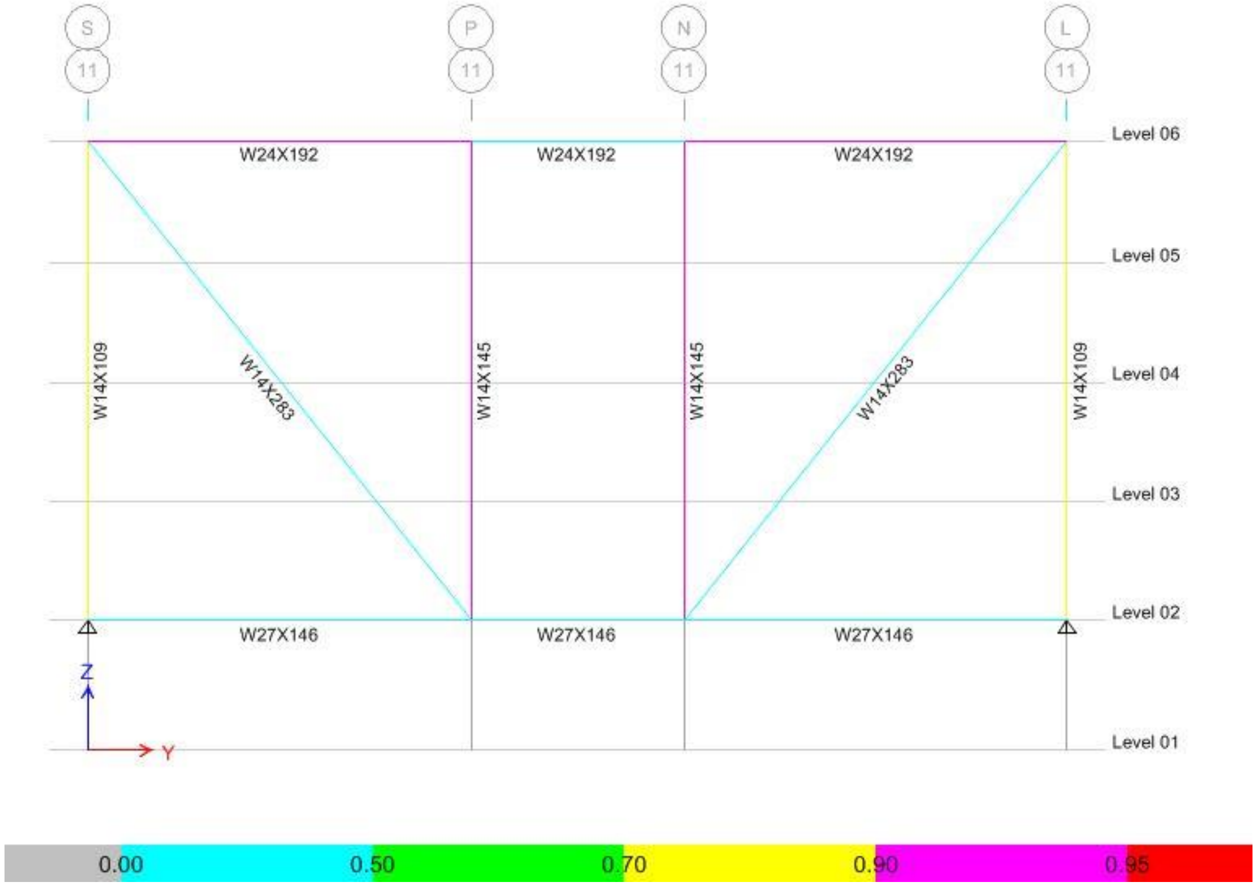


Figure 47: Truss T1 Code Check

Truss T2

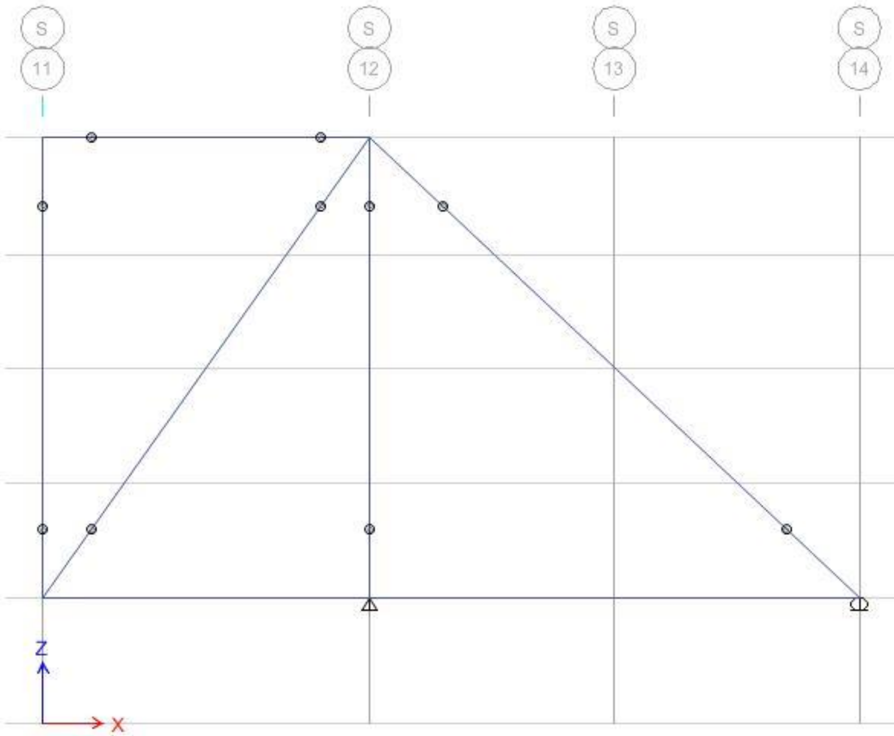


Figure 48: Truss T2 Model

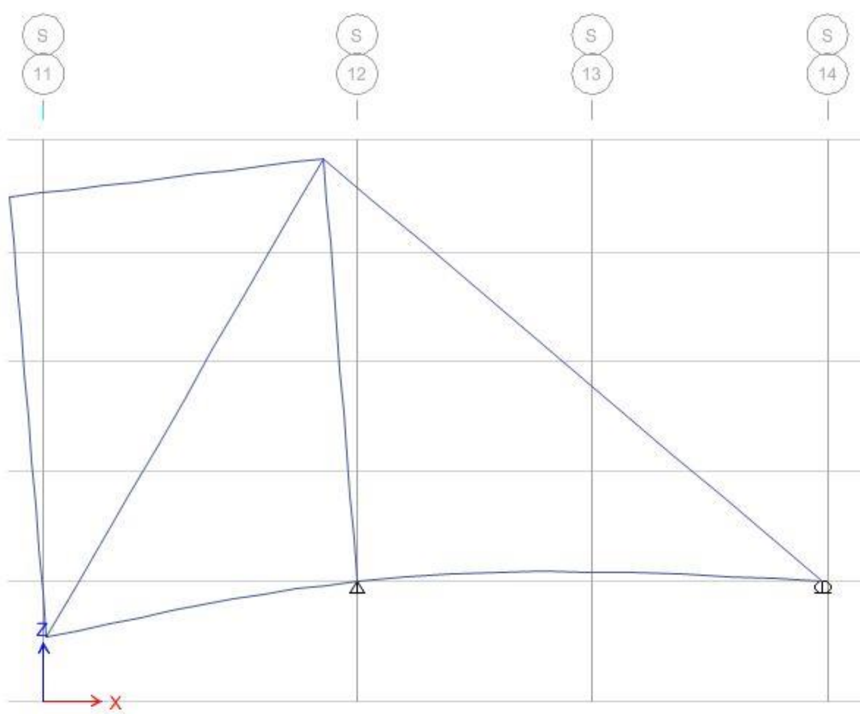


Figure 49: Truss T2 Deflected Shape

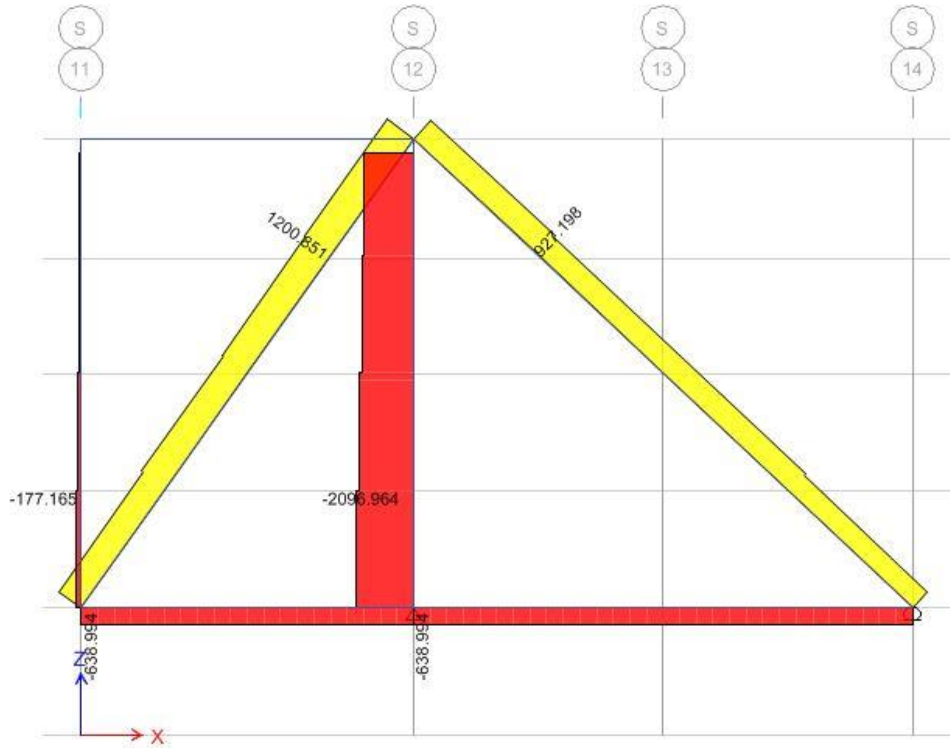


Figure 50: Truss T2 Axial Diagram

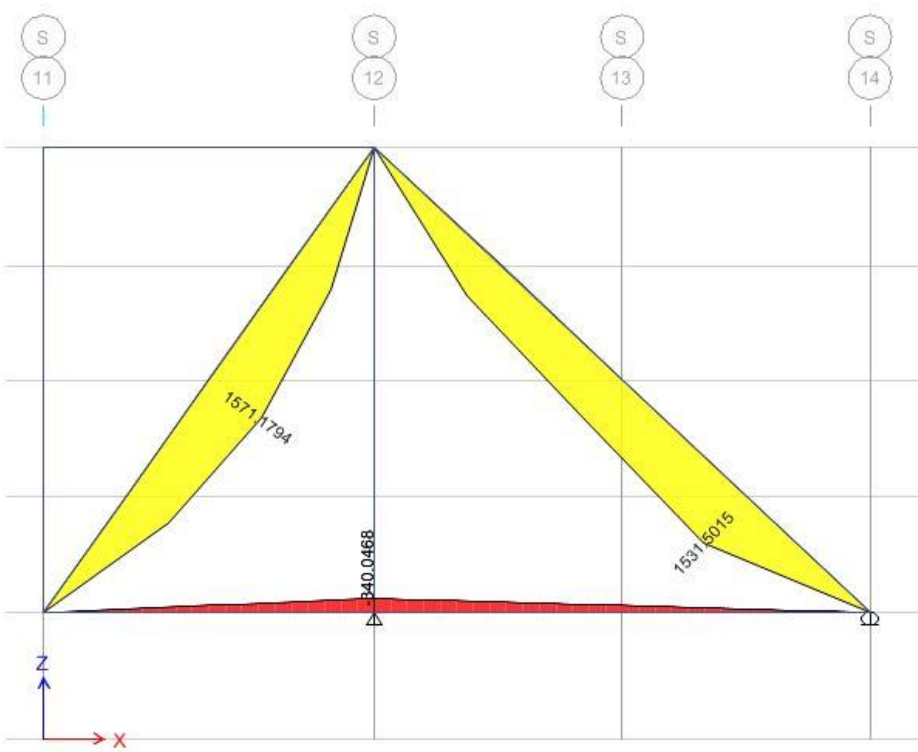


Figure 51: Truss T2 Moment Diagram

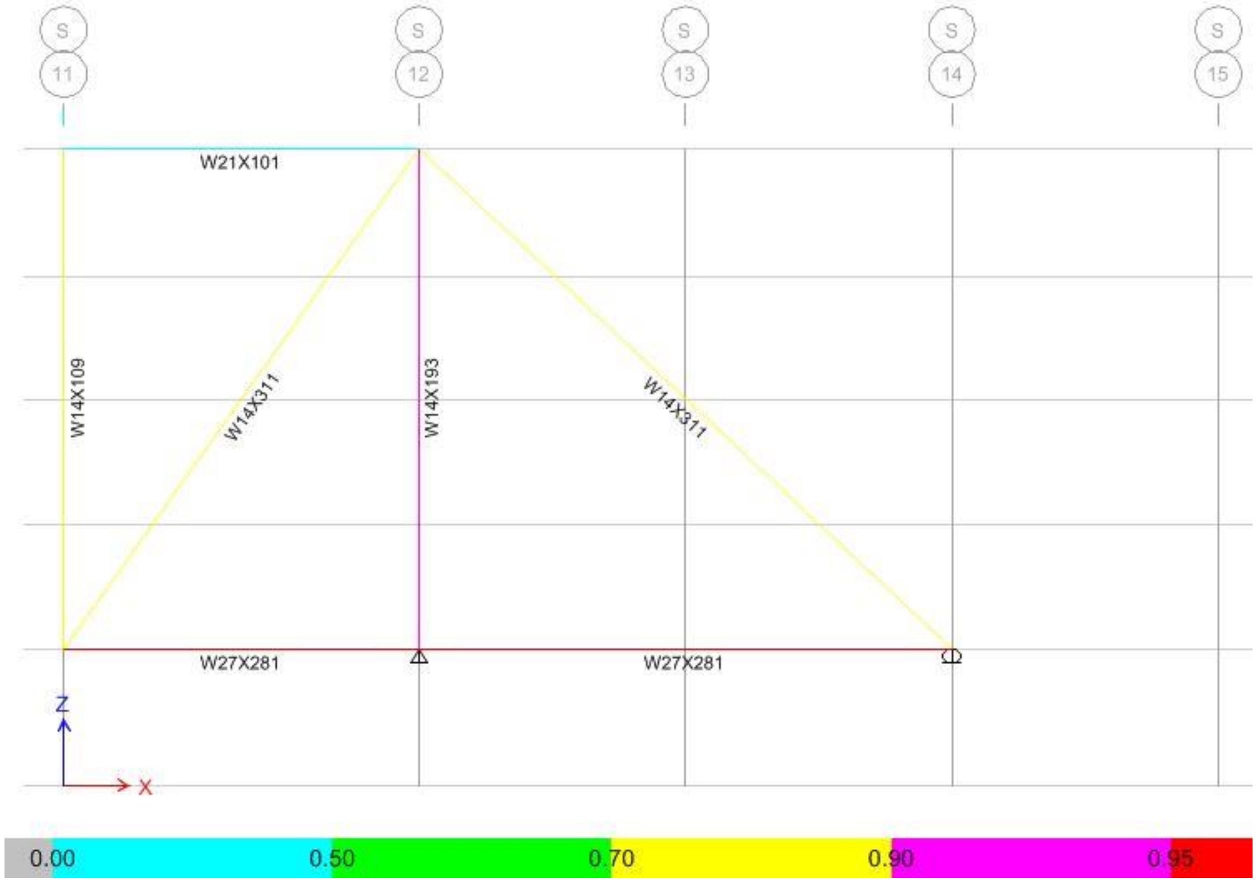


Figure 52: Truss T2 Code Check

Roof Truss

Note: Sections shown in roof model are those resulting from the truss analysis only. In all cases, the corresponding beams from the gravity roof design had a greater cross sectional area for resisting tension and compression as well as a greater flexural capacity and will pass the truss code check by inspection.

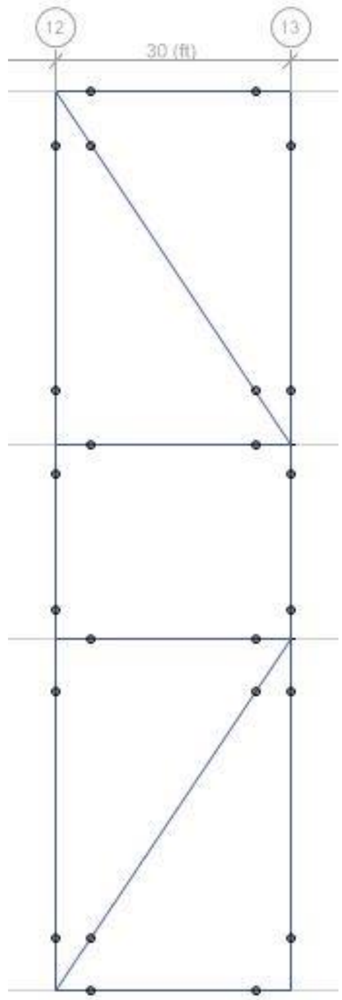


Figure 53: Roof Truss Model

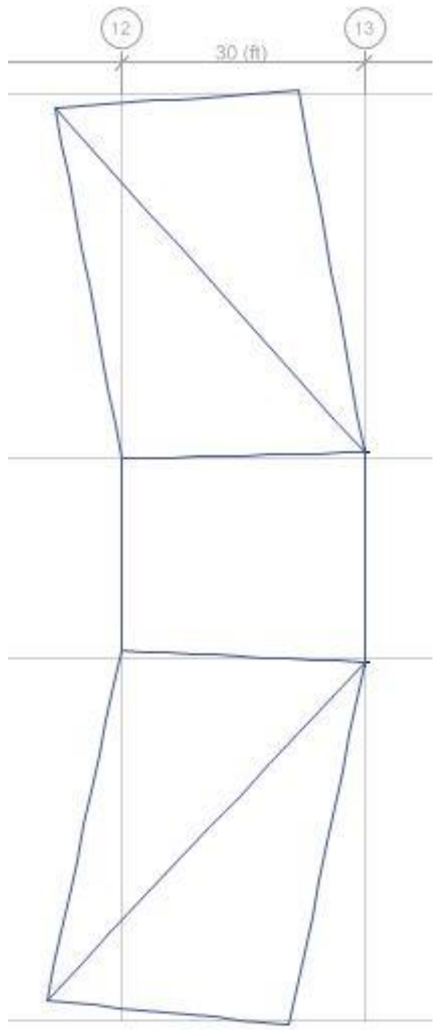


Figure 54: Roof Truss Deflected Shape

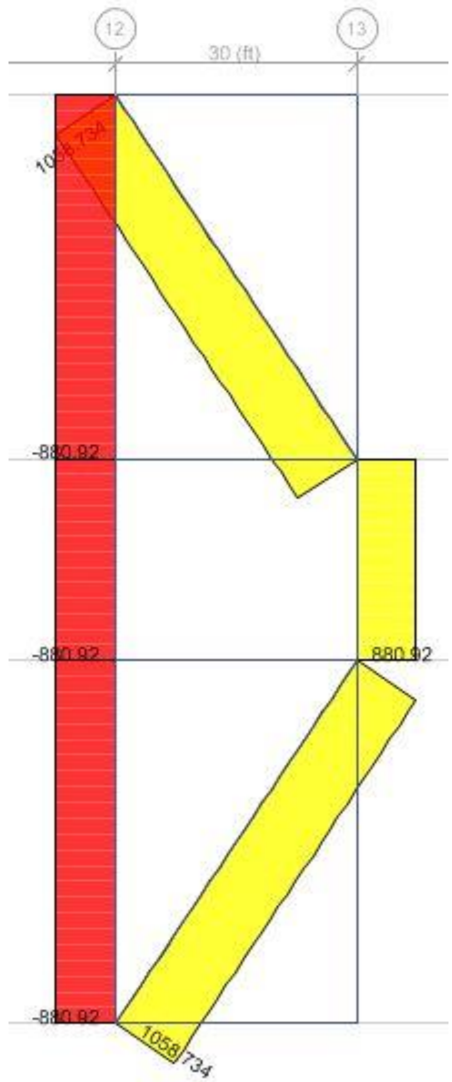


Figure 55: Roof Truss Axial Diagram

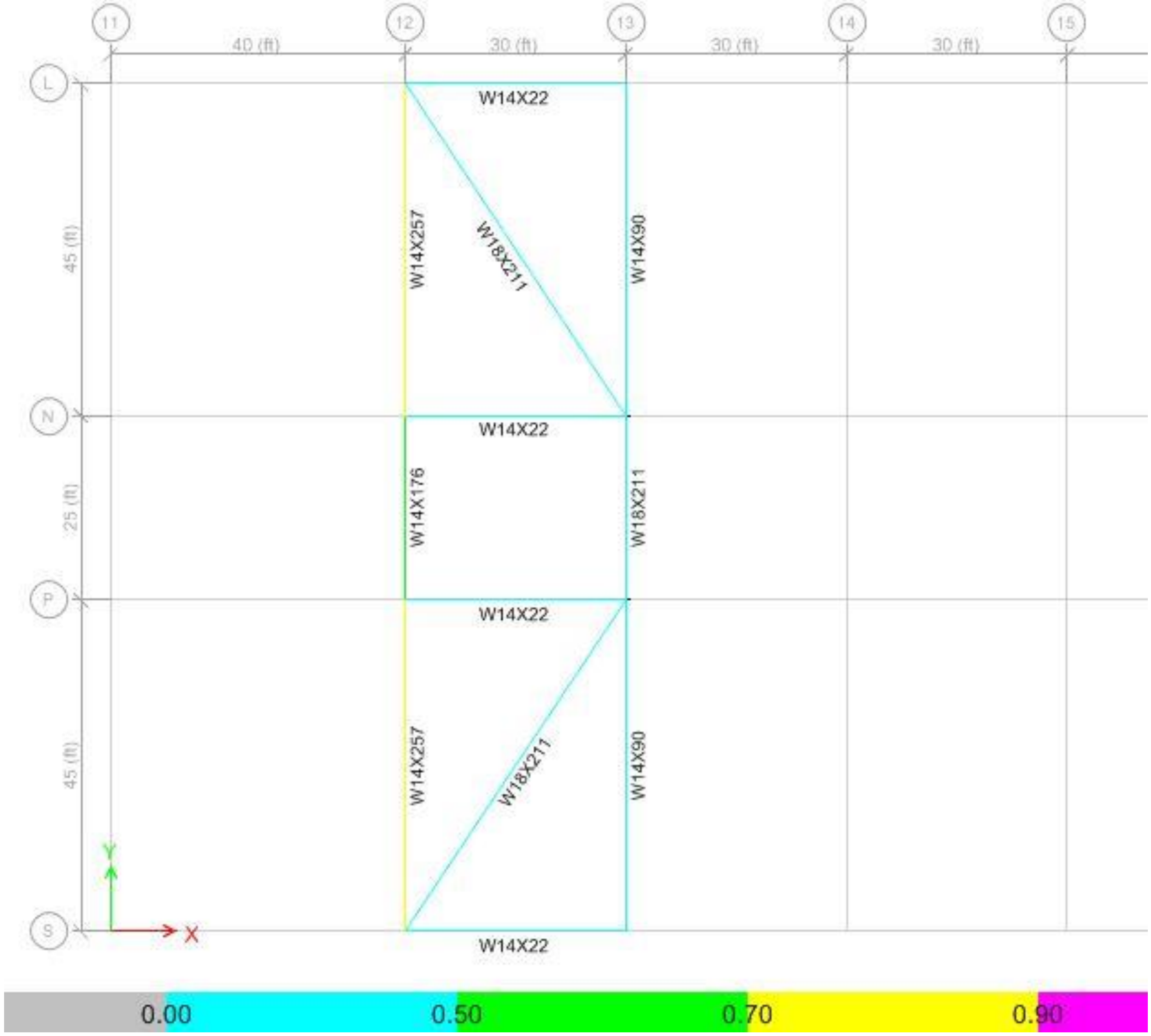


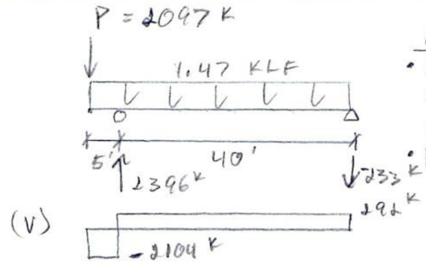
Figure 56: Roof Truss Code Check

Appendix H: Plate Girder Calculations and RISA Output

Begins on next page.

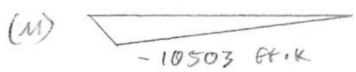
PL Girder Check 1/3

PLATE GIRDER CHECK



LOADS
- $1.2D + 1.6L = w$
  $w = 1.2(56\text{ PSF})(10') + 1.6(50)(10')$
  $w = 1.47$ KLF
- P from truss T2

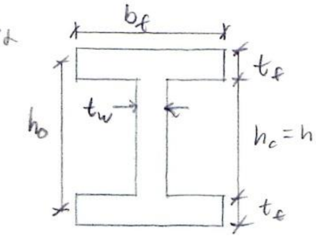
(V) From RISA



FRom RISA

- AS BUILT DIMENSIONS

ASTM 572 GR. 50



$h_o = 48''$
$b_f = 24''$
$t_w = 4''$
$h_c = h = 45''$
$t_f = 3''$

$E = 29000$ ksi
$F_y = 50$ ksi

- Check web slenderness

$$\lambda_w = \frac{h_c}{t_w} = \frac{45''}{4''} = 11.25$$

$$\lambda_p = 3.76\sqrt{\frac{E}{F_y}} = 3.76\sqrt{\frac{29000}{50}} = 90.6$$

$\lambda_w < \lambda_p$ ∴ AISC 360-10 Spec. Section F3

- Check unbraced length limit
  - Compression flange continuously braced by steel deck ∴ Not LTB, Compact web

- Check slenderness limits for flange local buckling

$$\lambda_f = \frac{b_f}{2t_f} = \frac{24''}{2 \cdot 3''} = 4$$

- From Table B4.1b Item 11:

$$\lambda_p = 0.38\sqrt{\frac{E}{F_y}} = 0.38\sqrt{\frac{29000}{50}} = 9.15$$

$$K_c = \frac{4}{\sqrt{h/t_w}} = \frac{4}{\sqrt{45/4}} = 1.19 > 0.76 \quad ∴ \text{ use } 0.76$$

For a symmetrical section $S_{xT}/S_{xc} = 1 > 0.7$ ∴

$F_L = 0.7F_y = 0.7(50) = 35$

$$\lambda_{rf} = 0.95\sqrt{\frac{K_c E}{F_L}} = 0.95\sqrt{\frac{0.76 \cdot 29000}{35}} = 23.8$$

$\lambda_f < \lambda_p$ ∴ compact flanges

PL Girder Check 2/3

Because compact section:

$\phi M_n = \phi M_p = \phi F_y Z_x$

$Z_x = 2(24)(3)(45/2 + 3/2) + 2(45/2)(4)(45/4) = 5481 \text{ in}^3$

$\phi M_n = 0.9(50)(5481)/12 = 20554 \text{ ft·k} > M_u = 10503$ OK

- Check shear (Non Tension Field Action)

$\lambda_{wv} = \frac{h}{t_w} = \frac{45}{4} = 11.25$

$\lambda_{wvp} = 1.1\sqrt{\frac{K_v E}{F_y}}$ → Assume unstiffened, $K_v = 5.0$

$= 1.1\sqrt{\frac{5 \cdot 29000}{50}} = 59.2$

$\lambda_{wv} < \lambda_{wvp} \therefore C_v = 1.0$

$V_n = 0.6 F_y A_w C_v = 0.6(50)(45 \cdot 4)(1) = 5400^k$

$\phi V_n = 0.9(5400) = 4860^k > V_u = 2104^k$ OK Unstiffened

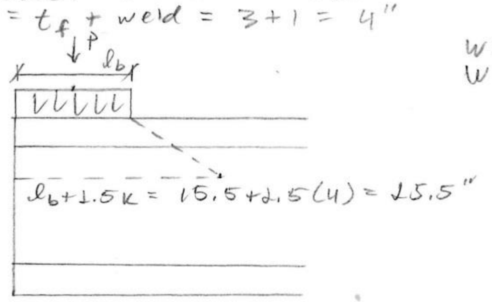
- Check Web Local Yielding Due to Point Load

S7.001 Gives 1" Weld

$k = t_f + \text{weld} = 3 + 1 = 4"$

W14x193; $l_b = 15.5"$

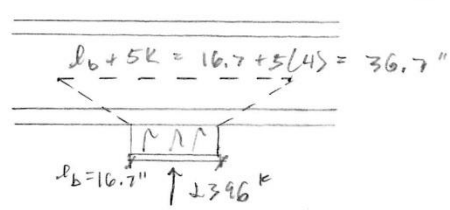
W14x283; $l_b = 16.7"$



$R_n = F_{yw} t_w (2.5k + l_b)$ $\phi = 1.0$

$= (50)(4)(25.5)(1.0) = \phi R_n = 5100^k$

$\phi R_n = 5100^k > P = 2097^k$ OK



Force applied @ 5' > 4' depth ∴

$R_n = F_{yw} t_w (5k + l_b)$ $\phi = 1.0$

$= (50)(4)(36.7)(1.0) = \phi R_n = 7340^k$

$\phi R_n = 7340^k > 2396^k$ OK

- Check web crippling

Point load @ < d/2 = 2'

$$\frac{l_b}{d} = \frac{15.5}{48} = 0.32 > 0.2$$

$$R_n = 0.40 t_w^2 \left[1 + \left(\frac{4 l_b}{d} - 0.2\right)\left(t_w/t_f\right)^{1.5}\right]\sqrt{\frac{E F_y t_f}{t_w}}$$

$$= 0.40 \cdot 4^2 \left[1 + \left(\frac{4 \cdot 15.5}{48} - 0.2\right)\left(4/3\right)^{1.5}\right]\sqrt{\frac{29000 \cdot 50 \cdot 3}{4}}$$

$$= 6.4(2.6807)(1042.8) = 17891^k$$

$$\phi R_n = 0.75(17891) = 13418^k > 2097^k \text{ OK}$$

Point load @ > d/2

$$R_n = 0.80 t_w^2 \left[1 + 3\left(\frac{l_b}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{E F_y \cancel{t_f}}{t_w}}$$

$$= 0.80 \cdot 4^2 \left[1 + 3\left(\frac{16.7}{48}\right)\left(\frac{4}{3}\right)^{1.5}\right]\sqrt{\frac{29000 \cdot 50 \cdot 3}{4}}$$

$$= 12.8(2.61)(1042.8) = 34838^k$$

$$\phi R_n = 0.75(34838) = 26128^k > 2396^k \text{ OK}$$

- Check deflection

- Service loads

$w = 56 \cdot 16 + 50 \cdot 16 = 1.06$ KLF

From ETABS: $P_{service} = 1554^k$

$$I = \frac{2 \cdot 24 \cdot 3^3}{12} + 2(24)(3)\left(\frac{45}{2} + \frac{3}{2}\right)^2 + \frac{4 \cdot 45^3}{12}$$

$I = 113427 \text{ in}^4$

$A = 2(24)(3) + 45(4) = 324 \text{ in}^2$

From RISA: $\Delta = -0.299" < 3/4"$ OK

3/4" limit for curtain wall attachment

∴ Current design adequate for increased loads. Additional load on this part of structure only additional 12% dead load and 18% live load.

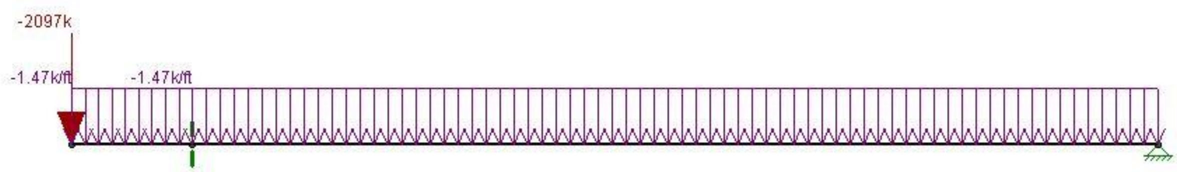


Figure 57: Plate Girder Loading

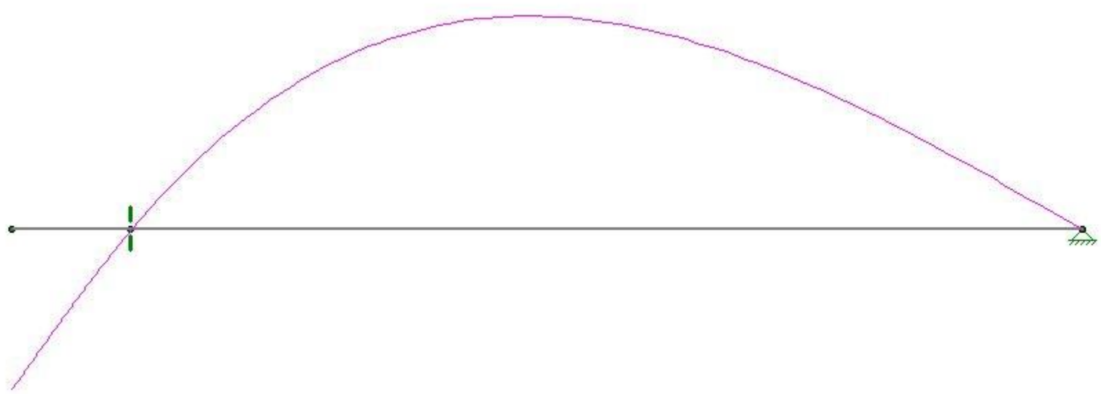


Figure 58: Plate Girder Deflected Shape

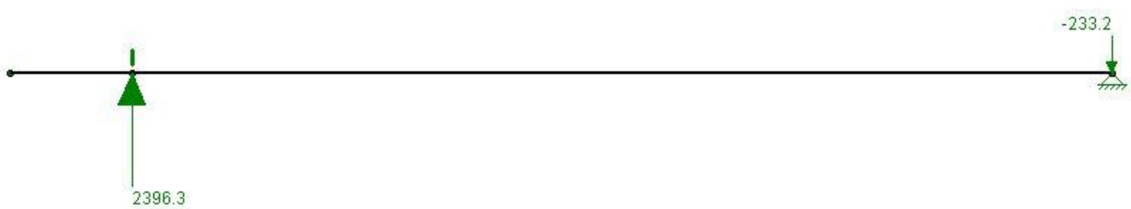


Figure 59: Plate Girder Reactions

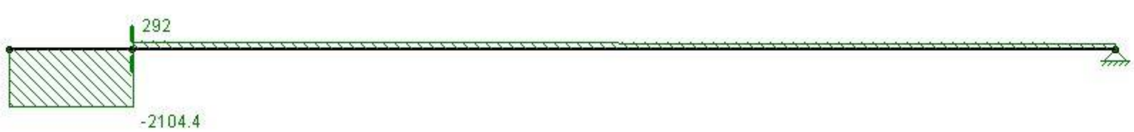


Figure 60: Plate Girder Shear Diagram

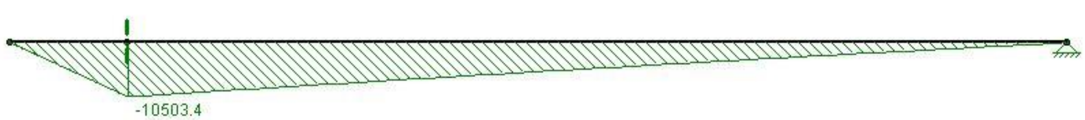


Figure 61: Plate Girder Moment Diagram

Appendix I: Seismic Loading Recalculations

COMBINED STORY WEIGHTS (k)						
	Parking Structure				Office	
Level	Walls	Columns	Slabs	Beams	Total	Total
B1	1286	431	5839	1412	0	8968
1	702	246	7201	3348	1881	13378
2	0	0	0	0	2521	2521
3	0	0	0	0	2527	2527
4	0	0	0	0	2527	2527
5	0	0	0	0	2531	2531
6	0	0	0	0	5421	5421
Penthouse	0	0	0	0	1543	1543

k_{office}=	1.03	
$k_{parking}$=	0.5	
V_{total}=	1552	k

SEISMIC STORY FORCES						
Level	w_x(k)	h_x(ft)	$w_xh_x^k$(ft-k)	C_{vx}	F_x(k)	M_{OT}(ft.-k)
B1	8968	11.2	29972	0.014	**21**	240
1	13378	26.0	Weight Lumped to Level 2			0
2	15899	41.3	733262	0.339	**526**	21680
3	2527	55.3	157491	0.073	**113**	6237
4	2527	69.3	198740	0.092	**142**	9864
5	2531	83.3	240549	0.111	**172**	14353
6	5421	97.7	607354	0.281	**435**	42509
PH Roof	1543	111.3	197690	0.091	**142**	15764
		$\Sigma w_xh_x^k$=	2165059	1	**1552**	**110647**

Seismic Overturning and Resisting Moments (ft.k.)		
Story	M_{resist}=wh	
Above B2	100173	
Above B1	347840	
Above 1	655838	
2	139632	
3	175014	
4	210665	
5	529361	
T.O. Steel	171631	
	2330154	**> 110647 OK**

SEISMIC LOAD CASE ECCENTRICITIES									
Level	Force(k)	XCM(ft.)	XCR(ft.)	ex (ft.)	5%Bx(ft.)	YCM(ft.)	YCR(ft.)	ey(ft.)	5%By(ft.)
6	577	130	134.6	4.59	5.750	57.5	57.5	0	14.5
5	172	127.5	137.1	9.61	5.750	57.5	57.5	0	14.5
4	142	130	140.3	10.34	5.750	57.5	57.5	0	14.5
3	113	127.5	142.9	15.45	5.750	57.5	57.5	0	14.5
2	526	130	144.9	14.93	5.750	57.5	57.5	0	14.5
B1	21	213.9734	330.1601	116.19	17.500	-108.5944	-103.6872	-4.9072	18.55
Sum=	1552								

Mtax=Fx(ey+5%Bx)
Mtay=Fx(ex+5%By)

100+30			
EX	30%EY	30%EX	EY
21	6.44	6.44	21

SEISMIC LOAD CASES														
	Case 1			Case 2			Case 3				Case 4			
Level	EX	Mtax(ft-k)+	Mtax(ft-k)-	EY	Mtay+	Mtay-	EX	Mtax+	Mtax-	EY	EY	Mtay+	Mtay-	EX
6	577	3318	-3318	577	11017	-5716	577	5968	-667	577	577	5716	-11017	577
5	172	991	-991	172	4158	-843	172	2649	666	172	172	843	-4158	172
4	142	819	-819	142	3538	-593	142	2292	654	142	142	593	-3538	142
3	113	649	-649	113	3381	107	113	2393	1095	113	113	-107	-3381	113
2	526	3022	-3022	526	15470	228	526	10871	4827	526	526	-228	-15470	526
B1	21	271	-481	21	2895	2098	21	2977	2225	21	21	-2203	-3000	21

Appendix J: BRBF Calculations and ETABS Output

Brace Axial Force from ETABS Analysis for BRB Sizing														
Level 6	Axial (k)	Case	Level 5	Axial (k)	Case	Level 4	Axial (k)	Case	Level 3	Axial (k)	Case	Level 2	Axial (k)	Case
D1	207	SeismicX	D1	266	SeismicX	D1	311	SeismicX	D1	345	SeismicX	D1	549	SeismicX
	-563	DStlS17		-621	DStlS17		-664	DStlS17		-696	DStlS17		-923	DStlS17
D2	559	DStlS17	D2	616	DStlS17	D2	660	DStlS17	D2	697	DStlS17	D2	923	DStlS17
	-206	SeismicX		-265	SeismicX		-309	SeismicX		-346	SeismicX		-549	SeismicX
D3	386	SeismicY-5%X	D3	448	SeismicY-5%X	D3	481	SeismicY-5%X	D3	440	SeismicY-5%X	D3	580	SeismicY-5%X
	-742	DStlS31		-804	DStlS31		-834	DStlS31		-791	DStlS31		-953	DStlS31
D4	736	DStlS31	D4	809	DStlS31	D4	836	DStlS31	D4	812	DStlS31	D4	972	DStlS31
	-383	SeismicY-5%X		-457	SeismicY-5%X		-485	SeismicY-5%X		-461	SeismicY-5%X		-598	SeismicY-5%X
D5	329	DStlS30	D7	352	DStlS55	D5	588	SeismicY-5%X	D7	397	DStlS55	D5	1257	SeismicY-5%X
	-329	DStlS31		-484	DStlS22		-680	DStlS31		-845	DStlS22		-1556	DStlS31
D6	329	DStlS31	D8	485	SeismicY-5%X	D6	438	DStlS55	D8	837	SeismicY-5%X	D6	735	DStlS55
	-329	DStlS30		-575	DStlS31		-572	DStlS22		-1142	DStlS31		-1174	DStlS22
D9	196	DStlS18	D11	247	DStlS19	D9	299	DStlS18,34,50	D11	324	DStlS19,35,51	D9	567	DStlS18,34,50
	-196	DStlS19		-247	DStlS18		-299	DStlS19,35,51		-324	DStlS18,34,50		-567	DStlS19,35,51
D10	196	DStlS19	D12	247	DStlS18	D10	299	DStlS19,35,51	D12	324	DStlS18,34,50	D10	567	DStlS19,35,51
	-196	DStlS18		-247	DStlS19		-299	DStlS18,34,50		-324	DStlS19,35,51		-567	DStlS18,34,50

Load Combination Key	
DStlS17	1.28D+L+0.2S-1.0E
DStlS31	1.28D+L+0.2S-1.0E
DStlS22	1.28D+L+0.2S+1.0E
DStlS18,34,50	1.28D+L+0.2S+1.0E
DStlS19,35,51	1.28D+L+0.2S-1.0E
DStlS22	1.28D+L+0.2S+1.0E

STEEL BEAM SUMMARY					
Label	Story	Section	PMM Controlling Ratio	PMM Combo	Class
B2	Level 06	W24X76	0.016 = 0 + 0.016 + 0	DStlS22	Seismic
B3	Level 06	W24X76	0.046 = 0 + 0.046 + 0	DStlS22	Seismic
B1	Level 06	W24X76	0.021 = 0 + 0.021 + 0	DStlS31	Seismic
B4	Level 06	W24X76	0.064 = 0 + 0.064 + 0	DStlS31	Seismic
B6	Level 06	W12X22	0.106 = 0 + 0.106 + 0	DStlS30	Seismic
B7	Level 06	W12X22	0.106 = 0 + 0.106 + 0	DStlS31	Seismic
B9	Level 06	W12X22	0.057 = 0 + 0.057 + 0	DStlS51	Seismic
B10	Level 06	W12X22	0.057 = 0 + 0.057 + 0	DStlS51	Seismic
B2	Level 05	W24X68	0.086 = 0 + 0.086 + 0	DStlS22	Compact
B3	Level 05	W24X68	0.067 = 0 + 0.067 + 0	DStlS30	Compact
B1	Level 05	W24X68	0.104 = 0 + 0.104 + 0	DStlS31	Compact
B4	Level 05	W24X68	0.07 = 0 + 0.07 + 0	DStlS31	Compact
B5	Level 05	W21X68	0.093 = 0 + 0.093 + 0	DStlS31	Seismic
B8	Level 05	W21X68	0.048 = 0 + 0.048 + 0	DStlS51	Seismic
B2	Level 04	W24X76	0.058 = 0 + 0.058 + 0	DStlS22	Seismic
B3	Level 04	W24X76	0.067 = 0 + 0.067 + 0	DStlS17	Seismic
B1	Level 04	W24X76	0.086 = 0 + 0.086 + 0	DStlS31	Seismic
B4	Level 04	W24X76	0.075 = 0 + 0.075 + 0	DStlS31	Seismic
B6	Level 04	W21X68	0.091 = 0 + 0.091 + 0	DStlS30	Seismic
B7	Level 04	W21X68	0.095 = 0 + 0.095 + 0	DStlS31	Seismic
B9	Level 04	W21X68	0.041 = 0 + 0.041 + 0	DStlS35	Seismic
B10	Level 04	W21X68	0.041 = 0 + 0.041 + 0	DStlS35	Seismic
B2	Level 03	W24X76	0.124 = 0 + 0.124 + 0	DStlS22	Seismic
B3	Level 03	W24X76	0.101 = 0 + 0.101 + 0	DStlS31	Seismic
B1	Level 03	W24X76	0.177 = 0 + 0.177 + 0	DStlS31	Seismic
B4	Level 03	W24X76	0.083 = 0 + 0.083 + 0	DStlS17	Seismic
B5	Level 03	W21X68	0.098 = 0 + 0.098 + 0	DStlS31	Seismic
B8	Level 03	W21X68	0.047 = 0 + 0.047 + 0	DStlS35	Seismic
B2	Level 02	W24X84	0.075 = 0 + 0.075 + 0	DStlS22	Seismic
B3	Level 02	W24X84	0.098 = 0 + 0.098 + 0	DStlS31	Seismic
B1	Level 02	W24X84	0.118 = 0 + 0.118 + 0	DStlS31	Seismic
B4	Level 02	W24X84	0.081 = 0 + 0.081 + 0	DStlS31	Seismic
B6	Level 02	W21X68	0.095 = 0 + 0.095 + 0	DStlS22	Seismic
B7	Level 02	W21X68	0.123 = 0 + 0.123 + 0	DStlS31	Seismic
B9	Level 02	W21X68	0.04 = 0 + 0.04 + 0	DStlS19	Seismic
B10	Level 02	W21X68	0.04 = 0 + 0.04 + 0	DStlS19	Seismic

STEEL COLUMN SUMMARY				
Story	Section	PMM Controlling Ratio	PMM Combo	Class
Level 06	W14X74	0.14 = 2.254E-04 + 0.059 + 0.081	DStlS22	Seismic
Level 06	W14X74	0.074 = 0.001 + 0.058 + 0.015	DStlS25	Seismic
Level 06	W14X74	0.172 = 1.784E-04 + 0.063 +	DStlS31	Seismic
Level 06	W14X74	0.088 = 0.002 + 0.068 + 0.018	DStlS31	Seismic
Level 06	W14X74	0.051 = 3.209E-04 + 0.031 + 0.02	DStlS29	Seismic
Level 06	W14X74	0.051 = 3.618E-04 + 0.035 +	DStlS18	Seismic
Level 05	W14X74	0.927 = 0.831 + 0.073 + 0.023	DStlS31	Seismic
Level 05	W14X74	0.499 = 0.444 + 0.026 + 0.029	DStlS22	Seismic
Level 05	W14X74	0.767 = 0.692 + 0.069 + 0.006	DStlS22	Seismic
Level 05	W14X74	0.68 = 0.605 + 0.044 + 0.03	DStlS31	Seismic
Level 05	W14X74	0.466 = 0.4 + 0.052 + 0.014	DStlS19	Seismic
Level 05	W14X74	0.466 = 0.4 + 0.052 + 0.014	DStlS18	Seismic
Level 04	W14X21	0.397 = 0.362 + 0.017 + 0.018	DStlS31	Seismic
Level 04	W14X21	0.32 = 0.297 + 0.023 + 3.721E-04	DStlS17	Seismic
Level 04	W14X21	0.304 = 0.26 + 0.002 + 0.042	DStlS17	Seismic
Level 04	W14X21	0.437 = 0.416 + 0.021 + 0.001	DStlS31	Seismic
Level 04	W14X21	0.067 = 0.06 + 0.006 + 0.001	DStlS19	Seismic
Level 04	W14X21	0.067 = 0.06 + 0.006 + 0.001	DStlS18	Seismic
Level 03	W14X21	0.796 = 0.717 + 0.046 + 0.033	DStlS31	Seismic
Level 03	W14X21	0.511 = 0.461 + 0.048 + 0.002	DStlS17	Seismic
Level 03	W14X21	0.607 = 0.556 + 0.046 + 0.005	DStlS22	Seismic
Level 03	W14X21	0.695 = 0.629 + 0.039 + 0.028	DStlS31	Seismic
Level 03	W14X21	0.34 = 0.288 + 0.037 + 0.015	DStlS19	Seismic
Level 03	W14X21	0.34 = 0.288 + 0.037 + 0.015	DStlS18	Seismic
Level 02	W14X21	0.941 = 0.903 + 0.026 + 0.012	DStlS31	Seismic
Level 02	W14X21	0.647 = 0.633 + 0.013 + 0.001	DStlS17	Seismic
Level 02	W14X21	0.689 = 0.647 + 0.024 + 0.018	DStlS22	Seismic
Level 02	W14X21	0.867 = 0.828 + 0.021 + 0.018	DStlS31	Seismic
Level 02	W14X21	0.327 = 0.295 + 0.019 + 0.013	DStlS19	Seismic
Level 02	W14X21	0.327 = 0.295 + 0.019 + 0.013	DStlS18	Seismic

STEEL BRACE SUMMARY					
Label	Story	Section	PMM Controlling Ratio	PMM Combo	Class
D1	Level 06	STARBRB-23.5	0.65 = 0.65 + 0 + 0	DStlS22	Non-Compact
D2	Level 06	STARBRB-23.5	0.644 = 0.644 + 0 + 0	DStlS22	Non-Compact
D3	Level 06	STARBRB-23.5	0.886 = 0.886 + 0 + 0	DStlS31	Non-Compact
D4	Level 06	STARBRB-23.5	0.877 = 0.877 + 0 + 0	DStlS31	Non-Compact
D5	Level 06	STARBRB-10.0	0.83 = 0.83 + 0 + 0	DStlS31	Non-Compact
D6	Level 06	STARBRB-10.0	0.83 = 0.83 + 0 + 0	DStlS30	Non-Compact
D9	Level 06	STARBRB-10.0	0.442 = 0.442 + 0 + 0	DStlS51	Non-Compact
D10	Level 06	STARBRB-10.0	0.442 = 0.442 + 0 + 0	DStlS50	Non-Compact
D1	Level 05	STARBRB-23.5	0.644 = 0.644 + 0 + 0	DStlS17	Non-Compact
D2	Level 05	STARBRB-23.5	0.639 = 0.639 + 0 + 0	DStlS17	Non-Compact
D3	Level 05	STARBRB-23.5	0.84 = 0.84 + 0 + 0	DStlS31	Non-Compact
D4	Level 05	STARBRB-23.5	0.848 = 0.848 + 0 + 0	DStlS31	Non-Compact
D7	Level 05	STARBRB-21.5	0.63 = 0.63 + 0 + 0	DStlS22	Non-Compact
D8	Level 05	STARBRB-21.5	0.758 = 0.758 + 0 + 0	DStlS31	Non-Compact
D11	Level 05	STARBRB-21.5	0.246 = 0.246 + 0 + 0	DStlS50	Non-Compact
D12	Level 05	STARBRB-21.5	0.246 = 0.246 + 0 + 0	DStlS51	Non-Compact
D1	Level 04	STARBRB-25.5	0.635 = 0.635 + 0 + 0	DStlS17	Non-Compact
D2	Level 04	STARBRB-25.5	0.631 = 0.631 + 0 + 0	DStlS17	Non-Compact
D3	Level 04	STARBRB-25.5	0.827 = 0.827 + 0 + 0	DStlS31	Non-Compact
D4	Level 04	STARBRB-25.5	0.827 = 0.827 + 0 + 0	DStlS31	Non-Compact
D5	Level 04	STARBRB-24.5	0.761 = 0.761 + 0 + 0	DStlS31	Non-Compact
D6	Level 04	STARBRB-24.5	0.634 = 0.634 + 0 + 0	DStlS22	Non-Compact
D9	Level 04	STARBRB-24.5	0.27 = 0.27 + 0 + 0	DStlS19	Non-Compact
D10	Level 04	STARBRB-24.5	0.27 = 0.27 + 0 + 0	DStlS50	Non-Compact
D1	Level 03	STARBRB-22.5	0.751 = 0.751 + 0 + 0	DStlS17	Non-Compact
D2	Level 03	STARBRB-22.5	0.752 = 0.752 + 0 + 0	DStlS17	Non-Compact
D3	Level 03	STARBRB-22.5	0.859 = 0.859 + 0 + 0	DStlS31	Non-Compact
D4	Level 03	STARBRB-22.5	0.878 = 0.878 + 0 + 0	DStlS31	Non-Compact
D7	Level 03	STARBRB-36.0	0.625 = 0.625 + 0 + 0	DStlS22	Non-Compact
D8	Level 03	STARBRB-36.0	0.849 = 0.849 + 0 + 0	DStlS31	Non-Compact
D11	Level 03	STARBRB-36.0	0.099 = 0.099 + 0 + 0	DStlS50	Non-Compact
D12	Level 03	STARBRB-36.0	0.099 = 0.099 + 0 + 0	DStlS19	Non-Compact
D1	Level 02	STARBRB-30.0	0.748 = 0.748 + 0 + 0	DStlS17	Non-Compact
D2	Level 02	STARBRB-30.0	0.746 = 0.746 + 0 + 0	DStlS17	Non-Compact
D3	Level 02	STARBRB-30.0	0.777 = 0.777 + 0 + 0	DStlS31	Non-Compact
D4	Level 02	STARBRB-30.0	0.788 = 0.788 + 0 + 0	DStlS31	Non-Compact
D5	Level 02	STARBRB-48.0	0.846 = 0.846 + 0 + 0	DStlS31	Non-Compact
D6	Level 02	STARBRB-48.0	0.636 = 0.636 + 0 + 0	DStlS22	Non-Compact
D9	Level 02	STARBRB-48.0	0.271 = 0.271 + 0 + 0	DStlS51	Non-Compact
D10	Level 02	STARBRB-48.0	0.271 = 0.271 + 0 + 0	DStlS50	Non-Compact

Buckling-Restrained Brace Frames 5 and 6

All of the following diagrams for BRBF 5 and 6 are representative of the controlling load combination 1.28D+L+0.2S+1.0E.

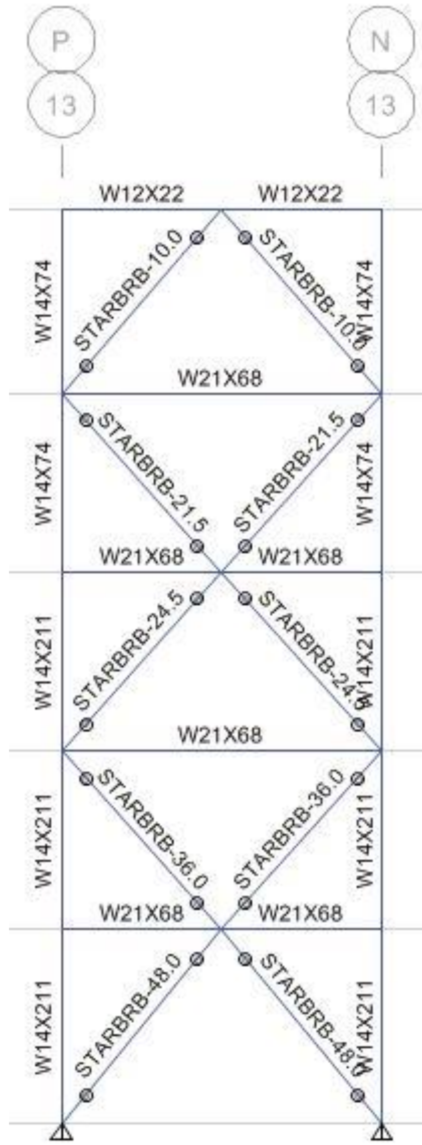


Figure 62: BRBF 5 and 6 Model

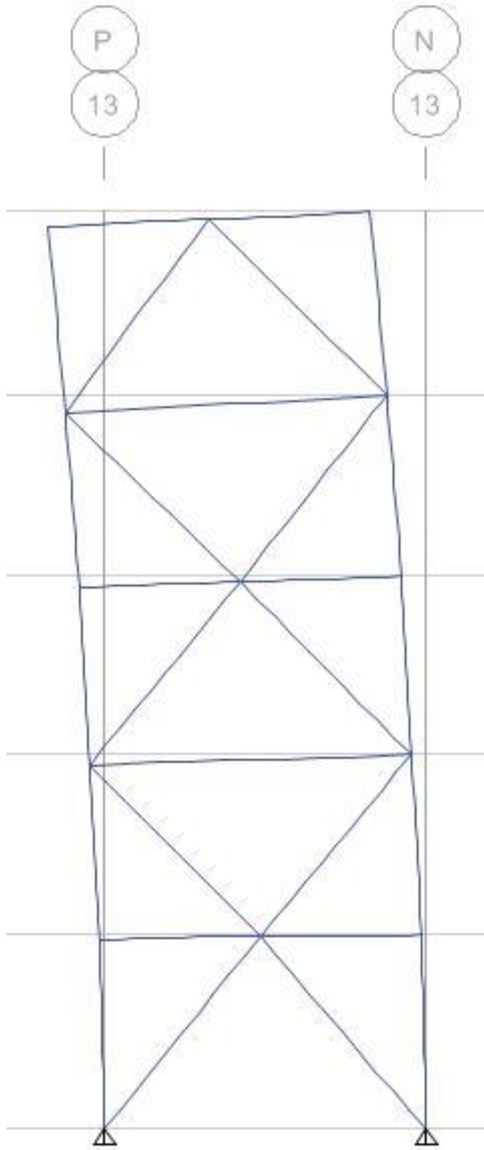


Figure 63: BRBF 5 and 6 Deflected Shape

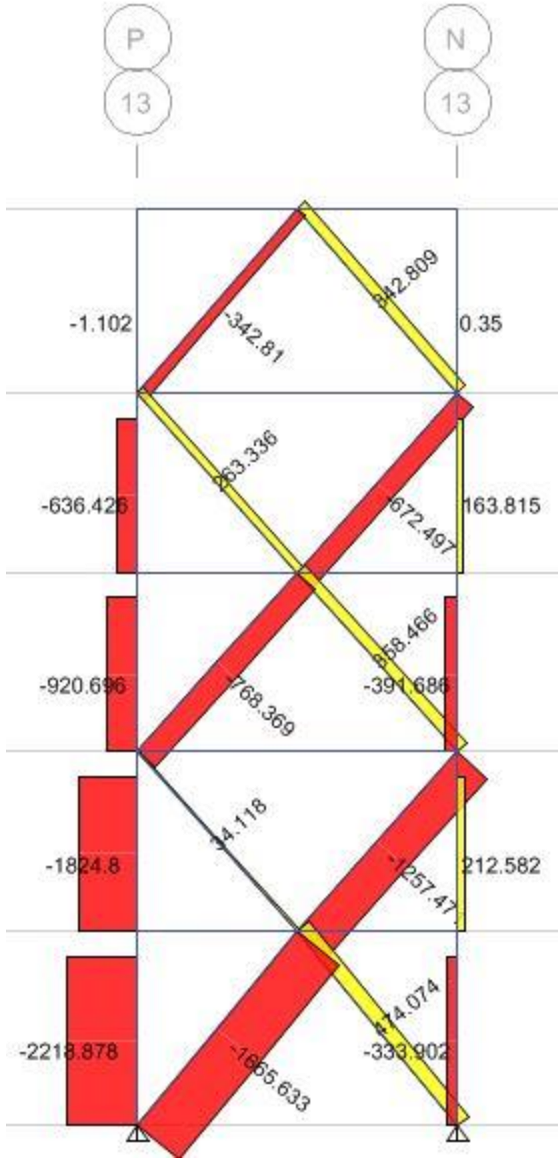


Figure 64: BRBF 5 and 6 Axial Force Diagram

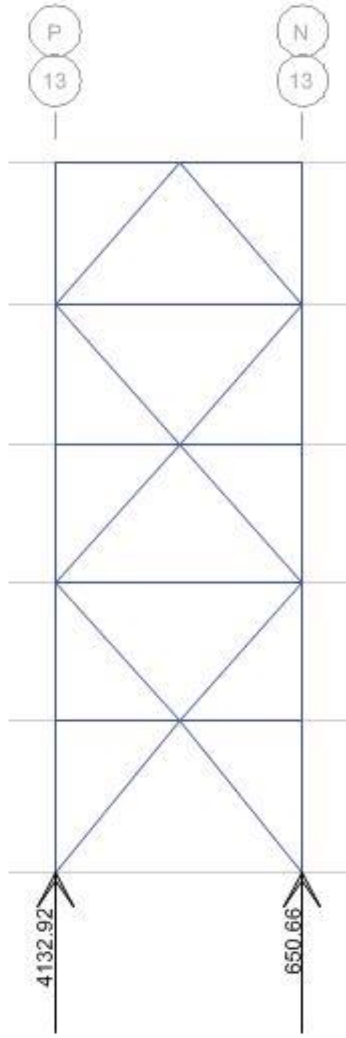


Figure 65: BRBF 5 and 6 Vertical Reactions under Controlling Load Combination

Buckling-Restrained Brace Frames 7 and 8

All of the following diagrams for BRBF 5 and 6 are representative of the controlling load combination 1.28D+L+0.2S+1.0E.

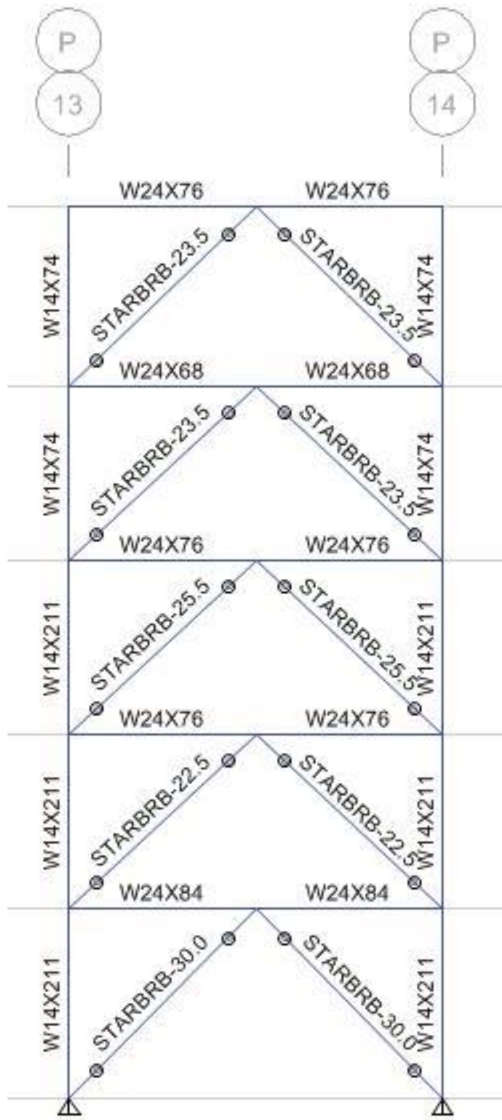


Figure 66: BRBF 7 and 8 ETABS Model

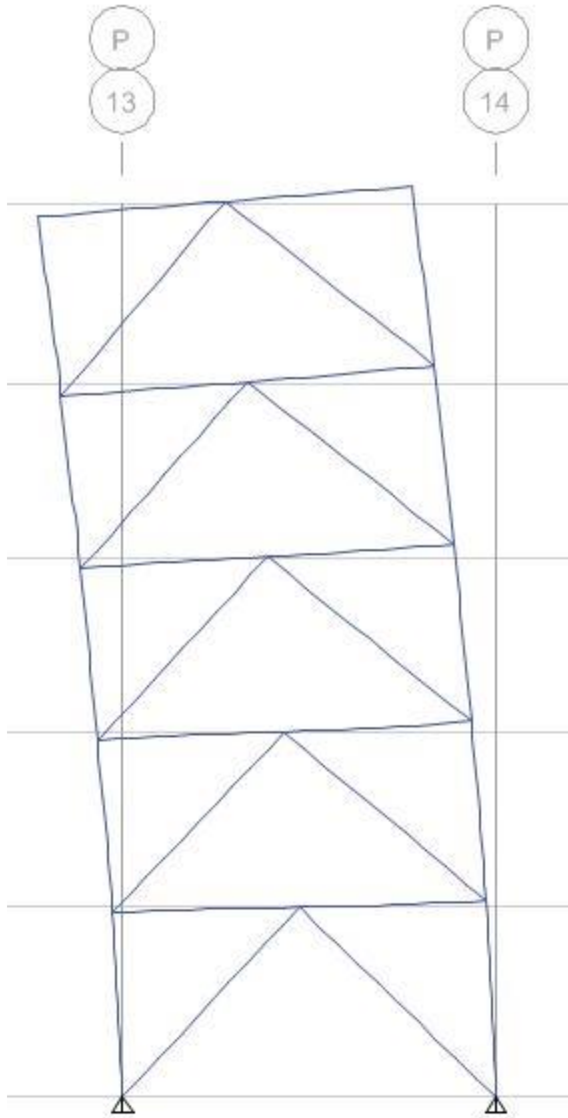


Figure 67: BRBF 7 and 8 Deflected Shape

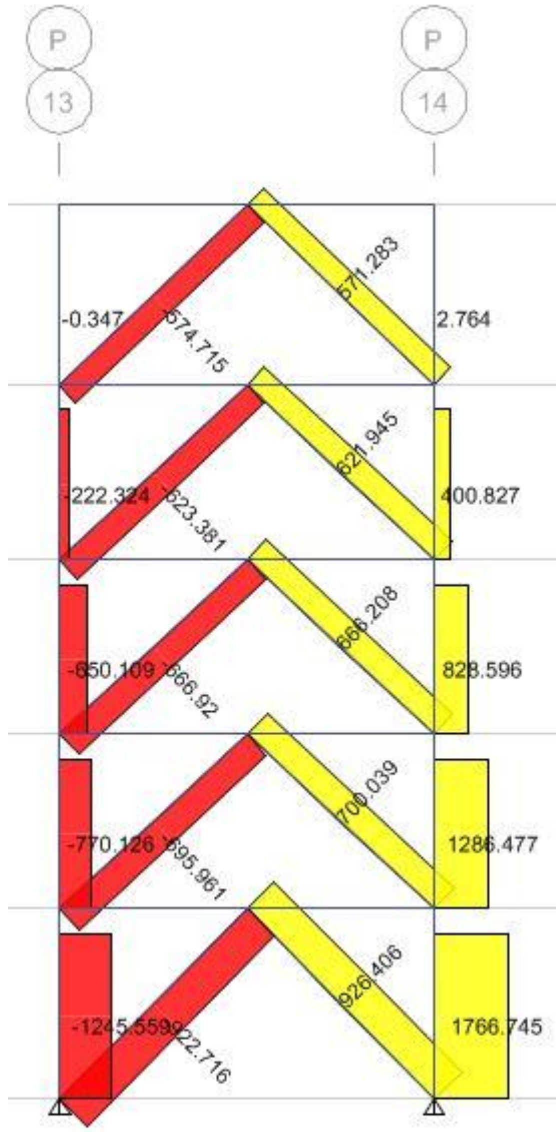


Figure 68: BRBF 7 and 8 Axial Diagram

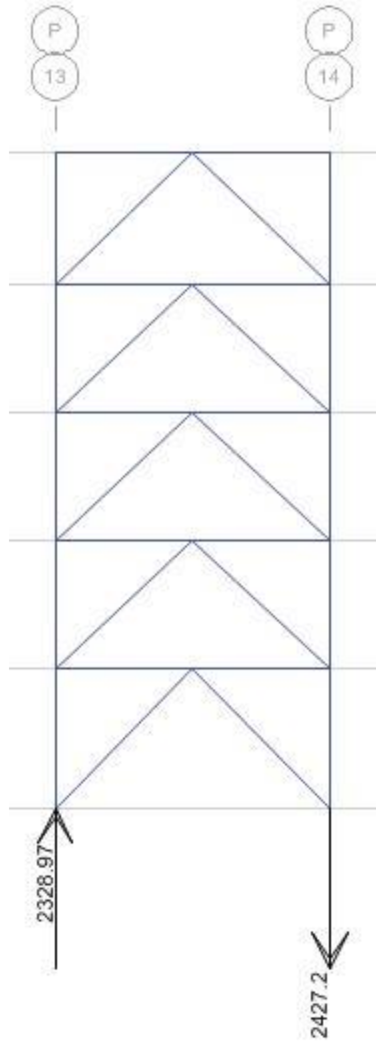


Figure 69: BRBF 7 and 8 Vertical Reactions under Controlling Load Combination

Appendix K: Construction Breadth Calculations

As Built Project

Schedule Duration for As Built Truss Assemblies							
Assembly	Item	Units	Quantity	Crew	Daily Output	Labor Hours	Duration (Days)
Truss T1	W14X90	LF	113	E-2	740	0.076	0.152
Truss T1	W24X94	LF	115	E-2	1080	0.074	0.106
Truss T1	W27X129	LF	115	E-2	1150	0.07	0.100
Truss T1	W14X159	LF	144	E-2	700	0.08	0.206
Truss T2	W14X120	LF	113	E-2	720	0.078	0.157
Truss T2	W21X73	LF	280	E-2	1024	0.078	0.273
Truss T2	W14X176	LF	303	E-2	683	0.082	0.443
Truss T2	W14X193	LF	113	E-2	671	0.083	0.168
Truss T2	W27X94	LF	200	E-2	1190	0.067	0.168
						Total=	**1.77**

Schedule Duration for As Built Roof System										
Assembly	Item	Units	Quantity	Crew	Daily Output	Labor Hours	Duration (Days)	# Crews	New Daily Output	New Duration
Deck & Insul.	3N20	SF	28450	E-4	3600	0.009	7.903	5	18000	1.58
TPO Single Ply Sheet	Roofing	Sq	285	G-5	25	1.6	11.400	5	125	2.28
Precast Conc. Unit Paving	Roof Pavers	SF	28450	2 Bric	250	0.064	113.800	18	4500	6.32
						Total=	**133**		**Total=**	**10.18**

Time Factor Calculation			
Month/Year	BCI	Modifier	
March '14	5335.54	1.074	> 1 OK
Jan. '11	4968.61		
Assume 3% Inflation		1.03	
Time Factor=		**1.106**	

Schedule Duration for As Built Roof Framing							
Assembly	Item	Units	Quantity	Crew	Daily Output	Labor Hours	Duration (Days)
Roof Framing	W24X94	LF	115	E-2	1080	0.074	0.106
Roof Framing	W24X62	LF	30	E-2	1110	0.072	0.027
Roof Framing	W21X68	LF	90	E-2	1036	0.077	0.087
Roof Framing	W21X57	LF	60	E-2	1048	0.076	0.057
Roof Framing	W24X68	LF	220	E-2	1110	0.072	0.198
Roof Framing	W24X55	LF	200	E-2	1110	0.072	0.180
Roof Framing	W21X50	LF	165	E-2	1064	0.075	0.155
Roof Framing	W21X44	LF	2625	E-2	1064	0.075	2.467
Roof Framing	W18X46	LF	25	E-2	960	0.083	0.026
Roof Framing	W18X35	LF	25	E-2	960	0.083	0.026
Roof Framing	W16X26	LF	50	E-2	1000	0.056	0.050
Roof Framing	W14X34	LF	108	E-2	810	0.069	0.133
Roof Framing	W14X30	LF	25	E-2	900	0.062	0.028
Roof Framing	W14X22	LF	375	E-2	1080	0.052	0.347
Roof Framing	W12X19	LF	250	E-2	880	0.064	0.284
Roof Framing	W12X14	LF	160	E-2	880	0.064	0.182
Roof Framing	W16X45	LF	25	E-2	800	0.07	0.031
Roof Framing	C12X20.7	LF	310	E-2	600	0.093	0.517
Roof Framing	W21X48	LF	180	E-2	1064	0.075	0.169
Roof Framing	W8X10	LF	20	E-2	600	0.093	0.033
Roof Framing	W8X18	LF	280	E-2	600	0.093	0.467
						Total=	**5.57**

COST ESTIMATE FOR THE AS BUILT PROJECT FOR SOUTH OFFICE TOWER

Cost Code	Assembly	Item	Units	Quantity	Waste/Accessory	Mat'l Unit Cost	Labor Unit Cost	Equip. Unit Cost	Unit Total	O&P Unit Total	Total Cost
05 12 23.75 2380	Truss T1	W14X90	LF	113	1.1	124	3.65	2.02	129.67	144	17867.52
05 12 23.75 5720	Truss T1	W24X94	LF	115	1.1	129	3.61	1.5	134.11	150	18975
(Interpolate)	Truss T1	W27X129	LF	115	1.1	179	3.39	1.41	183.8	203.5	25742.75
(Interpolate)	Truss T1	W14X159	LF	144	1.1	206	3.85	2.14	211.99	238	37751.56
05 12 23.75 2500	Truss T2	W14X120	LF	113	1.1	165	3.75	2.08	170.83	191	23699.28
(Interpolate)	Truss T2	W21X73	LF	280	1.1	100	3.81	1.58	105.72	119	36652
(Interpolate)	Truss T2	W14X176	LF	303	1.1	242	3.94	2.19	247.66	279	92929.32
(Interpolate)	Truss T2	W14X193	LF	113	1.1	265	3.99	2.23	270.99	305	37844.4
05 12 23.75 5900	Truss T2	W27X94	LF	200	1.1	129	3.28	1.36	133.64	149	32780
05 12 23.75 5720	Roof Framing	W24X94	LF	115	1.1	129	3.61	1.5	134.11	150	18975
05 12 23.75 5100	Roof Framing	W24X62	LF	30	1.1	85.5	3.52	1.46	90.48	102	3366
05 12 23.75 4700	Roof Framing	W21X68	LF	90	1.1	93.5	3.77	1.56	98.83	111	10989
(Interpolate)	Roof Framing	W21X57	LF	60	1.1	78.63	3.73	1.54	83.9	94	6204
05 12 23.75 5300	Roof Framing	W24X68	LF	220	1.1	93.5	3.52	1.46	98.48	111	26862
05 12 23.75 4900	Roof Framing	W24X55	LF	200	1.1	75.5	3.52	1.46	80.48	90.5	19910
05 12 23.75 4300	Roof Framing	W21X50	LF	165	1.1	69	3.67	1.52	74.19	83.5	15155.25
05 12 23.75 4100	Roof Framing	W21X44	LF	2625	1.1	60.5	3.67	1.52	65.69	74.5	215118.75
05 12 23.75 3520	Roof Framing	W18X46	LF	25	1.1	63.5	4.07	1.69	69.26	78.5	2158.75
05 12 23.75 3300	Roof Framing	W18X35	LF	25	1.1	48	4.07	1.69	53.76	62	1705
05 12 23.75 2700	Roof Framing	W16X26	LF	50	1.1	36	2.7	1.5	40.2	46	2530
05 12 23.75 2300	Roof Framing	W14X34	LF	108	1.1	47	3.34	1.85	52.19	59	7009.2
05 12 23.75 2100	Roof Framing	W14X30	LF	25	1.1	41.5	3	1.66	46.16	52.5	1443.75
(Interpolate)	Roof Framing	W14X22	LF	375	1.1	30.5	2.46	1.36	34.32	39.5	16293.75
(Interpolate)	Roof Framing	W12X19	LF	250	1.1	26.25	3.07	1.7	31.02	35.75	9831.25
(Interpolate)	Roof Framing	W12X14	LF	160	1.1	19.17	3.07	1.7	23.94	27.83	4898.08
(Interpolate)	Roof Framing	W16X45	LF	25	1.1	62	3.38	1.87	67.25	76	2090
(Interpolate)	Roof Framing	C12X20.7	LF	310	1.1	12.4	4.5	2.49	19.39	24	8184
(Interpolate)	Roof Framing	W21X48	LF	180	1.1	66	3.67	1.52	71.36	81	16038
05 12 23.75 0300	Roof Framing	W8X10	LF	20	1.1	13.75	4.5	2.49	20.74	25.5	561
(Interpolate)	Roof Framing	W8X18	LF	280	1.1	24.75	4.5	2.49	31.74	37.75	11627
05 31 23.50 3350	Deck & Insul.	3N20	SF	28450	1.1	2.775	0.44	0.03	3.25	3.80	118764.525
07 54 23.10 0200	TPO Single Ply Sheet	Roofing	Sq	285	1.1	79	54.5	7.15	140.65	186	58311
32 14 13.16 0800	Precast Conc. Unit Paving	Roof Pavers	SF	28450	1	7.1	2.82	0	10.02	12.58	357901

Green Roof Garden

Schedule Duration for Green Roof Garden System										
Assembly	Item	Units	Quantity	Crew	Labor Hours	Daily Output	Duration (Days)	# Crews	New Daily Output	New Duration
Deck	3VLI19	SF	28450	E-4	0.011	2850	9.98	5	14250	2.00
Concrete Topping	ltwt, 3.5" Top	CY	439	-	-	-				
Concrete Formwork	4 use	SF	28450	C-2	0.086	560	50.80	10	5600	5.08
Concrete Placement	Elev., crane & bucket	CY	439	C-7	0.758	95	4.62	2	190	2.31
Concrete Finishing	Ride on screed...	SF	28450	C-10E	0.006	4000	7.11	5	20000	1.42
Welded Wire Fabric	6x6-W2.1xW2.1	CSF	28450	2 Rodm	0.516	31	917.74	50	1550	18.35
Expanded Polystyrene Insulation	6" Thick	SF	28450	1 Rofc	0.008	1000	28.45	10	10000	2.85
Waterproof Membrane	215 mil, reinf	SF	28450	G-5	0.114	350	81.29	20	7000	4.06
Root Barrier	-	SF	28450	2 Rofc	0.021	775	36.71	10	7750	3.67
Moisture Retention Barrier and Resevoir		SF	15672	2 Rofc	0.18	900	17.41	10	9000	1.74
Separation Fabric	-	SF	15672	2 Rofc	0.021	775	20.22	10	7750	2.02
M3 Growth and Drainage Media	10" Thick	SF	15672	B-13C	0.035	1600	9.80	2	3200	4.90
M3 Growth and Drainage Media	12" Thick	SF	15672	B-13C	0.042	1335	11.74	2	2670	5.87
Wind Blanket	-	SF	15672	2 Rofc	0.021	775	20.22	10	7750	2.02
55 ton crane mobilization	-	Ea.	1	1 Eqhv	2.222	3.6	0.28	1	3.6	0.28
Roof edging, treated lumber	4"x6"	LF	1278	2 Carp	0.04	400	3.20	5	2000	0.64
pedestal pavers		SF	6470	D-1	0.178	90	71.89	18	1620	3.99
planting sedum	-	SF	5974	1 Clab	0.019	420	14.22	10	4200	1.42
Planting Wildflower	Ajuga, 1 yr	C (100)	119	B-1	2.667	9	13.22	5	45	2.64
Planting Garden	Vinca Minor, 1 yr	C (100)	113	B-1	2.4	10	11.30	5	50	2.26
Fence	3 rail	LF	115	B-1	0.16	150	0.77	1	150	0.77
Fence	fence pole	Ea.	24	B-1	0.25	96	0.25	1	96	0.25
						Total=	1331.22		Total=	68.55

Schedule Duration for Green Roof Garden Truss Assemblies							
Assembly	Item	Units	Quantity	Crew	Labor Hours	Daily Output	Duration (Days)
Truss T1	W14X145	LF	113	E-2	0.08	703	0.161
Truss T1	W24X192	LF	115	E-2	0.076	1050	0.110
Truss T1	W27X146	LF	115	E-2	0.07	1150	0.100
Truss T1	W14X283	LF	144	E-2	0.089	611	0.236
Truss T2	W21X101	LF	80	E-2	0.08	1000	0.080
Truss T2	W14X311	LF	303	E-2	0.091	593	0.511
Truss T2	W14X193	LF	113	E-2	0.083	671	0.168
Truss T2	W27X281	LF	200	E-2	0.07	1150	0.174
						Total=	1.54

Schedule Duration for Green Roof Garden Roof Framing							
Assembly	Item	Units	Quantity	Crew	Labor Hours	Daily Output	Duration (Days)
Roof Framing	W27X129	LF	700	E-2	0.07	1150	0.609
Roof Framing	W24X76	LF	120	E-2	0.072	1110	0.108
Roof Framing	W24X62	LF	230	E-2	0.072	1110	0.207
Roof Framing	W21X55	LF	380	E-2	0.076	1050	0.362
Roof Framing	W27X84	LF	170	E-2	0.067	1190	0.143
Roof Framing	W24X68	LF	180	E-2	0.072	1110	0.162
Roof Framing	W24X55	LF	1890	E-2	0.072	1110	1.703
Roof Framing	W21X44	LF	240	E-2	0.075	1064	0.226
Roof Framing	W18X40	LF	45	E-2	0.083	960	0.047
Roof Framing	W18X35	LF	528	E-2	0.083	960	0.550
Roof Framing	W16X26	LF	110	E-2	0.056	1000	0.110
Roof Framing	W14X30	LF	25	E-2	0.062	900	0.028
Roof Framing	W12X22	LF	175	E-2	0.064	880	0.199
Roof Framing	W14X22	LF	325	E-2	0.057	990	0.328
Roof Framing	W12X14	LF	25	E-2	0.064	880	0.028
Roof Framing	W8X18	LF	280	E-2	0.093	600	0.467
Roof Framing	W18X46	LF	25	E-2	0.083	960	0.026
Roof Framing	W14X257	LF	90	E-2	0.087	629	0.143
Roof Framing	W18X211	LF	133.2	E-2	0.089	900	0.148
Roof Framing	W14X176	LF	25	E-2	0.082	683	0.037
						Total=	5.63

COST ESTIMATE FOR THE GREEN ROOF GARDEN FOR SOUTH OFFICE TOWER											
Cost Code	Assembly	Item	Units	Quantity	Waste/Accessory	Mat'l Unit Cost	Labor Unit Cost	Equip. Unit Cost	Unit Total	O&P Unit Total	Total Cost
(Extrapolate)	Truss T1	W14X145	LF	113	1.1	181	3.76	2.3	186.73	206	25605.8
(Extrapolate)	Truss T1	W24X192	LF	115	1.1	238	3.65	1.65	243.38	270	34155
05 12 23.75 5940	Truss T1	W27X146	LF	115	1.1	181	3.33	1.51	185.84	206	26059
(Extrapolate)	Truss T1	W14X283	LF	144	1.1	355	4.22	2.58	362.27	395	62568
05 12 23.75 4760	Truss T2	W21X101	LF	80	1.1	125	3.83	1.73	130.56	146	12848
(Extrapolate)	Truss T2	W14X311	LF	303	1.1	391	4.32	2.63	397.88	433	144223.64
(Extrapolate)	Truss T2	W14X193	LF	113	1.1	241	3.92	2.4	247.79	272	33749.76
(Extrapolate)	Truss T2	W27X281	LF	200	1.1	343	3.33	1.51	347.84	386	84920
(Extrapolate)	Roof Framing	W27X129	LF	700	1.1	161	3.33	1.51	165.84	184	141680
05 12 23.75 5500	Roof Framing	W24X76	LF	120	1.1	94	3.45	1.56	99.01	111	14652
05 12 23.75 5100	Roof Framing	W24X62	LF	230	1.1	76.5	3.45	1.56	81.51	92	23276
(Extrapolate)	Roof Framing	W21X55	LF	380	1.1	69.5	3.65	1.65	74.55	82.5	34485
05 12 23.75 5800	Roof Framing	W27X84	LF	170	1.1	104	3.22	1.45	108.67	121	22627
05 12 23.75 5300	Roof Framing	W24X68	LF	180	1.1	84	3.45	1.56	89.01	100	19800
05 12 23.75 4900	Roof Framing	W24X55	LF	1890	1.1	68	3.45	1.56	73.01	82.5	171517.5
05 12 23.75 4100	Roof Framing	W21X44	LF	240	1.1	54.5	3.6	1.63	59.73	68	17952
05 12 23.75 3500	Roof Framing	W18X40	LF	45	1.1	49.5	3.99	1.8	55.29	63.5	3143.25
05 12 23.75 3300	Roof Framing	W18X35	LF	528	1.1	43.5	3.99	1.8	49.29	56.5	32815.2
05 12 23.75 2700	Roof Framing	W16X26	LF	110	1.1	32	2.65	1.62	36.27	42	5082
05 12 23.75 2100	Roof Framing	W14X30	LF	25	1.1	37	2.95	1.8	41.75	48	1320
05 12 23.75 1300	Roof Framing	W12X22	LF	175	1.1	27	3.01	1.84	31.85	37	7122.5
05 12 23.75 1900	Roof Framing	W14X22	LF	325	1.1	32	2.68	1.64	36.32	42	15015
05 12 23.75 1100	Roof Framing	W12X14	LF	25	1.1	19.8	3.01	1.84	24.65	29	797.5
05 12 23.75 0350	Roof Framing	W8X18	LF	280	1.1	26	4.42	2.7	33.12	39	12012
6 12 23.75 3520	Roof Framing	W18X46	LF	25	1.1	57	3.99	1.8	62.79	71.5	1966.25
(Extrapolate)	Roof Framing	W14X257	LF	90	1.1	323	4.14	2.52	329.19	359	35541
(Extrapolate)	Roof Framing	W18X211	LF	133.2	1.1	262	4.26	1.92	268.43	295	43223.4
(Extrapolate)	Roof Framing	W14X176	LF	25	1.1	220	3.87	2.36	226.16	249	6847.5

COST ESTIMATE FOR THE GREEN ROOF GARDEN FOR SOUTH OFFICE TOWER											
Cost Code	Assembly	Item	Units	Quantity	Waste/Accessory	Mat'l Unit Cost	Labor Unit Cost	Equip. Unit Cost	Unit Total	O&P Unit Total	Total Cost
05 31 13.50 5900	Deck	3VLI19	SF	28450	1.1	2	0.55	0.04	2.59	3.21	100456.95
03 31 16.10 0820	Concrete Topping	ltwt, 3.5" Top	CY	439	1.1	141	0	0	141	155	74849.5
03 11 13.35 1150	Concrete Formwork	4 use	SF	28450	1.1	1.03	3.59	0	4.62	6.65	208111.75
03 31 05.70 1450	Concrete Placement	Elev., crane & bucket	CY	439	1	0	28	13.45	41.45	57.5	25242.5
03 35 29.30 0350	Concrete Finishing	Ride on screed...	SF	28450	1	0	0.23	0.06	0.29	0.4	11380
03 22 05.50 0200	Welded Wire Fabric	6x6-W2.1xW2.1	CSF	28450	1.1	18.9	25	0	43.9	61	1908995
07 22 16.10 1932	Expanded Polystyrene Insulation	6" Thick	SF	28450	1	1.52	0.29	0	1.81	2.16	61452
07 33 63.10 0560	Waterproof Membrane	215 mil, reinf	SF	28450	1	0.26	3.79	0.48	4.53	7.1	201995
07 33 63.10 0570	Root Barrier	-	SF	28450	1	0.7	0.75	0	1.45	2.03	57753.5
07 33 63.10 0580	Moisture Retention Barrier and Resevoir		SF	15672	1	2.7	0.65	0	3.35	4.05	63471.6
07 33 63.10 0570	Separation Fabric	-	SF	15672	1	0.7	0.75	0	1.45	2.03	31814.16
07 33 63.10 0385	M3 Growth and Drainage Media	10" Thick	SF	15672	1	6	1.3	1.03	2.93	3.78	59240.16
07 33 63.10 0390	M3 Growth and Drainage Media	12" Thick	SF	15672	1	0.72	1.56	1.24	3.52	4.52	70837.44
07 33 63.10 0570	Wind Blanket	-	SF	15672	1	0.7	0.75	0	1.45	2.03	31814.16
07 33 63.10 0350	55 ton crane mobilization	-	Ea.	1	1	0	103	0	103	154	154
07 33 63.10 0365	hoisting cost 6-10 stories/day	avg 21 picks/day	Day	14	1	0	2075	1650	3725	5000	70000
07 33 63.10 0410	Roof edging, treated lumber	4"x6"	LF	1278	1	2.46	1.72	0	4.18	5.35	6837.3
09 63 13.10 0590	pedestal pavers		SF	6470	1	5.55	6.95	0	12.5	16.5	106755
07 33 63.10 0600	planting sedum	-	SF	5974	1.1	4.5	0.65	0	5.15	5.95	39099.83
32 93 13.20 0100	Planting Wildflower	Ajuga, 1 yr	C (100)	119	1.1	130	93.5	0	223.5	286	37437.4
32 93 13.20 0800	Planting Garden	Vinca Minor, 1 yr	C (100)	113	1.1	110	84	0	194	250	31075
32 31 23.20 9018	Fence	3 rail	LF	115	1.05	6.25	5.6	0	11.85	15.5	1871.625
33 31 23.20 9030	Fence	fence pole	Ea.	24	1	17.75	8.75	0	26.5	33	792