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Image Courtesy RTKL

CORPORATE HEADQUARTERS

Great Lakes Region, U.S.A.

FINAL REPORT

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STRUCTURAL OPTION
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08 APRIL 2015

Abstract

Corporate Headquarters
Great Lakes Region, U.S.A.

General Information

Building Height: 83'
Number of Stories: 5
Size: 659, 554 SF
Cost: Withheld at the request of Owner
Dates of Construction: August 2014-Spring 2016
Project Delivery Type: Design-Bid-Build

Project Team

RTKL Ohio Corp	Architect, Structural Engineer, Mechanical Engineer, Electrical Engineer, Plumbing, Telecommunications
Mark G. Anderson Constultants, Inc	Project Management
Bialosky + Partners Architects	Supporting Architect
Neff and Associates	Civil Engineer
Mahan Rykiel Associates, Inc	Landscape Architect
Code Consultants, Inc	Fire Protection and Code Consultant
Michael Blades and Associates, LTD	Elevator Consultant
Keith Davis Group, LLC	Roof and Waterproofing Consultant

Architecture

The Corporate Headquarters was designed to mimic the architecture of the existing outdoor mall directly to the North of the site. The building's façade is broken up into several segments in order to mimic the classic storefront look of the outdoor mall. With its large windows, curtain wall, and brick façade, the Corporate Headquarters strives to serve as a model of Classic Modern American Architecture.

Sustainability

The building's primary sustainability feature is the central courtyard, which begins on the third floor. It features an intensive green roof, a seating area for building occupants, and trees to help provide shade.

Structural

Foundation:	spread footings and grade beams, some of which are supported by aggregate piers
Framing:	steel framing, featuring w shapes for most beams, girders, and columns
Lateral:	8 braced frames near the core of the building

Lighting/Electrical

Integrated Power Center: housed on first floor of building
More Information and Electrical Drawings Requested from Architect.

Mechanical

Variable Air Volume system
14 Rooftop Air Handling Units providing up to 37,500 CFM
CRAC and Split Systems utilized in other areas of building.

Renderings Courtesy RTKL

M. Julia Haverty | Structural Option | Advisor Heather Sustersic

Acknowledgements

I would like to thank the following people for all their help their help and support this past year:

- The AE Faculty, in particular my advisor Heather Sustersic, for helping me through any obstacle I came across this year.
- The engineers at WJE Cleveland for all their time in helping me to secure permission to use the Corporate Headquarters as my senior thesis building, especially Mike Kotheimer, who helped me to obtain the necessary plans and information needed to complete the project.
- The project team at RTKL Ohio Corp for generously agreeing to let me use the Corporate Headquarters for my thesis.
- My fellow AE 2015 classmates, in particular the Power Players. Thank you for filling the past five years with memories I'll never forget.
- The rest of my friends and family, especially my Mom, for always supporting me and believing in me.
- French Vanilla coffee and Coke Zero for picking me up whenever I'm feeling down.

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Executive Summary

The Corporate Headquarters, located in the Great Lakes Region of the United States, is a new five story office and retail space designed to serve as new home base for an established and successful US based company. The building will serve as a focal point for the south entrance of an existing retail park. The building's existing structural system is composed of W-shape steel beams, girders, and columns. The composite beams and girders, along with the concrete on metal floor deck, make up the building's gravity system. The Corporate Headquarters relies on eight braced frames as its lateral force resisting system. Within the building lies an open air courtyard featuring an intensive green roof garden.

Purpose and Scope

The purpose of this report is to examine and investigate an alternate structural system for the Corporate Headquarters. Though the existing structural system was adequate to fit the building's needs, a scenario was developed in which the courtyard green roof's geometry and composition were changed in order to help increase office space and to aid in the design process. To accommodate this change, the building's gravity system was redesigned using long span steel joists and joist girders. The columns remained as w-shapes but were resized in accordance with the new loads.

The changes in the gravity system resulted in a lower total building weight, which required the building's seismic loads to be recalculated. Once these loads were determined, it was found that wind controls over seismic. The building's lateral system was redesigned with reinforced concrete shear walls taking the place of the existing steel moment frames. The new shear walls were placed in the same locations as the existing steel braced frames in order to maximize floor space and to maintain the integrity of the existing architectural design, which put walls on either side of the braces.

A green roof redesign was completed to help lower the dead loads on the building. The tree area was removed and the entire intensive green roof courtyard was redesigned with grass, garden, and patio areas. A focal garden was created in a shape symbolic to the building owners and it was filled with planters featuring native flowers.

Finally, the watertight enclosure of the main roof and courtyard levels were examined. New waterproofing membranes, application types, and water tests were researched in order to determine what would be the best fit for the courtyard green roof and the main roof level. First, a new drainage plan was created for the courtyard green roof, Membrane manufacturers were compared, assembly types were considered, and a system was found that best suits the needs of each level. Water tests were considered based on feasibility of the test, time to conduct the test, and appropriateness for the material.

Building Introduction

The Corporate Headquarters is in the midst of construction at the South end of an existing retail park in the Great Lakes Region of the Midwestern United States. It is a five story office a retail space designed to serve as the new headquarters for an established and successful US based company. The new 659,000 gross square foot building's architecture was designed to blend in with the style of the surrounding buildings in the retail park. It was designed in the contemporary "Americana" style, serving as the last component of the planned retail area. Ground broke in August 2014 and the project is anticipated to reach substantial completion in Spring 2016.

The building features an interior open green roof courtyard with entry access on the third floor and many large view windows, allowing workers within the offices to bring the atmosphere of the outside in. This courtyard is meant to help enrich the sense of creativity and community within employees. The courtyard features an intensive green roof with a variety of plantings and walking paths. To achieve this courtyard, the structural engineer chose to laterally brace the building with steel braced frames, which are tied at the base by grade beams at the foundation.

The Corporate Headquarters serves as the south port of entry into a retail park and will incorporate retail space on its ground floor and second floor. The upper levels are dedicated to larger open office spaces that allow for spatial flexibility and mobility. Pending acquisition of land adjacent to the site, a proposed bridge will connect the upper two floors of the Corporate Headquarters with a parking structure, as is commonplace in the rest of the retail park. The proposed face brick and curtain wall façade mimics the "Main Street America" feel of the retail park but speaks to how the company has evolved throughout the generations to stay classic, but feel current.

Site Plan and Location

Building Location: Great Lakes Region, U.S.A.

-exact location map not permitted

Site Map

Design Codes and Resources

The following documents were used to evaluate the building's existing structural system.

- **Ohio Building Code 2011**

  -incorporates IBC 2009

- **American Society of Civil Engineers**

  -ASCE 7-05: Minimum Design Loads for Buildings

- **Corporate Headquarters**

  -Construction Documents

  -Technical Specifications

- **Boise- Cascade**

  -Weight of Building Materials Technical Note

Structural System Overview

Foundation System

A geotechnical report of the future site of the Corporate Headquarters was written by in February 2012 by Geo-Sci, Inc. Following the completion of the report, the geotechnical engineer determined that the original soil bearing capacity of 4ksf would not be sufficient to support the weight of the building. In order to increase the soil bearing capacity, aggregate pier soil reinforcement system was recommended. These piers are to be placed below each column footing. Aggregate pier sizing varies with column footing size, with an average diameter of approximately 18”.

The geotechnical report required that all footings, both column and wall, be excavated and poured on the same day. If this cannot be achieved, a 3” concrete mud mat must be poured over all of the excavated soil. The foundation is comprised of spread footings, wall footings, column piers, and grade beams.

The foundation of the Corporate Headquarters required the use of grade beams in order to resolve the large dead load of the courtyard trees into the site soil below. This is evident due to the placement of the grade beams near the areas with courtyard access, namely, the southwestern corner of the courtyard and the northwestern corner. The grade beams take the load from the large columns located near the building core.

The typical spread footings (Figure 1) are centered under the base of the steel columns and are placed directly above the aggregate piers used for soil reinforcement. Since there are no moment frames within the structure of the building, it can be reasonably assumed that the connections are pinned. For columns that sit on both a spread footing and concrete pier (Figure 2), the connection can also be assumed to be pinned. All spread footings in this building are supported by aggregate piers due to the poor soil quality on the site.

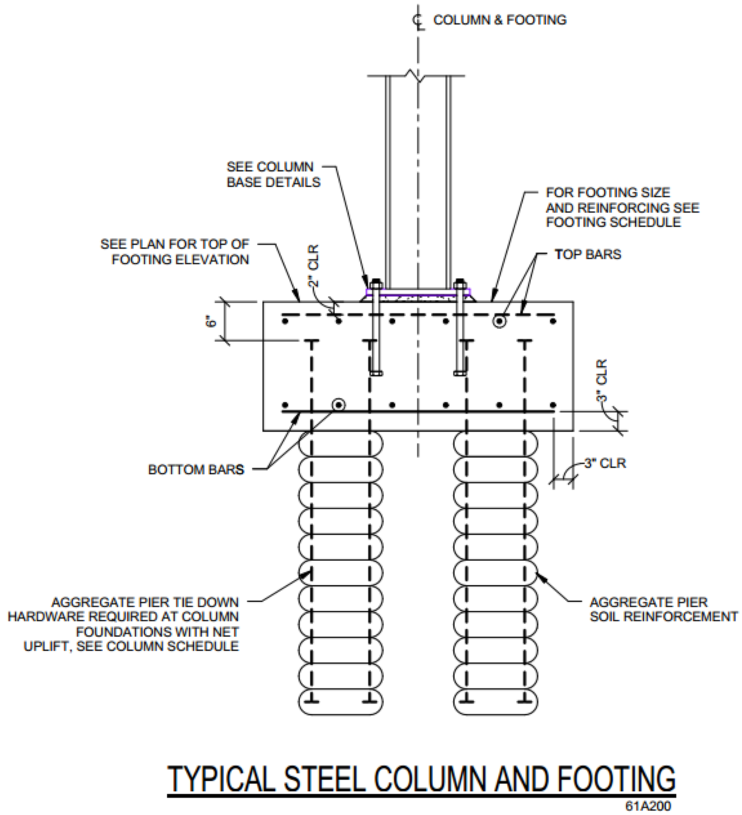


Figure 1- typical steel column and footing

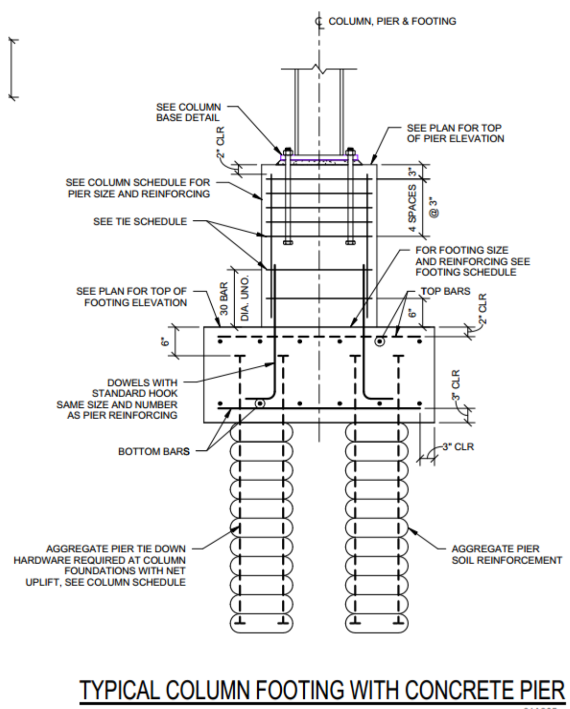


Figure 2- Typical column footing with concrete pier

Wall footings are used at all exterior cavity wall locations along the perimeter of the building, and the building rests on two different types of slab on grade. The larger slab depth (Type S-2 in) is used throughout the northern half of the building since it is slightly below grade and carries larger dead loads. Slab Type S-1 is used primarily near the center of the building, near the area of the courtyard, and is typical slab on grade construction. Both slab types can be seen in Figure 3.

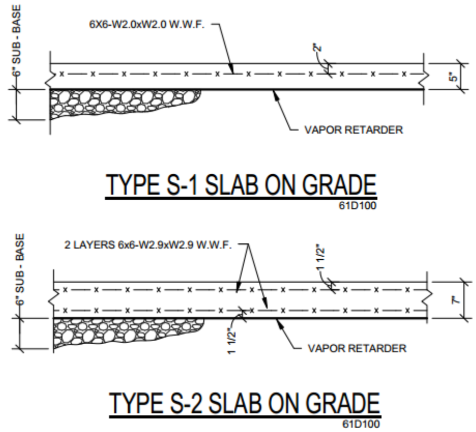


FIGURE 3- SLAB ON GRADE DETAILS

Roof System

The roofing system of the Corporate Headquarters is comprised of two different types of roof assemblies. The majority of the main roof is roof type R-1. Roof R-1 has 3” 18 gauge galvanized roof deck with no concrete topping while roof type R-2 features 3” 16 gauge composite metal deck with 6” of normal weight concrete slab topping. Deck is perpendicular in both assembly types.

Floor System

The Corporate Headquarter features two different construction assemblies for the floor system. The first assembly (F-1) features 3 ¼” lightweight concrete with 6x6-W1.4xW1.4 welded wire fabric reinforcement on top of a 2” 18 gage composite metal deck. Assembly F-2 has 4 ¼” of lightweight concrete reinforced with 6x6-W2.0xW2.0 draped welded wire fabric on 3” 16 gage composite metal deck. The decking runs perpendicular to the wide flange beams.

Typical Floor Bay

Many of the bays in the Corporate Headquarters are rectangular, and shapes only differ near the edges of the building and the interior courtyard area. A typical bay is 38’x40’. Two typical member sizes used in all levels of floor framing are W21x44 and W24x55, with slight variation in depth (+/- 3”) and weight (+/- 13 psf) when spans differ. In smaller span areas, such as around stair and elevator openings and the courtyard, W18 shapes and W21 shapes are common. Typical interior girders for a standard bay are

W24x68, and in areas with smaller bays are typically W21 shapes or lighter W24 shapes. Figure 4 below shows a typical 38' bay and W24x55 beams.

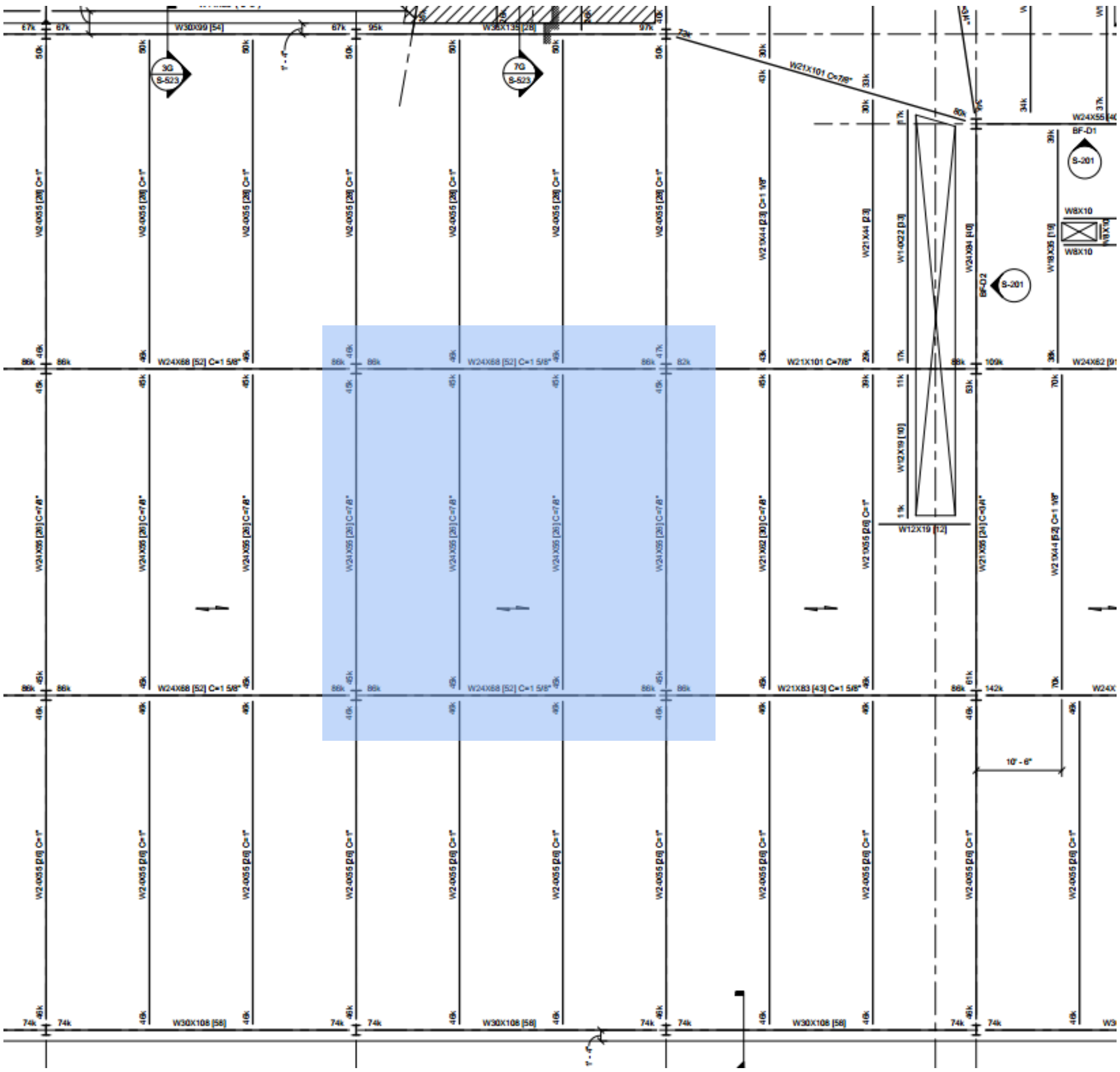
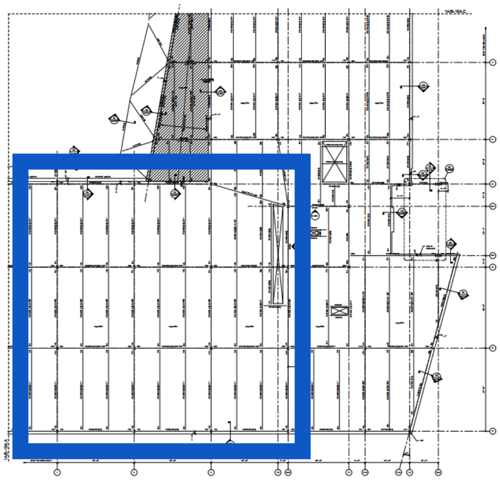


FIGURE 4: LEVEL 4 FRAMING PLAN SHOWING TYPICAL BAY (S104.D)



Gravity Framing

The gravity framing of the building is composed of steel wide flange columns. All columns are W14 or W12, with the majority of weights between 61 and 170. One exception to this is a column that extends from the first floor to the roof. Nearly every column in the building has a column splice, all of which have larger shapes on the bottom than the top. Every combination of column splices varies slightly in size, with no predominant size majority. The columns are spliced between level 2 and level 3, and eleven columns in the building have tension spices. The columns are tension spliced because they are part of braced frames and carry a large axial load. The column schedule may be found in the figure below, and supplementary floor plans and elevations may be found in Appendix X.

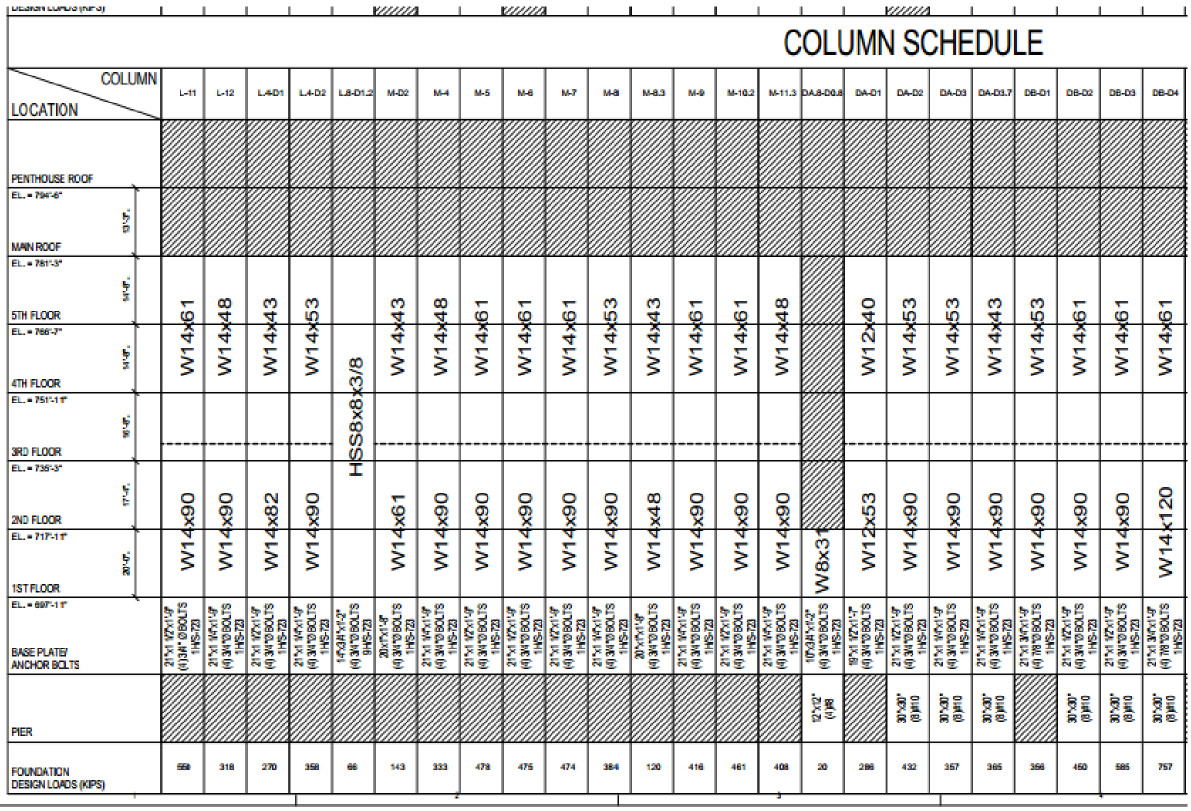


FIGURE 5- COLUMN SCHEDULE

Lateral System

The lateral system of the Corporate Headquarters is made up of eight braced frames near the core of the building (Figure 7). In six locations braced frames extend from the first floor to the roof, and in two locations the braced member begins on the second floor level. These two frames do not have braced members on level one to accommodate a future retail shaft. The load of these frames is transferred using heavier columns than those used in the other six braced frames. The columns in turn transfer the load to the grade beams in the foundation system.

The braced members are made of Hollow Structural Sections varying from HSS8x8x1/4 to HSS 16x16x5/8. In two locations, the bottom member of the brace is made of a W14 shape. The braces take a diagonal shape in five locations, a chevron shape in one location, and an inverted chevron shape in two locations.

The braced frames were chosen as the lateral force resistance system for the actual construction process due to their strength and stiffness properties. Additionally, braced frames use less material than moment resisting frames and don't require formwork, as concrete shear walls do.

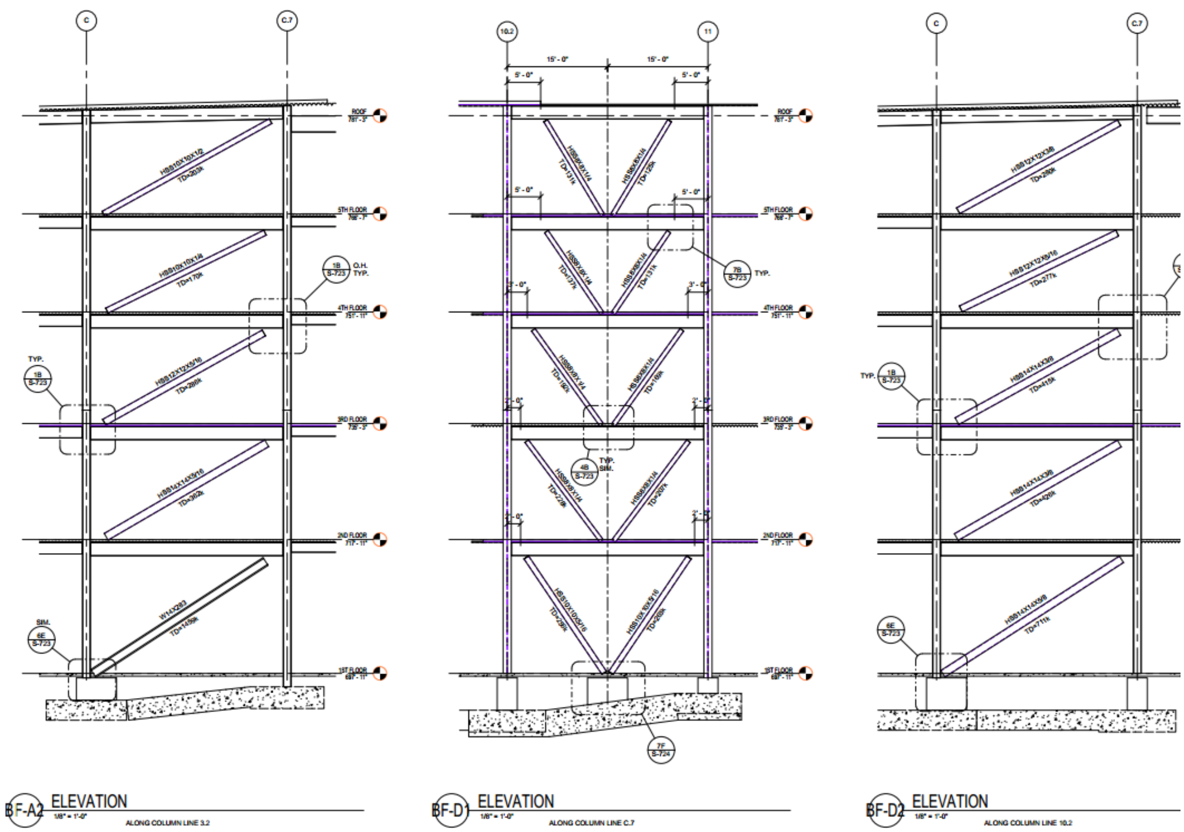


FIGURE 6- SAMPLE BRACED FRAME ELEVATIONS

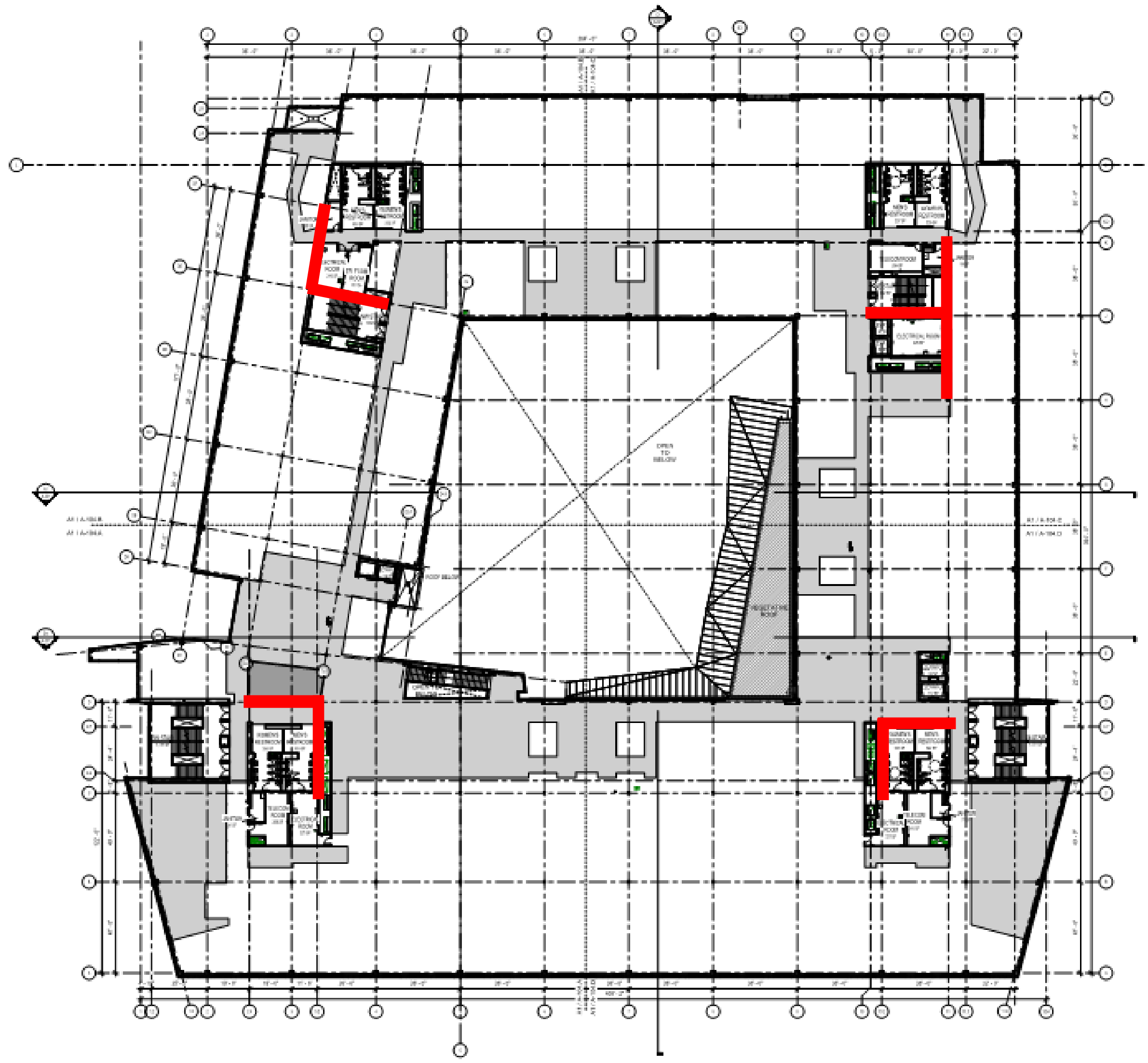


FIGURE 7-BRACED FRAME LOCATIONS

Existing Loading

This portion of the report will summarize the design loading for the existing project as determined from the project drawings and previous technical reports.

Gravity Loading

The loads in the tables below were taken from the sheet S-001 of the structural drawings. Hand calculations of snow loads may be found in in Appendix C. Many of the values calculated were similar to those found in the drawings, with a one psf discrepancy between the calculated and actual dead load values for the office floor areas. Verification of these loading conditions may be found in Technical Report 2.

Superimposed Design Loads		
	Dead Load (PSF)	Live Load (PSF)
Office Areas	61	65
Public Areas	61	100
Libraries	61	150
Main Server Room	76	250
Courtyard Grass Area	201	100
Courtyard Tree Area	441	100
Typical Roof	18	25
RTU Roof	117	25
Kitchen	144	150
A/V Suite	100	221

TABLE 1-SUPERIMPOSED DESIGN LOADS

Snow Load	
Ground Snow Load	Pg= 20 psf
Exposure Factor	Ce= 1.0
Importance Factor	I= 1.1
Thermal Factor	Ct= 1.1
Flat Roof Snow Load	**Pf= 17 psf**

TABLE 2-SNOW LOAD

Lateral Loading

This portion of the report shows the results of wind load and seismic load investigations for the existing project.

Wind

Wind calculations were performed using ASCE 7-05 and completed during the analysis of the building's existing structural system. A summary table of results of the calculations is listed below. The calculations may be viewed in full in Appendix F. Wind pressure in the east-west direction was found to be the prevailing wind case, creating a maximum base shear of 432.16 kips. Wind pressure in the North- South direction causes a base shear of 354.62 kips.

Wind Load Factors	
Basic Wind Speed	V=90 mph
Importance Factor	I=1.0
Exposure	B
Internal Pressure Coefficient	Gcpi=+/- 0.18
Topographic Factor	Kzt=1.0
Gust Effect Factor	Gf=.9

TABLE 3- WIND LOAD FACTORS

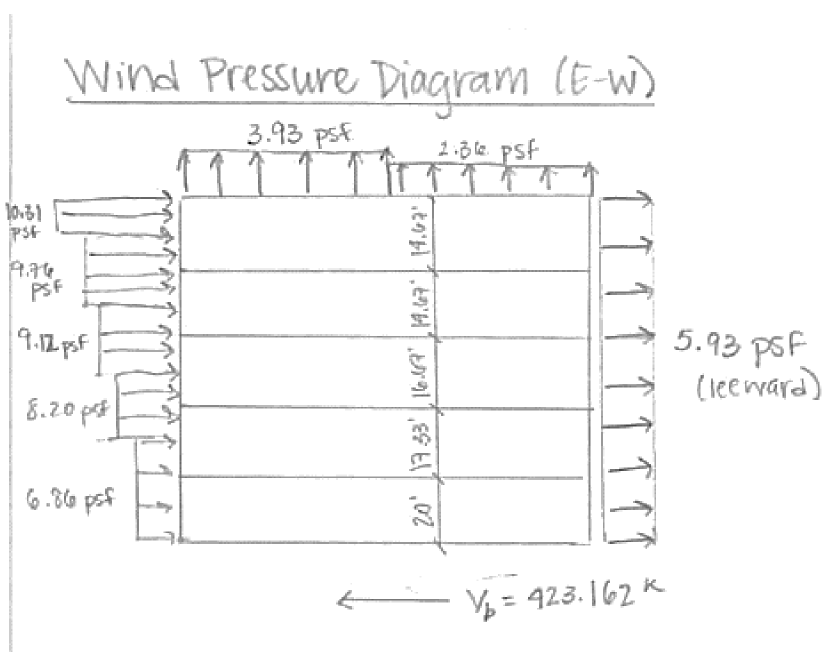


FIGURE 8-EAST WEST WIND PRESSURE DIAGRAM

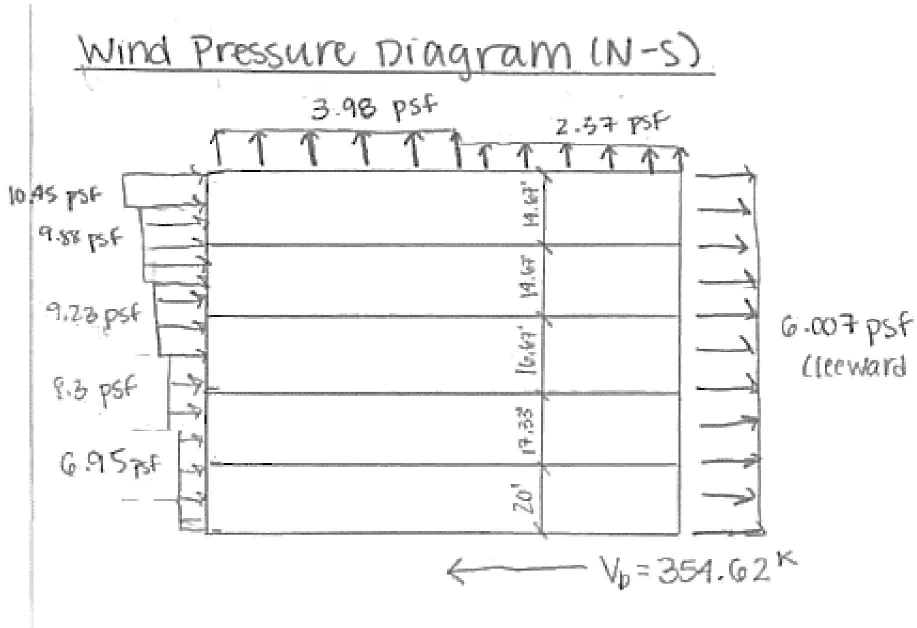


FIGURE 9-NORTH SOUTH WIND PRESSURE DIAGRAM

Seismic

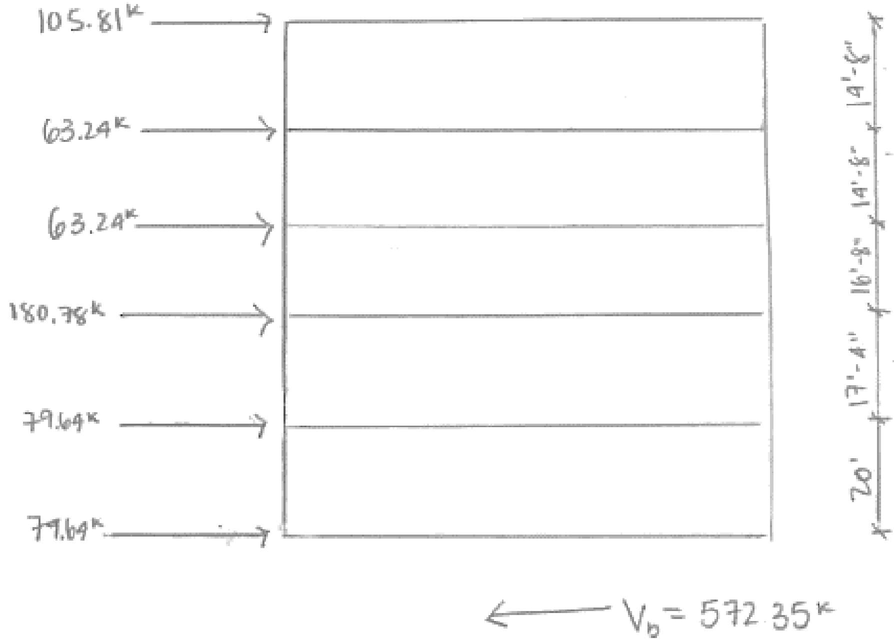
Seismic calculations were performed using the Equivalent Lateral Force Procedure found in ASCE 7-05. The building was analyzed as a true rectangle for ease of calculations. Full calculations may be found in Appendix G, and the figure below shows the vertical distribution of seismic forces. It was found that seismic force controls over wind force and the maximum base shear was found to be 572 kips. The building is located in Site Class C and was found to belong to seismic design category A. A brief summary of seismic design parameters and spectral response factors may be found in the tables below.

Seismic Parameters	
Site Class	C
Occupancy	II
Importance	1
SDC	A

TABLE 4-SEISMIC DESIGN PARAMETERS

Spectral Response Factors	
SS	0.175g
Sds	0.14
S1	0.051 g
Sd1	0.0578

TABLE 5-SPECTRAL RESPONSE FACTORS



Total Building Load = 57235 K

FIGURE 10-VERTICAL DISTRIBUTION OF SEISMIC FORCES

Problem Statement

The existing steel structure of the Corporate Headquarters meets all strength and serviceability requires. Though this system works and will continue to perform well in the future, the large floor to floor height within the space and relatively small lateral loads allowed for creative exploration of alternative structural systems. For this project, a scenario was created in which the shape of the courtyard green roof would be changed in order to gain more office space on upper floors and to simply the structural redesign process. The courtyard's current shape is similar to that of a parallelogram, so by changing into a rectangle, it allowed for more regularized bays in one corner of the building and more office space on the building's third, fourth, and fifth floors.

This change was implemented in order to best meet the needs of the building owner. The new Corporate Headquarters aims to hold more employees than the owner's previous office location, so creating additional office space by slightly decreasing the size of the courtyard green roof is a reasonable way to accomplish this. Adding to the overall gross square footage of the building will increase the building's weight, so in order to keep the building's total weight similar to the existing weight, a newer more lightweight structural system should be implemented.

Proposed Solution

In order to meet the challenges put in place by the created scenario, the courtyard green roof was reshaped into a true rectangle. This change allowed for more office space on the upper floors of the building and more regular bay shapes in the building's northwest corner. The new structural framing layout of the Corporate Headquarters may be found in Appendix B.

In order to best suit the building's new shape, steel joists and joist girders were used for the gravity system redesign. This system helped to decrease the building's weight as steel joists are typically lighter than traditional composite steel beams. The system is suitable because there are no floor to floor height restrictions in the building. The typical floor to floor height is 16.67', so joists and joist girders with large depths will have little impact on the functionality of the space below. The steel columns were resized in accordance with the new gravity loads.

The lateral force resisting system of the corporate headquarters was changed to eight reinforced concrete shear walls, which were placed in the same locations as the steel braced frames used in the current building design. These locations were chosen so that the building's architecture would not be disrupted, as each of the braced frames is currently contained within a wall.

The changes made to the geometry of the green roof courtyard required the changing of the green roof's design. The area was redesigned with a focus on local plants and the building owner's history with the site location. To help keep dead loads to a minimum, tree areas in the space were removed and replaced with a traditional grass space, though growing materials and the paving system were changed.

Lastly, to ensure that the new green roof remained water tight, courtyard and main roof's enclosure system were investigated, with a focus on the waterproofing system. The waterproofing membrane and installation type were changed, and water testing procedures were examined to see what would be the best fit for the building.

Structural Depth

The structural redesign of the Corporate Headquarters included the redesign of both the gravity and lateral system for the building. First, new gravity and seismic loads were determined and new roof and floor deck were selected. Next, the gravity system was designed. A gravity model was created in RAM, the building loads were input into the program, and member sizes were calculated. The member sizes were verified using hand checks, which can be found in Appendix D. Many of the building's existing bay sizes were retained, with the exception of a few bays near the northwest side of the courtyard. The average bay size is 38'x40'.

Following the completion of the gravity system, the building's lateral system was designed. Wind and seismic loads were input into RAM, along with constraints and criteria for the design of the shear walls. Walls were reinforced and spot checks were conducted, the results of which may be found in Appendix H.

Load Combinations

Basic load combinations were taken from ASCE 7-05 and all members were sized using load and resistance factor design.

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

RAM Modeling Process

The proposed building redesign was analyzed using RAM Structural System. The criteria used in the design of the gravity and lateral system included ASCE 7-05, IBC 2009, and ACI 318-11. Within the model, each diaphragm was considered to be rigid and each column was considered to have a pinned connection at its base. RAM Frame and RAM Concrete were used to develop the lateral system while RAM Beam and RAM Column programs were used to develop the gravity system.

Gravity System Redesign

The proposed gravity system of the Corporate Headquarters is comprised of long span steel joists, joist girders, and w-shaped columns, with the exception of the courtyard, which utilizes W shaped steel beams and girders due to its heavy load. An overview of the gravity redesign of the courtyard area will be covered in a later section of this report but a summary of courtyard loading can be seen in Table 8. In the first draft of the gravity redesign, K-series open web steel joists were the preferred framing material, but due to the large spans of the members, it was determined that long span joists would be a better option.

Gravity Loading

The dead and live loads for the gravity framing are summarized in the tables below. An isometric view of the gravity model can be viewed in Figure 11.

Dead Loads		
	Office	Roof
Concrete Slab (PSF)	31	50
Metal Deck (PSF)	3	3
MEP (PSF)	5	10
Ceiling (PSF)	2	2
Flooring (PSF)	3	-
Sprinklers (PSF)	3	3
Framing Allowance (PSF)	5	10
Adhered Membrane (PSF)	-	1
Roof Board (PSF)	-	1
Insulation (PSF)	-	3
Vapor Retarder (PSF)	-	1
Total Load (PSF)	**52**	**84**

TABLE 6- REDESIGN DEAD LOADS

Live Loading		
	Office	Roof
Live Load (PSF)	50	20
Partitions (PSF)	15	-
Snow (PSF)	-	17
Total Load (PSF)	65	20
Reduced LL	**41**	**20 (unreducable)**

TABLE 7- REDESIGN LIVE LOADS

Courtyard Green Roof Dead Loads (PSF)		
Material	Garden Area	Paver Area
Deck	3	3
Concrete Topping	31	31
Vegitation	20	
Engineered Fill (fully saturated)	55	55
Filter Fabric	1	1
Drainage Layer	2	2
Root Barrier	1	1
Waterproofing Membrane	1	1
Planter Allowance	10	10
Concrete Pavers		30
Total	**124**	**134**

TABLE 8- COURTYARD DEAD LOADS

Design Process

The roof deck and floor deck were selected after performing hand calculations, which may be found in Appendix C. Concrete topping thicknesses, which were specified in the structural drawings, were retained in order to maintain a two hour rating for the assembly. The gauge of metal deck was also retained due to a special provision in the project specifications. The Vulcraft floor and deck catalog was used in order to determine the floor and roof deck assemblies. The floor deck was found to be 1.5VLR18 with 3.25" LW concrete topping. Roof deck was found to be 1.5VL18 with 4" of normal weight concrete topping in areas in order to support the roof top mechanical units. In both the roof and floor deck, unshored 2 span conditions were utilized for economy. Though both of these decks are capable of handling a much larger load than is applied to them, it was important that the project maintain the depth of concrete topping and the gauge of the metal deck.

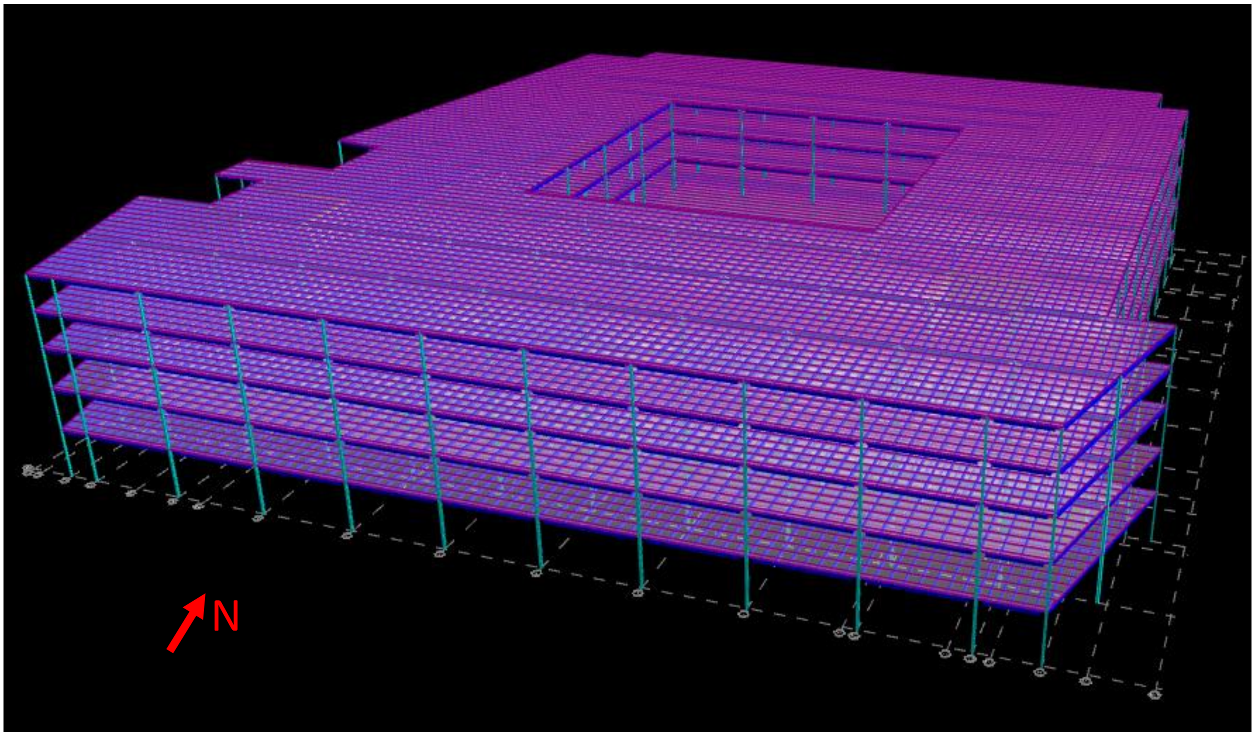


FIGURE 11- RAM GRAVITY MODEL

Using RAM Structural System, the loads shown above were input into the program and member sizes were calculated. Member sizing was controlled by live load deflection limitations and a desire to keep the joists and girders at a depth less than 36”. The choice to limit the depth of joists and joist girders was done to maintain the architectural integrity of the space. Though a depth of 36” may seem large, the building’s average floor to floor height was roughly 16.33’, therefore it was determined that the integrity of the space would be minimally disturbed with a three foot decrease in ceiling height.

In order to achieve the conditions set forth, joists were spaced at 4.75’ and have a maximum depth of 28”. This spacing correlated to the maximum number of spaces permitted when using framing into a joist girder spanning 38’. Spacing joists so closely together greatly helped to reduce member deflections, and the joist girders were found to have a deflection that was nearly have the allowable limit. Using spacing Joist girders were also limited to a maximum depth of 36”. A typical floor bay is shown in Figure 12. A typical roof bay was found to be similar, utilizing 28LH10 joists and 36G8N26.2K joist girders. A typical bay can be found in Figure 13 Spot checks of floor member sizes and RAM output samples may be found in Appendix D.

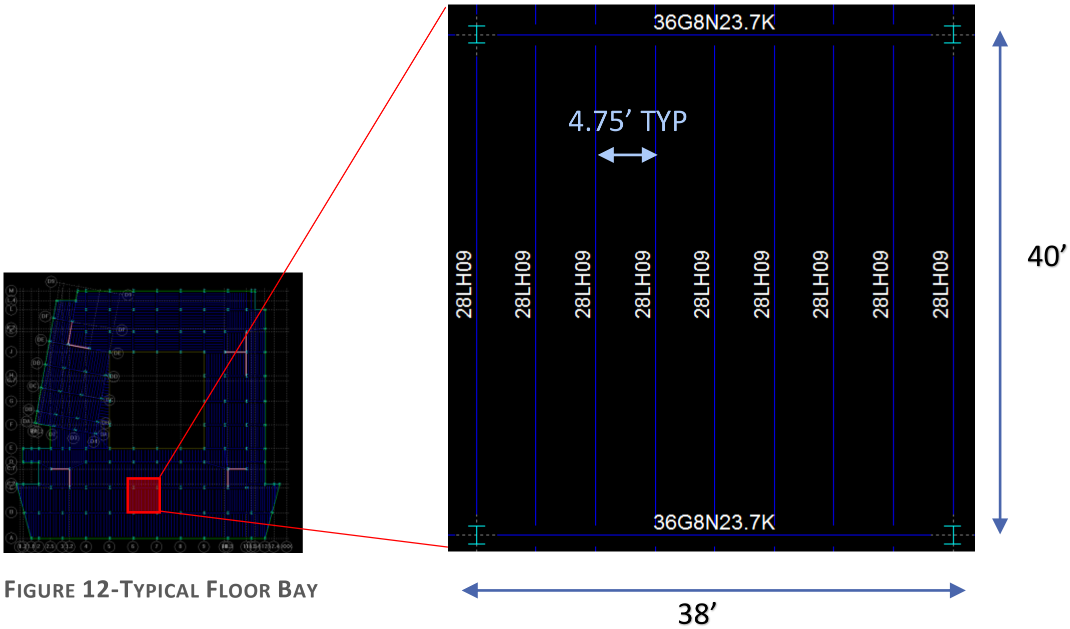


FIGURE 12-TYPICAL FLOOR BAY

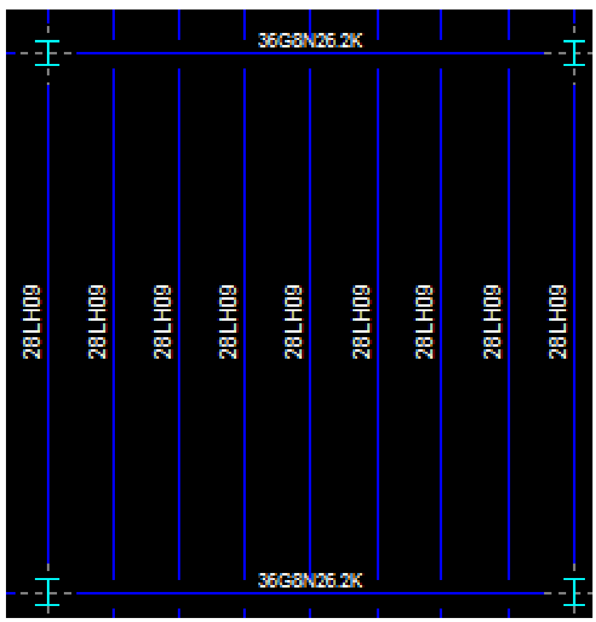


FIGURE 13-TYPICAL ROOF BAY

Steel columns were selected for the gravity system for relative ease of constructability and to help maximize floor area. Additionally, steel joists and joist girders in other buildings are more typically framed into steel columns rather than concrete columns or CMU columns, making steel a more appropriate choice. Columns were also sized using RAM Structural, and columns were spliced on the third level. Interior columns were typically a W14 while exterior columns were typically W12. Exterior columns were found to be suitable for both shear and flexure. Column spot checks and a sample of RAM outputs may be found in Appendix D. An isometric and plan view of the gravity columns may be found in Figure 14 and Figure 15, respectively. The columns are highlighted in lime green.

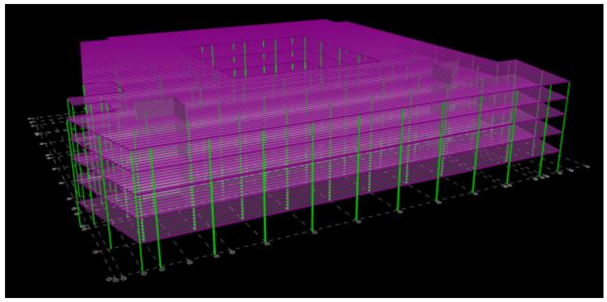


FIGURE 14-GRAVITY COLUMN ISOMETRIC VIEW

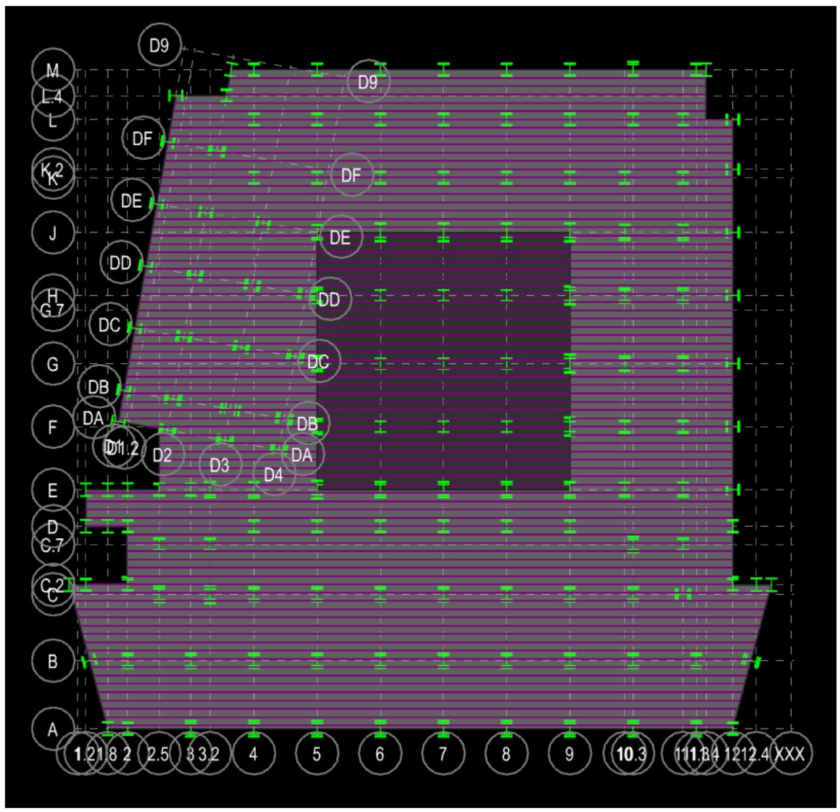


FIGURE 15- GRAVITY COLUMN PLAN VIEW (4^{TH} FLOOR)

Vibration Concerns

Since steel joists and joist girders were used for the gravity redesign, floor vibrations due to walking were a major concern. Using Design Guide 11, Chapter 4, Design for Walking Excitation, it was determined that the system as redesigned was suitable to meet recommended criterion. The system's frequency (fn) was determined to be 2.66 Hz and the acceleration limit was determined to be .0015, far less than the limit of .005. The vibration analysis calculations may be viewed in full in Appendix E. A reason for the low acceleration limit is due to the close spacing of the steel joists and the thickness of the concrete topping used in the floor deck.

Impact on Foundations

The overall weight of the Corporate Headquarters decreased as a result of the changed gravity system, so it is assumed that column footing sizes may decrease to help reduce building costs. A full analysis of new foundation sizing is outside the scope of this thesis.

Lateral System Redesign

The original lateral system of the Corporate Headquarters was governed by seismic load despite its seismic design category (A) and its location. When the gravity system was changed to long span steel joists and joist girders and the courtyard tree area was removed the building weight decreased. This decrease in weight lead to a decrease in seismic base shear and seismic loads were recalculated. A summary of the calculations may be viewed in Tables 10-12. Wind loads remained the same as in the original building design. The results of the calculations are summarized below in the Wind Loading section of this report and can be viewed in their entirety in Appendix F. As a result, the building is now controlled by wind forces in the east west direction.

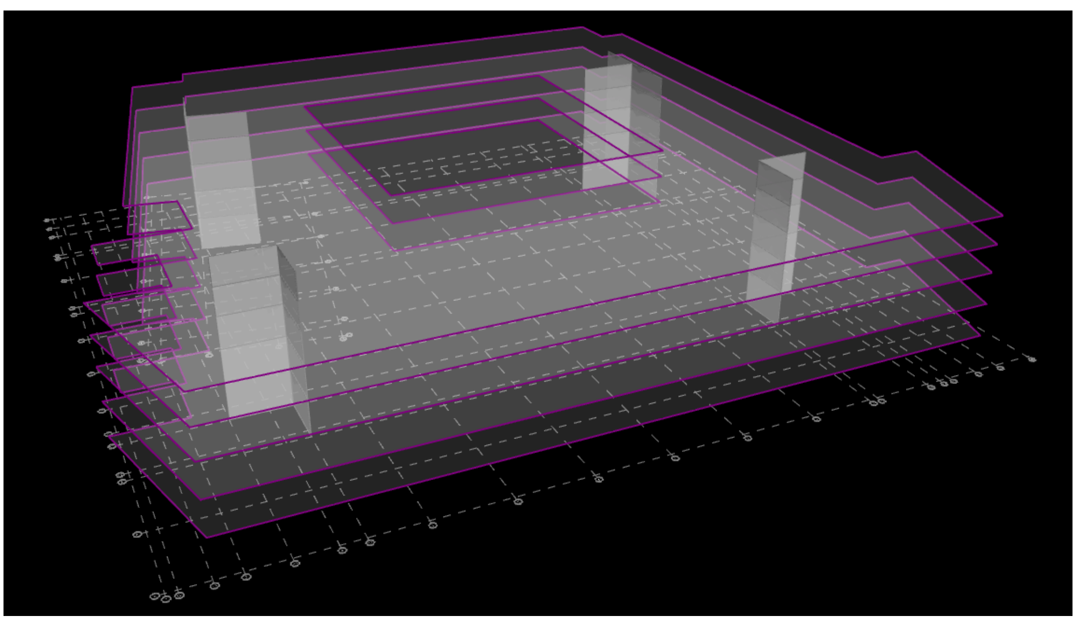


FIGURE 16- LATERAL SYSTEM ISOMETRIC VIEW

The building's proposed lateral system redesign is comprised of 8 reinforced concrete shear walls. The shear walls were placed in the locations of the existing system's steel braced frames for architectural integrity. The braced frames were each fully contained within a wall, so placing the shear walls in the same location seemed like a logical choice. The location of the shear walls can be seen in Figure 18. Each shear wall is 6" thick and is reinforced with the minimum #4 @12"O.C. in both directions (Figure 17- #4's at 12" O.C. Vertical and Horizontal). This reinforcement is the minimum required reinforcement and is used due to the light seismic loads the building is subjected to. A spot check was conducted to ensure that shear wall reinforcing was adequate. This calculation may be found in Appendix H.

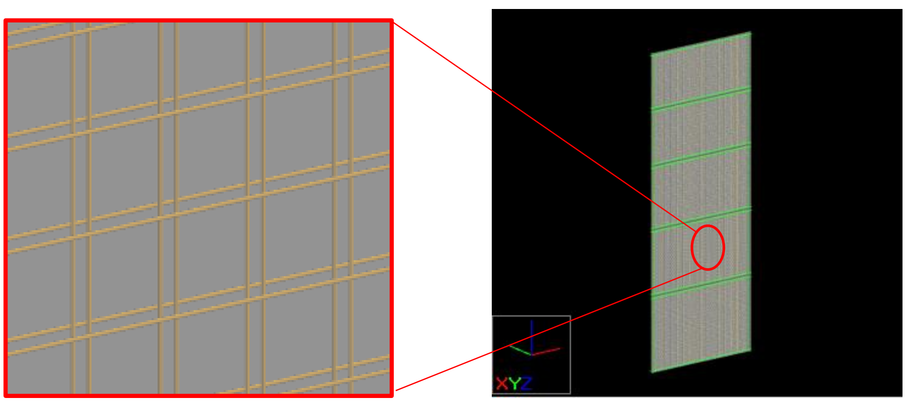


FIGURE 17- #4'S AT 12" O.C. VERTICAL AND HORIZONTAL

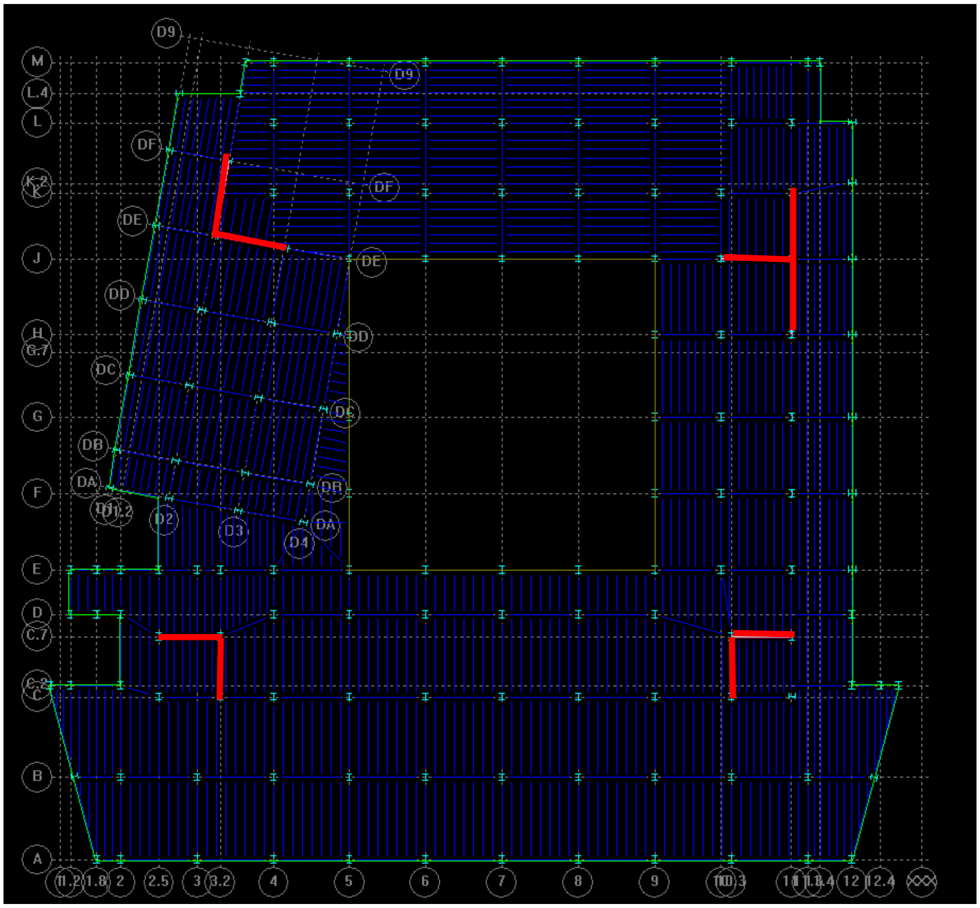


FIGURE 18- LOCATIONS OF REINFORCED CONCRETE SHEAR WALLS

Wind Loading

Wind Pressure (North-South Direction)							
Floor	z (ft)	qz (PSF)	Windward Pressure (PSF)	Leeward Pressure (PSF)	Tributary Area	Force (K)	Overturning Moment (ft-k)
2	20	11	6.952	-6.007	6096	78.998	1579.962
3	37.33	13.14	8.304	-6.007	5542	79.314	2960.800
4	54	14.61	9.234	-6.007	5314	80.988	4373.359
5	68.67	15.64	9.884	-6.007	4782	75.993	5218.444
roof	83.33	16.53	10.447	-6.007	2390	39.325	3276.950
					Base	**354.618**	**17409.515**

TABLE 9- NORTH SOUTH WIND PRESSURES

Wind Pressure (East-West Direction)							
Floor	**z (ft)**	**qz (PSF)**	**Windward Pressure (PSF)**	**Leeward Pressure (PSF)**	**Tributary Area**	**Force (K)**	**Overturning Moment (ft-k)**
2	20	11	6.864	-5.931	7368	94.273	1885.466
3	37.33	13.14	8.199	-5.931	6698	94.645	3533.094
4	54	14.61	9.117	-5.931	6422	96.636	5218.328
5	68.67	15.64	9.759	-5.931	5780	90.690	6227.687
roof	83.33	16.53	10.315	-5.931	2888	46.918	3909.638
					Base	**423.162**	**20774.214**

TABLE 10- EAST WEST WIND PRESSURES

Seismic Loading

Spectral Response Factors	
SS	0.175g
Sds	0.14
S1	0.051 g
Sd1	0.0578

TABLE 11- REDESIGN SEISMIC PARAMETERS

Seismic Parameters	
Site Class	C
Occupancy	II
Importance	1
SDC	A

TABLE 12- REDESIGN SPECTRAL RESPONSE FACTORS

Seismic Pressures				
Level	Area (SF)	Weight (K)	Force (K)	Overturning Moment (ft-k)
Main Roof	121940	10658	106.58	8882.38
5	121940	6341	63.41	4354.36
4	121940	6341	63.41	3424.14
3	145500	8777	87.77	3276.45
2	145500	7566	75.66	1513.2
Total Weight (K)=	**39683**	**Base Shear (K)**	**397**	**21450.53**

TABLE 13- REDESIGN SEISMIC WEIGHT AND FORCES

Modeling Process and Drift Results

Using the calculated loads, values were input into RAM Frame and RAM Concrete in order to design the shear walls. Due to the low height of the building (relative to its width) and the small loads it is subjected to, reinforcement in the shears walls was governed by minimum reinforcing requirements. Shear walls were originally 10" thick, but after a few iterations, it was determined that they could be 6" thick and support the building against lateral loads.

Using RAM Frame analysis software, story drifts were calculated for both wind and seismic loading. The drift results were then compared to the story drifts of the existing building. The values were compared and results are summarized in Table. It was found that drift in the proposed lateral system was significantly lower than the drift in the existing design. Wind drift was found to be well within the drift limits of h/400, as set forth in ASCE 7-05. The wind drift limit of the main roof was as follows;

$$\Delta_{max} = (83.33' \times 12''/1')/400 = 2.5''$$

The seismic drift limit of the main roof was also well below the limit. Per ASCE 7-05, story drift is limited to two percent of the total building height, which limits the total drift of the main roof level to the equation shown below:

$$\Delta_{max} = (83.33' \times 12''/1') \times 0.02 = 20.0''$$

The RAM output of drifts for both the existing system and the proposed system may be found in Appendix I.

Redesign Wind Drifts (N-S)		
	Story Drift (in)	Total Drift (in)
Main Roof	0.194	0.592
Level 5	0.16	0.398
Level 4	0.123	0.238
Level 3	0.079	0.115
Level 2	0.036	0.036

Existing Wind Drifts (N-S)		
	Story Drift (in)	Total Drift (in)
Main Roof	0.409	1.329
Level 5	0.363	0.92
Level 4	0.285	0.557
Level 3	0.188	0.272
Level 2	0.084	0.084

TABLE 14- NORTH SOUTH WIND DRIFTS

Redesign Wind Drifts (E-W)		
	Story Drift (in)	Total Drift (in)
Main Roof	0.272	0.816
Level 5	0.222	0.544
Level 4	0.169	0.322
Level 3	0.106	0.153
Level 2	0.047	0.047

Existing Wind Drifts (E-W)		
	Story Drift (in)	Total Drift (in)
Main Roof	0.555	1.764
Level 5	0.488	1.209
Level 4	0.38	0.721
Level 3	0.241	0.341
Level 2	0.1	0.1

TABLE 15-EAST WEST WIND DRIFTS

Redesign Seismic Drift		
	Story Drift (in)	Total Drift (in)
Main Roof	0.136	0.404
Level 5	0.11	0.268
Level 4	0.083	0.158
Level 3	0.052	0.075
Level 2	0.023	0.023

Existing Seismic Drift		
	Story Drift (in)	Total Drift (in)
Main Roof	0.244	0.751
Level 5	0.208	0.507
Level 4	0.158	0.299
Level 3	0.1	0.141
Level 2	0.041	0.041

TABLE 16- SEISMIC DRIFTS

Though the existing building is subjected to larger seismic forces than wind forces, wind drift in the East- West direction is most severe. In the proposed redesign, wind forces in the East-West direction control over North-South wind forces and seismic forces. Total drift is lower in the redesign than in the existing building as a result of the change from steel braced frames to concrete moment frames.

Center of Mass and Center of Rigidity

The center of mass and center of rigidity also changed as the lateral system was redesigned. Despite the fact that the proposed concrete shear walls and the existing braced frames are placed in the same location, the center of mass and center of rigidity of the building changed. These changes are due to the change in material. Concrete shear walls are heavier and more rigid than the steel braced frames. There is more concrete near the East side of the building, which is one of the reasons that the center of rigidity shifted left. The centers of mass of the two systems are in approximately the same location. The center of mass and center of rigidity for both systems is shown in Figure 19. The existing building is represented with the light blue circles while the centers of mass and rigidity for the redesign are show in red. The yellow circle represents the building's origin point from which all measurements are taken.

Centers of Mass and Rigidity				
	COM (x)	COM (y)	COR (x)	(COR (y)
Proposed Redesign	70.91	-105.04	115.9	-108.87
Existing Building	70.06	-107.23	68.44	-46.05

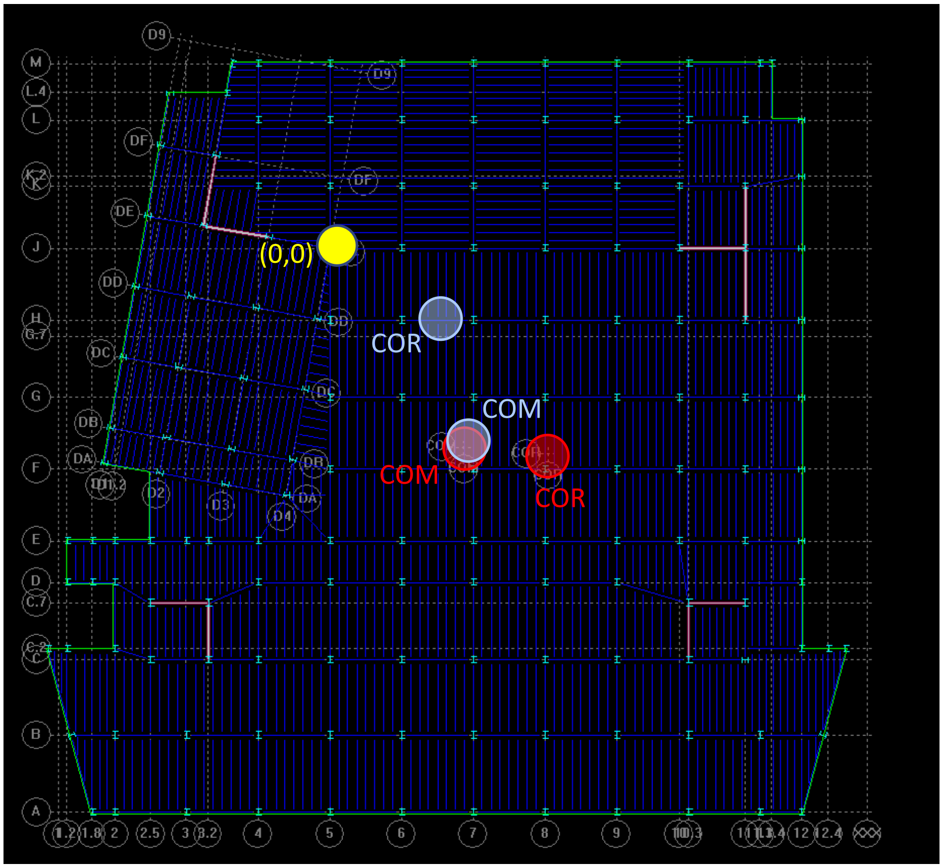


FIGURE 19- CENTER OF MASS AND CENTER OF RIGIDITY

Green Roof Breadth

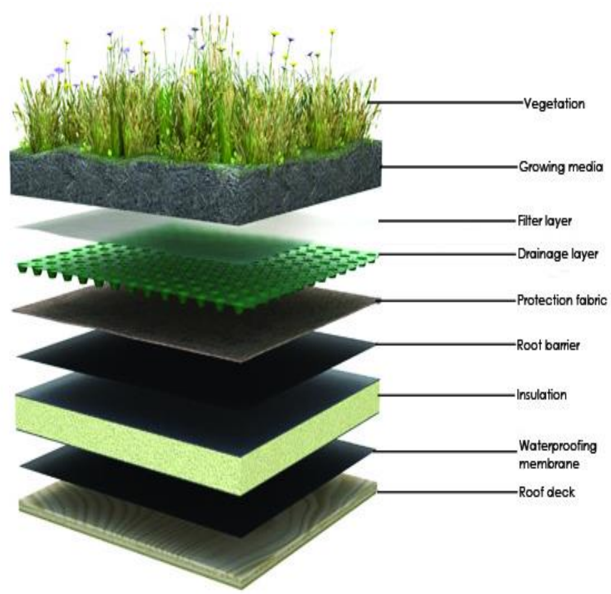
The idea to change the courtyard's design first stemmed from the decision to change the shape of its perimeter in order to simplify structural analysis. The courtyard's original shape was irregular, and didn't lend itself to an easy drainage pattern. While looking at the building loads provided on sheet S001 of the structural drawings, it was noted that the area of heaviest dead load was the courtyard tree area. A traditional green roof grass area is relatively heavy, but the tree area load was three times higher than that of the grass area. In order to reduce the dead loads on this portion of the building, it was determined that the courtyard design and planting pattern would change and the tree area would be eliminated and replaced with a regular green roof system. The scope of this breadth included a redetermination of system dead loads, a redesign of the space, and selection of new plants for an area of the garden.

Green Roof Loading

In the existing building, the courtyard green roof is the area with the highest dead load. In order to lower the overall weight of the building and to ease in the design process, it was determined that the courtyard tree area would be removed. The removal of this area significantly reduced the superimposed dead load in the space, however, the green roof load was still very heavy. Due to depth limitations set forth by the designer and extensive deflections within the members, steel joists and joist girders were deemed unfit to carry the load. Many iterations were carried out in which joist spacing and depth were changed, but overall, it was determined that w-shaped steel beams and girders would be a better system for this area of the building. Dead loads for the space are summarized in Table 17 and the design live load was 100 psf since the area could be classified as an assembly space. The courtyard green roof redesign includes two different areas, the concrete paver area and the garden areas. In order to maintain maximum flexibility in the space, the more conservative dead load value of 134 psf was used to design the entire area. This was done to ensure that concrete paver locations could be changed in the future. Using these dead load values and an assembly area live load value of 100 psf, beam and girder sizes were calculated in RAM structural system.

Courtyard Green Roof Dead Loads (PSF)		
Material	Garden Area	Paver Area
Deck	3	3
Concrete Topping	31	31
Vegitation	20	
Engineered Fill (fully saturated)	55	55
Filter Fabric	1	1
Drainage Layer	2	2
Root Barrier	1	1
Waterproofing Membrane	1	1
Planter Allowance	10	10
Concrete Pavers		30
Total	**124**	**134**

TABLE 17- GREEN ROOF DEAD LOADS



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Green Roof Framing

The courtyard green roof floor framing was redesigned using steel beams and girders since joists and joist girders were deemed unfit to carry the load and meet live load deflection criteria. Through a series of design iterations, it was noted that in order to carry the load of the courtyard green roof, joist girder depths would have to exceed 36". The area below the courtyard level is used for retail space, so to maintain the architectural integrity of that space, it was decided that member depths should not exceed 36". Thusly, steel beams and girders were used for framing. Beams were spaced at 6.33' and with a typical size of W24x55 with a 1" camber. Girders had a typical size of W40x167 and camber between ½" and ¾". Bay sizing remained at 38'x40'. A typical bay is shown in Figure 20 and a framing plan of level three can be found in Appendix B.

Vibrations were not taken into consideration in this area of the building due to time limitations, however, due to the large dead load of the green roof, it is assumed that the acceleration limit would be less than the minimum acceptable standard for walking excitation.

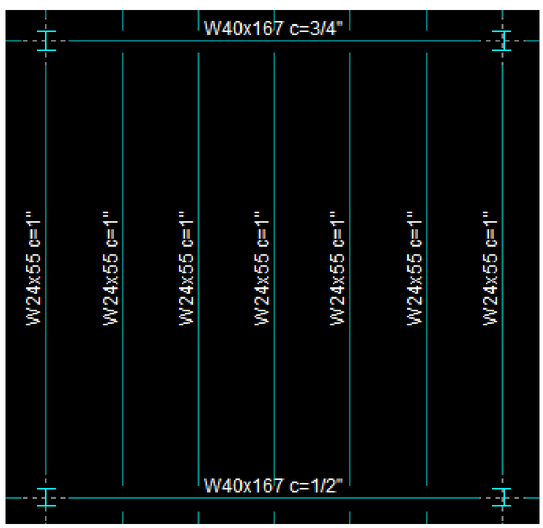


FIGURE 20- TYPICAL COURTYARD GREEN ROOF BAY

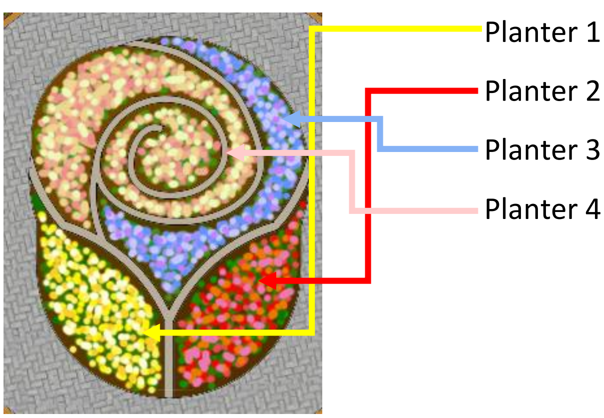
Design Narrative

The inspiration behind the courtyard's new design was a rose. The rose is symbolic to the building owner and therefore it was decided that planters in the shape of a rose would be the focal point of the area. Each individual planter in the rose is at a different height at 6" intervals (planter 1 is lowest, planter 4 is highest), so that it forms a spiral leading up to the top swirled planter. The spiral leading up to the sky is a represents strength and rising to the top, which symbolizes the building owner's success in their industry. Located in a circle around the rose are built in benches so that building occupants may enjoy a quick break outside during the warmer months. Additional seating is provided on the patio, where tables and umbrellas will be set out. The redesigned courtyard garden may be seen in Figure 21.

The flowers used in the rose planters will mainly be plants that are native to the area of the site. Though other plants will be added to the planters, the primary focus will be the local plants. The building owners are active members of the community and really love being a symbol of local pride, so local flowers seemed like a natural choice. Since the plants are local to the area, it is assumed that they will thrive in the location of the site. For security reasons regarding the building's location, the USDA plant hardiness map was not used in this report. The focal plants used in the rose planters are detailed in a later section of this report.

The entire area sits above engineered fill to ensure flexibility in the future of the space. What this means is that the concrete pavers used in the patio sit above a layer of highly compacted fill and the grass and planter areas will be above traditional engineered fill. The walkways and upper patio will be topped in concrete pavers using a Holland paving system (Appendix J).

FIGURE 21-COURTYARD REDESIGN



Green Roof Materials

Engineered Growing Medium (Appendix J)

LiveRoof Engineered Green Roof Soil (Appendix

- 55 psf when fully saturated at 10" depth
- Filters rainwater and buffers acid rain

Holland Pavers

Anchor – Holland Plus Pavers (Appendix J)

- Suitable for walkways and patios small and large areas
- Can be combined in a variety of patterns
- Approximately 30 psf
- Easily purchased through landscape distributors
- Easy snow removal due to smooth surface

Local Plants Used

Planter 1



1. ***Silphium perfoliatum*** (cup plant)
2. ***Viola blanda*** (sweet white violet)
3. ***Asclepias syriaca*** (common milkweed) *also found in planter 4

Planter 2



1. ***Cladonia cristatella*** (British soldier lichen)
2. ***Asclepias incarnate*** (swamp milkweed)
3. ***Asclepias tuberosa*** (butterfly milkweed)

Planter 3


1. ***Erigenia bulbosa*** (harbinger-of-spring)
2. ***Gentianopsis crinite*** (greater fringed gentian)

Planter 4

1. ***Epigaea repens*** (trailing arbutus)
2. ***Asclepias syriaca*** (common milkweed) *also found in planter 1

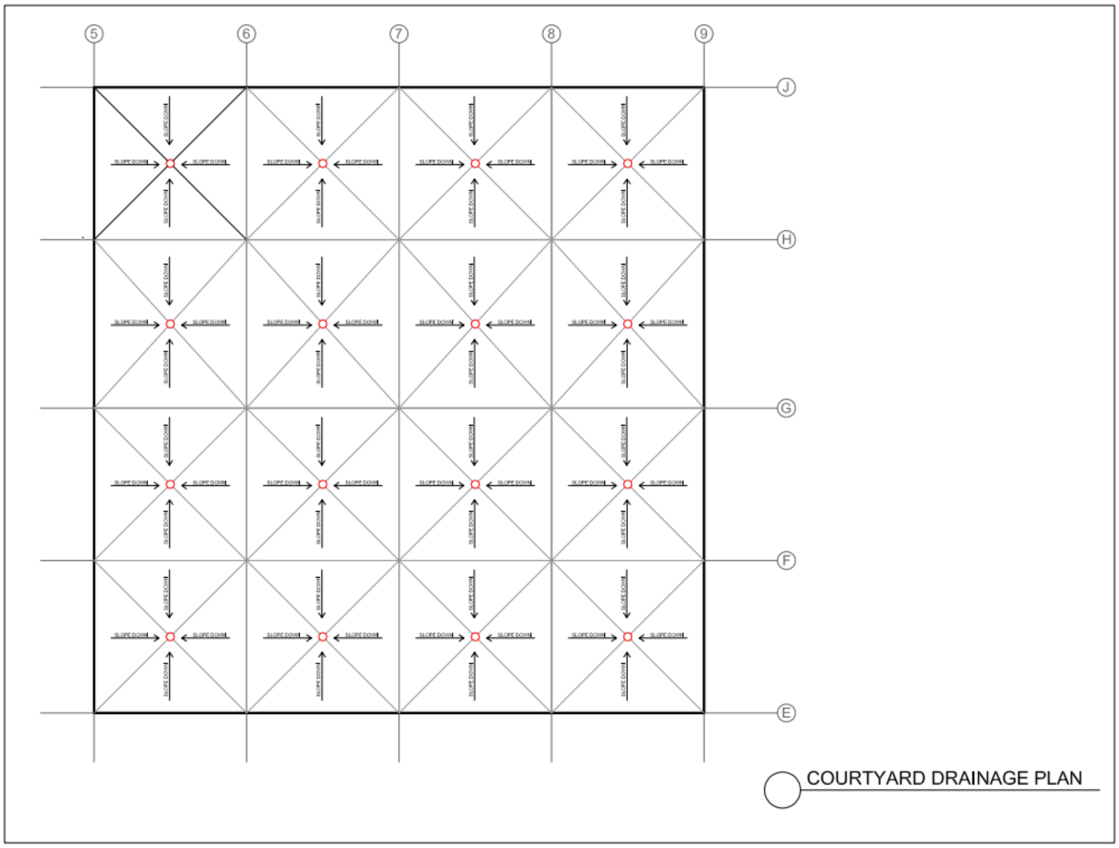
Enclosures Breadth

The watertight enclosure of the main roof level and the green roof courtyard are examined in the following section, with a heavier emphasis on the green roof courtyard due to time constraints.

The waterproofing system within the courtyard and on the main roof were changed due to the change in the courtyard's shape. With the new courtyard shape, a new drainage plan had to be made, which lead to an exploration of different waterproofing membranes. After creating a new drainage plan for the courtyard area, different waterproofing membrane manufacturers were researched to determine the best fit for the project. The manufacturers' cost, relative accessibility of the product, application process, and membrane material properties were compared. Once a manufacturer was chosen, water testing methods were selected. Different test methods were researched and selected based on cost, time, and feasibility of testing. Eventually, two test methods were selected, with the roof membrane and the courtyard membrane each requiring a different type of test.

Courtyard Drainage Plan

The location of the drains on the green roof courtyard was changed with the geometry of the space. Each drain now serves a square area of 1444 square feet. Though the drainage plan was changed to better suit the geometry of the space, drains will be tied into the existing drainage system. The building's drainage system was not changed and investigation of the system was determined to be outside the scope of this report.



Membrane Manufacturer Comparison

The following waterproofing membranes were considered for use based on the list of approved membrane manufacturers in Section 070413 of the project specifications.

American Hydrotech: MM6125

Barret Company: ram-Tough 250

Tremco: TREMproof 6100 (previously called TREMproof 150)

American Hydrotech MM6125

Monolithic Membrane 6125 by American Hydrotech is a thermoplastic, self-healing membrane made of asphalts and synthetic rubbers. It can be applied to plazas, roofs, and planters, making it a very reasonable choice for the courtyard and main roof of the Corporate Headquarters. The product has not experienced a material failure in 50 years. The membrane is installed at 215 mils thick, which assists in its self-healing properties. MM6125 can either be installed as a fabric reinforced assembly or as standard assembly.

The product's technical data, which can be found in Appendix K, was the most detailed of any of the three choices, showing that MM6125 performed extremely well under water resistance testing, heat stability tests, fertilizer resistance tests, and animal waste resistance over a three year period, in addition to meeting or exceeded the test requirements of many other fields.

The fertilizer resistance tests were conducted similar to ASTM D896: Standard Practice for Resistance of Adhesive Bonds to Chemical Reagents. The test was modeled after ASTM D896 since the fertilizer tested was undiluted 15/5/5 nitrogen/phosphorus/potash. At the conclusion of testing, there was no delamination, blistering, emulsification, or deterioration of the material, making it a great choice for the courtyard level, where fertilizer will most likely be used in each planter and garden space.

Hydrotech requires that MM6125 be applied by a trained and authorized Hydrotech applicator, and the product is not sold through a distributor but rather direct through the company. These factors make the product more expensive and harder to get to the job site since the distributor is not local to the project site and authorized Hyrdotech applicators typically charge a higher installation rate than traditional applicators. This higher cost can be justified by the product's reputation of 50 years with no material failure.

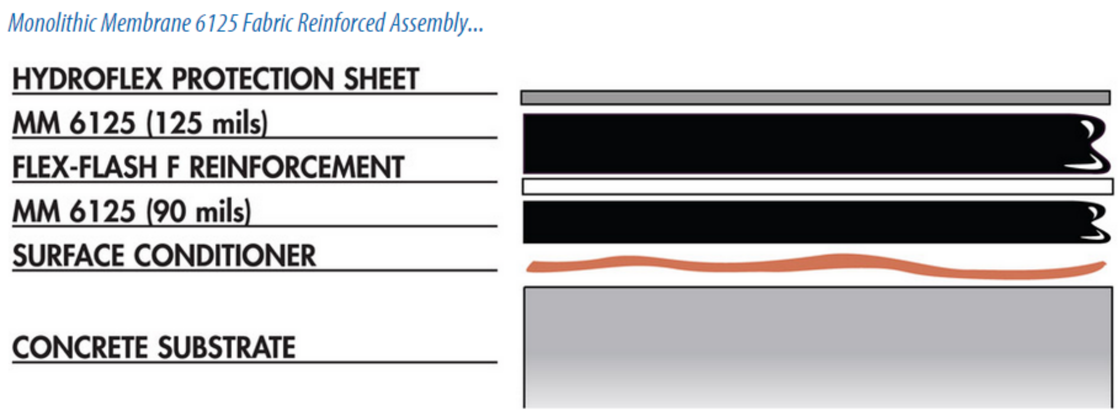


FIGURE 22-MM6125 FABRIC REINFORCED ASSEMBLY

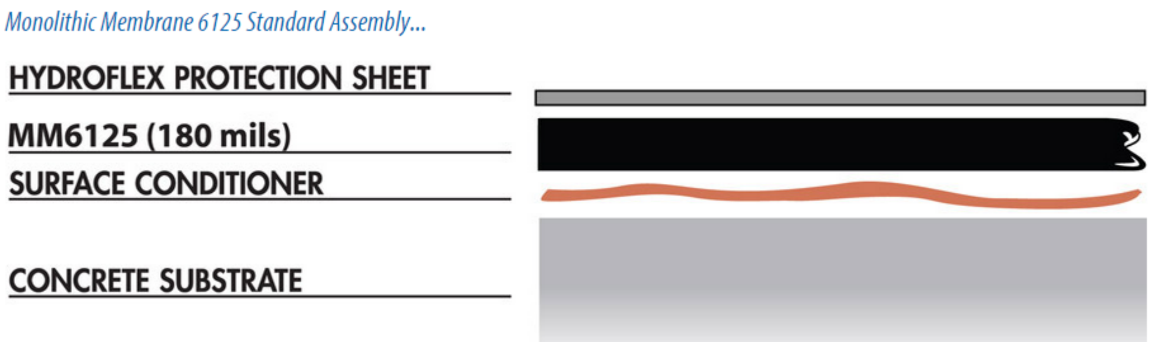


FIGURE 23-MM6125 STANDARD ASSEMBLY

Product Installation

The membrane application process first begins by melting the rubberized asphalt at a temperature between 350°F and 400°F. Appropriate rubber melters include air jacketed, oil-bath melters. A thin layer of surface conditioner is then applied to the concrete slab before laying down the membrane material. Each construction joint, control joint, and crack are sealed with 125 mil of the asphalt material. Using a squeegee tool, the MM6125 hot rubberized asphalt is applied to the remainder of the surface. In the standard assembly, the continuous membrane is applied at 180 mils with a minimum thickness of 125 mils (Figure 23). In the fabric reinforced assembly, an initial layer of the material is laid at 90 mil. While that layer is still warm and tacky, a thin layer of fabric reinforcing is laid down into the membrane. Above the fabric another layer of MM6125 is applied with a minimum thickness of 125 mils (Figure 22).

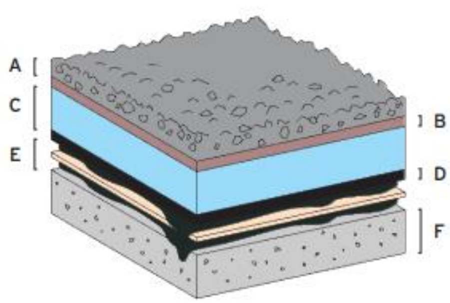
Barrett Roofs ram-Tough 250

Similar to the MM6125 membrane, the ram-Tough 250 is made of thermoplastic rubberized asphalt and has self-healing properties. Unlike the MM6125, the asphalt in this membrane is made of mineral filler and recycled tire rubber, making it a more environmentally friendly choice. The product can either be applied as a single or double membrane and is reinforced with neoprene flashing and polyester. The membrane is 215 mil thick and sets instantly.

Though there were fewer tests conducted on the ram-Tough 250 than the MM6125, this membrane passed each test it was subjected to. The summary of these tests can be found in the product specifications in Appendix K. Additionally, it has a substantially higher flash point and a slightly higher softening point than the MM6125. Though it is unlikely that the membrane would ignite, the large size of the building and its maximum number of occupants makes fire safety a primary concern. With a flashpoint of nearly 620 °F, the membrane would be difficult to ignite.

In addition to having a high flashpoint, the ram-Tough 250 has adhesion properties of 20% higher than the standard passing rate. The material is suitable for use in plaza deck waterproofing, greenroof applications, and protected membrane roofs, making it a good choice for the courtyard level. The product costs $35-$40 per sq. ft for standard installation.

Prior to installing the single membrane (SM) system, the asphalt mix is melted in an air jacketed melter between 375°F and 400°F. Next, the underlying concrete slab is checked for cracks, cold joints, expansion joints and construction joints. Cracks and joints are then primed using a primer/surface conditioner prior to membrane installation. Once these areas are conditioned, the remainder of the concrete surface is treated with primer. Using a Hudson type garden spray, the surface conditioner is applied at a rate of 200-600 square feet per gallon. Once the surface is completely dry, application of the ram-Tough 250 membrane can begin. Using a roller map or squeegee, the melted asphalt is spread over the surface. The material shall have an average thickness of 180 mils with a minimum thickness of 125 mils. During the application process, the material's adhesion and thickness shall be tested once per hour.



CROSS-SECTION

A. Ballast
B. Filter Fabric
C. Extruded Insulation
D. Protection Course
E. Fluid-applied RAM-Tough 250 (2) and Polyester Reinforcement
F. Primed Substrate

FIGURE 24-RAM TOUCH 250 CROSS SECTION WITH INSULATION, FILTER FABRIC, AND BALLAST APPLIED

Product Installation

The installation process for the double membrane (DM) system follows nearly the same procedure as the SM system. The systems differ because the double membrane system has a layer of Poly-Felt 125 VP reinforcement roll fabric between two layers of ram-Tough 250. The layer is 125 mils below the top of the membrane and 90 mils above the primed concrete.

Tremco TREMproof 6100

TREMproof 6100 (TP 6100), which replaced Tremco's TREMproof 150, is a hot applied, rubberized asphalt waterproofing membrane. The product is best used in horizontal waterproofing applications such as plaza decks and roof decks. Though it is extremely similar to the other two membranes studied, it is unique in that it may only be applied as a multi-layered, fabric-reinforced assembly (Figure 25). The material has a total nominal thickness of 215 mils.

The material performed similarly to its competitors when subjected to the same ASTM and CGSB tests (see Appendix K for physical properties), having a higher flashpoint than the MM6125 but a lower flashpoint than the ram-Tough 250. Additionally, the material performed well under a pinholing test that was not conducted on the other two membranes. The TP 6100 did not exhibit any pinholes when prodded during testing.

This material was previously considered the heavy favorite for the waterproofing membrane due to the manufacturer's close proximity to the project site. The manufacturing plant is less than 30 miles from the project site, and the owners of the Corporate Headquarters have always appreciated supporting local business. Upon further investigation of the product, it was found that special permissions from the manufacturer are required if the membrane is to be applied over the top of lightweight concrete. Though the main roof uses normal weight concrete, the courtyard level uses both normal weight and lightweight concrete. If the material were to only be used in the normal weight concrete sections of the courtyard slab, then the product would be forced to have seams and would lose its monolithic quality. For this reason, the product was deemed unsuitable to be the waterproofing choice on this project.

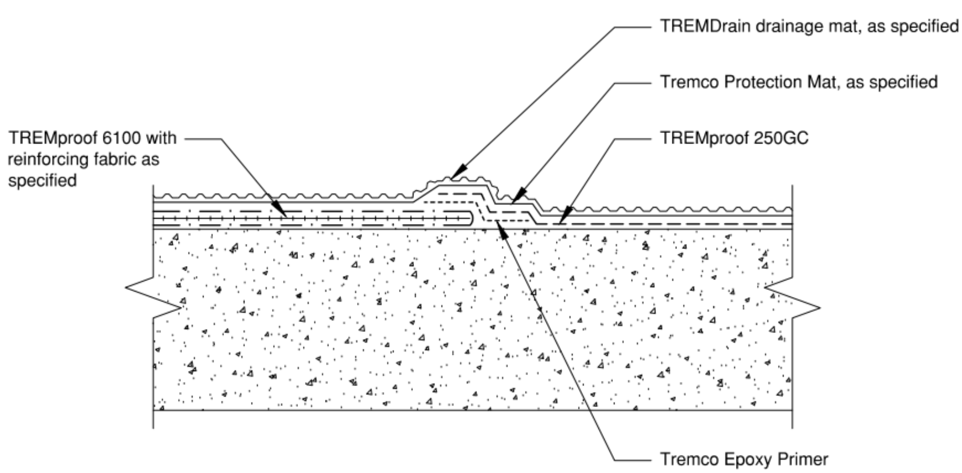


FIGURE 25-TREMPROOF 6100 CROSS SECTION

Product Selection

After careful consideration, it was decided that American Hydrotech's MM6125 would be the best type of membrane for making the courtyard and main roof level watertight. This membrane was chosen for its excellent reputation and performance during testing. One test that was instrumental in the selection of this membrane was the animal waste resistance test. Though the courtyard level will not be exposed to animal waste due to the insulation, filter fabric, and ballast above the membrane, the main roof level will leave the membrane exposed to the elements. The membrane also performed well during water resistance testing, which is a critical concern in an area where ponding water may occur.

The MM6125 membrane will be installed as instructed above, and the melter used to heat the material will be the A-380 from A&A Melters. The specifications for this product may be found in Appendix L. This melter was chosen due to its large capacity, its ability to quickly heat material, and its efficiency. The A-380 has been approved for use by the American Hydrotech corporation.

Water Testing

In order to test the adequacy of the waterproofing material, two different water testing methods will be used. A flood test will be conducted on the courtyard level after the installation of the membrane and a leakage test will be conducted on a section of the main roof waterproofing membrane prior to installation.

ASTM D5957-98

The courtyard flood test will be performed under the guidelines set forth in ASTM D5957-98: Standard Guide for Flood Testing Horizontal Waterproofing Installations. This test is suitable because it is intended for use in areas that are over habituated spaces, just as the courtyard lies over office and retail space.

Testing Procedure

In order to have a successful water test, each drain in the testing area must be plugged using an approved drain plug. Following drain plugging, a temporary containment device must be constructed. Per ASTM D5957-98, there are four different containment assemblies that may be used. For the purpose and ease of this test, containment assembly option number 4 shall be used and constructed as illustrated below (Figure 26). This particular assembly appears the easiest to construct due to the type of materials used.

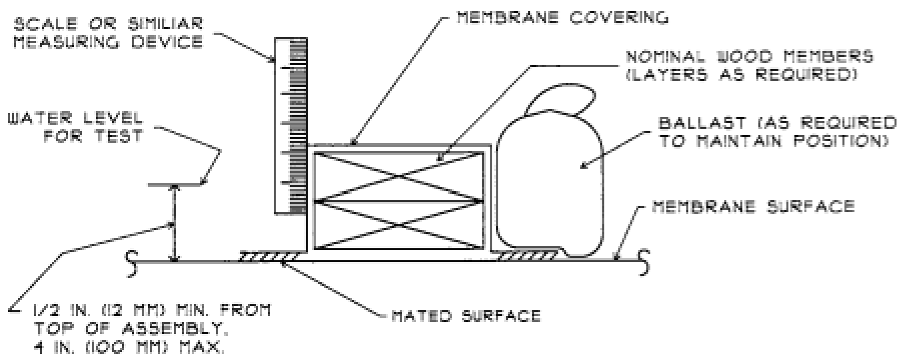


FIG. 4 Containment Assembly—Option No. 4

FIGURE 26-ASTM D5957-98 CONTAINMENT OPTION 4

Following the construction of the containment assembly, potable water shall be added to the contained area using hoses. The depth of the water should be a minimum of 1” and a maximum of 4”. Water depth cannot be within 2” of the top of the upturned flashing. Once the desired depth of water is achieved, the test may begin. Water shall be left in the flooded area for a minimum of 24 hours and a maximum time of 72 hours, making sure that there is someone there to constantly monitor the apparatus. Observed conditions below the water level must be documented every four hours until the test is completed. If there is observed leakage in the waterproofing membrane at any point during the testing interval, the test must be stopped, water must be drained from the area, and the leak point must be repaired.

At the completion of the test, the ponded water shall be removed from the area by slowing removing the drain plugs. If the plugs are removed too quickly, the drainage system may be damaged. If there are no visible leaks then the membrane and there are no visible blisters or other deformations, the testing is complete. The final step in the flood test is writing a detailed report of the test procedure and the results.

ASTM D7281-07

The main roof waterproofing membrane shall be tested in accordance with ASTM D7281-07: Standard Test Method for Determining Water Migration Resistance Through Roof Membranes. This test was designed to assist in determining water migration in built up built-up or single ply roof membranes. It is meant to simulate both ponding water on a roof membrane and the deterioration caused by the sun's UV rays.

Testing Procedure

The first step in this testing procedure is to construct the leakage test apparatus (Figure 27). Once the apparatus has been constructed, a 2'x4' piece of the roofing membrane is selected. Due to the membrane's monolithic quality, there will be no field seams present in the material and therefore they do not need to be used in a 2'x4' sample of membrane. The material sample will be conditioned for 1000 hours in a fluorescent UV condensation weather apparatus, as outlined in Practice G154[1]. After the required time in the weathering apparatus, the sample shall be inspected for signs of distress and damage. The sample is then to be placed in the leakage test apparatus in between the two flexible foam gaskets, which are above the support plate. At that point, a 6" of water is applied to the sample for 7 days. After the 7 days, pressurized air (6.9 kPa) is introduced into the bottom portion of the leakage test apparatus, and then immediately released. This process of imputing and releasing air is repeated 25 times. At the end of the 25 cycles, the sample is inspected for water leakage and detailed report is written.

[1]ASTM Practice G154-12a: Standard Practice for Operating Fluorescent Ultraviolet (UV) Lamp Apparatus for Exposure of Nonmeta

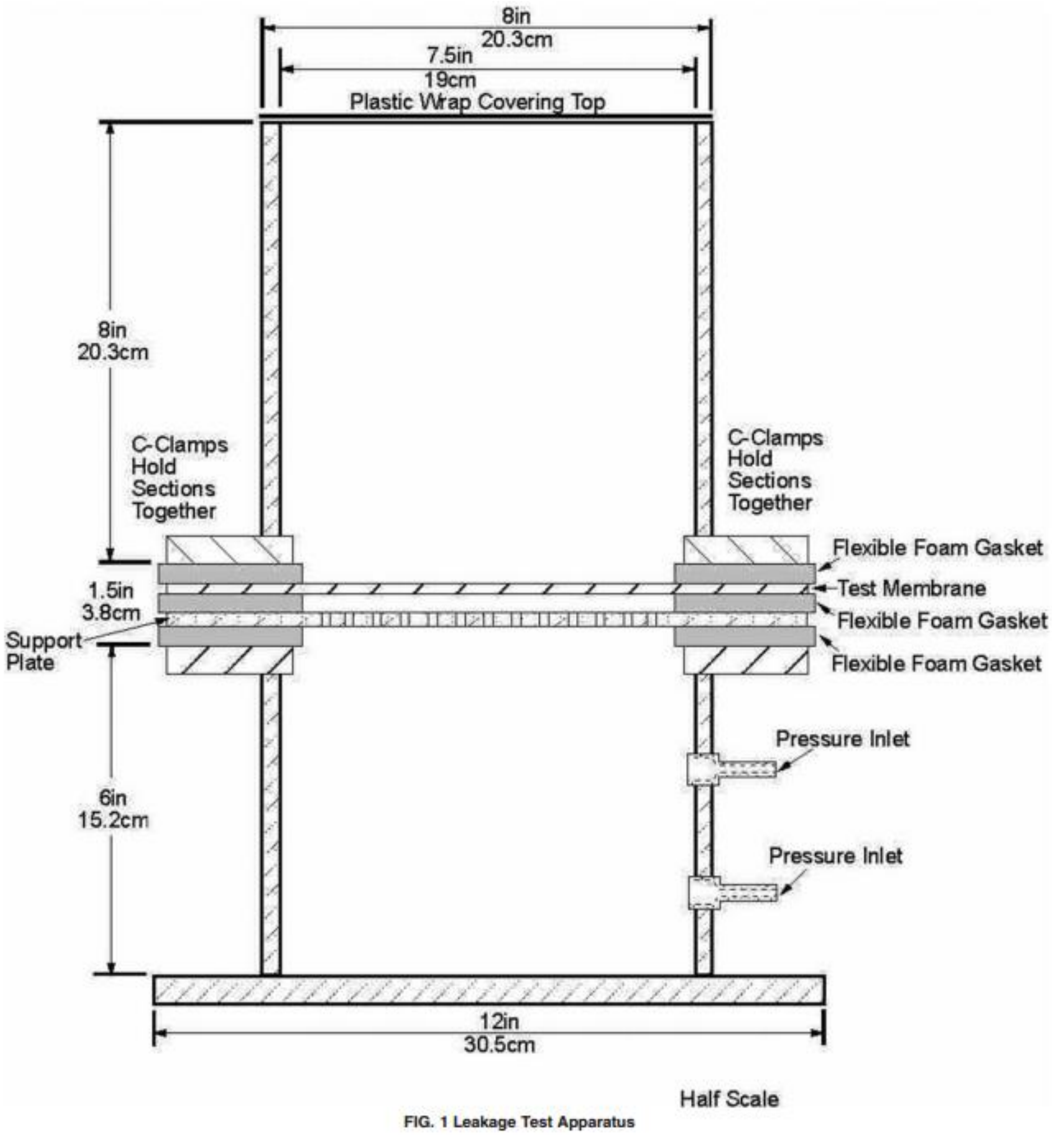


FIGURE 27-ASTM D7281-07 LEAKAGE TEST APPARATUS

System Comparison

After completing a gravity and lateral redesign of the Corporate Headquarters, comparison between the existing and proposed system was conducted. It was noted that the overall weight of the building decreased as a result of the redesign. The total seismic weight in the existing system was 57,235 kips while the weight of the redesigned system was 39,683 kips. In addition to the decrease in building weight, the story drifts for each lateral loading condition decreased. For these reasons, the redesigned gravity and lateral systems of the corporate headquarters may be considered as viable design alternatives for the building.

Conclusion

This report included an overview of redesign and analysis of the Corporate Headquarters. The building's existing gravity and lateral systems were analyzed during the fall semester of this course and found to be adequate to meet the needs of the building. A fictitious scenario was created in which the building owner requested more office space. In order to meet this request, and to help simply the design process, the geometry of the interior courtyard green roof was changed into a rectangle. The change in the shape of the space lead to a redesign of the area, a change in the roofing materials, and thusly, a change in loading.

To accommodate the loading change and change in shape of the courtyard, new structural plans were created for the space and a new gravity system was designed. The proposed gravity system uses long span steel joists and joist girders which were designed in RAM Structural System and verified using spot checks and information from the Vulcraft joist catalog. Steel gravity columns were resized in RAM Column in accordance with the new loading conditions and verified using spot checks. Floor vibrations due to walking were a concern with this system since steel joists have a history of poor performance in this field. In order to ensure that vibrations wouldn't be an issue in the space, a calculation was completed using Design Guide 11. It was found that the proposed system is suitable under vibration standards

The gravity system of the building was changed from eight steel braced frames into eight reinforced concrete shear walls. The shear walls were placed in the same locations as the existing braced frames for architectural purposes. Seismic loading on the building changed due to the changes in the courtyard area, and the new forces were used to design the reinforcement of the shear walls. The walls were sized and designed using RAM Concrete and RAM Frame, and sizes were verified using spot checks.

Following the lateral redesign, the courtyard green roof (breadth one) was redesigned. Though the gravity system of this space was designed at the same time as the gravity system for the rest of the building, this system had different loads and therefore required the use of steel beams and girders rather than a joist and joist girder system. A new layout for the space was created and a new planting pattern was developed that highlighted plants local to the building location. The local plants are featured in a focal garden in the middle of the space. Other materials such as new engineered fill and new concrete pavers were also selected for the space.

Finally, the watertight enclosure of the courtyard green roof and the main roof level were redesigned. This served as the second breadth topic. First, the drainage plan of the courtyard level was changed. Then, different waterproofing manufacturers were compared before one was selected, and different application techniques were researched. The new waterproofing membrane was used on both the courtyard and main roof level. Water tests were researched in order to test the watertight barrier of the membrane. The roof membrane required a different test than the courtyard membrane as the roof membrane will be exposed to the element and the courtyard membrane will be covered in by the green roof assembly.

Redesigning the gravity and lateral system of this building, as well as having an opportunity to change the courtyard green roof and watertight enclosure, was a wonderful learning experience. It was extremely beneficial to see how certain decisions could impact the entire design process and I am grateful that I got to explore areas that I am interested in working in in the future.

Resources

ASCE 7-05: Minimum Design Loads for Buildings

International Building Code 2009

AISC Steel Construction Manual, Fourteenth Edition

ACI 318-11: Building Code Requirements for Structural Concrete and Commentary

AISC Design Guide 11: Floor Vibrations Due to Human Activity

Vulcraft Steel Joists and Joist Girders Catalog

Vulcraft Deck Catalog

ASTM D5957-98: Standard Guide for Flood Testing Horizontal Waterproofing Installations

ASTM D7281-07: Standard Test Method for Determining Water Mitigation Resistance Through Roof Membranes

American Hydrotech Product Specifications

Barrett Roof Product Specifications

Tremco Product Specifications

Virtual Herbarium- The Native Plant Society of Northeastern Ohio

Anchorblock Product Specifications

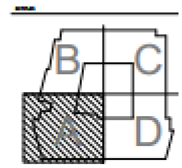
A&A Melters Product Specifications

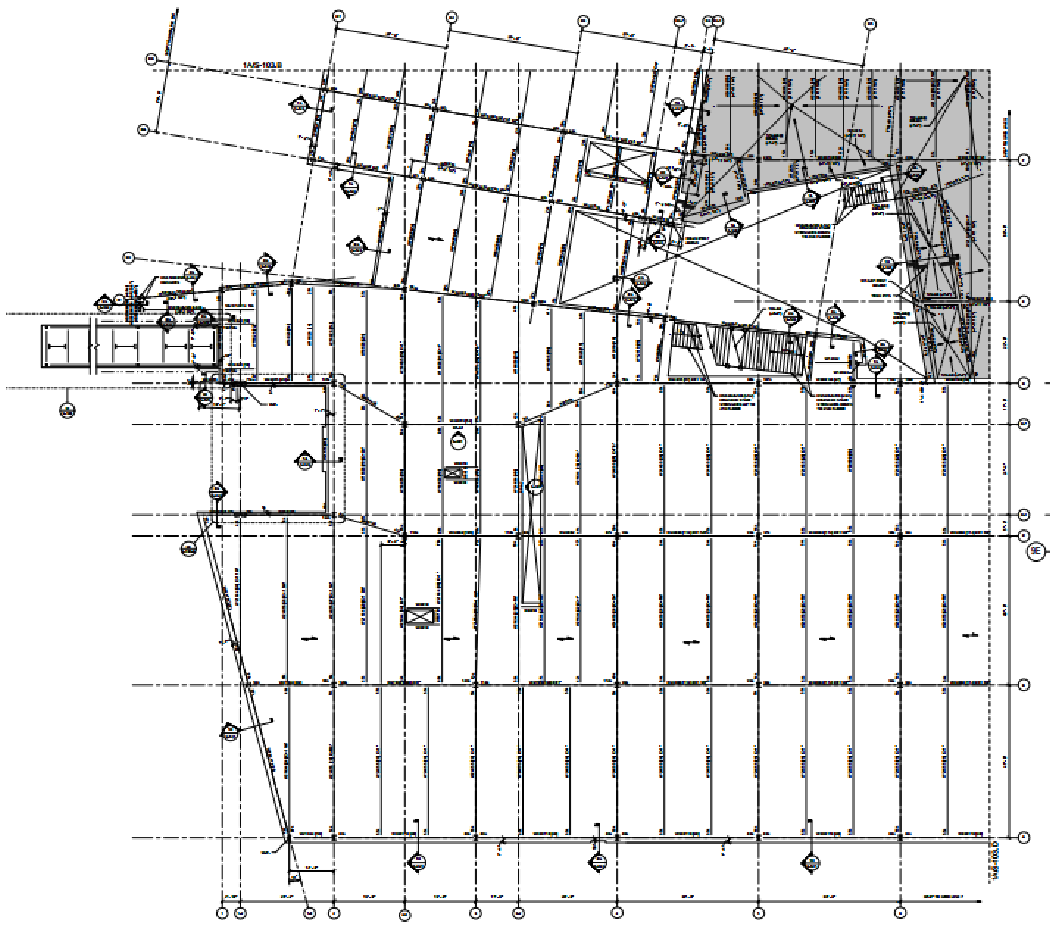
Appendices

Appendix A

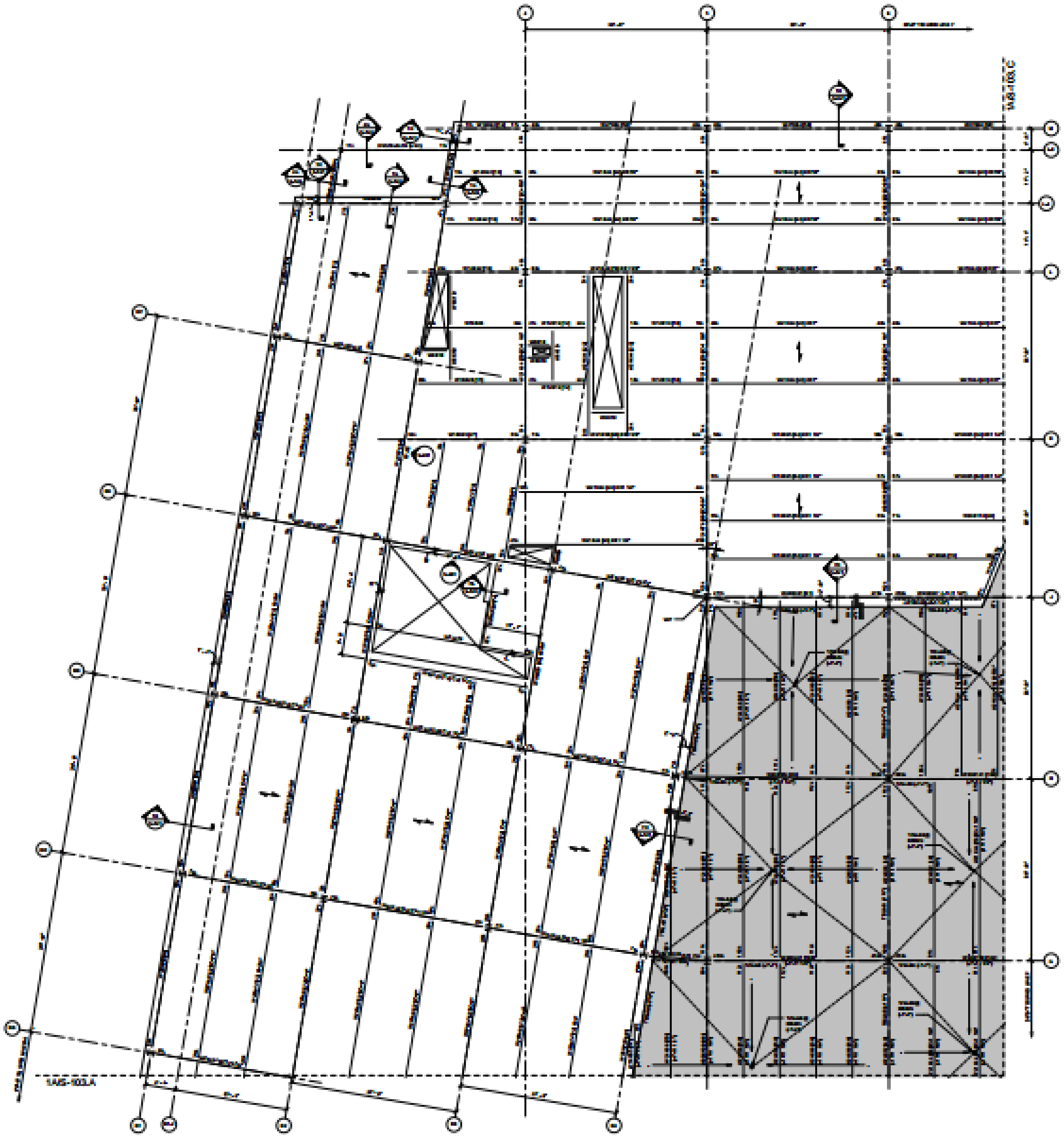
Sample Existing Building Floor Plans and Elevations

Building Key Plan

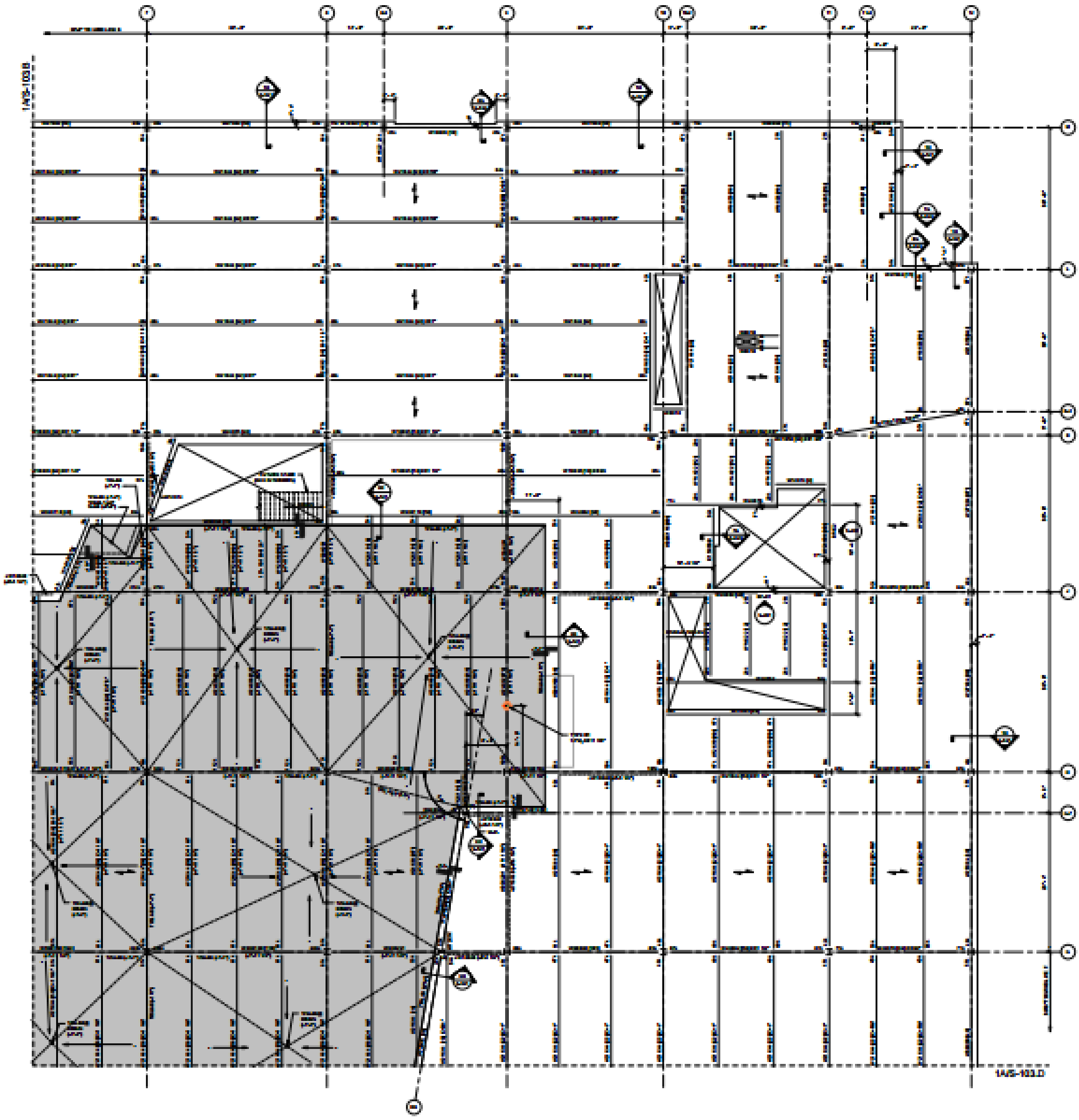




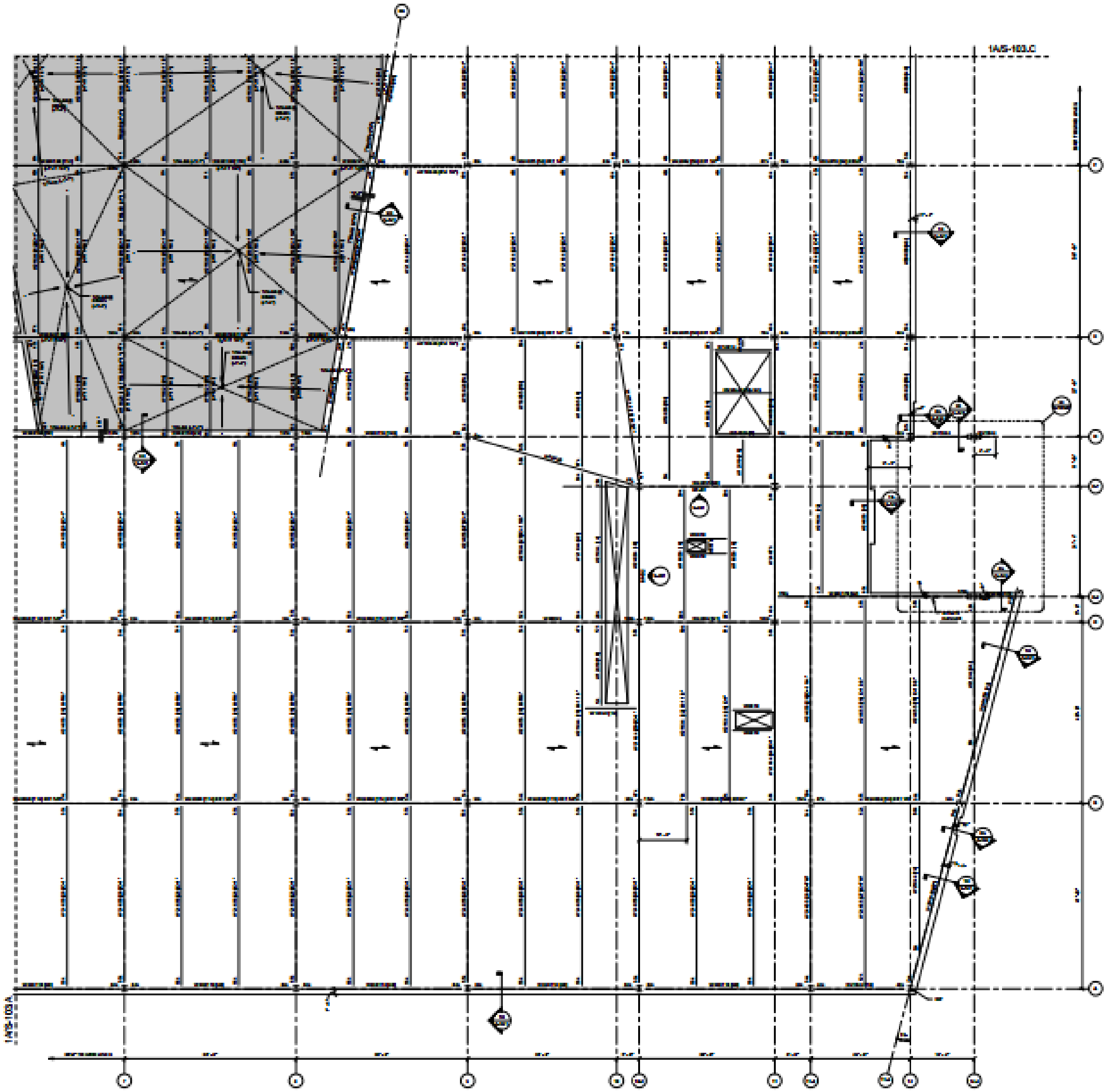
Area A Third Floor Framing



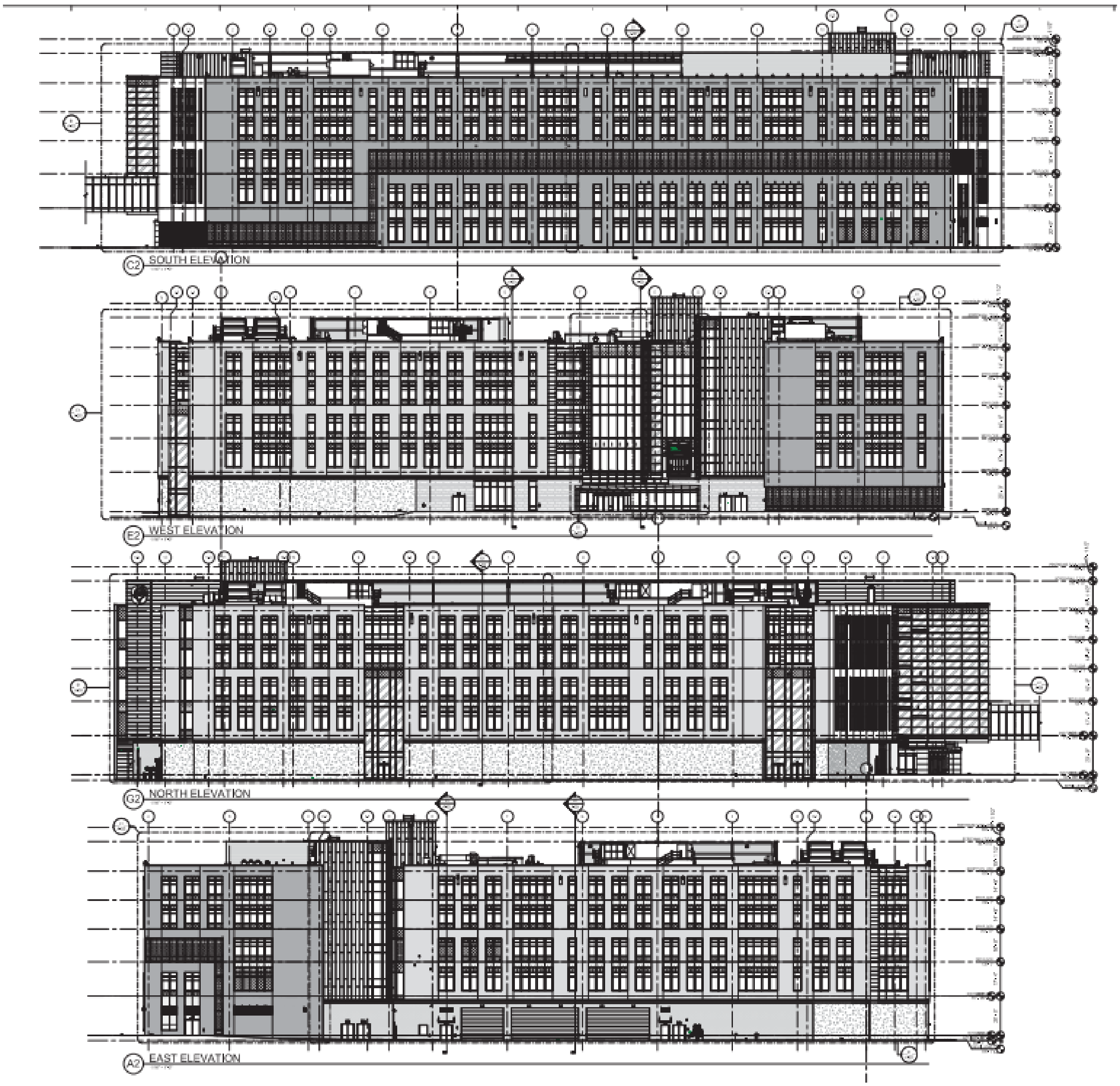
Area B Third Floor Framing



Area C Third Floor Framing



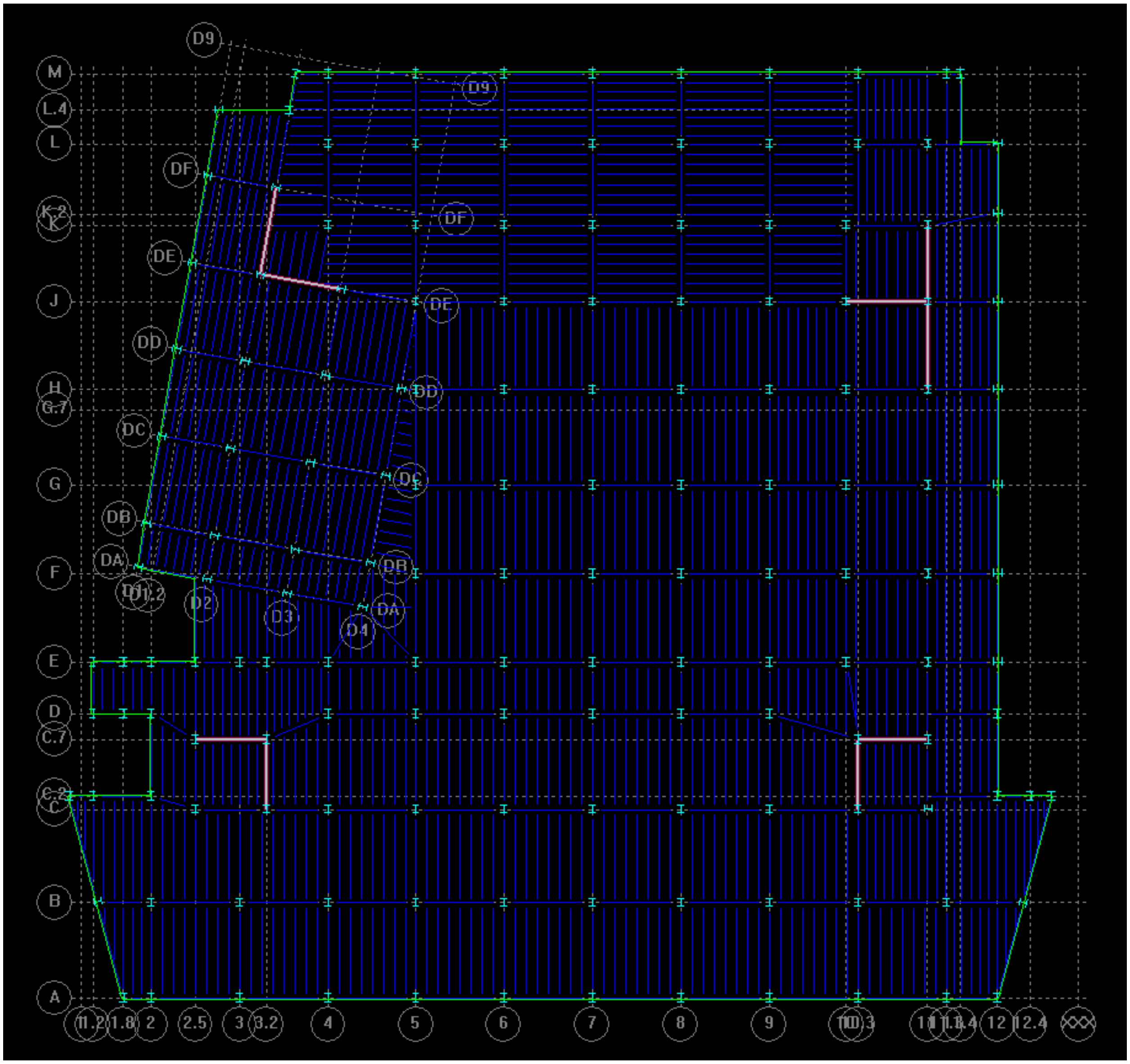
Area D Third Floor Framing



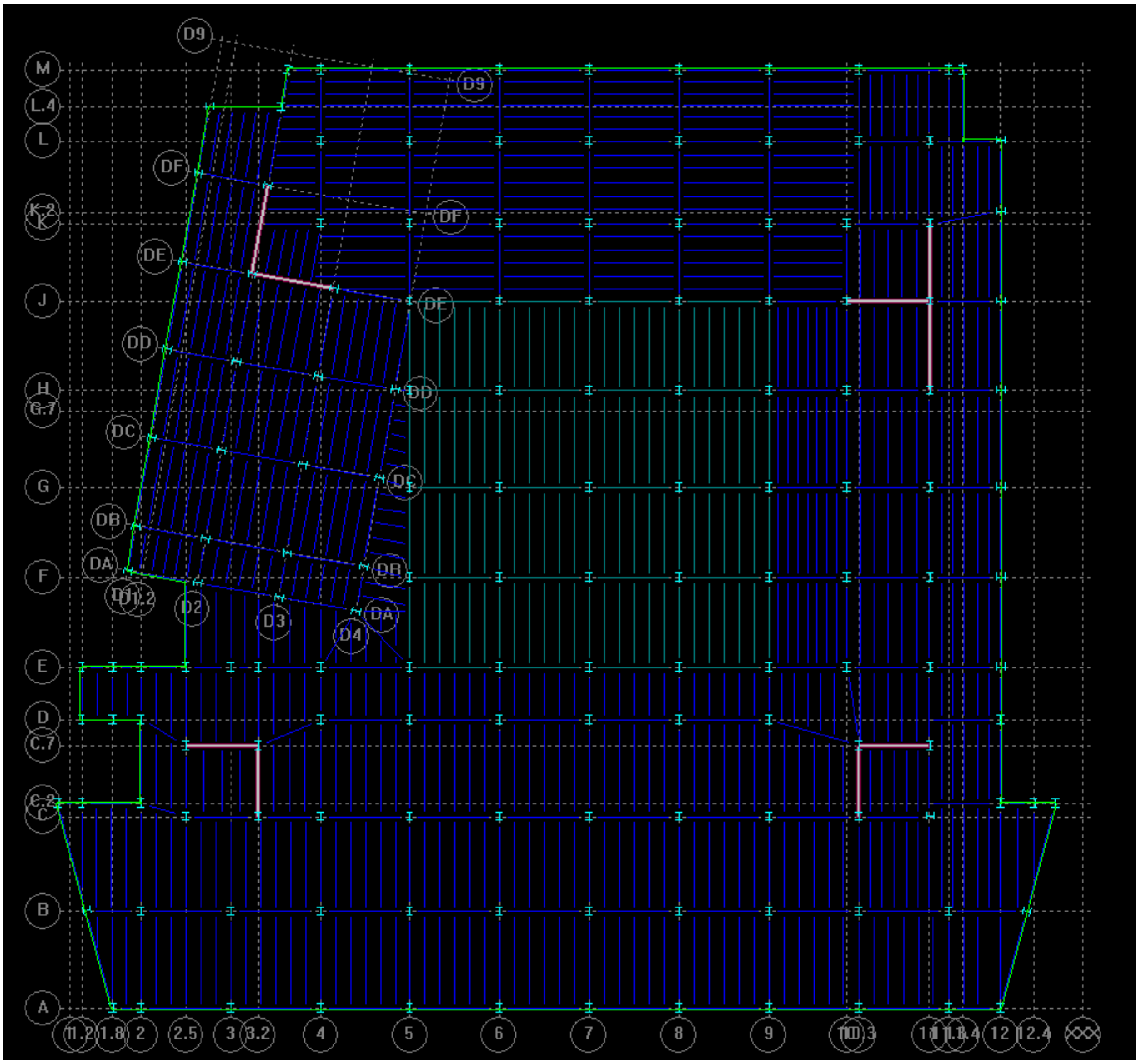
Building Elevations

Appendix B

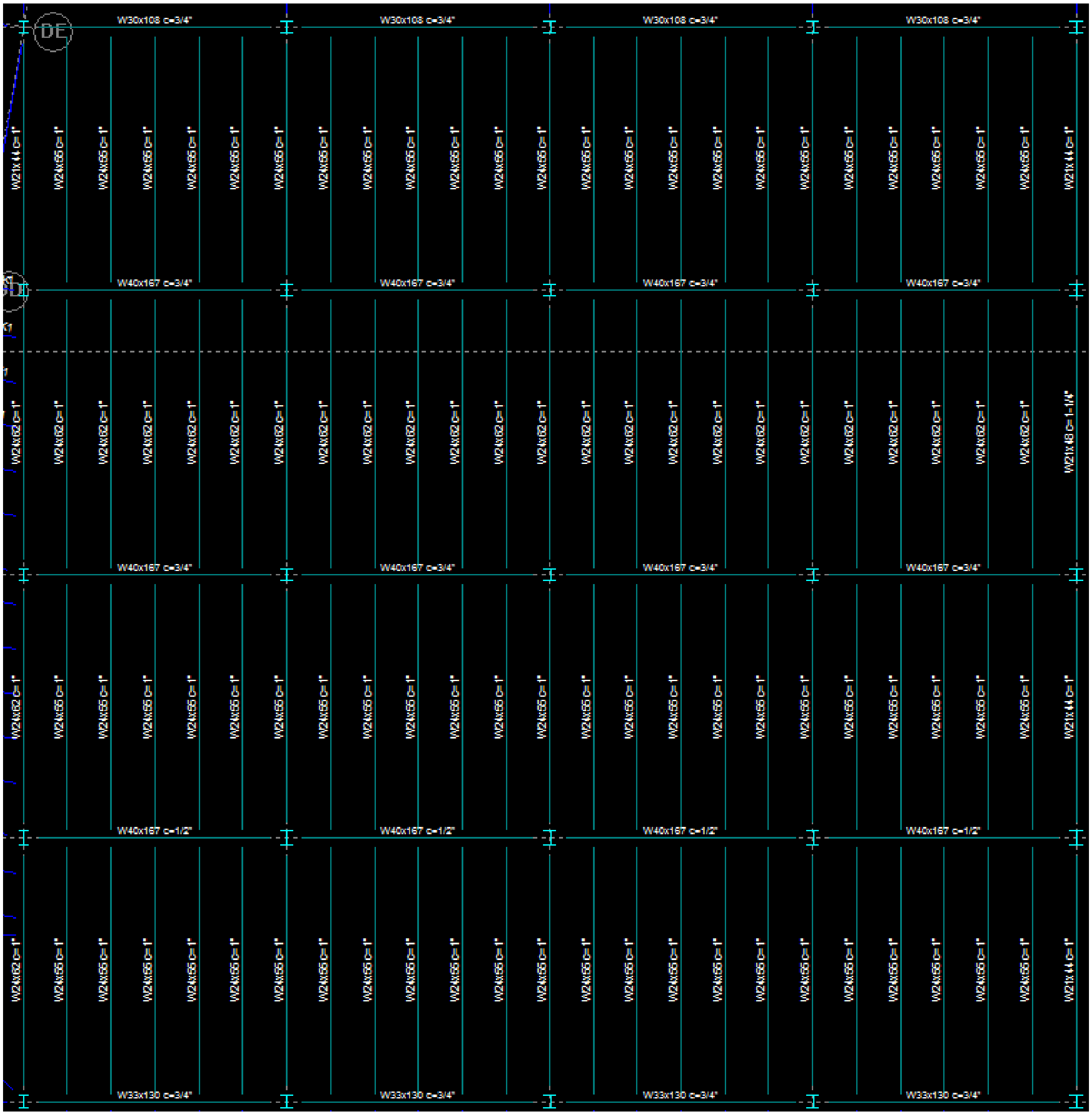
Redesign Structural Framing Plans



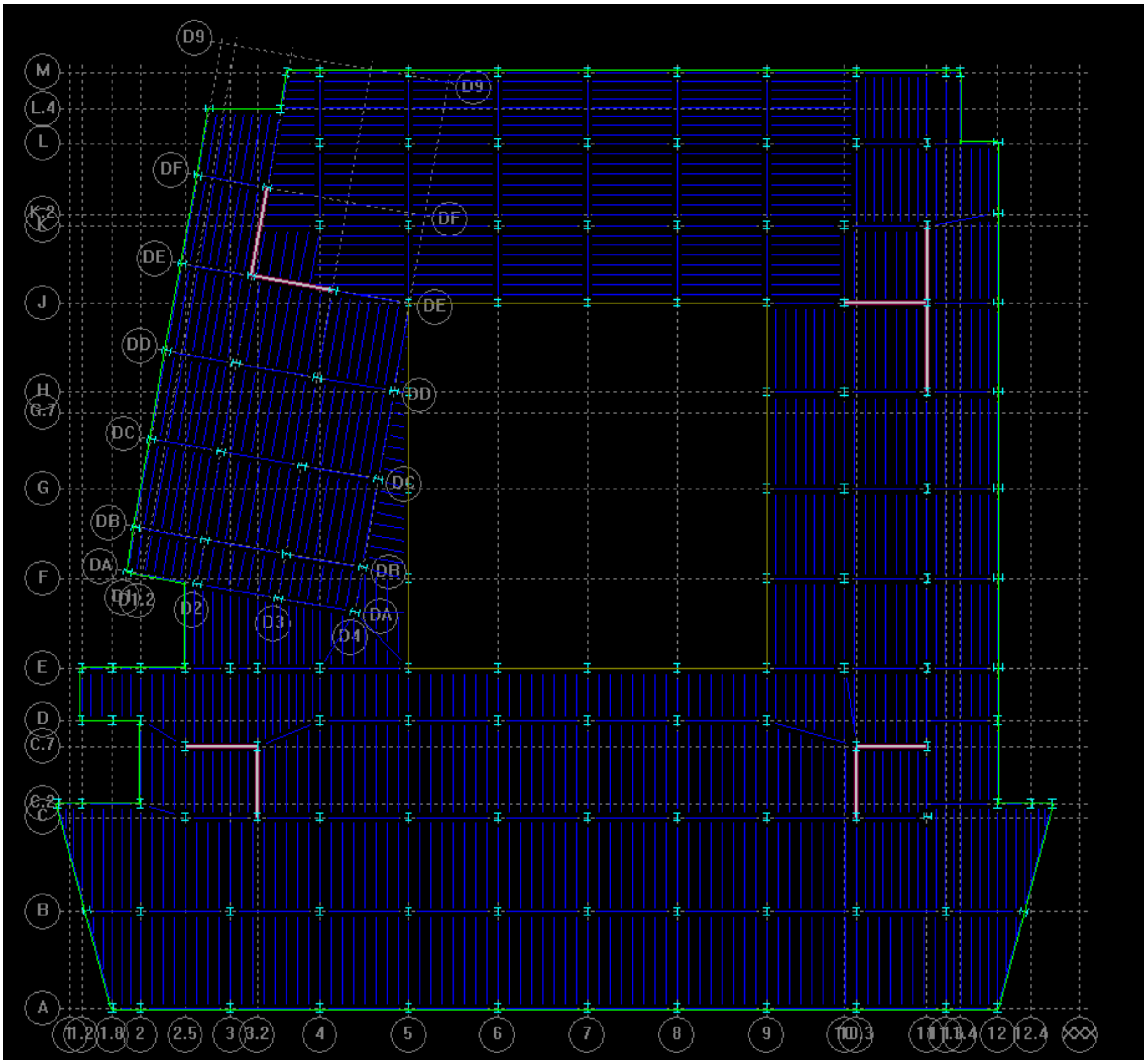
Level 2 Framing Plan



Level 3 (Courtyard Level) Framing Plan



Courtyard Green Roof Framing Plan, sample bay size shown in Figure 20 (clearer member sizes)



Level 4 and 5 Framing Plan (roof framing plan looks identical in plan view)

Appendix C

Gravity Loading Calculations

Gravity System | Roof Deck Calculation | 1/1

Typical Bay Loading

Live Load = 20 psf per ASCE 7-05 - Table 4-1

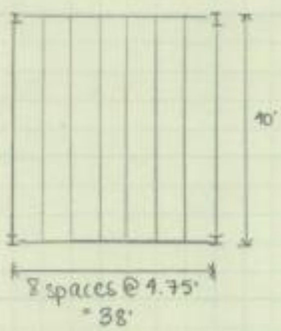
Dead Load

adhered membrane = 1 psf
1/4" roof board = 1.2 psf
insulation = 3 psf
vapor retarder = 1 psf
ceilings = 2 psf
MEP = 10 psf
sprinklers = 3 psf
framing = 10 psf

31.2 ~31 psf

Total Load = 51 psf

Snow Load = 17 psf   20 > 17, live load controls



Determine Deck Size

- try 4" NW concrete for fire rating
- use unshored construction

Try 1.5VL18 (same as floor deck for ease of constructability)

1.5VL18 max unshored clear span
1 span 7'-0"
2 span 9'-1"
3 span 9'-4"

Superimposed live load for 5'-0 span (closest to 4.75') = 400 psf > 51 ✓

Total deck weight

$(4" \times \frac{1}{12})(150\,pcf) = 50$ psf
deck SW = 2.82 psf

Total Deck Weight = 52.82 ~53 psf

Use 1.5VL18 w/ 4" Normal Weight Topping

Total Roof DL = 31 + 53 = 84 psf

Gravity System | Floor Deck Calculation | 1/1

Typical Bay Loading

Live Load

office = 50 psf
partitions = 15 psf

65 psf

Live Load Reduction

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$

$= 65\left(.25 + \frac{15}{\sqrt{(2)(38)(40)}}\right)$

$L = 41$ psf

Dead Load
Framing = 5 psf
ceiling = 2 psf
flooring = 3 psf
MEP = 5 psf
sprinklers = 3 psf

18 psf

TL = 41 + 18 = 59 psf

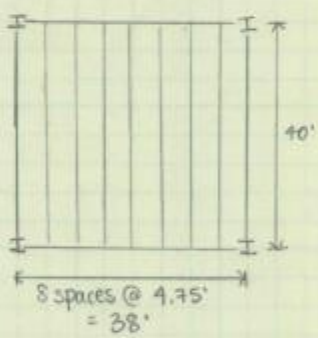
Courtyard Area Loading

Live Load

assembly area = 100 psf
↳ unreducable

Dead Load
MEP = 5 psf
Ceiling = 2 psf
Sprinklers = 3 psf
Framing = 5 psf
plantings | pavers = 80 psf
Adhered Membrane = 1 psf
Insulation = 3 psf
Vapor Retarder = 1 psf
TL = 200 psf

*ideally, find a deck to suit both loading patterns



Determine Deck Size

- need 3 1/4" LW conc for 2 hr fire rating
- use unshored construction for economy

Try 1.5VLR18 w/ 3.25" topping
max unshored clear spans
1 span = 8'-2"
2 span = 10'-1"
3 span = 10'-5"
} > 4.75' ∴ ok
use 2 span condition

superimposed Live Load for 5' span = 400 psf
400 > 200 > 59

Total Deck weight

$(3.25" \times \frac{1}{12}) \times 115$ pcf = 31.15 psf
deck SW = 2.82 psf

Total Deck weight = 34 psf

Total Dead Weight = 52 psf

Use 1.5VLR18 w/ 3.25" LW concrete

M. Julia Haverty | Gravity Loads | Tech Report 2

SNOW LOAD CALCULATIONS

Flat Roof Snow Load, p_f

$p_f = 0.7 C_e C_t I p_g$ (ASCE 7-05 Eq. 7-1)

From S-001
- $p_g = 20$ psf
- occupancy category II
- $I = 1.1$
- $C_e = 1.0$
- $C_t = 1.1$
- exposure B

$p_f = 0.7(1.0)(1.1)(1.1)(20)$ $p_f = 16.94 \sim 17$ psf $\underline{p_f = 17 \text{ psf}}$

Snow Drift

- calculated for drift from mechanical penthouse roof
- windward snow drift

$h_b = \frac{p_s}{\gamma}$ in this case, $p_s = p_f = 17$ psf

$\gamma = 0.13 p_g + 14$ but cannot exceed 30 pcf
$= 0.13(20) + 14$
$\gamma = 16.6$

$h_b = \frac{17 \text{ psf}}{16.6 \text{ pcf}}$ $h_b = 1.02$ ft $h_c/h_b > 0.2 \rightarrow$ drift loads must be calculated

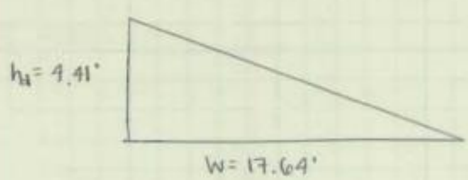
$h_c = 15'-1.5"$

$\ell_u = 394' \rightarrow$ from Figure 7-9, $h_d \sim 5.5$ ft

$h_d = 0.43 \sqrt[3]{\ell_u} \sqrt[4]{p_g + 10} - 1.5 = 0.43 \sqrt[3]{394} \sqrt[4]{20+10} - 1.5$
$h_d = 5.88'$
$3/4\, h_d = 4.41'$

$h_d > h_b$ $h_d < h_c \rightarrow w = 4h_d$ $w = 4(4.41)$ $w = 17.64'$

Drift Density $p_d = h_d \gamma$ $p_d = 4.41(16.6)$ $\underline{p_d = 73.21 \text{ psf}}$

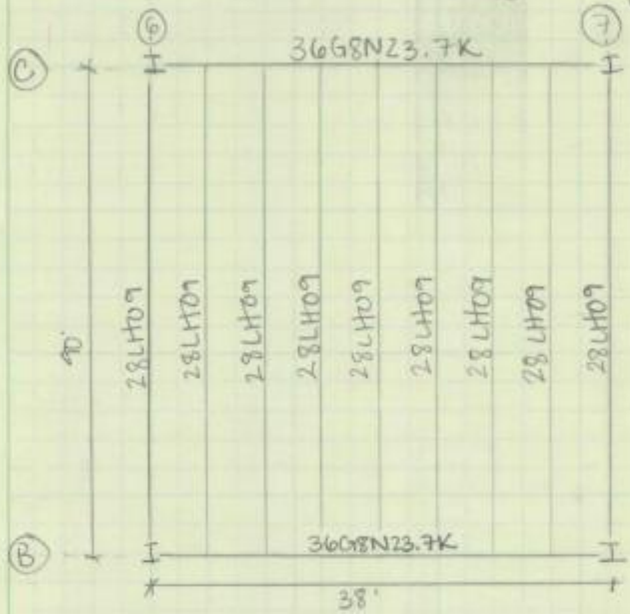


Appendix D

Gravity Member Spot Checks

Gravity System | Steel Joist Design | 1/3

Joist Spot Check (members designed using RAM)



From 4th floor plan

Dead Load

$3\frac{1}{4}$" LW conc slab $= 115\left(\frac{3.25}{12}\right) = 31.15$ psf
18 GA Composite Metal Deck = 2.8 ~ 3 psf
Ceiling = 5 psf
MEP = 10 psf
Sprinklers = 3 psf

DL = 52 psf + joist self wt

Live Load

offices = 50 psf
partitions = 15 psf
65 psf

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$ $K_{LL} = 1.0$

$= 65\left(.25 + \frac{15}{\sqrt{1(38)(40)}}\right)$ $L = 41$ plf

Average Joist Spacing = 4.75'
Average Joist Span = 40'

28LH09 = 21 plf
max load < 1232 plf

$w_{u+l} = (1.2(52) + 1.6(65))\,4.75 = 790.4 + 1.2\,SW$
$w_{tl} = (52 + 65)\,4.75 = 555.75$ plf + joist weight

From Vulcraft Catalog, 28LH09 @ 42' span (smallest dimension listed in table)

$w_{utl} = 790.4 + 1.2(21) = 815.6$ plf
$w_{tl} = 555.75 + 21 = 576.75$

w for L/360 = 428 plf
w for L/240 = 428 × 1.5 = 642 642 > 576.75 ✓

$\boxed{\text{use 28LH09 joists spaced at 4.75'}}$

Gravity System | Joist Girder Design | 2/3

Joist Girder Spot Check

36G8N23.7K
SW = 69 plf for 24K

38' x 40' Bays
8 Joist Spaces (N=8), 4.75' spacing

DL = 52 plf + 69 plf = 121 plf

LL = 41 plf (reduced

TL = 162 plf

4.75' (162 plf) = 770 plf

P = 770 x 40' tTrib width = 30.8 K , P given in Ram = 23.7K

Check Live Load Deflection (governs)
LL = 41 x 40' = 1640 plf = w = 1.64 klf = .137 k/in

Approximate Joist Girder Moment of Inertia

$I = 0.018NPLd$
$= 0.018(8)(23.7)(38)(36)$
$I = 4668.7 \text{ in}^4$

allowable deflection = $L/360 = 38(12)/360 = 1.27"$

deflection = $1.15\left[\frac{5wL^4}{384EI}\right] = 1.15\left[\frac{5 \times (0.137)(38 \times 12)^4}{384(29000)(4668.7)}\right] = 0.66"$

$0.66" < 1.27"$ ✓

$\boxed{\text{Use 36G8N23.7K Joist Girders}}$

Beam Deflection Summary

RAM Steel 14.06.01.00 Page 23/77
DataBase: RAM Model Gravity and Lateral March 29 04/06/15 20:14:04
Academic License. Not For Commercial Use.

Bm #	Beam Size	Dead	Live	Total
2346	26K9	0.660	0.531	1.191
2255	24LH06	0.946	0.762	1.708
2248	26K9	0.660	0.531	1.191
2331	28LH10	1.089	0.877	1.965
2354	24LH07	0.825	0.664	1.490
2200	28LH09	0.869	0.699	1.568
2207	28LH09	0.869	0.699	1.568
2214	28LH09	1.092	0.879	1.971
2221	28LH09	0.869	0.699	1.568
2228	24LH07	0.825	0.664	1.490
2326	28LH08	1.069	0.860	1.929
2347	26K9	0.660	0.531	1.191
2256	24LH06	0.946	0.762	1.708
2249	26K9	0.660	0.531	1.191
2332	28LH10	1.089	0.877	1.965
2355	24LH07	0.825	0.664	1.490
2201	28LH09	0.869	0.699	1.568
2208	28LH09	0.869	0.699	1.568
2215	28LH09	1.092	0.879	1.971
2222	28LH09	0.869	0.699	1.568
2229	24LH07	0.825	0.664	1.490
2327	28LH08	1.069	0.860	1.929
2348	26K9	0.660	0.531	1.191
2257	24LH06	0.946	0.762	1.708
2250	26K9	0.660	0.531	1.191
2333	28LH10	1.089	0.877	1.965
1437	28LH08	1.069	0.860	1.929
1383	24LH09	1.004	0.808	1.813
1384	24LH09	1.004	0.808	1.813
1396	28LH09	1.014	0.816	1.831
1431	24LH06	0.946	0.586	1.708
1433	24LH09	0.670	0.539	1.209
2334	28LH10	1.089	0.877	1.965
2314	32LH09	1.223	0.984	2.207
2307	24LH06	0.977	0.787	1.764
2300	22K4	0.405	0.326	0.731
2293	28LH07	0.977	0.787	1.764
2286	28LH07	0.977	0.787	1.764
2279	28LH09	0.936	0.754	1.690
2364	28LH07	0.977	0.787	1.764
2371	24LH06	0.823	0.510	1.487
2378	24LH06	0.888	0.550	1.604
2315	32LH09	1.223	0.984	2.207
2308	24LH06	0.977	0.787	1.764
2301	22K4	0.405	0.326	0.731

Sample of RAM Member Deflection Output

Gravity System | Column Design | 3/3

Column Spot Check

Main Roof

5	W14×68
4	W14×68
3	W14×132
2	W14×132
1	W14×132

Column 6B - Interior Column W14×132

Worst case loading from level 3, look at W14×132

$l_u = 16.67'$

From RAM output, $P_u = 710.19^k$

AISC Table 4-1: 16.67' unbraced length (check 16' & 17')

$KL = 16'$ $\phi P_n = 1440^k > 710^k$ ✓

$KL = 17'$ $\phi P_n = 1410^k > 710^k$ ✓

W14×132 is acceptable for design

Roof

5	W12×45
4	W12×45
3	W12×79
2	W12×79
1	W12×79

Column 8A - Exterior Column W12×79

$l_u = 16.67'$

From RAM output, $P_u = 371.75 = 372^k$

$M_{ux} = 7.48$ k-ft $M_{uy} = 7.16$ k-ft

Check Table 6-1 in AISC, exposed to shear & flexure

$KL = 16'$: $p = 1.28\times10^{-3}$ $b_x = 2.13\times10^{-3}$ $b_y = 3.51\times10^{-3}$

→ $KL = 17'$: $p = 1.33\times10^{-3}$ $b_x = 2.16\times10^{-3}$

$pP_u = 1.33\times10^{-3}(371.75) = .49 \geq .2$ ∴ use Eqn 6-1

$$pP_u + b_x M_{ux} + b_y M_{uy} \leq 1.0$$

$$1.33\times10^{-3}(371.75) + 2.16\times10^{-3}(7.48) + 3.51\times10^{-3}(7.16) = .54 < 1.0 \checkmark$$

W12×79 is acceptable for design

Gravity Column Design

RAM Steel 14.06.01.00
DataBase: RAM Model Gravity and Lateral March 29 03/30/15 23:12:21
Building Code: IBC Steel Code: AISC 360-10 LRFD
Academic License. Not For Commercial Use.

Story level 4th Floor, Column Line 6-B

Fy (ksi) = 50.00 Column Size = W14X132
Orientation (deg.) = 90.0

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	16.67	16.67
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	9.85	9.85
Bottom	9.85	9.85

CONTROLLING COLUMN LOADS - Skip-Load Case 4:

	Dead	Live	Roof
Axial (kip)	438.57	107.73	23.08
Moments Top Mx (kip-ft)	-0.07	-0.03	0.00
My (kip-ft)	0.00	0.00	0.00
Bot Mx (kip-ft)	-0.11	-1.90	0.00
My (kip-ft)	-0.00	-9.39	0.00

Reverse curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	= 710.19	0.90*Pn (kip)	= 1419.31
Mux (kip-ft)	= 3.17	0.90*Mnx (kip-ft)	= 877.50
Muy (kip-ft)	= 15.02	0.90*Mny (kip-ft)	= 423.75
Rm	= 1.00		
Cbx	= 1.72		
Cmx	= 0.58	Cmy	= 0.60
Pex (kip)	= 10943.48	Pey (kip)	= 3919.63
B1x	= 1.00	B1y	= 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.500
Eq H1-1a: 0.500 + 0.003 + 0.032 = 0.535

Gravity Column Design

RAM Steel 14.06.01.00 Page 2/3
DataBase: RAM Model Gravity and Lateral March 29 03/30/15 23:12:21
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Story level 3rd Floor, Column Line 6-B

Fy (ksi) = 50.00 Column Size = W14X132
Orientation (deg.) = 90.0

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		17.33	17.33
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	9.85	9.85
	Bottom	9.85	9.85

CONTROLLING COLUMN LOADS - Skip-Load Case 3:

			Dead	Live	Roof
Axial (kip)			565.13	161.59	23.08
Moments	Top	Mx (kip-ft)	-0.10	-0.04	0.00
		My (kip-ft)	-0.00	0.00	0.00
	Bot	Mx (kip-ft)	-0.08	-1.50	0.00
		My (kip-ft)	0.00	10.36	0.00

Reverse curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	= 948.25	0.90*Pn (kip)	= 1395.76
Mux (kip-ft)	= 2.50	0.90*Mnx (kip-ft)	= 877.50
Muy (kip-ft)	= 16.58	0.90*Mny (kip-ft)	= 423.75
Rm	= 1.00		
Cbx	= 1.76		
Cmx	= 0.57	Cmy	= 0.60
Pex (kip)	= 10125.81	Pey (kip)	= 3626.76
B1x	= 1.00	B1y	= 1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.679
Eq H1-1a: 0.679 + 0.003 + 0.035 = 0.717

Gravity Column Design

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Story level 2nd floor, Column Line 6-B

Fy (ksi) = 50.00 Column Size = W14X132
Orientation (deg.) = 90.0

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		20.00	20.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	9.85	9.85
	Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Skip-Load Case 6:

			Dead	Live	Roof
Axial (kip)			692.04	191.89	23.08
Moments	Top	Mx (kip-ft)	-0.07	-0.03	0.00
		My (kip-ft)	0.00	8.98	0.00
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	= 1149.02	0.90*Pn (kip)	= 1295.81
Mux (kip-ft)	= 0.14	0.90*Mnx (kip-ft)	= 877.50
Muy (kip-ft)	= 14.91	0.90*Mny (kip-ft)	= 423.75
Rm	= 1.00		
Cbx	= 1.67		
Cmx	= 0.60	Cmy	= 0.60
Pex (kip)	= 7602.68	Pey (kip)	= 2723.05
B1x	= 1.00	B1y	= 1.04

INTERACTION EQUATION

Pu/0.90*Pn = 0.887
Eq H1-1a: 0.887 + 0.000 + 0.031 = 0.918

Gravity Column Design

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Story level 4th Floor, Column Line 8-A

Fy (ksi)	=	50.00	Column Size	=	W12X79
Orientation (deg.)	=	90.0			

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	16.67	16.67
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	8.70	8.55
Column Eccentricity (in) Bottom	8.70	8.55

CONTROLLING COLUMN LOADS - Skip-Load Case 2:

	Dead	Live	Roof
Axial (kip)	226.28	58.89	11.97
Moments Top Mx (kip-ft)	2.67	1.22	0.00
My (kip-ft)	-0.00	0.00	0.00
Bot Mx (kip-ft)	3.87	1.77	0.00
My (kip-ft)	0.00	4.48	0.00

Reverse curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	=	371.75	0.90*Pn (kip)	=	762.46
Mux (kip-ft)	=	7.48	0.90*Mnx (kip-ft)	=	446.25
Muy (kip-ft)	=	7.16	0.90*Mny (kip-ft)	=	203.63
Rm	=	1.00			
Cbx	=	2.21			
Cmx	=	0.32	Cmy	=	0.60
Pex (kip)	=	4735.02	Pey (kip)	=	1544.96
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.488
Eq H1-1a: 0.488 + 0.015 + 0.031 = 0.534

Gravity Column Design

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Story level 3rd Floor, Column Line 8-A

Fy (ksi)	= 50.00	Column Size	= W12X79
Orientation (deg.)	= 90.0		

INPUT DESIGN PARAMETERS:

	X-Axis	Y-Axis
Lu (ft)	17.33	17.33
K	1	1
Braced Against Joint Translation	Yes	Yes
Column Eccentricity (in) Top	8.70	8.55
Bottom	8.70	8.55

CONTROLLING COLUMN LOADS - Skip-Load Case 4:

	Dead	Live	Roof
Axial (kip)	291.55	84.21	11.97
Moments Top Mx (kip-ft)	3.73	1.62	0.00
My (kip-ft)	0.00	0.00	0.00
Bot Mx (kip-ft)	3.05	0.00	0.00
My (kip-ft)	-0.00	-4.70	0.00

Reverse curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	=	490.59	0.90*Pn (kip)	=	743.36
Mux (kip-ft)	=	7.07	0.90*Mnx (kip-ft)	=	446.25
Muy (kip-ft)	=	7.53	0.90*Mny (kip-ft)	=	203.63
Rm	=	1.00			
Cbx	=	2.18			
Cmx	=	0.39	Cmy	=	0.60
Pex (kip)	=	4381.23	Pey (kip)	=	1429.53
B1x	=	1.00	B1y	=	1.00

INTERACTION EQUATION

Pu/0.90*Pn = 0.660
Eq H1-1a: 0.660 + 0.014 + 0.033 = 0.707

Gravity Column Design

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Story level 2nd floor, Column Line 8-A

Fy (ksi)	= 50.00	Column Size	= W12X79
Orientation (deg.)	= 90.0		

INPUT DESIGN PARAMETERS:

		X-Axis	Y-Axis
Lu (ft)		20.00	20.00
K		1	1
Braced Against Joint Translation		Yes	Yes
Column Eccentricity (in)	Top	8.70	8.55
	Bottom	0.00	0.00

CONTROLLING COLUMN LOADS - Skip-Load Case 10:

			Dead	Live	Roof
Axial (kip)			357.04	99.50	11.97
Moments	Top	Mx (kip-ft)	2.65	1.15	0.00
		My (kip-ft)	-0.00	-4.06	0.00
	Bot	Mx (kip-ft)	0.00	0.00	0.00
		My (kip-ft)	0.00	0.00	0.00

Single curvature about X-Axis
Single curvature about Y-Axis

CALCULATED PARAMETERS: (1.2DL + 1.6LL + 0.5RF)

Pu (kip)	= 593.63	0.90*Pn (kip)	= 664.12
Mux (kip-ft)	= 5.01	0.90*Mnx (kip-ft)	= 446.25
Muy (kip-ft)	= 8.72	0.90*Mny (kip-ft)	= 203.63
Rm	= 1.00		
Cbx	= 1.67		
Cmx	= 0.60	Cmy	= 0.60
Pex (kip)	= 3289.53	Pey (kip)	= 1073.32
B1x	= 1.00	B1y	= 1.34

INTERACTION EQUATION

Pu/0.90*Pn = 0.894
Eq H1-1a: 0.894 + 0.010 + 0.038 = 0.942

Appendix E

Vibration Analysis

Gravity Redesign | Vibration Analysis | 1/4

Vibration Check

28LH09 joists, 4.75' spacing 40' span SW = 21 plf
36G8N23.7* joist girders 38' span SW = 69 plf

DL = 52 psf + joist SW
LL = 65 psf = P_o → reduction = 41 psf

Slab: 3 1/4" LW conc
f'c = 3500
1.5" deck, 4.75" total thickness } 34 psf

f_{allow} = 50 ksi

Joist Properties

Determine I_{chord}

$M_{all} = \frac{wL_n^2}{8}$ w = 667 plf (ASD)
$L_n = 40' - .33' = 39.67'$

$M_{all} = \frac{667(39.67)^2}{8} = 131.21$ ft-k

$A_j = .85 A_{chord} = .85(A_{top} + A_{bot})$ $A_{top} = 1.25 A_{bot}$ $A_{chord} = A_{top} + A_{bot}$

$A_{bot} = \frac{M_{all}}{(d-1")f_{all}} = \frac{131.21 \times 12}{(28-1)(50)} = 1.17\ in^2$ $A_{top} = 1.46\ in^2$ $A_{chord} = 2.63\ in^2$

$A_j = .85(2.63) = 2.24\ in^2$ $A_j = 2.24\ in^2$

$y_c = .5" + \frac{A_{bot}(d-1")}{A_{chord}} = .5 + \frac{1.17(28-1)}{2.63}$ $y_c = 12.51"$

$I_{chord} = A_{top}(y_c - .5")^2 + A_{bot}(d - y_c - .5")^2$
$= 1.46(12.51 - .5)^2 + 1.17(28 - 12.51 - .5)^2 = 473.5\ in^4 = I_{chord}$

$n = E_s / 1.35E_c = 29000 / 1.35 w_c^{1.5}\sqrt{f'c} = 29000 / [1.35(115)^{1.5}\sqrt{3.5}]$ n = 9.31

$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{[(4.75' \times 12"/ft)/9.31](3.25)(2) + 2.63(4.75 + 12.51)}{[(4.75 \times 12)/9.31](3.25) + 2.63}$ $\bar{y} = 3.67"$

$I_{comp} = \Sigma I + \Sigma Ad^2$
$= [(4.75 \times 12)/9.31](3.25)^3 + 473.5 + 2.63(12.51 + 4.75 - 3.67)^2$
$+ [(4.75 \times 12)/9.31](3.25)(3.67 - 1.5)^2$

$I_{comp} = 1263.1\ in^4$

Gravity Redesign | Vibration Analysis | 2/4

Using Design Guide 11, pg 15 $L/d = 40(12)/28 = 17.14 \rightarrow$ use eqn 3.16

$C_r = 0.90(1-e^{-.28(L/d)})^{2.8} = 0.90(1-e^{-.28(17.14)})^{2.8}$ $C_r = .88$

$\gamma = \frac{1}{C_r} - 1 = \frac{1}{.88} - 1$ $\gamma = .136$

$I_j = \frac{1}{\frac{\gamma}{I_{chord}} + \frac{1}{I_{comp}}} = \frac{1}{\frac{.136}{4735} + \frac{1}{10634}}$ $I_j = 926.85$ $I_j < I_{comp}$ ✓

Determine Δ_j

$W_j = 4.75(52+65) = 555.75$ plf

$\Delta_j = \frac{5 w_j L_j^4}{384 E I_j} = \frac{5(555.75)(40)^4(1728)}{384(29e^6)(926.85)} = 1.19''$

$D_s = \frac{12 d_e^3}{12n} = \frac{12(3.25+.5)^3}{12(9.31)} = 5.66 \text{ in}^3$

$D_j = I_j/S = 926.85/(4.75 \times 12) = 16.26 \text{ in}^3$

$B_j = C_j (D_s/D_j)^{1/4} L_j$ $C_j = 2.0$ $= 2.0(5.66/16.26)^{1/4}(40') = 61.44'$

$W_j = (w_j/s) B_j L_j = (555.75/4.75)(61.44)(40)$ $W_j = 287.58 \sim 288^k$

Girder Properties

Determine I_{chord}

$M_{all} = \frac{w L_n^2}{8}$ $w = 23.7(7)/38'$ $w = 4.37$ klf $= 4366$ plf
$L_n = 38' - .33 = 37.67'$

$M = \frac{4366(37.67)^2}{8}$ $M_{all} = 774.4$ ft-k

$A_g = .85 A_{chord} = .85(A_{top} + A_{bot})$ $A_{top} = 1.25 A_{bot}$ $A_{chord} = A_{top} + A_{bot}$

$A_{bot} = \frac{M_{all}}{(d-1)f_{all}} = \frac{774.4 \times 12}{(36-1)(50)} = 5.31 \text{ in}^2$ $A_{top} = 6.64 \text{ in}^2$ $A_{chord} = 11.95 \text{ in}^2$

$A_g = .85(11.95)$ $A_g = 10.16 \text{ in}^2$

$y_c = .5 + \frac{A_{bot}(d-1'')}{A_{chord}} = \frac{.5 + 5.31(36-1)}{11.95}$ $y_c = 15.59''$

$I_{chord} = A_{top}(y_c - .5'')^2 + A_{bot}(d - y_c - .5'')^2$

$= 6.64(15.59-.5)^2 + 5.31(36-15.59-.5)^2$ $I_{chord} = 3616.91 \text{ in}^4$

Gravity Redesign | Vibration Analysis | 3/4

$n = E_s / 1.35E_c = 29000/1.35(115)^{1.5}\sqrt{3.5} - 9.31$

$b_{eff} = span/8 + span/8 = span/4 = 38/4 = 9.5'$

$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{((9.5\times12)/9.31)(3.25)(1.5) + 11.95(4.75+15.59)}{[(9.5\times12)/9.31](3.25) + 11.95}$ $\bar{y} = 5.85''$

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

$= [(9.5\times12)/9.31)(3.25)^3 + 3616.91 + 11.95(15.59+4.75-5.85)^2$

$+ [(9.5\times12)/9.31)](3.25)(5.85-1.5)^2$

$I_{comp} = 420.34 + 3616.91 + 2509.023 + 753.04$ $I_{comp} = 4044.31\ in^4$

$L/d = 38(12)/36 = 12.67 \rightarrow$ eqn 3.16

$C_r = 0.90(1-e^{-.28(L/d)})^{2.8} = 0.90(1-e^{-.28(12.67)})^{2.8}$ $C_r = .83$

$\gamma = \frac{1}{C_r} - 1 = \frac{1}{.83} - 1$ $\gamma = .21$

$I_g = \frac{1}{\frac{\gamma}{I_{chord}} + \frac{1}{I_{comp}}} = \frac{1}{\frac{.21}{3616.91} + \frac{1}{4044.31}} = 3287.66 = I_g$ $I_g < I_{comp}$

Determine Δ_g

$W_g = 9.5(52+65) + 69$ plf SW $= 1180.5$ plf

$\Delta_g = \frac{5 w_g L_g^4}{384 E I_g} = \frac{5(1180.5)(38)^4(1728)}{384(29e^6)(3287.66)} = .58''$

$D_s = 12 d_e^3/12n = \frac{12(3.25+5)^3}{12(9.31)} = 5.66\ in^3$

$D_g = I_g/L_g = 3287.66/(38\times12) = 7.21$

$B_g = C_g(D_j/D_g)^{1/4} L_g = 2.0(16.26/7.21)^{1/4}(38') = 93.13$

$W_g = (w_g/b_{eff}) B_g L_g = (1180.5/9.5)(93.13)(38) = 439.76^k \sim 440$

$W = \frac{\Delta_j}{\Delta_j + \Delta_g} W_j + \frac{\Delta_g}{\Delta_j + \Delta_g} W_g = \frac{1.19}{1.19+.58}(287.58) + \frac{.58}{1.19+.58}(439.76)$ $W = 337.44^k$

$f_n = 0.18\sqrt{\frac{g}{\Delta_g + \Delta_j}}$ $g = 386.4\ in/sec^2$

Gravity Redesign | Vibration Analysis | 9/4

$f_n = 0.18\sqrt{\frac{386.4}{.58+1.19}}$ $f_n = 2.66$ Hz

$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W} \leq \frac{a_o}{g}$

$\frac{a_o}{g} = 0.005$

$\beta = 0.05$

$\frac{a_p}{g} = \frac{65 \exp(-0.35 \times 2.66)}{0.05 \times (337.44 \times 1000)} = .0015 < .005 \therefore$ OK

System vibrations are within walking excitation limits.

Appendix F
Wind Loads

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WIND LOAD CALCULATIONS

- calculated using ASCE 7-05

1) Occupancy Category – II from drawings

confirmed in table 1-1

2) Wind Load Importance Factor

From drawings, V = 90 mph, category II
$I = 1.00$, From drawing, $I = 1.00$ → match

3) Basic Wind Speed, from Figure 6-1, confirmed on dwgs

$V = 90$ mph

4) Wind Load Parameters

a. Wind Directionality Factor, K_d, from Table 6-4, confirmed on dwgs.

$K_d = 0.85$

b. Exposure Category (§6.5.6.3)

Exposure B → confirmed in drawings (case 2, not low rise building)

c. Topographic Factor, K_{zt} (§6.4.2.1 & §6.5.7, Table 6-4)

$K_{zt} = 1.0$ → confirmed on dwgs → $K_{zt} = 1.0$ (2-D escarpment) no hill

d. Gust Effect factor (§6.5.8)

From commentary

$n_1 = 100/H$ → average value $= 100/83 = 1.20$

$n_1 = 75/H$ → lower bound $= 75/83 = .90$ ← use this value to be conservative

exposure B factors (from Table 6-2)

$\alpha = 7.0$
$z_g(ft) = 1200$
$\hat{a} = 1/7$
$\hat{b} = 0.84$
$\bar{\alpha} = 1/4.0$
$\bar{b} = 0.45$
$c = 0.30$
$\ell(ft) = 320$
$\bar{\epsilon} = 1/3.0$
$z_{min}(ft) = 30$

$\bar{z}$ = equivalent height of structure $= .6h = .6(83) = 49.8'$

For buildings without concrete shear walls, a simplified procedure can be used

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$$N_1 = \frac{n_1 L_{\bar z}}{\bar V_{\bar z}} = \frac{.90(367.05)}{74.80} \qquad N_1 = 4.417$$

$$L_{\bar z} = \ell\left(\frac{\bar z}{33}\right)^{\bar\epsilon} = 320\left(\frac{49.8}{33}\right)^{1/3} = 367.05 \text{ ft}$$

$$\bar V_{\bar z} = \bar b\left(\frac{\bar z}{33}\right)^{\bar \alpha} V\left(\frac{88}{60}\right) = 0.45\left(\frac{49.8}{33}\right)^{1/4} 90\left(\frac{88}{60}\right) = 65.84 \text{ ft/s}$$

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

h = mean roof height = 83'
B = horizontal dimension of building normal to wind direction
BNS = 326'
BEW = 394'

$$Q_{NS} = \sqrt{\frac{1}{1+.63\left(\frac{326+83}{367.05}\right)}} = .77$$

$$Q_{EW} = \sqrt{\frac{1}{1+.63\left(\frac{394+83}{367.05}\right)}} = .74$$

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

From §6.5.8.1, g_Q & g_v = 3.4

$$I_{\bar z} = c\left(\frac{33}{\bar z}\right)^{1/6} = .30\left(\frac{33}{49.8}\right)^{1/6} = .28$$

$$G_{NS} = 0.925\left(\frac{1+1.7(3.4)(.28)(.77)}{1+1.7(3.4)(.28)}\right) \qquad G_{NS} = .79$$

$$G_{EW} = 0.925\left(\frac{1+1.7(3.4)(.28)(.74)}{1+1.7(3.4)(.28)}\right) \qquad G_{EW} = .78$$

1.6184

e. Enclosure Classification (§6.5.9 & §6.2)

- Building is enclosed as it does not meet "open" and "partially enclosed" conditions

f. Internal Pressure Coefficient Figure 6-5

$GC_{pi} = \pm 0.18$ for enclosed buildings

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CP Values (Figure 6-6)

- Wind typically blows Southwest in the area the Corporate Headquarters is located. It tends to hit more vertically than horizontally, so for this reason, the north side of the building will be considered windward and the south side will be considered leeward.

L = horizontal dimension of building parallel to wind direction
B = horizontal dimension of building normal to wind direction

Wall C_p Values

Windward wall: $C_p = 0.8$ use with q_z

leeward wall: $B = 326'$ $L = 394'$ $L/B = 394/326 = 1.21$

- need to interpolate to find C_p

$$x = \frac{(1.21-1)(-0.3+0.5)}{(2-1)} + -0.5$$

L/B	Cp
0-1	-0.5
1.21	x
2	-0.3

$C_p = -0.46$ use with q_h

Side wall: $C_p = -0.7$, use with q_h

Roof C_p Values

- Roof has 0° slope
  - horizontal distance from windward edge = 394' $h = 83'$

$394 > 2h$

$C_p = -0.3, -0.18$

Find Wind Pressures

$K_z = 2.01(z/z_g)^{2/\alpha}$ z = height of floor above ground

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$

$p = q G_f C_p$

Floor #	z (ft)	K_z	q_z (psf)
2	20	0.62	11.0
3	37.33	0.75	13.14
4	54	0.83	14.61
5	68.67	0.89	15.64
roof	83.33	0.94	16.53

Excel Equations: $p = qCP_i$ $G_{NS} = .79$ $G_{EW} = .78$ $q_h \sim 16.5'$

Building width NS = 326' Building width EW = 394'

* See excel sheet for values

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Wind Pressure for Roof

$P = q_h G C_p$

$q_h = 0.00256[2.01(83.33/1200)^{2/7}](1.0)(.85)(90^2)(1.0) = 16.78$ psf

N-S direction

windward: $p = 16.78(.79)(-0.3) = -3.98$ psf

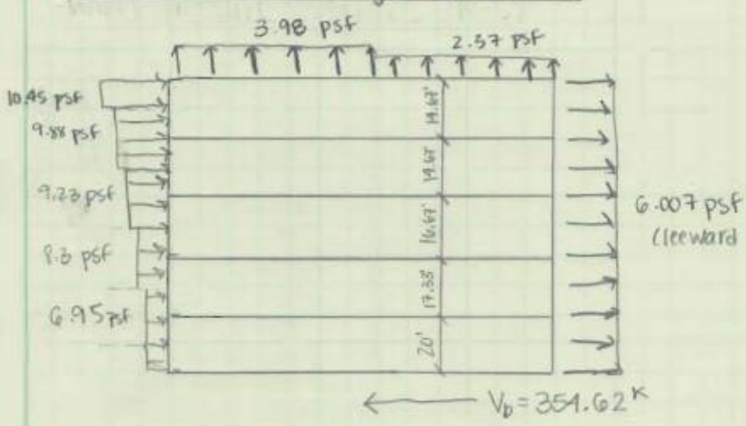
leeward: $p = 16.78(.79)(-0.18) = -2.37$ psf

E-W direction

windward: $p = 16.78(.78)(-0.3) = -3.93$ psf

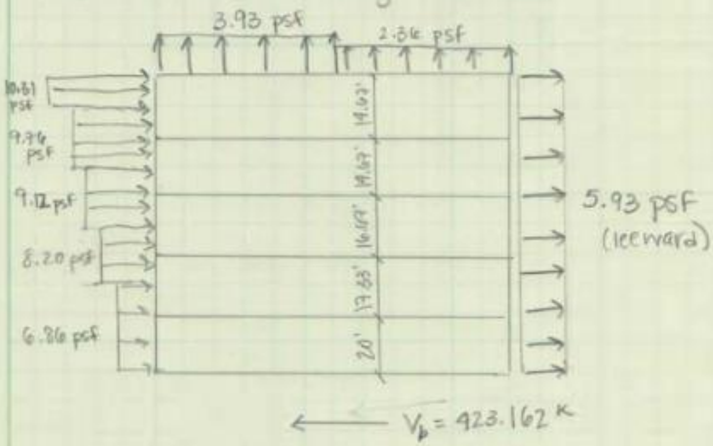
leeward: $p = 16.78(.78)(-0.18) = -2.36$ psf

Wind Pressure Diagram (N-S)



M.Julia Haverty | Wind Loads | Tech Report 2

Wind Pressure Diagram (E-W)



Wind Pressure (North-South Direction)							
Floor	z (ft)	qz (PSF)	Windward Pressure (PSF)	Leeward Pressure (PSF)	Tributary Area	Force (K)	Overturning Moment (ft-k)
2	20	11	6.952	-6.007	6096	78.998	1579.962
3	37.33	13.14	8.304	-6.007	5542	79.314	2960.800
4	54	14.61	9.234	-6.007	5314	80.988	4373.359
5	68.67	15.64	9.884	-6.007	4782	75.993	5218.444
roof	83.33	16.53	10.447	-6.007	2390	39.325	3276.950
					Base	354.618	17409.515

Wind Pressure (East-West Direction)							
Floor	z (ft)	qz (PSF)	Windward Pressure (PSF)	Leeward Pressure (PSF)	Tributary Area	Force (K)	Overturning Moment (ft-k)
2	20	11	6.864	-5.931	7368	94.273	1885.466
3	37.33	13.14	8.199	-5.931	6698	94.645	3533.094
4	54	14.61	9.117	-5.931	6422	96.636	5218.328
5	68.67	15.64	9.759	-5.931	5780	90.690	6227.687
roof	83.33	16.53	10.315	-5.931	2888	46.918	3909.638
					Base	423.162	20774.214

Loads and Applied Forces

RAM Frame 14.06.01.00
DataBase: RAM Model Gravity and Lateral March 29 04/01/15 15:30:07
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LOAD CASE: Wind 2

Wind ASCE 7-05 / IBC2006/2009
Exposure: B
Basic Wind Speed (mph): 90.0 Importance Factor: 1.000
Apply Directionality Factor, Kd = 0.85
Use Topography Factor, Kzt: 1.00
Use Calculated Frequency for X-Dir.
Use Calculated Frequency for Y-Dir.
Gust Factor for Rigid Structures, G: Use Calculated G for X-Dir.
Gust Factor for Rigid Structures, G: Use Calculated G for Y-Dir.
Damping Ratio for Flexible Structures = 0.01
Mean Roof Height (ft): Top Story Height = 83.34
Ground Level: Base

WIND PRESSURES:

X-Direction: Natural Frequency = 1.617 Structure is Rigid
Y-Direction: Natural Frequency = 2.270 Structure is Rigid
CpWindward = 0.80 qLeeward (qh) = 16.53 psf
GCpn (Parapet): Windward = 1.50 Leeward = -1.00

Height	Kz	Kzt	qz	Gust Factor G		CpLeeward		Pressure (psf)	
ft			psf	X	Y	X	Y	X	Y
83.34	0.938	1.000	16.534	0.786	0.783	-0.487	-0.500	16.724	16.827
68.67	0.888	1.000	15.645	0.786	0.783	-0.487	-0.500	16.165	16.269
54.00	0.829	1.000	14.606	0.786	0.783	-0.487	-0.500	15.512	15.619
37.33	0.746	1.000	13.144	0.786	0.783	-0.487	-0.500	14.593	14.704
20.00	0.624	1.000	10.998	0.786	0.783	-0.487	-0.500	13.243	13.359
0.00	0.575	1.000	10.130	0.786	0.783	-0.487	-0.500	12.698	12.816

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC09_1_X

Level	Diaph.#	Ht	Fx	Fy	X	Y
		ft	kips	kips	ft	ft
Main Roof Level	1	83.34	48.28	0.00	62.00	-100.50
5th Floor	1	68.67	94.29	0.00	62.00	-100.50
4th Floor	1	54.00	96.36	0.00	62.00	-100.50
3rd Floor	1	37.33	98.20	0.00	62.00	-100.50
2nd floor	1	20.00	99.21	0.00	62.00	-100.50

APPLIED STORY FORCES

Type: Wind_IBC09_1_X

Level	Ht	Fx	Fy
	ft	kips	kips
Main Roof Level	83.34	48.28	0.00
5th Floor	68.67	94.29	0.00
4th Floor	54.00	96.36	0.00
3rd Floor	37.33	98.20	0.00

Loads and Applied Forces

RAM Frame 14.06.01.00 Page 2/10
DataBase: RAM Model Gravity and Lateral March 29 04/01/15 15:30:07
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2nd floor	20.00	99.21	0.00
		436.34	0.00

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC09_1_Y

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Main Roof Level	1	83.34	0.00	51.72	62.00	-100.50
5th Floor	1	68.67	0.00	101.04	62.00	-100.50
4th Floor	1	54.00	0.00	103.31	62.00	-100.50
3rd Floor	1	37.33	0.00	105.35	62.00	-100.50
2nd floor	1	20.00	0.00	106.54	62.00	-100.50

APPLIED STORY FORCES

Type: Wind_IBC09_1_Y

Level	Ht ft	Fx kips	Fy kips
Main Roof Level	83.34	0.00	51.72
5th Floor	68.67	0.00	101.04
4th Floor	54.00	0.00	103.31
3rd Floor	37.33	0.00	105.35
2nd floor	20.00	0.00	106.54
		0.00	467.97

APPLIED DIAPHRAGM FORCES

Type: Wind_IBC09_2_X+E

Level	Diaph.#	Ht ft	Fx kips	Fy kips	X ft	Y ft
Main Roof Level	1	83.34	36.21	0.00	62.00	-40.80
5th Floor	1	68.67	70.72	0.00	62.00	-40.80
4th Floor	1	54.00	72.27	0.00	62.00	-40.80
3rd Floor	1	37.33	73.65	0.00	62.00	-40.80
2nd floor	1	20.00	74.40	0.00	62.00	-40.80

APPLIED STORY FORCES

Type: Wind_IBC09_2_X+E

Level	Ht ft	Fx kips	Fy kips
Main Roof Level	83.34	36.21	0.00
5th Floor	68.67	70.72	0.00
4th Floor	54.00	72.27	0.00

Appendix G

Seismic Loads

Seismic Loads | Final Report | 1/2

Seismic Load Calculation

1) Building not exempt (§11.1.2)

2) Design Spectral Response Acceleration (§11.4)

a) Site Class C

b) Acceleration Parameters

$S_s = 0.175g$
$S_1 = 0.051g$

c) site class effects (§11.4.3)

$F_a = 1.2$
$F_v = 1.7$

$S_{ms} = F_a S_s = 1.2(0.175) = 0.21g$ $\quad S_{sm} = 0.21g$
$S_{m1} = F_v S_1 = 1.7(0.051) = 0.0867g$ $\quad S_{m1} = 0.0867g$

d) Determine Spectral Acceleration Parameters (§11.4.4)

$S_{DS} = 2/3\, S_{ms} = 2/3(0.21)$ $\quad S_{DS} = 0.14$
$S_{D1} = 2/3\, S_{m1} = 2/3(0.0867)$ $\quad S_{D1} = 0.0578$

3) Find Seismic Design Category

occupancy category II
Importance factor = 1.0
$S_{DS} < 0.167 \rightarrow$ category A
-matches given SDC A given in drawings

4) Analysis Procedure Selection

§11.4 - buildings and other structures assigned to SDC A need only comply with the requirements of section 11.7

§11.7.1 - seismic loads shall be taken as "E" and combined with other load combinations from Sections 2.3 & 2.4

§1.4 - Eqn 1.4-1 $F_x = 0.01\, W_x$

5-9) Skip due to SDC A

10) Calculate effective total seismic weight (W) for each floor

Roof Area = 121940 SF

$W_{RF} = \text{area}[DL + .20S] = (121940[84 + .2(17)])/1000 = 10658^k$

Floor 5 & 4 area = 121940 SF $\quad W_4 = W_5 = 6341^k$

$W_{F_{4,5}} = (2)\,121940(52)/1000 = 12682\ K$

Floor 2 : area = 145,500 SF

$W_{F_2} = 145{,}500(52)/1000 = 7566\ K$

Seismic Loads cont Final Report 2/2

Courtyard level
regular floor loading area = 121940 SF
courtyard green roof area = 23560 SF

$W_{reg\ floor} = 121940(52)/1000 = 6341^k$
$W_{green\ roof} = 23560[100 + .2(17)]/1000 = 2436\ k$ } $8777^k = W_{F_3}$

Total Building Seismic Weight = ΣW

$W = 39683^k$ $W = 10658 + 12682 + 7566 + 8777 = 39683^k$

11) Calculate Base shear (V)

$V = C_s W$ $V = 0.01(39683)$ $V = 396.83 \sim 397^k$

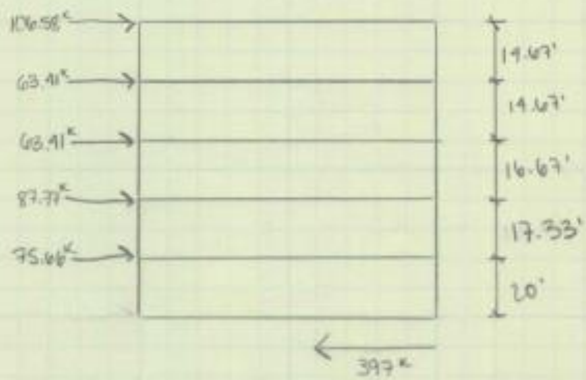
12) Vertical Distribution of Seismic Forces
SDCA → $F_x = 0.01\ W_x$

$F_{Roof} = 0.01(10658) = 106.58^k$

$F_{level\,4} = F_{level\,5} = 0.01(6341) = 63.41^k$

$F_{Courtyard\ level} = 0.01(8777) = 87.77^k$

$F_{level\ 2} = 0.01(7566) = 75.66^k$



Overturning Moment
8882.38 ft-k
4354.36 ft-k
3424.14 ft-k
3276.45 ft-k
1513.2 ft-k
$\Sigma M = 21450.53$ ft-k

Appendix H

Shear Wall Check

Lateral System | Shear Wall Check | 1/2

30.5' Shear wall — Base level check — $h_w = 83.3'$

$M_u = 4050.9$ k-ft — length = 30.5' — $f'_c = 3500$ psi

$V_u = 167.6^k$ — thickness = 12" — $f_y = 60$ ksi

$P_u = 760.57^k$ — #4's @ 12" horizontal & vertical reinf
use (8) #6 flexural reinforcing each end

Shear: $V_u \le \phi V_{n\,max} = \phi 10\sqrt{f'c}\, hd$ — $\phi = .75$ for shear

$d = .8\ell = .8(30.5 \times 12)$ $d = 292.8"$

$\phi V_n = 0.75(10)\sqrt{3500}(6)(292.8)/1000 = 779.5$ kips

$167.6 < 779.5$ ∴ ok

shear strength of concrete $V_c = 2\sqrt{f'c}\, hd$

$V_c = 2\sqrt{3500}(6)(292.8)/1000 = 207.9^k$

$\frac{1}{2}\phi V_c = \frac{1}{2}(.75)(207.9) = 77.95^k < 167.6^k$ ∴ shear reinforcing req'd

check reinforcing provided

for #4 bar, $A_s = 0.2\ in^2/ft$ $\frac{A_v}{S} = \frac{V_s}{f_y d}$ $V_s = \frac{A_v}{S} f_y d$

$V_s = \left(\frac{0.2}{12}\right)(60{,}000)(292.8) = 292.8^k$

$\phi V_n = \phi(V_c + V_s) = 77.95 + .75(292.8)$ $\phi V_n = 297.55$

$\phi V_n = 297.55 > 167.6 = V_u$ ∴ ok

$\rho_t = \frac{A_v}{Sh} = \frac{2(0.2)}{12(12)} = .00278 > .0025$ ∴ ok

spacing ok ✓

use (2)#4 bars @ 12" O.C. for horizontal shear reinforcing

check vertical shear reinforcing

$\rho_\ell = A_v/Sh \ge 0.0025 + 0.5\left(2.5 - \frac{h_w}{\ell_w}\right)(0.0028 - 0.0025)$

$\rho_\ell = A_v/Sh > 0.0025 + 0.5\left(2.5 - \frac{83.3}{30.5}\right)(0.0028 - .0025) = .0024 < .0025$

use $\rho_\ell \ge 0.0025$ as a minimum

Lateral System | Shear Wall Check | 2/2

$\frac{A_v}{Sh} = \frac{2(0.2)}{12(12)} = 0.0028 > 0.0025 \rightarrow$ ok

use (2) #4 bars @ 12" o.c. for vertical reinforcement

Check flexural reinforcement

$d = 0.8\ell_w = 0.8(30.5 \times 12) = 292.8$

$M_u = 4050.9$ ft-k

$M_u \leq \phi M_n = \phi A_s f_y jd$

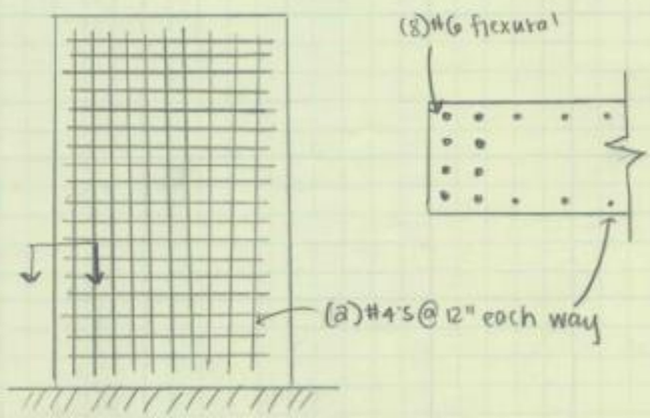
$a = \frac{A_s f_y}{.85 f'c\, b}$ for (8)#6, $A_s = 3.52\ in^2$

$a = \frac{3.52(60)}{.85(3.5)(6)} = 11.83$ $jd = d - a/2 = 292.8 - 11.83/2$ $jd = 286.88$

$4050.9(12) \leq (.85)(3.52)(60)(286.88)$

$48610.8 \leq 51500.70 \therefore$ ok

(8) #6 bars is adequate for flexural reinforcement.



Section Cut Design Summary

RAM Concrete Shearwall 14.06.01.00
Database: RAM Model Gravity and Lateral March 29 04/01/15 20:20:12
Design Code: ACI 318-08
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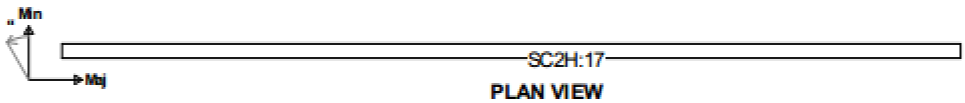
Section Cut ID: SC2H:17 (Horizontal)
Story: 2nd floor
Ag = 2196 in2 Imaj = 24513946 in4 Imin = 6588 in4
Major Axis Orientation: 0.00 degrees (CCW from global X-axis)
Wall Design Group: 2
Design Status: PASS



PLAN VIEW

Axial/Flexural Results:

Interaction:	0.249	OK	
Pu =	760.57 kips	phiPn =	3054.97 kips
Mu =	4050.9 kip-ft	at Beta =	-0.0 deg CCW from Major axis
Controlling Load Combo:	0.900 D - 1.600 W1 (LC 42)		
Code Ref:	10.3.7		

Shear Results:

Segment SC2H:17:
Length = 30.50 ft Thick = 6.00 in f'c = 3500 psi fy = 60 ksi
Vert Bar Pat: #4@12" oc Horiz Bar Pat: #4@12" oc
Vu = 167.6 kip phiVn = 587.1 kip OK
Controlling Load Combo: 0.900 D + 1.600 W1 (LC 30)
Code Ref: 14.2.3 & 11.9.5

Reinforcement Checks:

Min Vert Reinf Ratio: Limit: 0.250% Actual: 0.572% (11.9.4) OK
Segment SC2H:17:
Max Vert Bar Spacing Limit: 18.00 in Actual: 12.00 in (11.9.5) OK
Min Vert Bar Spacing Limit: 1.00 in Actual: 11.50 in (7.6.1) OK
Min Longit Reinf Ratio Limit: 0.545% Actual: 0.545% (21.9.4.3) OK
Min Number of Reinf Curtains: 1 Actual: 2 (14.3.4) OK
Min Number of Reinf Curtains: 1 Actual: 2 (21.9.2.2) OK

Appendix I

Story Drifts and Center of Rigidity

Center of Rigidity

RAM Frame 14.06.01.00
DataBase: Existing Building RAM Model April 7 04/07/15 16:25:16
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CRITERIA:

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

			Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Type**	**Xr**	**Yr**	**Xm**	**Ym**
			ft	**ft**	**ft**	**ft**
Main Roof Level	1	Rigid	61.14	-76.32	68.90	-111.43
5th Floor	1	Rigid	63.93	-66.54	68.63	-111.59
4th Floor	1	Rigid	65.36	-60.50	68.66	-111.59
3rd Floor	1	Rigid	69.30	-55.90	70.03	-107.25
2nd floor	1	Rigid	57.88	-42.54	70.27	-107.01

			Story Lateral Stiffness	
Level	**Diaph. #**	**Type**	**KX**	**KY**
			kips/ ft	**kips/ ft**
Main Roof Level	1	Rigid	15133.34	27342.70
5th Floor	1	Rigid	18677.74	28740.48
4th Floor	1	Rigid	23277.28	36810.86
3rd Floor	1	Rigid	32540.51	48020.54
2nd floor	1	Rigid	57339.36	71555.50

NOTES:
Center of rigidity (CR) values given above are only used for load cases that require explicit calculation of CRs for use in calculation of load eccentricities (for example, ASCE 7-05 Wind Load Case).

Note that this information is never used for analysis. On the other hand, it should be noted that analysis results always include any torsional effects due to having center of rigidity and mass center at different locations. In other words, the analysis always accounts for locations and stiffnesses of frame members and diaphragms. Hence, any torsional effects of the masses being offset from the stiffnesses (i.e., CR) are implicitly and correctly accounted in the analysis.

Existing Building Center of Rigidity

Story Displacements

RAM Frame 14.06.01.00
DataBase: Existing Building RAM Model April 7 04/07/15 16:25:16
Building Code: IBC
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CRITERIA:

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

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
E1	Seismic	EQ_User
W1	Wind	W_User
W2	ASCE 7 WIND	Wind_IBC09_1_X
W3	ASCE 7 WIND	Wind_IBC09_1_Y
W4	ASCE 7 WIND	Wind_IBC09_2_X+E
W5	ASCE 7 WIND	Wind_IBC09_2_X-E
W6	ASCE 7 WIND	Wind_IBC09_2_Y+E
W7	ASCE 7 WIND	Wind_IBC09_2_Y-E
W8	ASCE 7 WIND	Wind_IBC09_3_X+Y
W9	ASCE 7 WIND	Wind_IBC09_3_X-Y
W10	ASCE 7 WIND	Wind_IBC09_4_X+Y_CW
W11	ASCE 7 WIND	Wind_IBC09_4_X+Y_CCW
W12	ASCE 7 WIND	Wind_IBC09_4_X-Y_CW
W13	ASCE 7 WIND	Wind_IBC09_4_X-Y_CCW
E2	ASCE 7 Seismic	EQ_IBC09_X_+E_F
E3	ASCE 7 Seismic	EQ_IBC09_X_-E_F
E4	ASCE 7 Seismic	EQ_IBC09_Y_+E_F
E5	ASCE 7 Seismic	EQ_IBC09_Y_-E_F

Level: Main Roof Level, Diaph: 1

Center of Mass (ft): (68.90, -111.43)

LdC	Disp X	Disp Y	Theta Z
	in	in	rad
D	-0.01438	0.04909	0.00001
Lp	-0.01068	0.03566	0.00001
E1	0.65083	0.01278	0.00004
W1	0.52824	0.01184	0.00003
W2	0.55501	0.01241	0.00004
W3	0.01007	0.40886	-0.00000

Existing Building Story Displacements

Story Displacements

RAM Frame 14.06.01.00
DataBase: Existing Building RAM Model April 7
Building Code: IBC

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04/07/15 16:25:16

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W4	0.38845	0.00605	-0.00003
W5	0.44406	0.01257	0.00008
W6	0.03875	0.31031	0.00006
W7	-0.02365	0.30297	-0.00007
W8	0.42381	0.31595	0.00002
W9	0.40870	-0.29733	0.00003
W10	0.27360	0.23177	-0.00007
W11	0.36211	0.24216	0.00011
W12	0.26227	-0.22820	-0.00007
W13	0.35078	-0.21780	0.00011
E2	0.24413	0.00478	0.00001
E3	0.25496	0.00618	0.00003
E4	0.01169	0.17107	0.00002
E5	0.00018	0.16958	-0.00001

Level: 5th Floor, Diaph: 1

Center of Mass (ft): (68.63, -111.59)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.02653	0.06082	0.00002
Lp	-0.01873	0.04302	0.00001
E1	0.54512	0.01052	0.00004
W1	0.46494	0.00940	0.00004
W2	0.48845	0.00986	0.00004
W3	0.00983	0.36319	-0.00000
W4	0.33887	0.00507	-0.00002
W5	0.39380	0.00971	0.00008
W6	0.03818	0.27500	0.00005
W7	-0.02344	0.26978	-0.00006
W8	0.37371	0.27979	0.00003
W9	0.35897	-0.26500	0.00003
W10	0.23657	0.20614	-0.00006
W11	0.32399	0.21354	0.00010
W12	0.22552	-0.20245	-0.00006
W13	0.31293	-0.19505	0.00010
E2	0.20752	0.00390	0.00001
E3	0.21789	0.00484	0.00003
E4	0.01146	0.14819	0.00001
E5	0.00042	0.14718	-0.00001

Level: 4th Floor, Diaph: 1

Center of Mass (ft): (68.66, -111.59)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.02924	0.06461	0.00001

Existing Building Story Displacements cont'd

Story Displacements

RAM Frame 14.06.01.00
DataBase: Existing Building RAM Model April 7
Building Code: IBC

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04/07/15 16:25:16

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Lp	-0.02045	0.04493	0.00001
E1	0.41869	0.00853	0.00004
W1	0.36230	0.00740	0.00003
W2	0.38001	0.00777	0.00003
W3	0.00808	0.28532	-0.00000
W4	0.26255	0.00418	-0.00001
W5	0.30746	0.00747	0.00006
W6	0.03123	0.21584	0.00004
W7	-0.01911	0.21214	-0.00005
W8	0.29106	0.21982	0.00002
W9	0.27895	-0.20817	0.00003
W10	0.18258	0.16224	-0.00005
W11	0.25402	0.16748	0.00008
W12	0.17349	-0.15875	-0.00004
W13	0.24493	-0.15350	0.00008
E2	0.15800	0.00313	0.00001
E3	0.16629	0.00378	0.00002
E4	0.00926	0.11401	0.00001
E5	0.00044	0.11332	-0.00001

Level: 3rd Floor, Diaph: 1

Center of Mass (ft): (70.03, -107.25)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.02129	0.05413	0.00001
Lp	-0.01480	0.03718	0.00001
E1	0.27801	0.00788	0.00003
W1	0.22891	0.00668	0.00002
W2	0.24059	0.00702	0.00002
W3	0.00414	0.18801	-0.00000
W4	0.16648	0.00468	-0.00001
W5	0.19441	0.00585	0.00004
W6	0.01875	0.14166	0.00003
W7	-0.01254	0.14036	-0.00003
W8	0.18355	0.14627	0.00001
W9	0.17734	-0.13574	0.00002
W10	0.11546	0.10878	-0.00003
W11	0.15987	0.11063	0.00005
W12	0.11080	-0.10273	-0.00002
W13	0.15521	-0.10088	0.00006
E2	0.09959	0.00294	0.00001
E3	0.10476	0.00316	0.00002
E4	0.00537	0.07456	0.00001
E5	-0.00013	0.07433	-0.00000

Existing Building Story Displacements cont'd

Story Displacements

RAM Frame 14.06.01.00
DataBase: Existing Building RAM Model April 7
Building Code: IBC

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04/07/15 16:25:16

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Level: 2nd floor, Diaph: 1

Center of Mass (ft): (70.27, -107.01)

LdC	Disp X	Disp Y	Theta Z
	in	in	rad
D	-0.02086	0.04525	0.00001
Lp	-0.01441	0.03078	0.00000
E1	0.11116	0.00923	0.00002
W1	0.09501	0.00770	0.00001
W2	0.10017	0.00809	0.00001
W3	0.00159	0.08357	-0.00000
W4	0.06700	0.00534	-0.00000
W5	0.08325	0.00680	0.00002
W6	0.01029	0.06350	0.00001
W7	-0.00790	0.06186	-0.00001
W8	0.07632	0.06875	0.00001
W9	0.07393	-0.05661	0.00001
W10	0.04432	0.05040	-0.00001
W11	0.07016	0.05272	0.00003
W12	0.04253	-0.04362	-0.00001
W13	0.06837	-0.04129	0.00003
E2	0.04108	0.00339	0.00000
E3	0.04410	0.00363	0.00001
E4	0.00278	0.03307	0.00000
E5	-0.00043	0.03281	-0.00000

Existing Building Story Displacements cont'd

Center of Rigidity

RAM Frame 14.06.01.00
DataBase: RAM Model Gravity and Lateral March 29 04/01/15 17:33:48
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CRITERIA:

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

			Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Type**	**Xr** ft	**Yr** ft	**Xm** ft	**Ym** ft
Main Roof Level	1	Rigid	128.30	-127.04	69.30	-111.86
5th Floor	1	Rigid	125.77	-123.34	69.94	-110.29
4th Floor	1	Rigid	122.08	-118.03	69.98	-110.15
3rd Floor	1	Rigid	115.90	-108.87	70.91	-105.24
2nd floor	1	Rigid	107.29	-95.49	70.96	-105.04

			Story Lateral Stiffness	
Level	**Diaph. #**	**Type**	**KX** kips/ ft	**KY** kips/ ft
Main Roof Level	1	Rigid	39048.61	59632.67
5th Floor	1	Rigid	53978.85	82333.23
4th Floor	1	Rigid	68228.74	101710.96
3rd Floor	1	Rigid	104353.22	151009.51
2nd floor	1	Rigid	163043.16	218744.04

NOTES:
Center of rigidity (CR) values given above are only used for load cases that require explicit calculation of CRs for use in calculation of load eccentricities (for example, ASCE 7-05 Wind Load Case).

Note that this information is never used for analysis. On the other hand, it should be noted that analysis results always include any torsional effects due to having center of rigidity and mass center at different locations. In other words, the analysis always accounts for locations and stiffnesses of frame members and diaphragms. Hence, any torsional effects of the masses being offset from the stiffnesses (i.e., CR) are implicitly and correctly accounted in the analysis.

Redesign System Center of Rigidity

Story Displacements

RAM Frame 14.06.01.00
DataBase: RAM Model Gravity and Lateral March 29 04/01/15 17:33:48
Building Code: IBC
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CRITERIA:

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

LOAD CASE DEFINITIONS:

D	DeadLoad	RAMUSER
Lp	PosLiveLoad	RAMUSER
Rfp	PosRoofLiveLoad	RAMUSER
W1	Wind 2	Wind_IBC09_1_X
W2	Wind 2	Wind_IBC09_1_Y
W3	Wind 2	Wind_IBC09_2_X+E
W4	Wind 2	Wind_IBC09_2_X-E
W5	Wind 2	Wind_IBC09_2_Y+E
W6	Wind 2	Wind_IBC09_2_Y-E
W7	Wind 2	Wind_IBC09_3_X+Y
W8	Wind 2	Wind_IBC09_3_X-Y
W9	Wind 2	Wind_IBC09_4_X+Y_CW
W10	Wind 2	Wind_IBC09_4_X+Y_CCW
W11	Wind 2	Wind_IBC09_4_X-Y_CW
W12	Wind 2	Wind_IBC09_4_X-Y_CCW
E1	Seismic	EQ_IBC09_X_+E_F
E2	Seismic	EQ_IBC09_X_-E_F
E3	Seismic	EQ_IBC09_Y_+E_F
E4	Seismic	EQ_IBC09_Y_-E_F
ND1	Notional	NL_AISC360_DL_X
ND2	Notional	NL_AISC360_DL_Y
NL1	Notional	NL_AISC360_LL_X
NL2	Notional	NL_AISC360_LL_Y
NR1	Notional	NL_AISC360_Rf_X
NR2	Notional	NL_AISC360_Rf_Y

Level: Main Roof Level, Diaph: 1

Center of Mass (ft): (69.30, -111.86)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.01926	0.01092	0.00001

Redesign System Story Displacements

Story Displacements

RAM Frame 14.06.01.00 Page 2/5
DataBase: RAM Model Gravity and Lateral March 29 04/01/15 17:33:48
Building Code: IBC

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Lp	-0.00987	0.00575	0.00000
Rfp	-0.00141	0.00080	0.00000
W1	0.27178	0.03370	-0.00001
W2	0.03172	0.19373	-0.00004
W3	0.20766	0.04243	-0.00004
W4	0.20001	0.00812	0.00001
W5	0.01943	0.12573	-0.00000
W6	0.02815	0.16487	-0.00006
W7	0.22763	0.17058	-0.00004
W8	0.18005	-0.12002	0.00002
W9	0.17686	0.15548	-0.00007
W10	0.16458	0.10039	0.00001
W11	0.14117	-0.06247	-0.00003
W12	0.12890	-0.11756	0.00005
E1	0.13569	0.01891	-0.00001
E2	0.13395	0.01122	0.00000
E3	0.01374	0.08525	-0.00001
E4	0.01560	0.09343	-0.00002
ND1	0.08567	0.00951	-0.00000
ND2	0.00942	0.05680	-0.00001
NL1	0.01844	0.00191	-0.00000
NL2	0.00195	0.01231	-0.00000
NR1	0.00568	0.00068	-0.00000
NR2	0.00064	0.00375	-0.00000

Level: 5th Floor, Diaph: 1

Center of Mass (ft): (69.94, -110.29)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.01408	0.00784	0.00001
Lp	-0.00727	0.00439	0.00000
Rfp	-0.00102	0.00050	0.00000
W1	0.22215	0.02549	-0.00001
W2	0.02440	0.15954	-0.00003
W3	0.16955	0.03267	-0.00003
W4	0.16366	0.00556	0.00001
W5	0.01494	0.10419	0.00000
W6	0.02166	0.13512	-0.00005
W7	0.18491	0.13877	-0.00003
W8	0.14831	-0.10054	0.00002
W9	0.14341	0.12584	-0.00006
W10	0.13395	0.08231	0.00001
W11	0.11596	-0.05364	-0.00002
W12	0.10651	-0.09717	0.00004
E1	0.10981	0.01432	-0.00001
E2	0.10846	0.00830	0.00000

Redesign System Story Displacements cont'd

Story Displacements

RAM Frame 14.06.01.00 Page 3/5
DataBase: RAM Model Gravity and Lateral March 29 04/01/15 17:33:48
Building Code: IBC

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E3	0.01053	0.08963	-0.00001
E4	0.01196	0.07605	-0.00002
ND1	0.06871	0.00712	-0.00000
ND2	0.00720	0.04585	-0.00001
NL1	0.01541	0.00145	-0.00000
NL2	0.00151	0.01038	-0.00000
NR1	0.00442	0.00051	-0.00000
NR2	0.00048	0.00292	-0.00000

Level: 4th Floor, Diaph: 1

Center of Mass (ft): (69.98, -110.15)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.00918	0.00508	0.00000
Lp	-0.00478	0.00300	0.00000
Rfp	-0.00066	0.00029	0.00000
W1	0.16866	0.01760	-0.00001
W2	0.01668	0.12253	-0.00002
W3	0.12825	0.02322	-0.00002
W4	0.12474	0.00318	0.00001
W5	0.01051	0.08047	0.00000
W6	0.01451	0.10333	-0.00004
W7	0.13901	0.10510	-0.00002
W8	0.11399	-0.07870	0.00001
W9	0.10707	0.09491	-0.00004
W10	0.10144	0.06274	0.00001
W11	0.08831	-0.04294	-0.00002
W12	0.08268	-0.07511	0.00003
E1	0.08256	0.00993	-0.00001
E2	0.08175	0.00552	0.00000
E3	0.00725	0.05309	-0.00001
E4	0.00811	0.05779	-0.00001
ND1	0.05123	0.00484	-0.00000
ND2	0.00490	0.03455	-0.00001
NL1	0.01198	0.00100	-0.00000
NL2	0.00105	0.00817	-0.00000
NR1	0.00320	0.00035	-0.00000
NR2	0.00033	0.00214	-0.00000

Level: 3rd Floor, Diaph: 1

Center of Mass (ft): (70.91, -105.24)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.00472	0.00260	0.00000
Lp	-0.00248	0.00160	0.00000

Redesign System Story Displacements cont'd

Story Displacements

RAM Frame 14.06.01.00 Page 4/5
DataBase: RAM Model Gravity and Lateral March 29 04/01/15 17:33:48
Building Code: IBC

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Rfp	-0.00053	0.00013	0.00000
W1	0.10660	0.00941	-0.00000
W2	0.00955	0.07855	-0.00001
W3	0.08114	0.01301	-0.00001
W4	0.07876	0.00111	0.00001
W5	0.00581	0.05213	0.00000
W6	0.00852	0.06570	-0.00002
W7	0.08711	0.06597	-0.00001
W8	0.07278	-0.05186	0.00001
W9	0.06724	0.05903	-0.00003
W10	0.06343	0.03992	0.00001
W11	0.05649	-0.02934	-0.00001
W12	0.05268	-0.04845	0.00002
E1	0.05182	0.00537	-0.00000
E2	0.05128	0.00277	0.00000
E3	0.00407	0.03394	-0.00000
E4	0.00465	0.03672	-0.00001
ND1	0.03182	0.00254	-0.00000
ND2	0.00277	0.02178	-0.00000
NL1	0.00779	0.00053	-0.00000
NL2	0.00061	0.00540	-0.00000
NR1	0.00196	0.00018	-0.00000
NR2	0.00018	0.00133	-0.00000

Level: 2nd floor, Diaph: 1

Center of Mass (ft): (70.96, -105.04)

LdC	Disp X in	Disp Y in	Theta Z rad
D	-0.00140	0.00078	0.00000
Lp	-0.00074	0.00050	0.00000
Rfp	-0.00010	0.00004	0.00000
W1	0.04682	0.00293	-0.00000
W2	0.00288	0.03562	-0.00001
W3	0.03524	0.00462	-0.00001
W4	0.03499	-0.00021	0.00000
W5	0.00201	0.02396	0.00000
W6	0.00230	0.02947	-0.00001
W7	0.03727	0.02892	-0.00000
W8	0.03296	-0.02452	0.00000
W9	0.02816	0.02557	-0.00001
W10	0.02775	0.01781	0.00000
W11	0.02492	-0.01451	-0.00000
W12	0.02451	-0.02227	0.00001
E1	0.02254	0.00175	-0.00000
E2	0.02248	0.00070	0.00000
E3	0.00130	0.01537	-0.00000

Redesign System Story Displacements cont'd

Story Displacements

RAM Frame 14.06.01.00 Page 5/5
DataBase: RAM Model Gravity and Lateral March 29 04/01/15 17:33:48
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E4	0.00138	0.01649	-0.00000
ND1	0.01365	0.00076	0.00000
ND2	0.00084	0.00964	-0.00000
NL1	0.00357	0.00015	0.00000
NL2	0.00019	0.00255	-0.00000
NR1	0.00078	0.00006	-0.00000
NR2	0.00006	0.00055	-0.00000

Redesign System Story Displacements cont'd

Redesign Wind Drifts (E-W)		
	Story Drift (in)	Total Drift (in)
Main Roof	0.272	0.816
Level 5	0.222	0.544
Level 4	0.169	0.322
Level 3	0.106	0.153
Level 2	0.047	0.047

Existing Wind Drifts (E-W)		
	Story Drift (in)	Total Drift (in)
Main Roof	0.555	1.764
Level 5	0.488	1.209
Level 4	0.38	0.721
Level 3	0.241	0.341
Level 2	0.1	0.1

Redesign Wind Drifts (N-S)		
	Story Drift (in)	Total Drift (in)
Main Roof	0.194	0.592
Level 5	0.16	0.398
Level 4	0.123	0.238
Level 3	0.079	0.115
Level 2	0.036	0.036

Existing Wind Drifts (N-S)		
	Story Drift (in)	Total Drift (in)
Main Roof	0.409	1.329
Level 5	0.363	0.92
Level 4	0.285	0.557
Level 3	0.188	0.272
Level 2	0.084	0.084

Redesign Seismic Drift		
	Story Drift (in)	Total Drift (in)
Main Roof	0.136	0.404
Level 5	0.11	0.268
Level 4	0.083	0.158
Level 3	0.052	0.075
Level 2	0.023	0.023

Existing Seismic Drift		
	Story Drift (in)	Total Drift (in)
Main Roof	0.244	0.751
Level 5	0.208	0.507
Level 4	0.158	0.299
Level 3	0.1	0.141
Level 2	0.041	0.041

Appendix J

Green Roof Materials Technical Information

Basic Product Specs

Please contact a LiveRoof Representative for access to the LiveRoof® Spec Writer for customized 3 part green roof specifications.

Get a Detailed Spec

MODULE SIZE	**LiveRoof Standard:** 1' x 2' x 3-1/4" (soil height appx. 4-1/4") **LiveRoof Lite:** 1' x 2' x 1-3/4" (soil height appx. 2-1/2") **LiveRoof Deep:** 1' x 2' x 3-1/4" (soil height appx. 6") **LiveRoof Maxx:** 1' x 1' x 3-1/4" (soil height appx. 8") Soil fills soil elevator, plants and soil obscure module edges.
MODULE WEIGHT	**Standard and Deep:** 14 oz./sq. ft. **Lite:** 10.5 oz./sq. ft. **Maxx:** 14 oz./sq. ft.
MATERIAL	100% recycled polypropylene (avg. 10% post-consumer, 90% post-industrial) 100 mil. thick walls.
WATER DISPERSAL	Approx. 10.0 gal. per min. per lineal foot. *Hi-Flow option available with standard and deep module.*
MODULE COLOR	Black or gray
WEIGHT VEGETATED (fully saturated)	**LiveRoof Standard:** approx. 27-29 lbs./SF

LiveRoof Lite: approx. 15-17 lbs./SF

LiveRoof Deep: approx. 40-50 lbs./SF
LiveRoof Maxx 8": approx. 55-65 lbs/SFDRAINAGEPositive drain holes, at lowest point in module.

SOIL MEDIAProprietary LiveRoof specified engineered soil, based upon German FLL granulometric specifications, 94+% by dry weight inorganic content for minimal shrinkage/decomposition. (92% in British Columbia).
Dry weight approx. 60-65 lbs/cu.ft.
May vary somewhat with local grower.

ACCEPTABLE PROTECTIVE UNDERLYING MATERIALSModules to be placed directly upon heavy duty (HDPE, Polypropylene, TPO, EPDM or recyclable PVC) slip sheet/root barrier of 40-60 mil. thickness with effectively bonded seams. This is placed as an additional protective barrier above roof waterproofing membrane and extended 3 inches vertically along parapet to ward against edge abrasion. This may also be glued to parapet if manufacturer approves.

Confirm suitability of waterproofing membrane with manufacturer. Alternatively low profile drain boards work well and manufacturers of cold fluid applied reinforced urethane membranes typically warrant their systems for use in conjunction with the LiveRoof® system.

IRRIGATION SYSTEMIrrigation is recommended for backup during prolonged hot, dry and windy weather patterns.Simple overhead system is inexpensive and effective insurance. *Irrigation requirements are dependent on plant selection, climate and roof design.*

> In hot, humid or arid climates, irrigation systems should always be installed and used as needed given weather conditions.
> Similarly, irrigation systems are necessary on pitched green roofs and those in wind-challenged conditions, such as in coastal areas and on tall buildings.

> If LiveRoof Lite system is used, irrigation will be essential in all climates.
> If the Deep system is used and populated with non-succulents, irrigation is also essential.

EDGE TREATMENTSCoengineered RoofEdge® aluminum edging with adequate drain perforations recommended. Any edging should allow for adequate drainage (extending to the bottom of the edging) with sidewalls tall enough to completely cover the modules and contain the soil.PAVERSCoengineered LiveRoof RoofStone® recommended.WIND UPLIFTPatent-pending WindDisc™ method for improving wind uplift resistance is recommended for green roofs subject to high wind conditions.PLANTSDrought-tolerant, hardy RoofTop Proven™ plants recommended. Consult the Licensed Grower in your region for specific recommendations.CONVEYANCE METHODPrevegetated modules to be delivered by Hoppit® or other appropriately engineered conveyance device.

http://www.liveroof.com/basic-product-specs/

ANCHOR™
BUILD SOMETHING BEAUTIFUL

Paver Benefits

SEGMENTAL PAVING - The most versatile option featuring individual units placed by hand or machine. Superior design flexibility and an upgraded appearance stand out from typical paving applications.

PAVEMENT TYPES	APPEARANCE	INITIAL COST & INSTALLATION	MAINTENANCE	WINTER DURABILITY	SNOW REMOVAL
Concrete Pavers Best Choice	The widest range of surface finishes, colors, shapes, and sizes. Laying patterns can complement the architectural style of any home because of the wide variety of styles available.	Moderate - Tightly fitted, uniform units are placed over a sand bed and a compacted aggregate base. Immediately ready for use. Can be installed by homeowner or an ICPI Certified Professional.	Low—Stained or broken pavers can be easily replaced without patches. Dark colored pavers can help hide stains. Factory-made pavers last for decades.	High—Small, high density units resist cracking as well as damage from freeze-thaw cycles and salts. Pavers are stronger than ordinary or stamped concrete.	Smooth surface allows for easy snow removal. Darker colored pavers help melt snow faster. Snow-melt systems can be easily integrated to eliminate snow and ice removal.
Cobble Stone	Gives elegant, permanent, yet informal "Old World" feel.	Highest—Non-uniform units must be fitted together by hand.	Low—High quality stone lasts for decades. Wide joints may encourage weeds and ants. Rough surface makes walking and driving dificult.	High—High density stone resists cracking and salts.	Rough surface makes plowing difficult.
Clay Brick	Traditionally comes in shades of red and red-brown. Limited shapes and sizes.	Moderate—High-Mortar-set base may be used which increases costs. Natural variations in dimensions may slow installation or cause difficulty in maintaining straight pattern lines.	Low—Natural surface variations may lead to chipping or possible damage.	Moderate—Salts may cause deterioration in some clay pavers.	Smooth surface allows for easy snow removal. Darker colored pavers help melt snow faster.
Other Paving Options					
Stamped Concrete	Surface is usually colored. Patterns designed to give appearance of segmental paving but saw-cut joints may show.	High—Difficult for homeowner to install. Requires special equipment to stamp stone or paver patterns into surface. Surface sealer often used.	Moderate—Cracking may likely develop. Patched repairs may be hard to match to original color. Color fading also possible over time.	Low—Potential for deterioration from de-icing salts.	Uneven surface of some patterns and textures may make plowing difficult.
Ordinary Concrete	Grey or light brown. Can be colored throughout or on surface only.	Moderate—Difficult for homeowner to install and requires 5 to 7 days for hardening before use. Surface quality varies with weather and installation.	Moderate—Cracking may likely develop. Repairs and replaced sections may leave visible patches. Oil stains difficult to remove.	Low—Cracks from freeze-thaw cycles, settlement and salt deterioration may occur.	Smooth initial surface allows for easy removal. Light colored surface may not melt snow rapidly
Asphalt	Few color options. Achieving neat looking edges may be difficult. Stamped asphalt appears painted and artificial	Low—Installs quickly over compacted aggregate base. Must be professionally installed.	High—Wear and weather will break down surface. Black seal coat required every 2-3 years. Rut or pothole repairs leave visible patches. Subject to erosion from oil drippings.	Low—Cracks from freeze-thaw cycles, settlement and salt deterioration may occur.	Smooth surface allows for easy removal. Dark surface accelerates snow melting.
Crushed Stone or Gravel	Typically rustic look. Appearance varies with color and shape of stones.	Low—Dumped and spread over soil (no base required).	High—Scattered stone must be replaced and leveled regularly. Ruts from tires are likely to develop.	High—Stones resist freeze-thaw cycles and salts.	Stones and surface may become uneven during plowing.

Chart reference is taken from the ICPI's brochure "The Beauty of Choosing The Best Pavement. A comparison guide for consumers" and can be found at www.icpi.org.

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Appendix K

Waterproofing Membrane Specifications

MM6125 Physical Properties Chart | HIDE

PROPERTY	TEST METHOD	TYPICAL RESULTS
Flash Point	ASTM D-92, CGSB-37.50-M89	500 °F (260 °C)*
Low Temperature Crack Bridging Capability	CGSB-37.50-M89	No cracking, adhesion loss, or splitting
Water Vapor Permeability	ASTM E 96, Procedure E, CGSB-37.50-M89	1.6 ng/Pa(s)M^2, (0.018 perm)
Water Resistance (5 days/50 °C)	CGSB-37.50-M89	No delamination, blistering, emulsification, or deterioration
Water Absorption	CGSB-37.50-M89	0.22 g weight gain
Toughness	CGSB-37.50-M89	13.0 Joules
Ratio of Toughness to Peak Load	CGSB-37.50-M89	0.069
Viscosity	CGSB-37.50-M89	7.0 seconds
Heat Stability	CGSB-37.50-M89	No change in viscosity, penetration, flow or low temperature flexibility
Low Temperature Flexibility (- 25 °C)	CGSB-37.50-M89	No delamination, flexibility adhesion loss, or cracking
Penetration	ASTM D 1191, CGSB-37.50-M89	75.0 mm @ 77 °F (25 °C), 121.7 mm @ 122 °F (50 °C)
Flow	ASTM D 1191, GSB-37.50-M89	0.0 mm @ 140 °F (60 °C)
Softening Point	ASTM D 36	180 °F (82 °C)
Elongation	ASTM D 1191	1000 % minimum
Resiliency	ASTM D 3407	40% minimum
Bond to Concrete @ 0 °F, (18 °C)	ASTM D 3408	Pass
Hydrostatic Pressure Resistance	ASTM D-08.22, Draft 2	100 psi (=231 foot head of water)
Acid Resistance	ASTM D 896 Procedure 7.1 (N-8)	Pass 50% Nitric Acid, 50% Sulfuric Acid
Salt Water Resistance (20% sodium carbonate and calcium chloride)	ASTM D-896 similar	No delamination, blistering, emulsification, or deterioration
Fertilizer Resistance (undiluted 15/5/5 nitrogen /phosphorus/potash)	ASTM D-896 similar	No delamination, blistering, emulsification, or deterioration
Animal Waste Resistance	3 year exposure	No deterioration
Solids Content	NO	100% no solvents
Shelf Life		10 years (sealed)
Specific Gravity		1.23 ± .02
45 °F more than the application temperature recommended by the manufacturer.		

American Hydrotech MM6125

TYPICAL PHYSICAL PROPERTIES
(Meets or exceeds CGSB-37.50 M89 Standards)

Properties	Test Method	Test Requirement	Test Results	Comments
Color	NA	NONE	N.A.	Black
Softening Point	ASTM-D-36		83°C (181°)	Pass
Solids Content	CGSB-37-GP-50	100%	100%	Pass
Ratio of toughness to peak load	CGSB-37-GP-50	Min.0.040	0.059	Pass
Low temperature crack bridging capacity	CGSB-37-GP-50	No Cracking No Adhesion Loss No Splitting	No Cracking No Adhesion Loss No Splitting	Pass
Toughness, J	CGSB-37-GP-50	Min. 5.5	11.7	Pass
Penetration 0.1 mm	CGSB-37-GP-50	Max 110 @ 25°C (77°F) Max 200 @ 50° C (122°F)	80 @ 25° C 155 @ 50° C	Pass
Flow, MM	CGSB-37-GP-50	Max 3 @ 60°C (140°)	0.50 @ 60° C	Pass
Flash Point	CGSB-37-GP-50 ASTM-D-92	Min 260° C (500° F)	327°C (620°F)	Pass
Water Resistance 50° C (122°F) for 4 days	CGSB-37-GP-50 ASTM-D-92	No delamination No blistering No Emulsification No deterioration No pinholes	No delamination No blistering No Emulsification No deterioration No pinholes	Pass
Adhesion	CGSB-37-GP-50	Min. 1	1.2	Pass
Viscosity	CGSB-37-GP-50	Min 2, Max 15	4 Sec.	Pass
Water Vapor Permeability	CGSB-37-GP-50	Max 1.7 0.35 g max gain	0.18 ng/Pa.m2.s	Pass
Water absorption	CGSB-37-GP-50	Min 0.18 0.18 g max loss	0.22 g gain	Pass
Low Temperature flexibility & adhesion	CGSB-37-GP-50	No Cracking No delamination No adhesion lose	No Cracking No delamination No adhesion loss	Pass
Heat stability	CGSB-37-GP-50	Aged Samples, No change in viscosity, penetration flow or low temp	Aged Samples, No change in viscosity, penetration flow or low temp	Pass

Barret Roofing ram-Tough 250

TREMCO

TYPICAL PHYSICAL PROPERTIES

CAN/CGSB 37.50-M89 Specification for Hot-Applied, Rubberized Asphalt for Roofing and Waterproofing

Property	Requirement	Test Method	Result
Flash Point	Min. 500 °F (260 °C)	ASTM D92	>572 °F (300 °C)
Cone Penetration	Max. 110 dmm at 77 °F (25 °C) Max. 200 dmm at 122 °F (50 °C)	ASTM D3407	As received: <45 dmm at 77 °F (25 °C) <100 dmm at 122 °F (50 °C) After heat aging: <60 dmm at 77 °F (25 °C) <125 dmm at 122 °F (50 °C)
Flow	Max. 3 mm	ASTM D5329	As received: 0 mm After heat aging: 0 mm
Toughness	Min. 5.5 J	CAN/CGSB 37.50-M89; Section 4.4	>10J
Ratio of Toughness to Peak Load	Min. 0.040	CAN/CGSB 37.50-M89; Section 4.5	>0.15
Adhesion Rating	Threads shall be covered with membrane material	CAN/CGSB 37.50-M89; Section 4.6	Pass
Water Absorption	Max. 0.35 g gain in mass	CAN/CGSB 37.50-M89; Section 4.8	<0.3 g gain in mass
Pinholing	Shall not show more than one pinhole	CAN/CGSB 37.50-M89; Section 4.9	No pinholes
Low Temperature Flexibility	Shall not show any cracking	CAN/CGSB 37.50-M89; Section 4.10	As received: No cracking After heat aging: No cracking
Crack Bridging Capability	Shall not show any evidence of cracking, splitting or loss of adhesion	CAN/CGSB 37.50-M89; Section 4.11	Pass
Water Vapor Transmission – Dessicant Method	Max 1.7 ng/Pa*s*m²	ASTM E96	1.32 ng/Pa*s*m²
Viscosity Test	2 - 15 s	CAN/CGSB 37.50-M89; Section 4.13	Pass
Shelf Life			24 months when properly stored in original, unopened packaging
Specific Gravity			1.29

Tremco TREMproof 6100

Appendix L

Rubber Melter Specifications

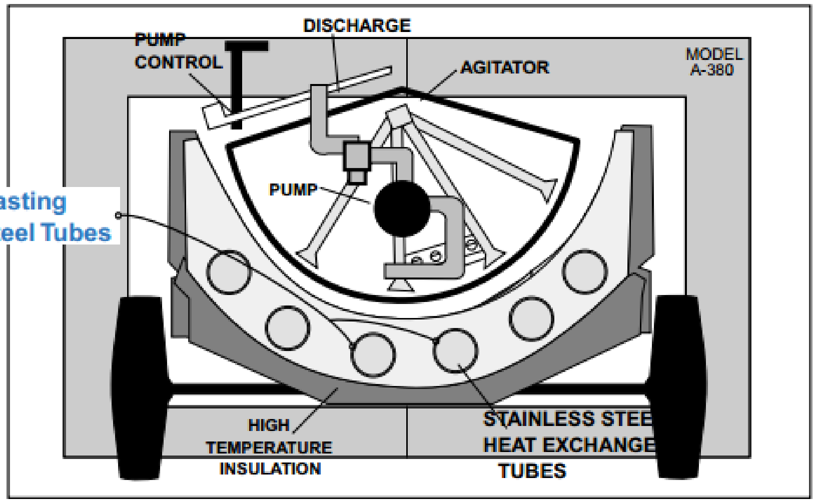
A&A MELTERS

1-888-469-4480

The Superior Air-Jacketed Hot Rubber Melters

380 FRONT END VIEW

Long Lasting Stainless Steel Tubes



SPECIFICATIONS: A-380 (360 US gallon capacity)

Heat Transfer Oil ....None Required Fire tubes..........5" Sch 80 Stainless steel
Capacity (Custom Sizes Available) .... Std. 360 US Gallons
Agitator ....Honda 9.0 H.P. Air-Cooled Engine or Optional Yanmar 7.0 HP Diesel Motor
Overall Dimensions.. L-172" W-80" H-70"
Burner.......Adjustable Liquid Propane 2 X 750,000 BTU or Optional Beckett Diesel Burners
Temperature Controls .... Optional
Inner Shell .... 1/4" Rolled Steel
Outer Shell .... 3/16" Rolled Steel
Insulated Jacket .... 1" Super High Temp. plus 2" High . Fiberglass
Suspension 4" Drop Axle .... 7000 lb. Axle and Springs
Tires .... 2 X 8.00 X 16" 10 Ply Rating
Tandem Axle .... Optional
Chassis .... 6" Steel Channel
Brakes .... Electric Standard
Tow Hitch .... As ordered c/w Safety Chains (2)
Pump .... Optional 2" Viking
Horizontal Wand Pumping System .... Optional
Shipping Weight .... 3,560 lbs.

Heated Material Output:
- material capacity: 360 US gallons 360 gal. /hr x 85.0% = **306 gal./hr**
- heat up time: 60 min's (1hr.) 1,639.3 L /hr x 85.0% = **1,393.3 L/hr.**

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