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Image Courtesy RTKL

CORPORATE HEADQUARTERS

Great Lakes Region, U.S.A.

FINAL REPORT

M. JULIA HAVERTY
STRUCTURAL OPTION
ADVISOR: H. SUSTERSIC
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

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

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.

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

<u>Building Location:</u> Great Lakes Region, U.S.A.

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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.

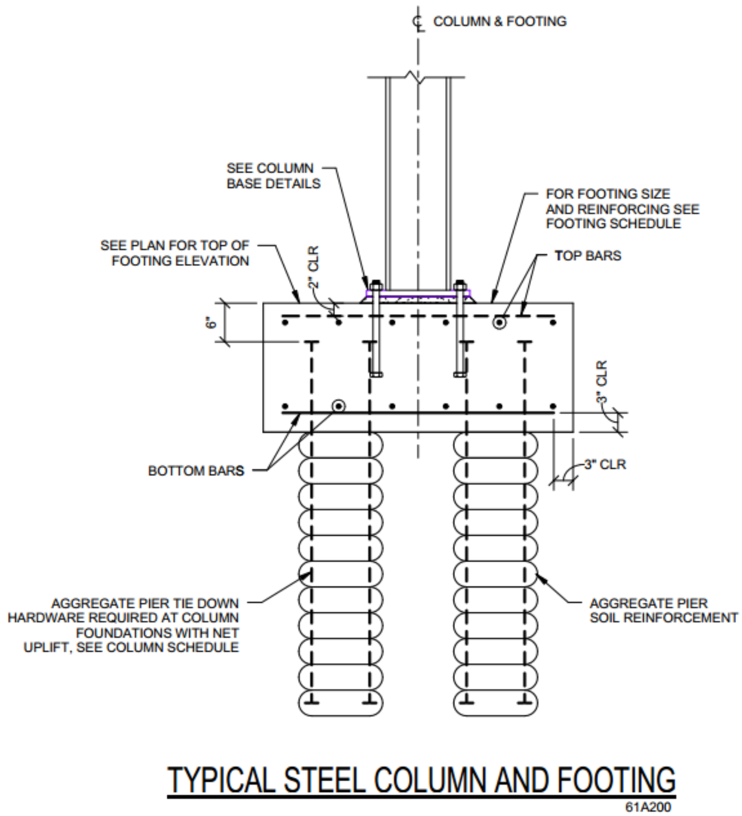


FIGURE 1- TYPICAL STEEL COLUMN AND FOOTING

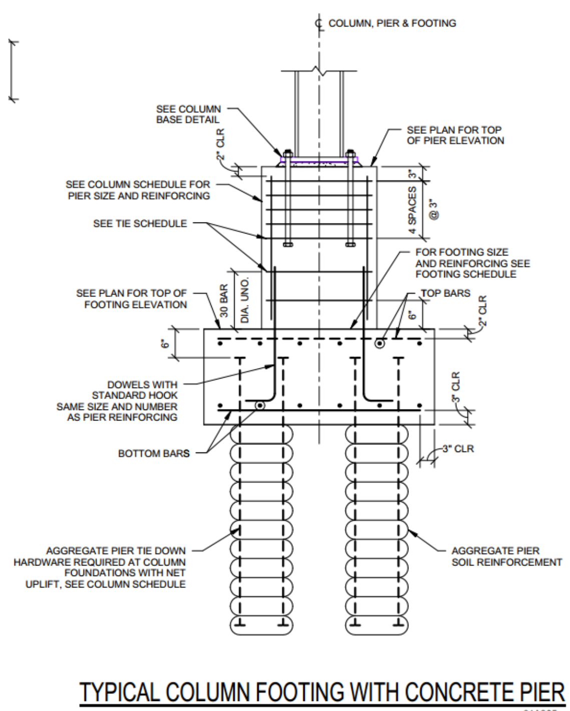


FIGURE 2- TYPICAL COLUMN FOOTING WITH CONCRETE PIER

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.

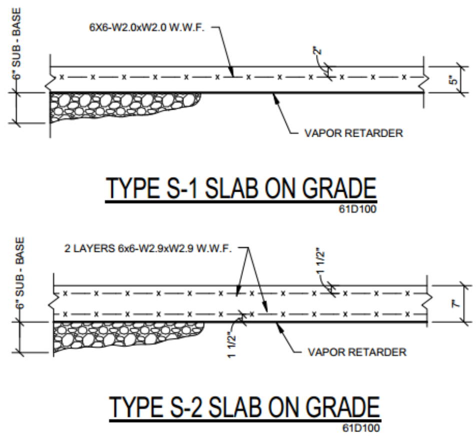


FIGURE 3- SLAB ON GRADE DETAILS

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.

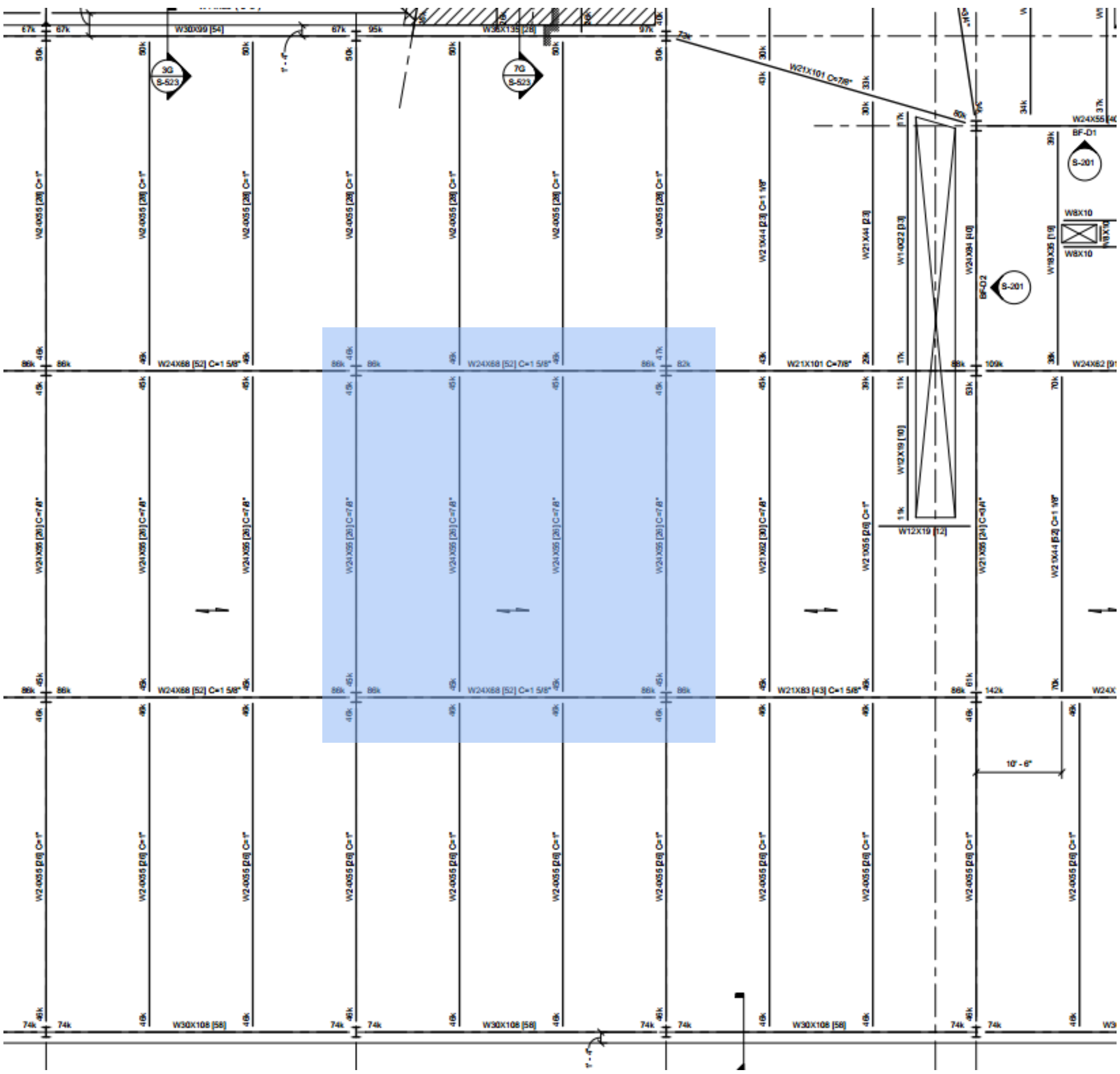
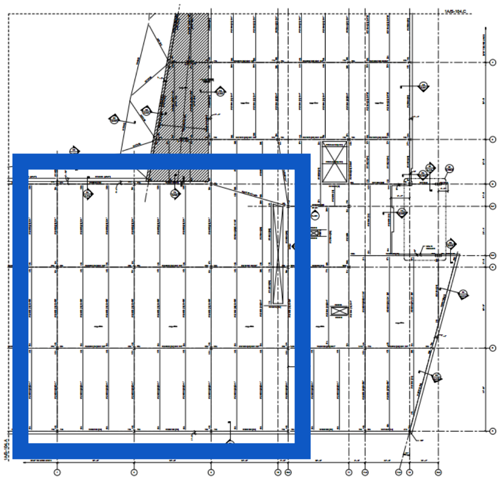


FIGURE 4: LEVEL 4 FRAMING PLAN SHOWING TYPICAL BAY (S104.D)



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.

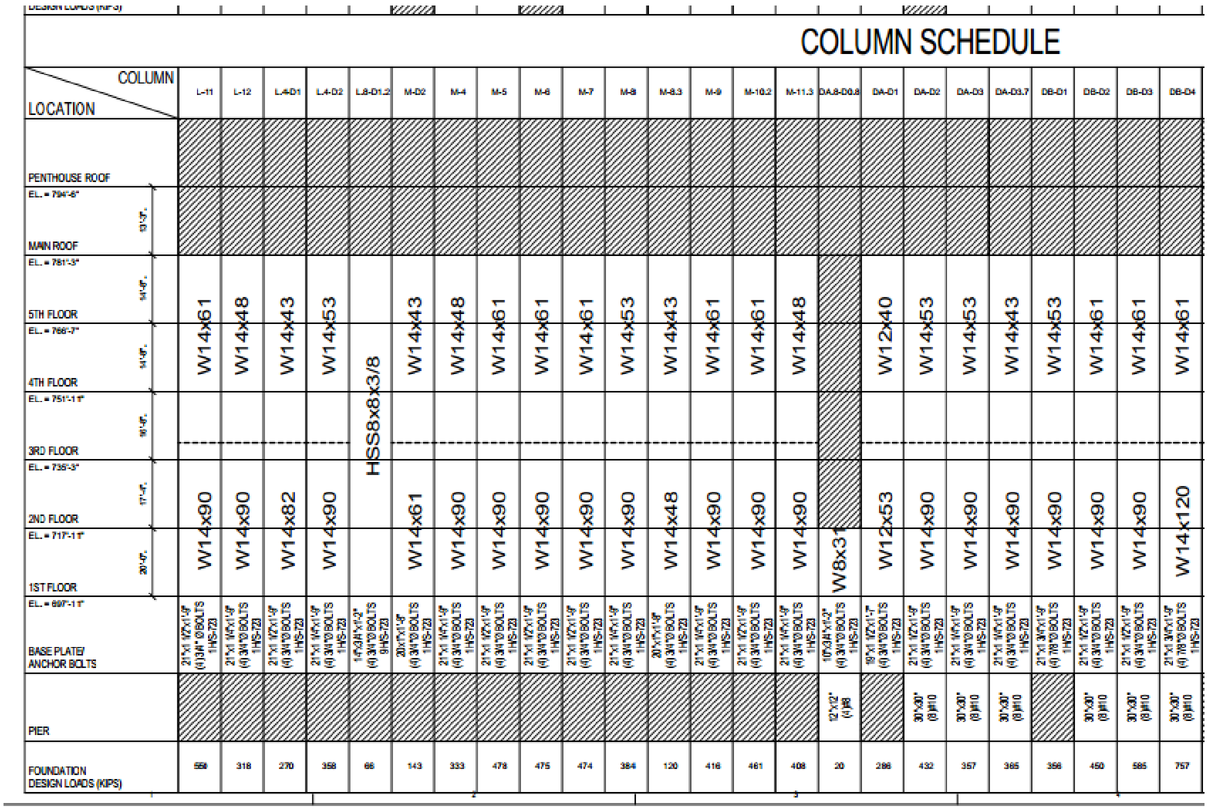


FIGURE 5- COLUMN SCHEDULE

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.

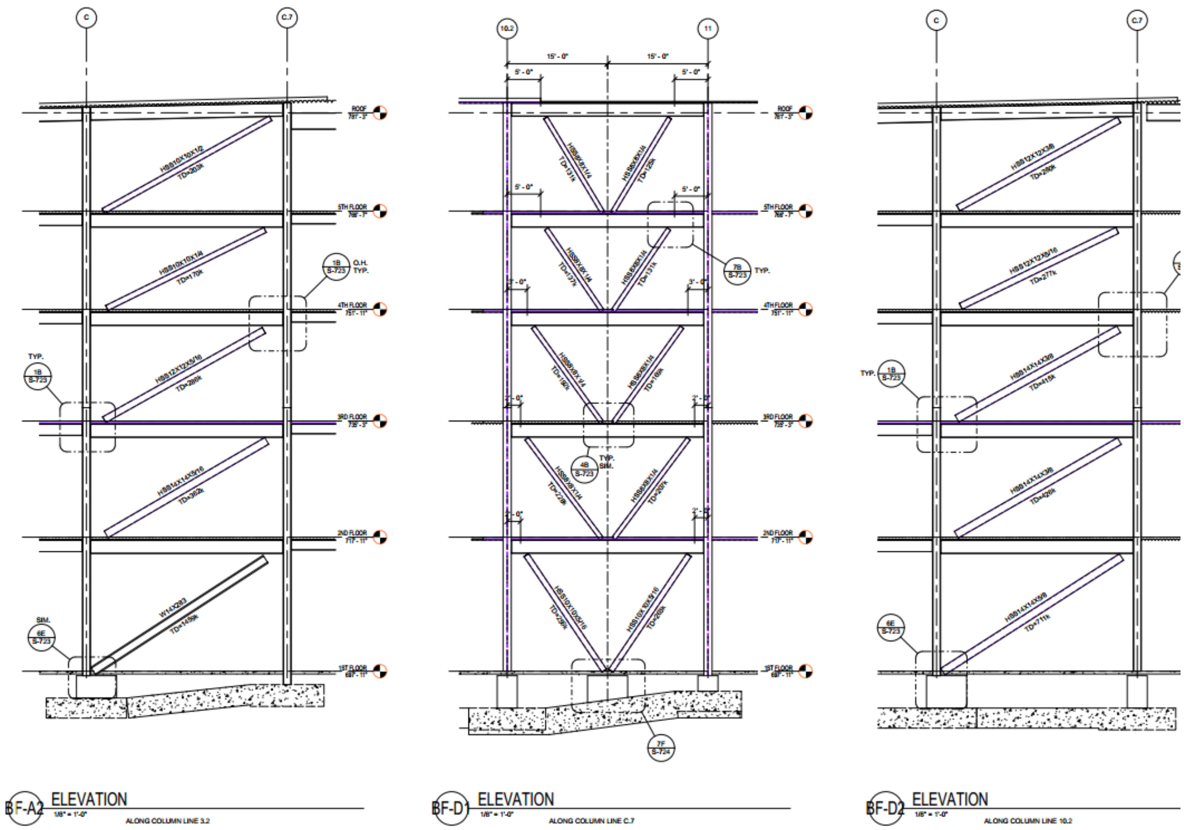


FIGURE 6- SAMPLE BRACED FRAME ELEVATIONS

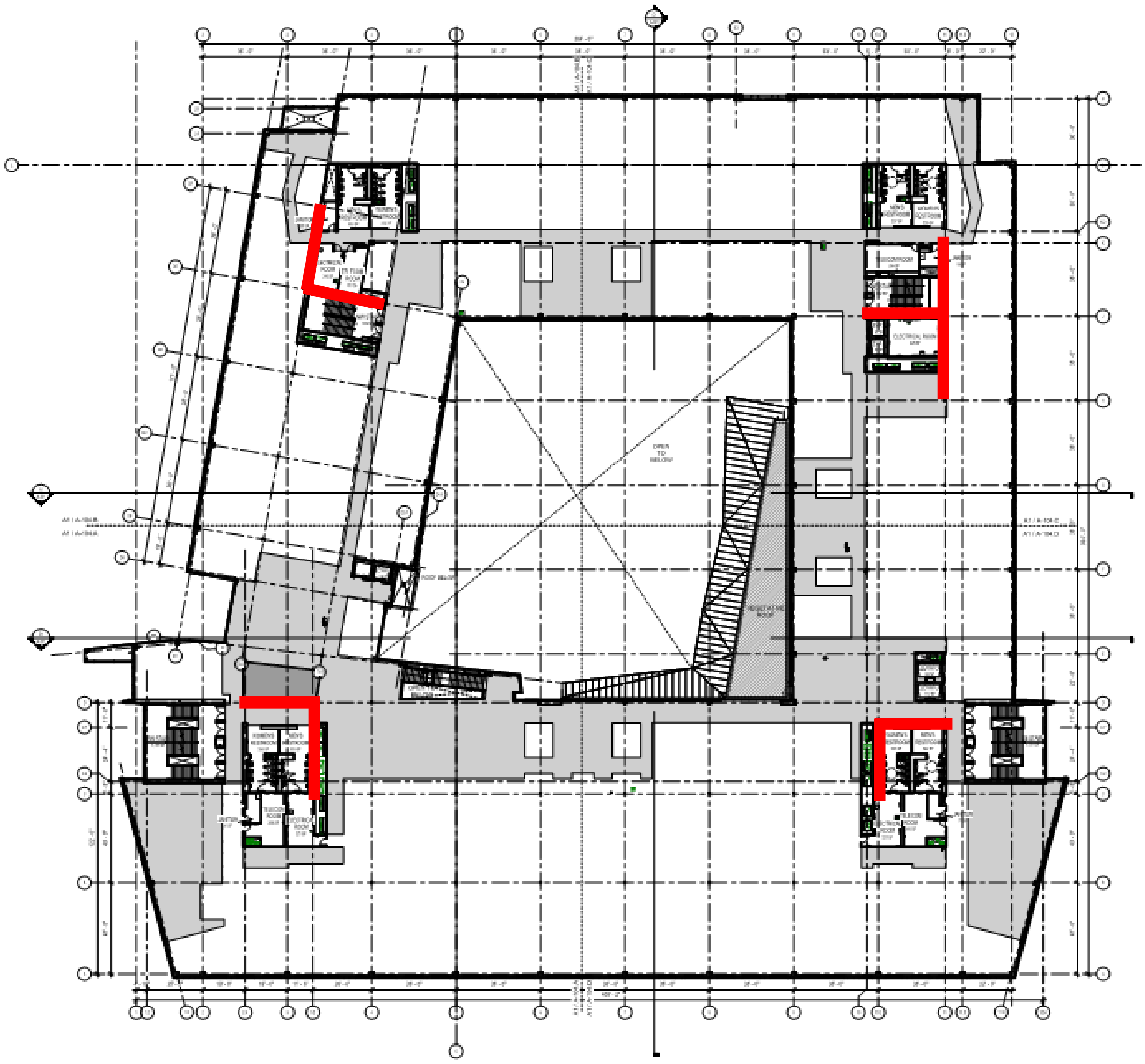


FIGURE 7-BRACED FRAME LOCATIONS

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

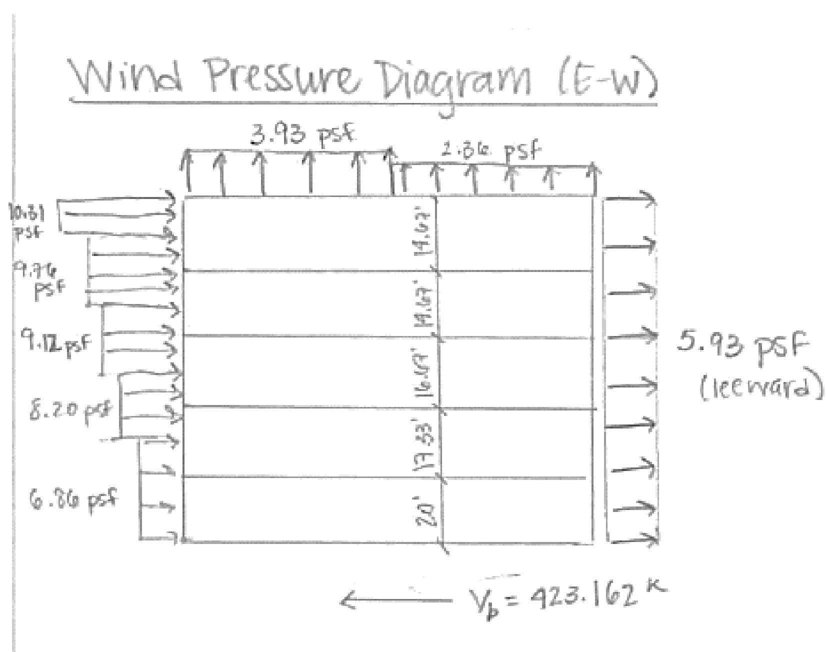


FIGURE 8-EAST WEST WIND PRESSURE DIAGRAM

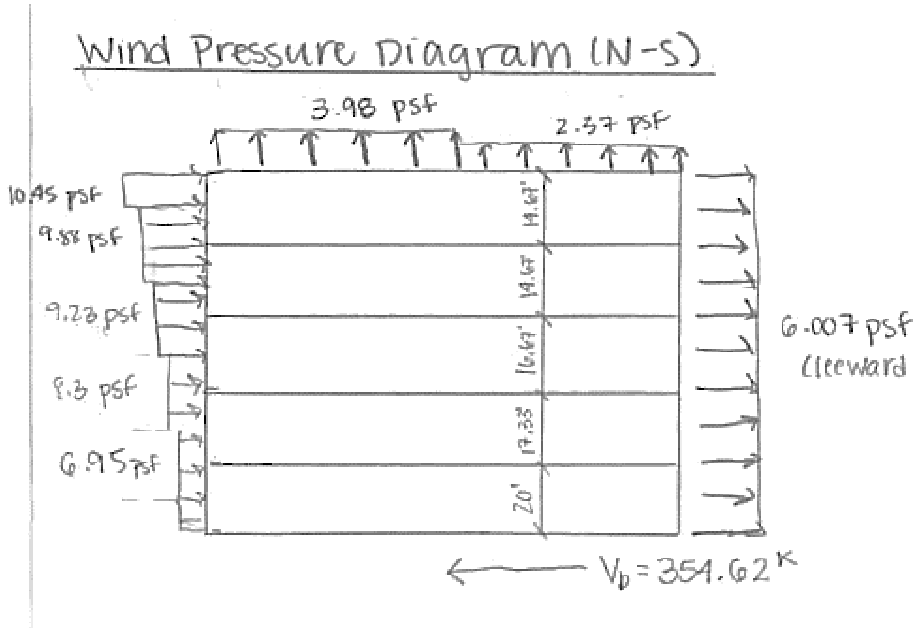


FIGURE 9-NORTH SOUTH WIND PRESSURE DIAGRAM

Seismic

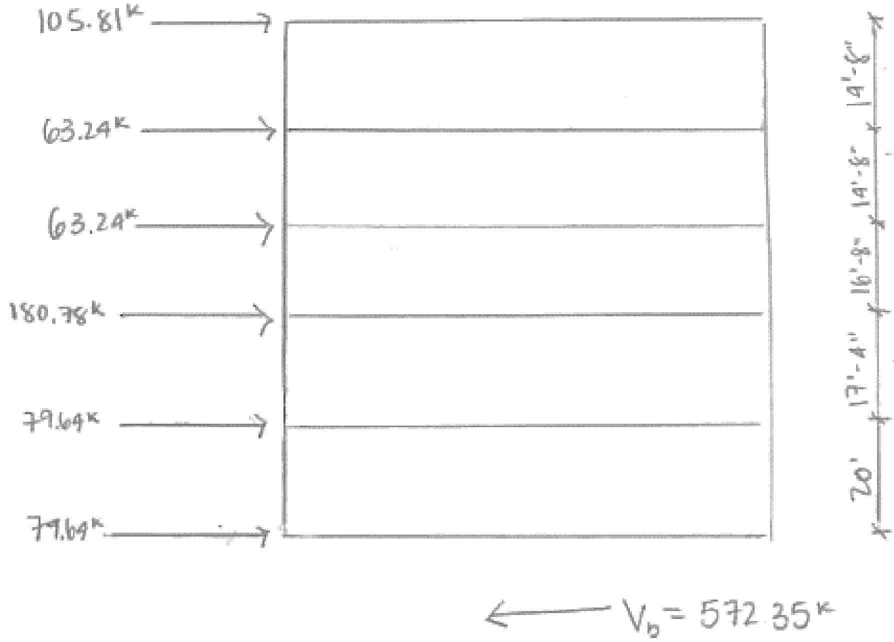
Seismic calculations were performed using the Equivalent Lateral Force Procedure found in ASCE 7-05. The building was analyzed as a true rectangle for ease of calculations. Full calculations may be found in Appendix G, and the figure below shows the vertical distribution of seismic forces. It was found 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.

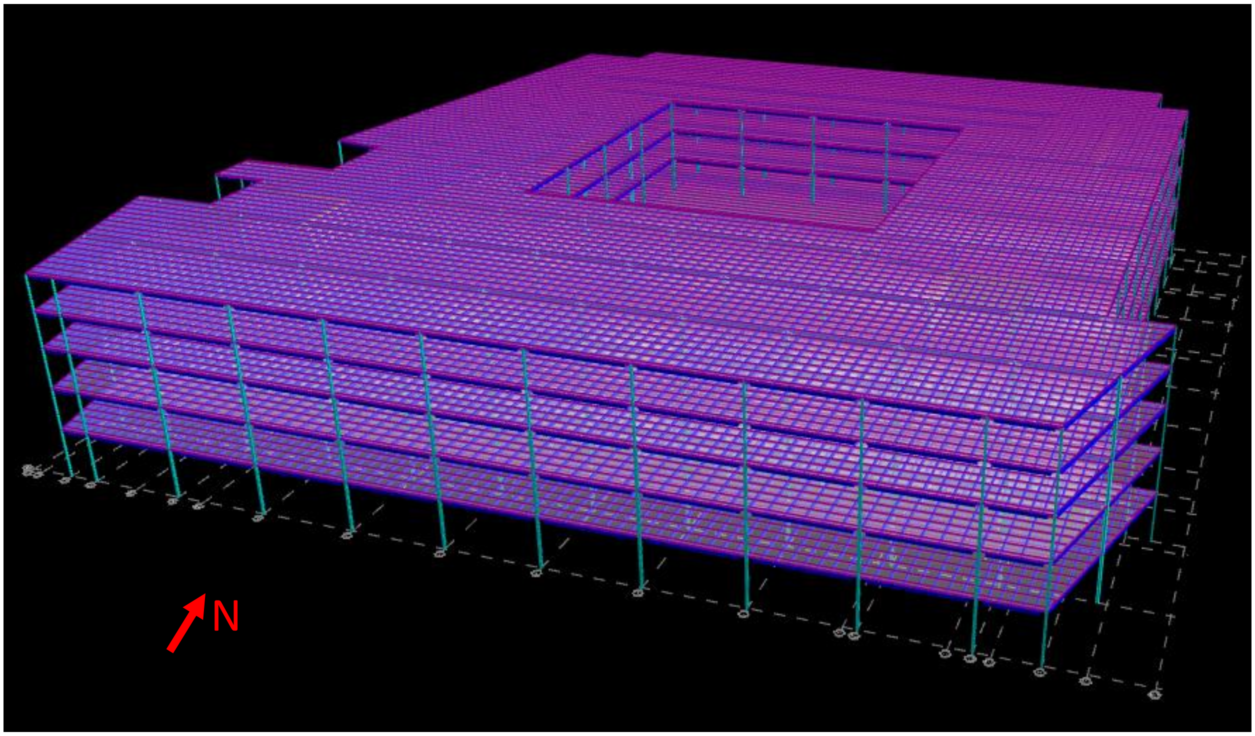


FIGURE 11- RAM GRAVITY MODEL

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 **Error! Reference source not found.** Spot checks of floor member sizes and RAM output samples may be found in Appendix D.

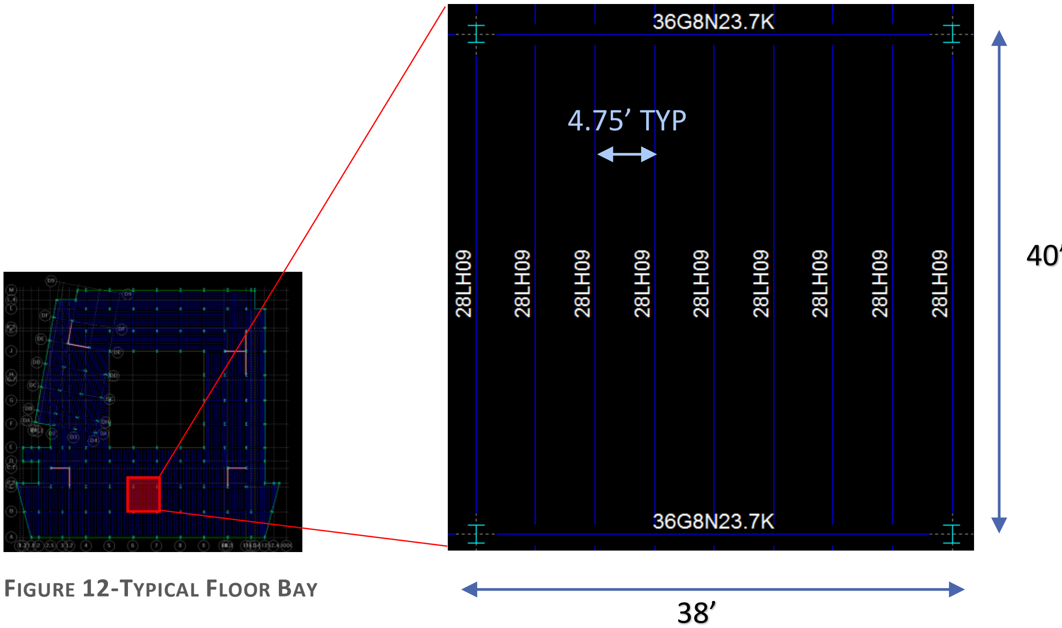


Figure 12-Typical Floor Bay

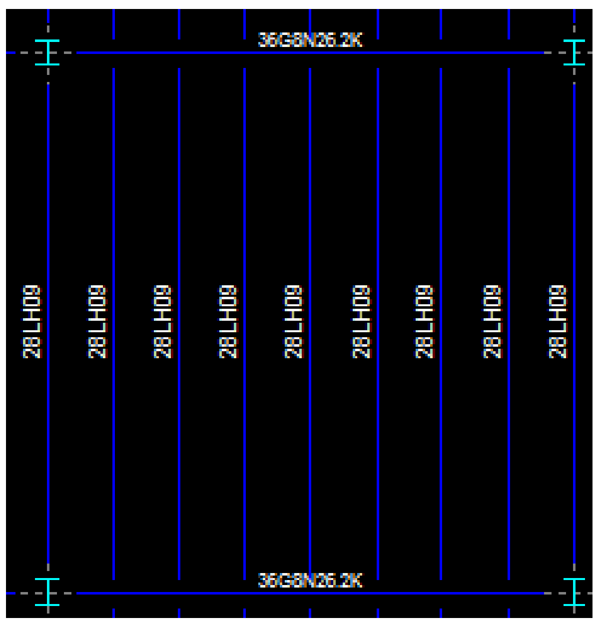


Figure 13- Typical Roof Bay

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.

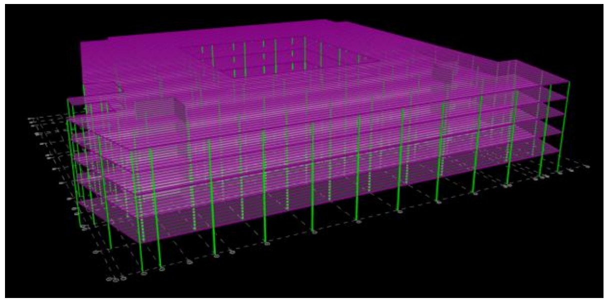


FIGURE 14-GRAVITY COLUMN ISOMETRIC VIEW

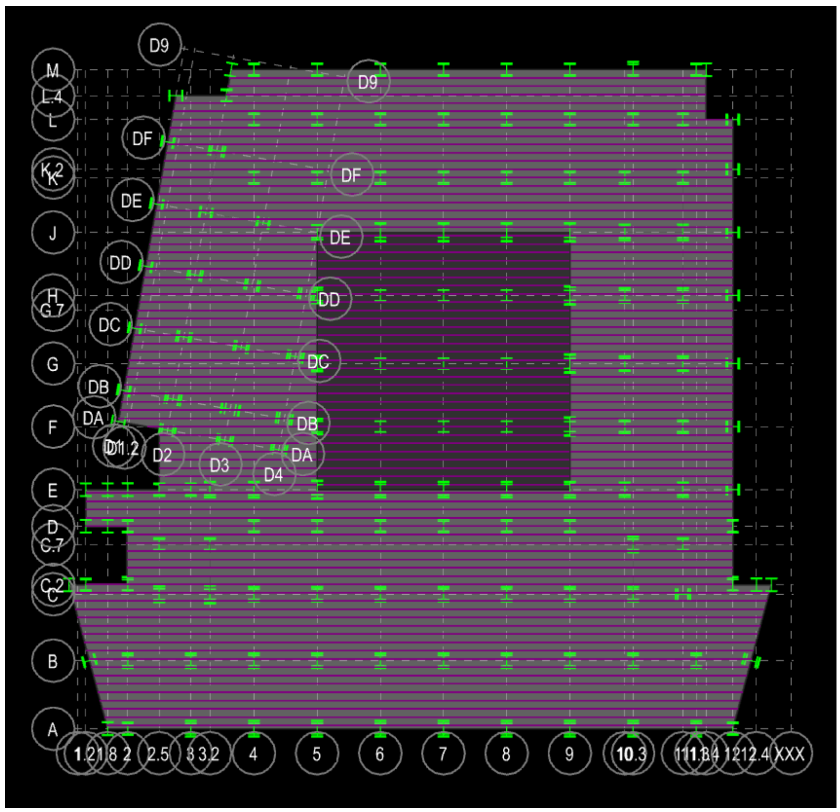


FIGURE 15- GRAVITY COLUMN PLAN VIEW (4TH FLOOR)

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.

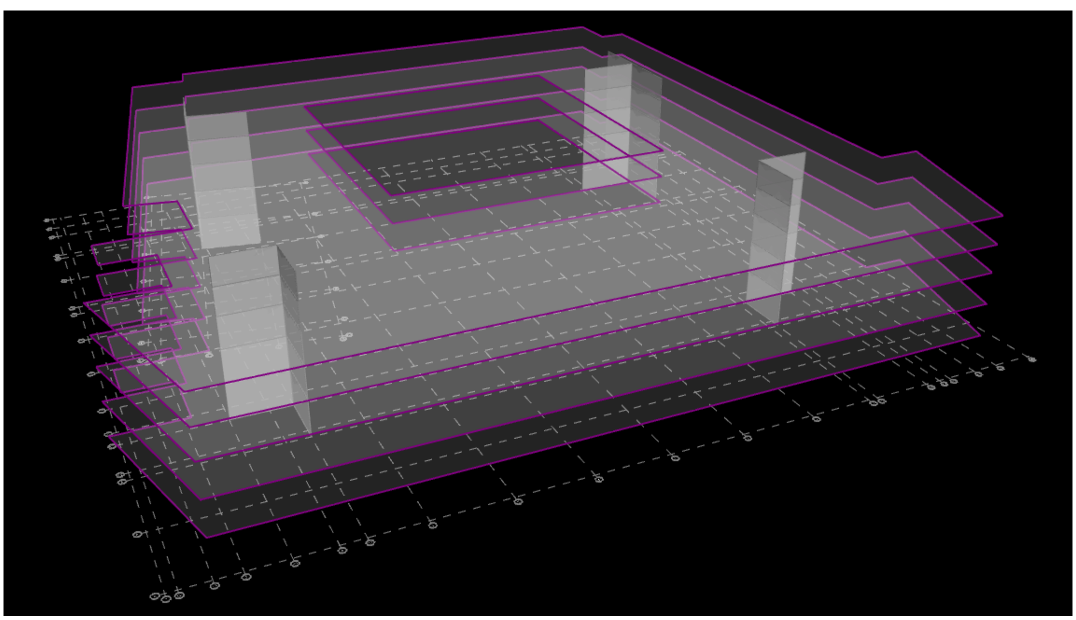


FIGURE 16- LATERAL SYSTEM ISOMETRIC VIEW

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**Error! Reference source not found.** . 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 HorizontalFigure 17**Error! Reference source not found.**). 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.

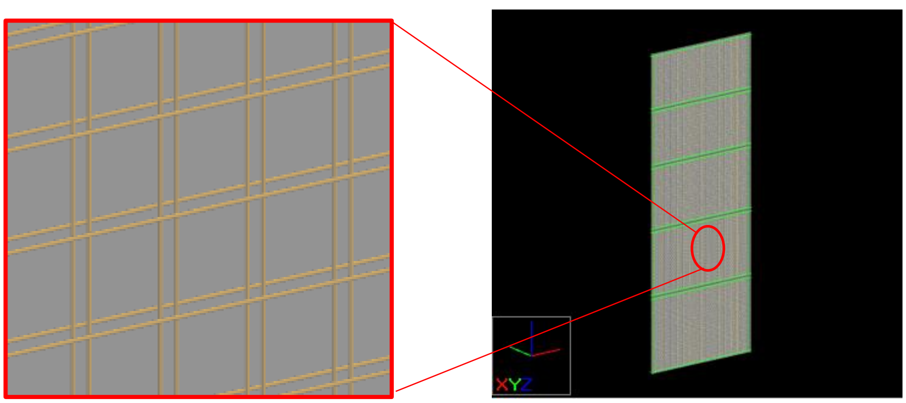


FIGURE 17- #4'S AT 12" O.C. VERTICAL AND HORIZONTAL

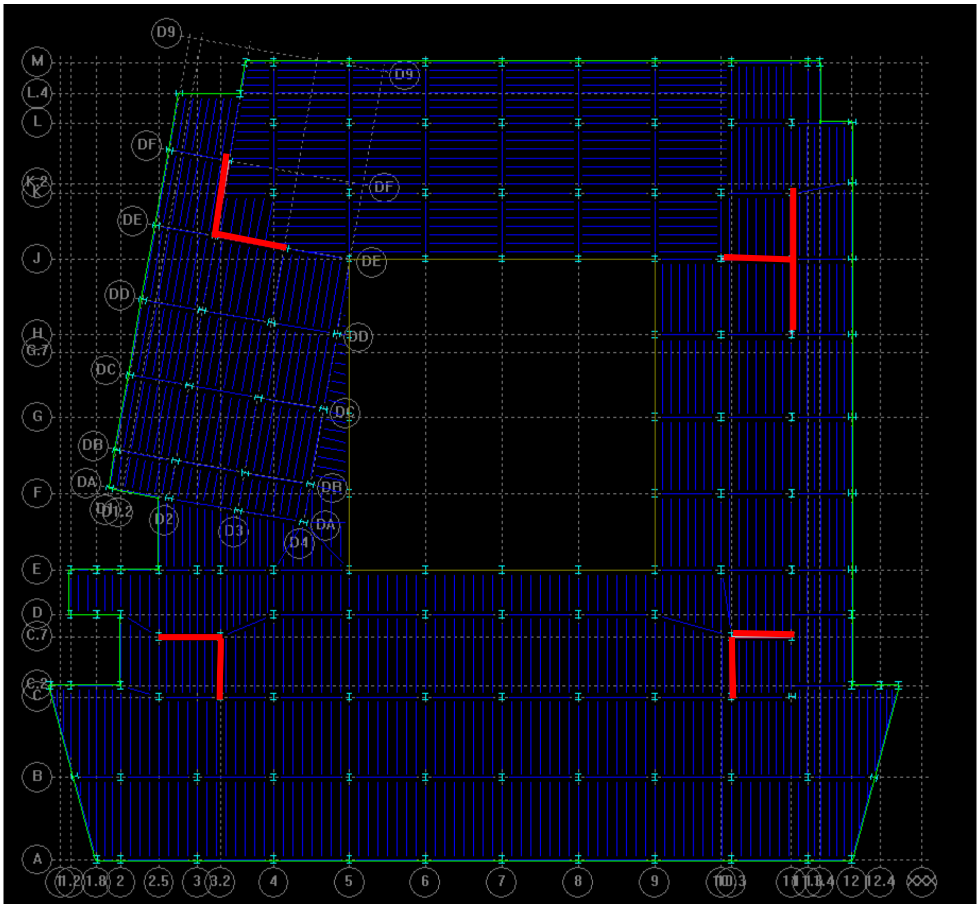


FIGURE 18- LOCATIONS OF REINFORCED CONCRETE SHEAR WALLS

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 19Figure 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

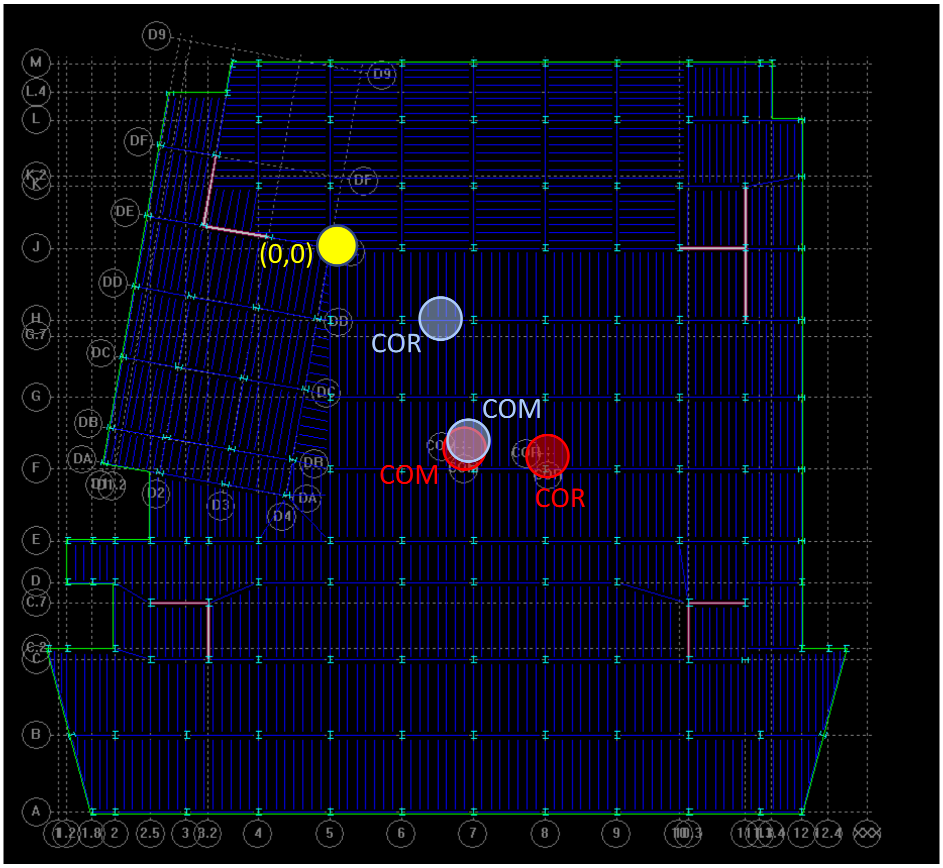


FIGURE 19- CENTER OF MASS AND CENTER OF RIGIDITY

Green Roof Breadth

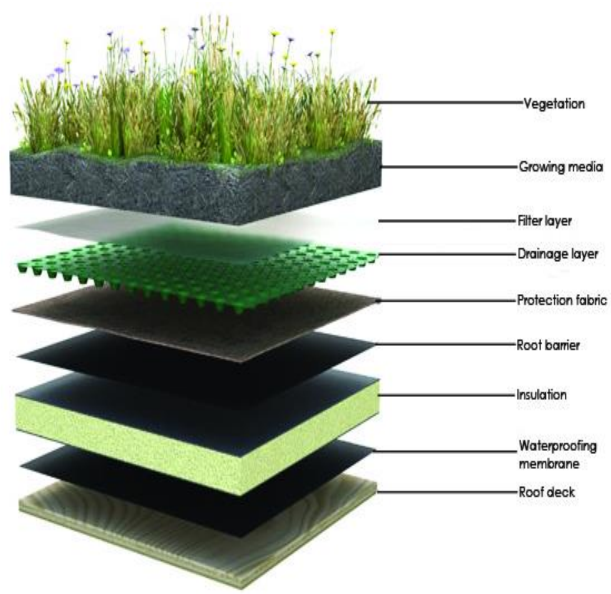
The idea to change the courtyard's design first stemmed from the decision to change the shape of its perimeter in order to simplify structural analysis. The courtyard's original shape was irregular, and didn't lend itself to an easy drainage pattern. While looking at the building loads provided on sheet 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.

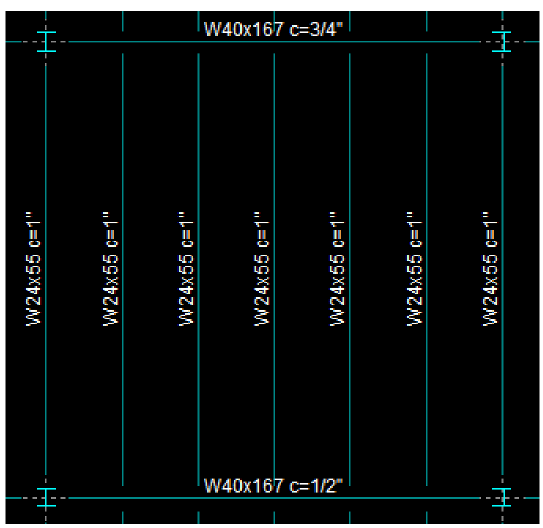


FIGURE 20- TYPICAL COURTYARD GREEN ROOF BAY

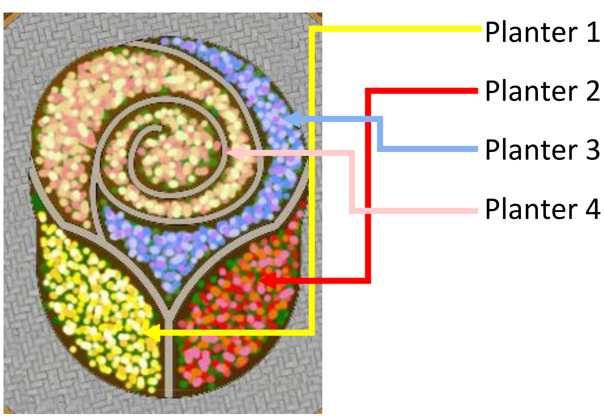
Design Narrative

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

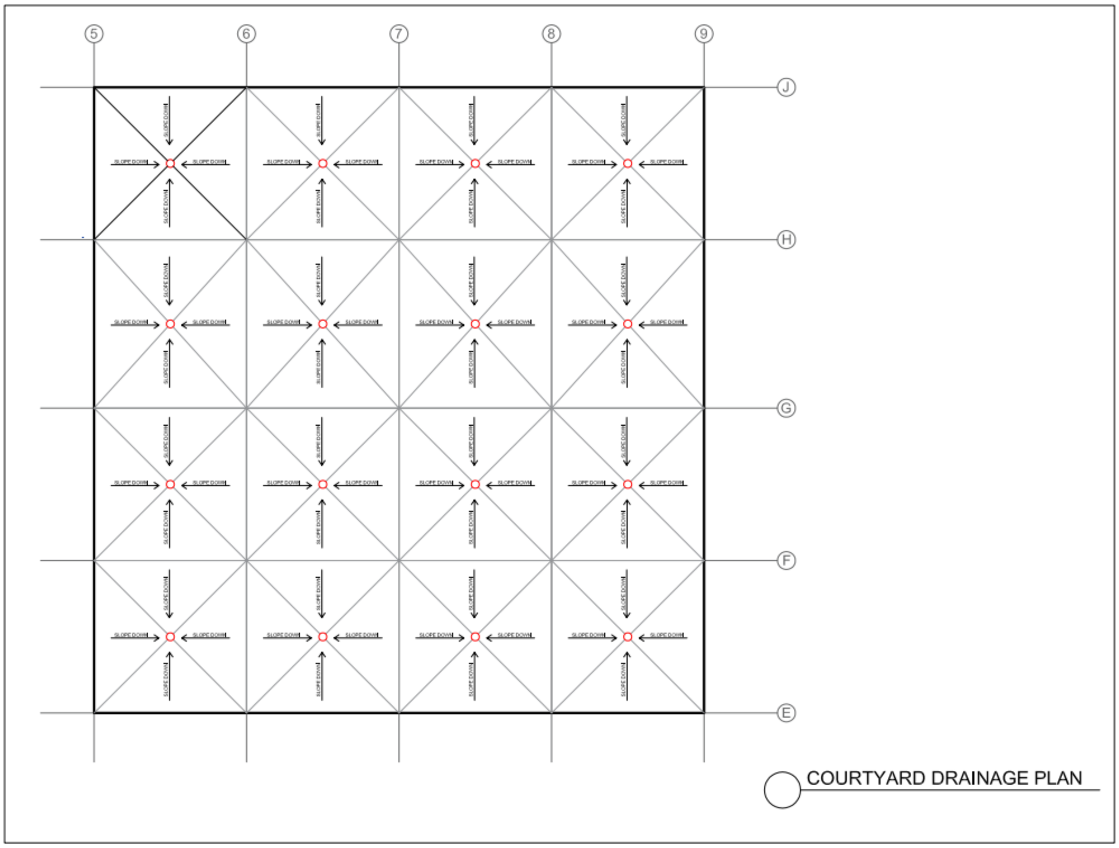
Enclosures Breadth

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.

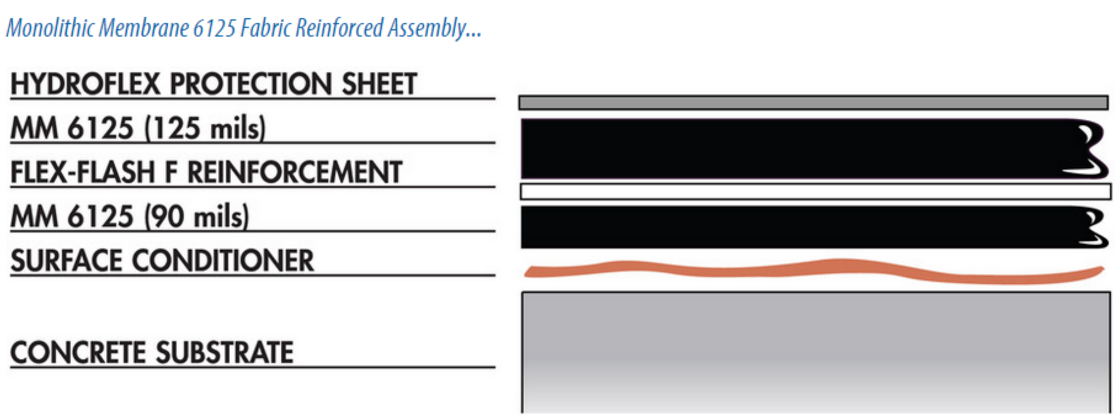


FIGURE 22-MM6125 FABRIC REINFORCED ASSEMBLY

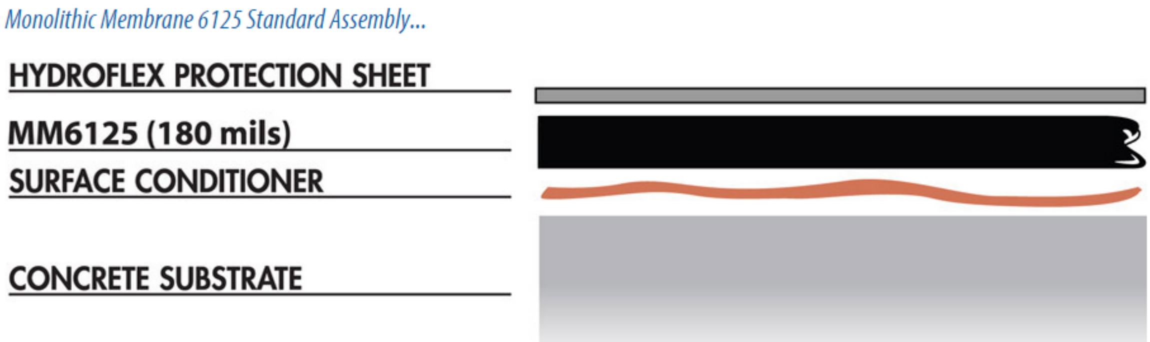


FIGURE 23-MM6125 STANDARD ASSEMBLY

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.

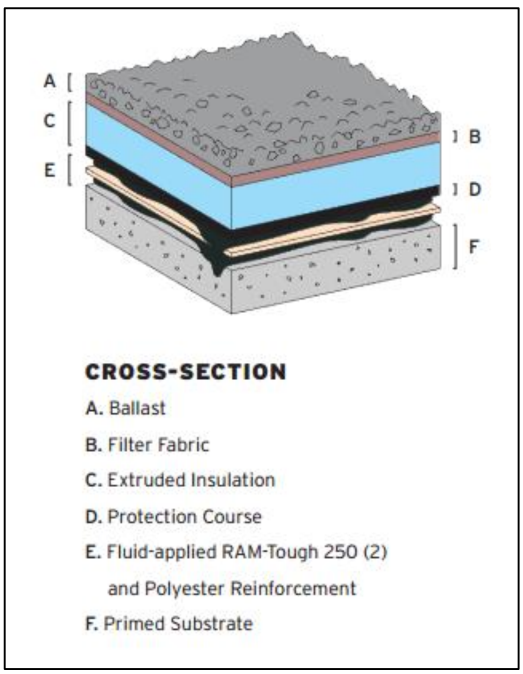


FIGURE 24-RAM TOUCH 250 CROSS SECTION WITH INSULATION, FILTER FABRIC, AND BALLAST APPLIED

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.

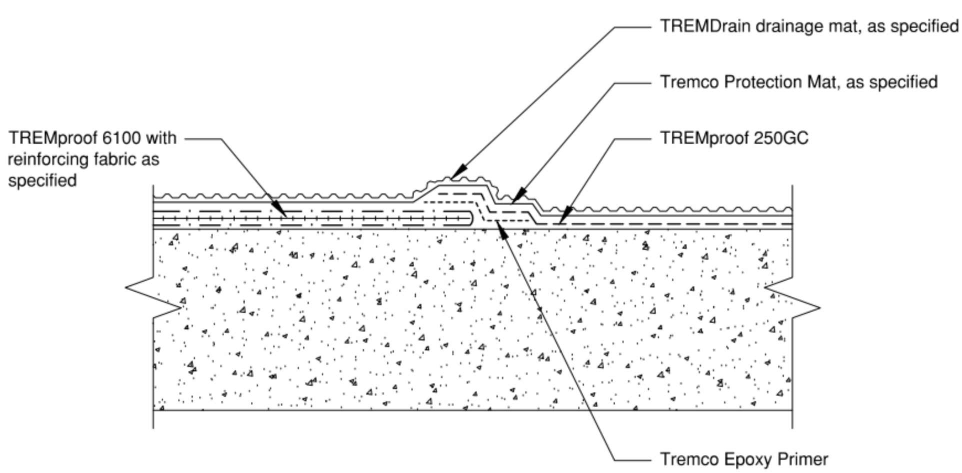


FIGURE 25-TREMPROOF 6100 CROSS SECTION

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.

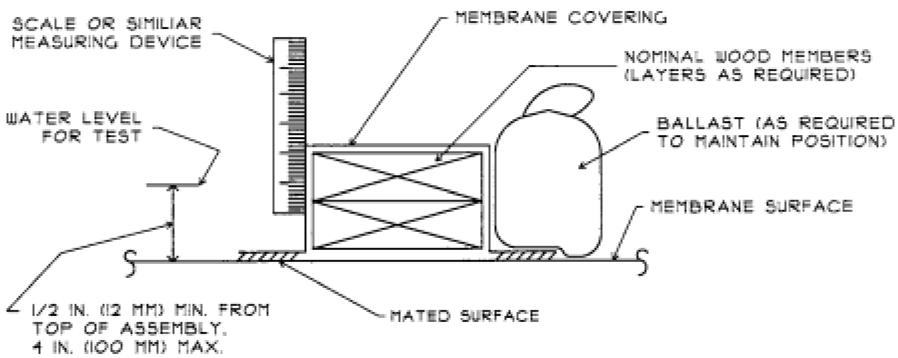


FIG. 4 Containment Assembly—Option No. 4

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

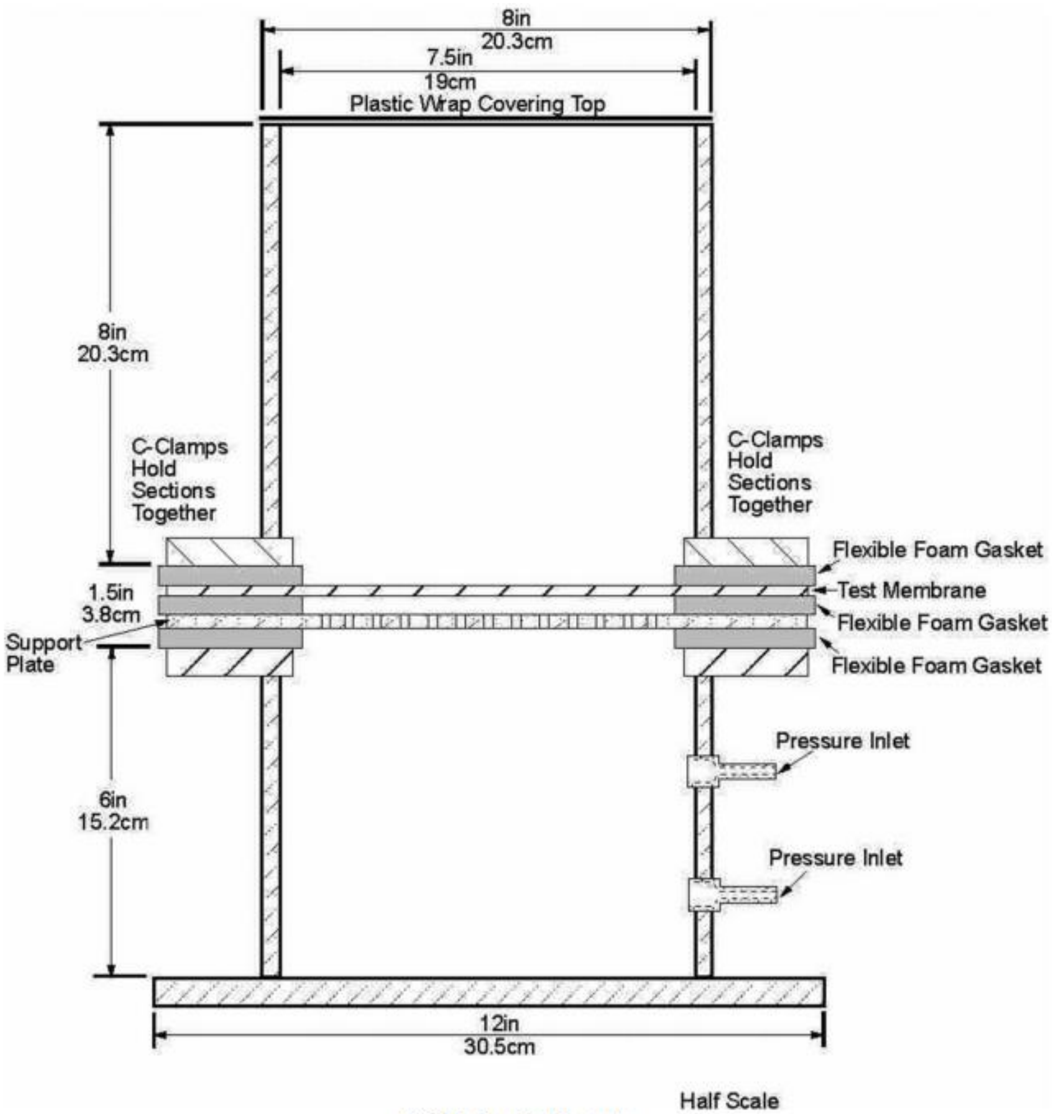


FIG. 1 Leakage Test Apparatus

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