Rockville Metro Plaza II

121 Rockville Pike
Rockville, Maryland

Senior Thesis Final Report

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Rockville Metro Plaza II

121 Rockville Pike, Rockville, Maryland

John M. Vais | Structural Option

Building Statistics

Size: 322,925 sq. ft. (GSF)
200,000 sq. ft. Office Space
14,000 sq. ft. Retail
114,000 sq. ft. Parking
No. of Stories: 10 Above Grade
3 Below Grade (parking)
Delivery Method: Design-Bid Build
Construction: Sept. 2011 – April 2013

Project Team

Development Team: Foulger-Pratt
Architect: WDG Architecture
Civil Engineer: Joyce Engineering
Structural: Cagley & Associates
MEP: WFT Engineering
Landscape Arch: Studio 39

Architectural Features

- Precast concrete panel façade
- Spacious/open office floor plans
- Large window for natural lighting
- LEED Platinum rating

Structural System

- Spread footings 10 ksi bearing capacity soil
- 8” thick one-way slabs
- Post-tensioned beams span building width

Mechanical System

- Two rooftop cooling towers
- VAV system on each floor
- Exhaust fans and CO detectors in garage

Lighting/Electrical Systems

- Central 2500A bus duct riser, 277/480V
- 250A panel boards serve each floor
- Fluorescent lamps illuminate office areas
- 450 kW diesel generator

Acknowledgements

I would like to thank the building developer, Foulger-Pratt, for allowing me to use Rockville Metro Plaza II for my senior thesis project.

Additionally, I would like to thank Mr. Frank Malits, Mr. Daniel Camp, and the office of Cagley and Associates for providing me with the drawings, information, and advice I needed to complete this project.

Thanks to all the PSU Architectural Engineering faculty, who provided me with the knowledge and tool which made this thesis possible.

And finally, I would like to thank my friends and family, whose support has made it possible to successfully complete these last five years of school. I cannot express how much your support is appreciated.

Contents

Executive Summary

The focus of this report is to investigate an alternative structural system for Rockville Metro Plaza II. The original concrete design inherently has its advantages and disadvantages. A new structural system comprised mainly of steel was chosen to compare to the original. This report will explore in depth the pros and cons of each system and compare the two against one another. This investigation will aim to minimize any impacts to architecturally important features such as open floor plans and occupant views. The investigation will also aim to keep the realities of economics, constructability, and scheduling in mind.

Figure 1: South West Corner – by JMV

For this report, the subgrade parking structure was left as originally designed and the seismic base was taken to be at grade. The levels above grade were redesigned using composite beams, lightweight concrete on composite metal decking, and steel supporting columns. A hybrid system of steel and concrete elements was employed as the lateral system.

The use of steel beams resulted in deeper floor depths than in the original design, and thus the redesigned structure's height was adjusted accordingly. This change in story height as well as the change in the building's mass at each floor elicited the need for recalculated seismic and wind loads. After the loads were recalculated and applied to the structure, it was determined that wind controlled the design of the structure's lateral system. Additionally, the design of the lateral system was governed by drift more so than by strength requirements. Overall building torsion and overturning were also investigated and found to be suitable for the redesign.

An architectural study was conducted in order to assess the realistic implications which inevitably come along with the alternative system. The layout of the lateral system was given great consideration and the resulting design was selected with the goal of keeping the floor plan open and the views unhindered. Implications regarding the constructability of the system were also considered. The economical and scheduling impacts of each of the two systems were determined and weighed. It was determined that the steel structure would have an approximate cost of $5.888 million versus the concrete structure, which was found to cost $6.23 million. This resulted in savings of approximately 5% on the total structure's cost. The schedule study proved the steel system to produce a shorter erection time as well.

Building Summary


Figure 2: Rockville Pike Entrance - JMV

Rockville Metro II is the second part of a three phase project that will aid in revitalizing its community. The building is planned to bring new retail venues and Class A office space to the Rockville, MD area. In September of 2011, construction began on this ten story structure.

The structure was planned to have three levels of below grade parking. An initial geotechnical report concluded that the soil at this level would be adequate to support the structure on concrete footings alone. The only concern found was that the water level could exceed this elevation. Thus damp-proofing measures were taken in the design.

The entire structural system is built using cast-in-place concrete. The lower levels of the structure (parking and retail levels) use flat plate, two-way slabs with mild reinforcing to support the floors. Columns which bear these levels incorporate drop caps for added flexural strength, deflection control, and better resistance to punching shear forces. The upper levels of the structure (the office spaces) also use a flat plate slab with mild reinforcing to support the floors. However, in order to facilitate a more flexible office space, larger column-to-column spans (40 feet) were designed. This required additional support of the slabs. To achieve this, wide, shallow post tensioned beams were added to the design. These aided in the control of deflection as well as reduced the potential for cracking. All live loading was determined using ASCE 7 as a guide.

In order to respond to the potential for lateral loads on the structure such as seismic and wind, shear walls were incorporated into the structural design. These walls were placed at the center of the structure about the elevator core. These walls were designed to be 12” thick with rebar reinforcing. ASCE 7 also aided in determining the loading conditions for these elements. The roof of the structure is specified as a green roof. MET II is set to achieve a LEED rating of Platinum, and the green roof is one of the attributes that will aid in this achievement.

In April of 2013, construction on MET II concluded, and MET II became the National Headquarters for Choice Hotels. The following report will describe the structural systems of MET II in more depth. The structure will be analyzed as originally designed and built. Cagley and Associates is responsible for the original design the structural system of MET II and has provided all structural drawings for this report.

Site Location

Rockville Metro Plaza II is located in Rockville, Maryland, just 20 miles northwest of the heart of Washington D.C. The site sits prominently on Rockville Pike which is one of the main routes through the area. Across from the lot is the Rockville Metro stop. With such close proximity to these passage ways, this site boasts a transportation convenience for both employees and visitors alike.

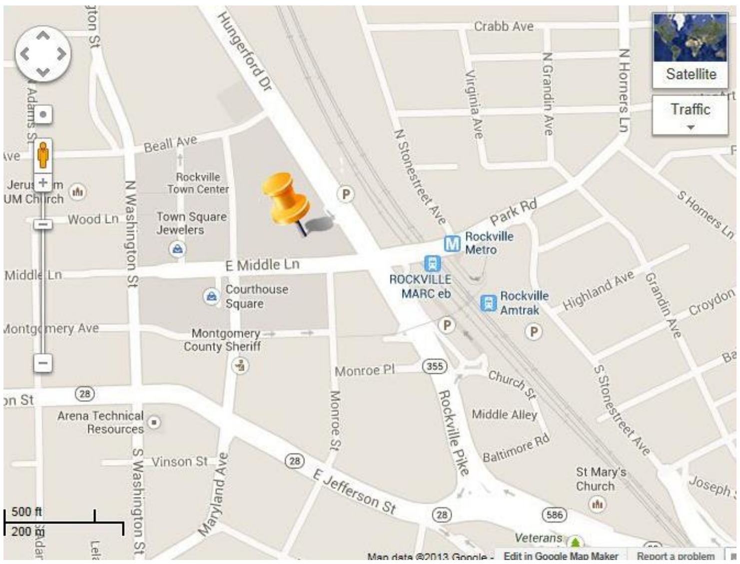


Figure 3: Map of Site Location – From "maps.google.com"

The bustling Rockville area is primarily occupied by businesses, retail, restaurants, and high rise apartments. It is an ever expanding and reawakening locale, as new construction projects continually rejuvenate the lively scene. Upon visiting the area, it can be quite evident why Choice Hotels would decide to make MET II the site of their new North American Headquarters.

The new construction of MET II would be an addition to the current Rockville Metro Plaza I to the Northwest. This posed a complication during construction, for impact on MET I's daily function had to be minimized as much as possible. Excavation of the addition would be required to yield to the existing structure as well.

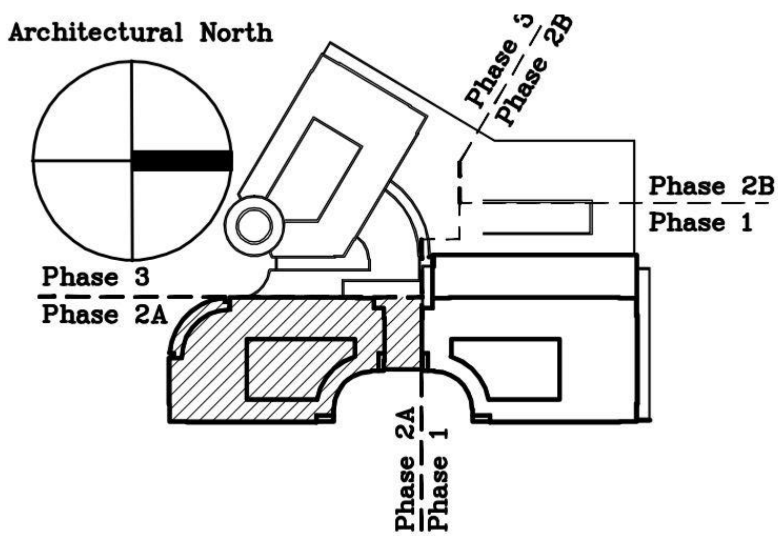


Figure 4: Map of Building Relations – by WDG Arch.

Design Codes

As defined on page S1.00 of the construction documents, the following codes are applicable to the design and construction of MET II's structural system and will also be used in the calculations included in this report:

- "The International Building Code-2009",
  International Code Council

- "Minimum Design Loads for Buildings and Other Structures" (ASCE 7),
  American Society of Civil Engineers

- "Building Code Requirements for Structural Concrete, ACI 318-02",
  American Concrete Institute

- "ACI Manual of Concrete Practice – Parts 1 Through 5",
  American Concrete Institute

- "Post Tensioning Manual",
  Post Tension Institute

The following were added for analysis:

- "Steel Construction Manual" – (14th ed.)
  American Institute of Steel Construction

Figure 5: Rockville Town Square Obelisk – by JMV

Existing Structural Systems Overview

Foundations

The foundation of MET II is comprised of concrete footings and strap beams. The depths, sizes, and reinforcing of footings vary greatly and are dependent upon the column load which it is supporting/distributing. All footings and strap beams were poured using 3000 psi concrete. A net allowable bearing pressure of 10,000 psf was used to design the foundations which are to be placed on undisturbed soil at foundation level. Strap beams had to be used in certain sections where the footing could not be placed centered under the column (e.g. property line abutment). The strap beam helps to distribute the weight of the eccentrically loaded column to adjacent footings and thus aids in resisting overturning. See Figure 6 below for an illustration of the foundation design.

Based on the geotechnical study conducted by Specialized Engineering, it was determined that at the proposed foundation level of this site, the soil was comprised mainly of decomposed and weathered rock. Their Subsurface Exploration and Geotechnical Evaluation report concluded that concrete footings would be adequate to support the anticipated load of the structure.

The one concern which was pointed out in the report was that ground water levels could be at or above the foundation level. In response, the foundation and its walls were designed with this in mind. A layer of granular fill was placed below the slab on grade, with drainage pipes placed throughout. These pipes direct the water to a sump pit which can expel the water when called upon. A vapor barrier lines the underside of the S.O.G. and water stops are installed at steps in the slab grade. Gravel and drains are installed similarly about the exterior foundation walls, as well as sheathing and coatings for damp-proofing.

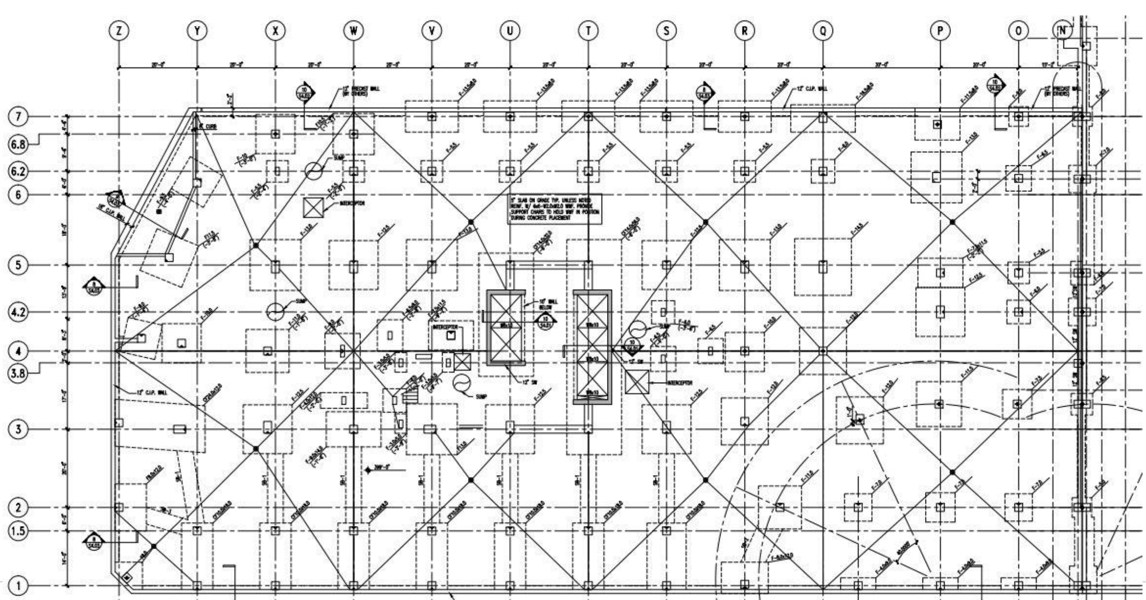


Figure 6: Foundation Plan (refer to Appendix for Enlarged Image)

Floor Systems

The structure's floor systems vary depending on the occupancy/function of the space which they are supporting as well as the distance being spanned. The concrete used for most slabs and beams was specified as 4500 psi normal weight concrete (unless noted otherwise). Refer to the Appendix for illustration of the floor systems as well as the typical bays.

Beginning at the slab on grade, we find a 5" thick concrete slab reinforced with 6x6 – W2.0 x W2.0 welded wire fabric. Two way flat slabs are employed on parking levels P2, P3, and P6. These slabs are 8" thick and use mild reinforcing which is distributed appropriately in order to resist positive midspan moment as well as negative moment created at slab-column intersections. A bottom mat is comprised of #4 bars running each way at 12" on center. The size, length, and spacing of top bars (and additional bottom bars) vary depending on loading and span distance. Drop caps are also incorporated around columns in order provide better flexural capacity, aid in deflections, and better resist punching.

The on-grade (Retail Level) level of the structure also uses only mild reinforcing in the construction of its slab. The slab thickness and elevation varies across this floor depending on the area and its use. Throughout the lobby and retail spaces, a 9" slab was found to be sufficient. However, the loading dock area and the courtyard require 10" and 12" slabs respectively. A bottom mat is comprised of #5 bars running each way at 14" on center. Once again, drop caps are used to add flexure and shear strength.

The remaining floors are designated to be office levels. These levels combine a mild-reinforced slab with post tensioned beams in order to achieve a larger slab bay (typically 40' x 20'). A bottom mat is comprised of #4 bars running each way at 12" on center. In order to achieve the large span of 40' while maintaining a relatively thin floor depth, the use of post tensioning in this design is critical (typical detail shown below in Figure 7). It allows for deflection and cracking to be controlled/reduced over these spans while the slab depth is kept to 8" thick and beams are kept to a typical 20" in depth.

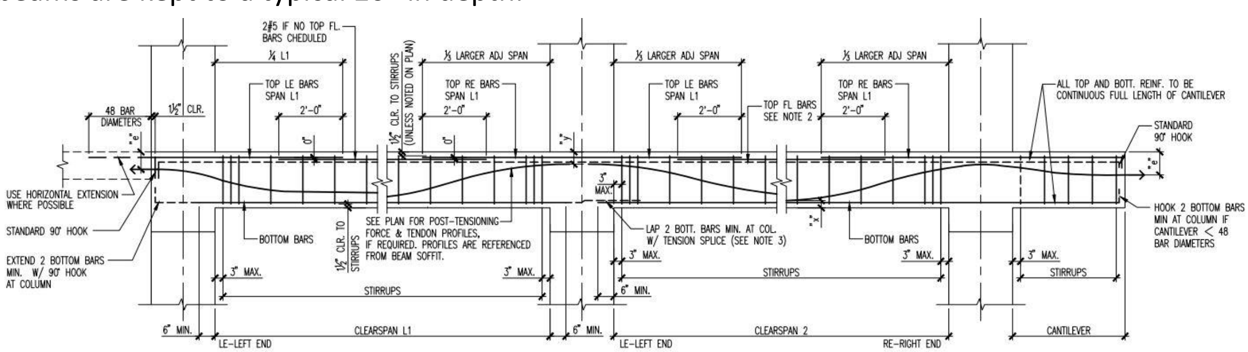


Figure 7: Post Tension Beam Detail Elevation

Column System

The structure of MET II is comprised of concrete columns. The majority of the building's columns are 24" x 24" in dimension and are reinforced with #10 and #11 rebar. The exterior of the building incorporates 30" diam. columns as architectural accents. The strength of concrete used to construct the columns is stepped down as the column rises: 5000/6000 psi ground through the 4th level, 4000 psi 5th through the 8th level, and 3000 psi 9th level and above.

The office portion of the structure achieves a fairly repetitive column layout (see the appendix for floor plan illustrations). However, the exterior-to-interior column span on each the East and West side of the structure is 40' in length. This architecturally driven span allows tenants to have a wider, more flexible floor plan. In response to this, post tension beams are used to transfer the slab load to the columns. Within these levels, these beams are typically 48" x 20" in dimension.

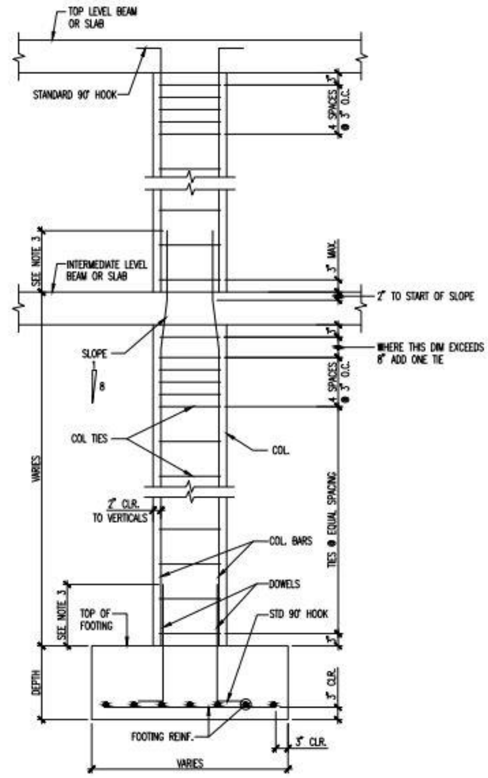


Figure 8: Column Detail Elevation

Within the parking levels an extra row of columns has been added on each the east and west sides. This divides the otherwise 40' span in two (thus eliminating the need for post tension beams as seen in the upper floors). Also, most interior columns in the parking areas also incorporate drop caps for added flexural, shear capacity, and deflection control.

In order to respond to architectural features that stood in the path of select columns, it was necessary to design some columns as sloped. On the plaza and P6 levels, interior columns are commonly sloped to accommodate the standard parking stall space in the garage levels below, as seen in figure 6 to the right.

Figure 9: Sloped Columns in Retail Space

Lateral System

Rockville Metro Plaza II uses shear walls and moment frames as the main lateral force resisting system. Lateral loads that are applied to the building are resisted by this shear wall and moment frame system as these elements transfer the force to the building's foundation.

Shear walls 12" in thickness frame the two elevator towers at the center of the structure and extend from the foundation to the roof of the structure (see figure 7 - shear wall locations are highlighted). Another 12" thick shear wall is present along part of the Northern face of the structure on the sub-grade levels. The strength of concrete used follows the same gradation as applied to the columns. As with most concrete structures, the rigid construction allows most of the building's frames to act as moment frames. This reduces the need for multiple shear walls and allows MET II to be designed with so few.

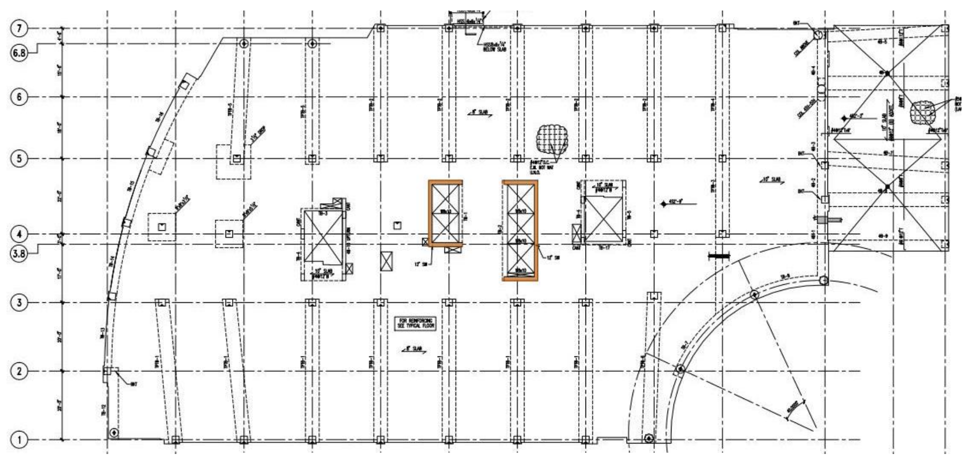


Figure 10: Shear Walls - 4th Floor

Roof System

In order to aid MET II in its pursuit of a LEED Platinum rating, a green roof system was designed as the main roofing system. The roof begins with a mildly reinforced, 8" concrete slab. A bottom mat is comprised of #4 bars running each way at 12" on center. Top bars and additional bottom bars are placed as needed. Next, a roof membrane and waterproofing layer are applied, on top of which rigid insulation is placed. A thin moisture retention mat is draped, followed by a drainage mat. Four inches of a light weight substrate soil mix is laid, in which a sedum mix is planted. Sedum is a genus of flowering plants of the family Crassulaceae and is widely used as an alternative to grass on green roofs. Refer to Figure 11 to the right for the green roof composition.

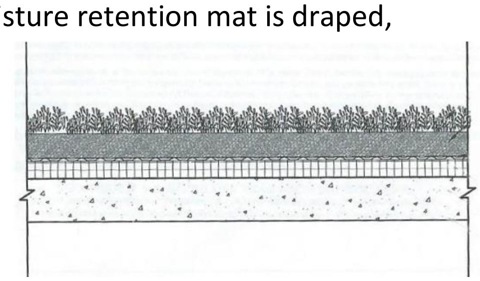


Figure 11: Green Roof Layers

Thesis Objectives

Problem Statement

Through past studies, the concrete structure of Rockville Metro Plaza II proved to be capable of withstanding the required design loads. The shallow floor system and long span beams create versatile rentable spaces on each level. However, the use of concrete creates a heavy structure which requires larger gravity members and foundations.

Proposed Solution

In Technical Report III, alternative floor systems were studied. Systems considered were assessed based on their ability to maintain the open floor plan as seen in the original system. Cost, fireproofing, and several other considerations were also measured in the comparison of systems. The study concluded with identifying a composite steel floor system as a viable alternative to the current concrete system.

A steel system for the office levels will likely reduce the overall weight of the structure. This may benefit the foundation of the building, resulting smaller foundation elements. Similarly, the building's gravity system may see benefits in member sizing as dead loads are reduced. This will also impact the lateral loads on the building, further reducing seismic loads. The parking levels will likely remain unchanged in the new design however.

The redesign of the structural system will also require that the lateral system be considered. The implementation of braced frames, moment frames, and/or shear walls will be investigated. Lateral forces will be recalculated and considered once again, incorporating any changes made to the structure.

Impacts that this redesigned system will have on other areas of the building will also need to be explored. One consideration is the change in floor depth due to the size of steel members. This will require coordination with MEP systems and may lead to increasing the overall building height. Additionally, the architecture of the office space will require analysis when placing lateral elements. Fireproofing steel elements will also be necessary, but will result in additional costs.

In conclusion, an entire redesign of the structural system will be completed. The alternative design will then be compared back to the original and pros and cons will be weighted to determine the feasibility of the alternative.

Cost and Schedule

Altering the main structural system of Rockville Metro Plaza II will have a significant effect on the cost and schedule of the project. The impact that this change has on the construction schedule will be assessed through calculations and comparisons. A cost analysis will also be investigated in order to determine the feasibility of the alternative system.

Architecture

The redesigned structural system will have many potential impacts on the architecture of Rockville Metro Plaza II. A deeper floor system will increase the floor to floor height of the structure. This issue will consider the routing of MEP system, local zoning requirements, and impacts regarding the façade. Additionally, the redesigned lateral force resisting system will be of significant focus. The placement of these elements must respect the interior flow of the office space as well as the intended aesthetics of the building's façade.

MAE Requirements

Knowledge gained from graduate level course work will be incorporated into the investigation, analysis, and design of work in the depth and breaths of the proposed project.

AE 530 – Computer Modeling of Building Structures - Knowledge from this course will be integral in creating effective and useful models. These models which will be created in ETABS and RAM will allow for the analysis and design of the gravity and lateral systems of the structure.

AE 534 – Analysis and Design of Steel Connections - Material from this course will be relied upon heavily as connection design will be necessary for the steel structural system redesign.

AE 538 – Earthquake Resistant Design of Buildings- Additionally, coursework from this class will be incorporated in designing the lateral system of the structure.

Figure 12: Exterior Perspective

Gravity Loads

In comparing the design values provided on the structural documents to those listed in the International Building Code and ASCE 7, it is evident that all live load requirements were met or exceeded. The main areas of where this trend is evident are mechanical rooms and office areas. Each of these spaces were designed with higher live loads most likely due to the owner's specification, anticipated actual loading, or the simply the office's standard practice for good design. The comparison of live load values may be seen in Table 4 below. These same values are used in the redesign in order to provide a better comparison between systems.

ASCE 7 was used in calculating the flat roof snow load of the structure. Using this document as a guide, the same value as presented on the structural documents was derived. This calculation can be seen in Table 5 below. Snow drift was not considered in this report. The super-imposed values presented below in Table 6 are also as listed on the structural documents.

Table 1: Floor Live Loads

Area	As Designed (psf)	ASCE 7-05 (psf)
Corridors (first level)	100	100
Corridors (above first)	100	80
Lobbies	100	100
Marquees/Canopies	75	75
Mechanical Room	150 (U)	125
Offices	80 + 20 (partitions)	50 + 20 (partitions)
Parking Garage	50	40
Retail – First Floor	100	100
Stairs/Exit Ways	100 (U)	100
Storage (Light)	125 (U)	125

Table 2: Flat Roof Snow Load

Ground Snow Load	P_g=	25 psf
Snow Exposure Factor	C_e=	1.0
(Terrain Category B)		
Thermal Factor	C_t=	1.0
Importance Factor	I_s=	1.0
$P_f = 0.7*P_g*C_e*C_t*I_s*P_g =$		17.5 psf

Table 3: Superimposed Dead Loads

Area	Design Value (psf)
Floor	5
Roof	10

Gravity Loads Continued

In determining the loading of the redesigned structure, the live loading from the original system was directly carried over. For example, in the office spaces, the occupancy live load as designed and defined in the IBC is an office load of 80 psf with an additional 20 psf for the possibility of partitions installed in the space. The main difference in load comes from the change in dead load due to the lighter redesigned system. In terms of loading, the slab itself and the supporting beams contribute most of the dead load to the floor system. Such items as flooring, hanging ceiling tiles, and mechanical/lighting equipment are relatively light and are accounted for in the super imposed dead load.

In pursuit of a LEED rating, the roof of MET II was designated as a green roof composition. Green roofs are a more environmentally friendly alternative to the standard roof. They reduce heat island effects, reduce rainwater runoff (which lessens the potential for sewer overflow), and provide a habitat for birds and insects, as well as many other benefits. For the structure, however, this can equate to a heavier roof as there will be more mass present than that of a standard roof. The roof is designated as an extensive green roof which means that the vegetation will mainly grasses and similar small plants (e.g. sedum). These plants have relatively shallow root systems and thus do not require a deep soil base, as only a 4" depth is used. The element is considered to be architecturally important to the structure and it's LEED Rating, thus the green roof is carried into the redesigned structure.

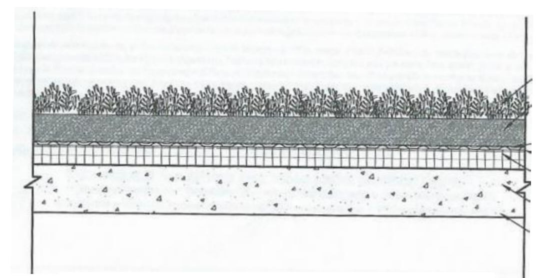


Figure 13: Green Roof Cross

Rockville Metro II is enclosed by a wall system comprised of precast concrete panels and aluminum framed glass windows. This system is attached to the structural system's slabs and columns. Within the original design, each precast panel spans between two exterior columns. Two connections are made at each column and to the slab at mid-span. These connections are both load bearing and non-load bearing. The load bearing connections (i.e. support weight of panel) only occur at the columns. Other connections act to tie back the panel to the structure and to resist loads perpendicular to the panel. The redesign steel system assumes that this same connection type (or similar) will be possible.

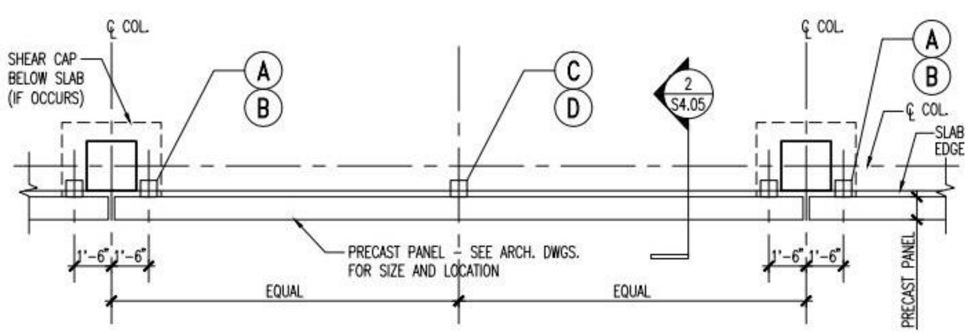


Figure 14: Precast Plan Detail – by Cagley and Assoc.

Gravity System Redesign

The design of the gravity system began by initially considering the typical office bay. Results from Tech III were revisited in order to aid in the bay's layout. It was determined that a composite beam system was most promising and thus this system was employed. Design began by determining the loading on the bay as well as its geometry. The long span configuration provides fewer connections and was thus chosen for its constructability. The deck was selected to meet unshored conditions thus bettering the constructability of the system once again. A 2" composite metal deck (2VLI20) with 3.25" of lightweight concrete topping (115 pcf) was selected from the Vulcraft Catalogue. This configuration provides the necessary two hour minimum fire rating for the space while aiming to minimize any impacts on the depth of the floor system.

The image below displays the RAM Structural System model which was used to design the gravity system of the structure. Blue elements designate the item as a gravity element whereas red designates it as part of the lateral system. Through an iterative process, the members of the gravity system were designed.

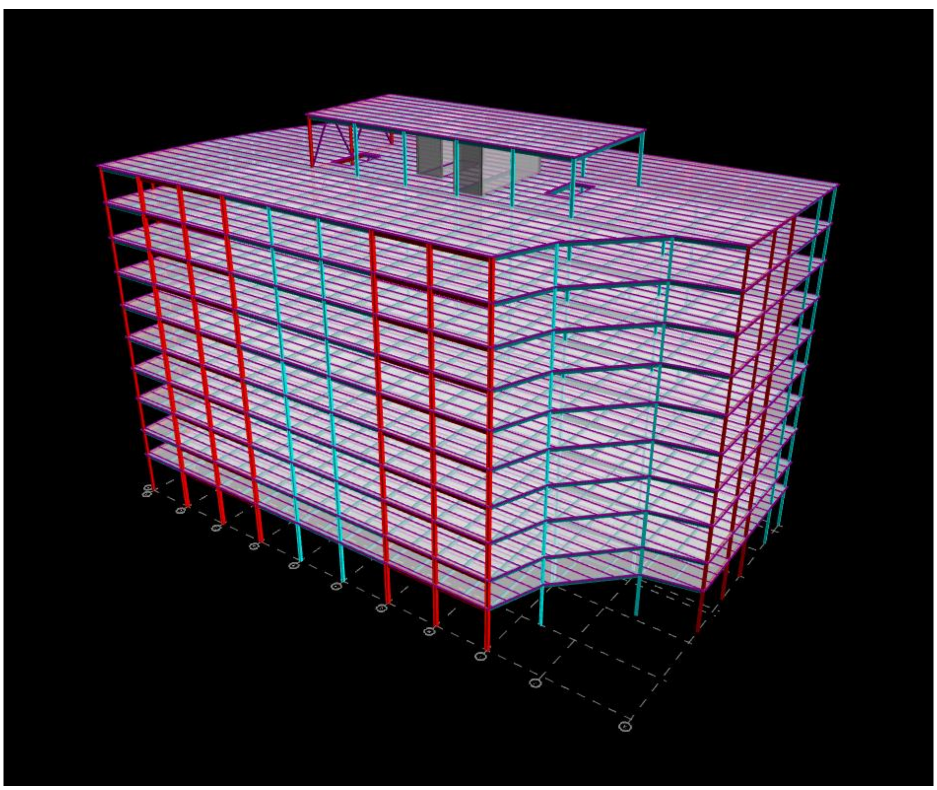


Figure 15: RAM Structural System Building

Gravity System Redesign Continued

Figure 16: Typical Redesigned Office Framing Plan

The above image displays the floor plan for a typical office level within Rockville Metro Plaza II. The redesigned floor system employs camber and composite action throughout many of the beam and girder elements. Identification of these features have however been removed from the image for clarity. Camber was reserved for those members spanning over 24' and requiring a minimum of ¾" of camber. The amount of camber was increased by ¼" as needed (with an upper bound of 4"). Studs were limited to 12" spacing and uniform distribution. While two rows of studs were allowed in the program, it was sought to provide only a single row on beams. These amendments to the programs criteria allow for ease in constructability and manufacture of elements.

The columns of the redesigned system were designed in RAM Structural System's Column Module. All columns were designed with appropriate splicing increments which in turn allow for ease in construction. Gravity columns are spliced on two level increments. This acknowledges constructability and transportation influences as well as factors regarding safety.

All hand calculations and spot checks for beams and columns are available in Appendix A.

Floor Depth Comparison

One key advantage of the original concrete system over the steel redesign is that it possesses a relatively shallow floor depth. Within the post-tensioned original design, the deepest structural component extends to 20” below the floor surface. The steel redesign however requires a typical depth of 23.25” below the top of slab. Also note that in the original design, the post-tensioned beams only span key distances and end prior to the building’s core. Thus the center of the building has a depth of only 8” due to the two-way slab type configuration in this section. This leaves a much greater margin for mechanical space. However the steel redesign requires that the new 23.25” structural depth be seen throughout the floor system. At maximum depth, the redesigned steel system extends 26.25” below the top of slab. This depth occurs only in certain areas where required by the loading and geometry. It was ensured that these few elements would not be interfering with the main HVAC ducts within the ceiling.

It was found necessary to resolve the difference in the floor depths, and so after examination of the mechanical drawings and consideration of the duct sizes, 10” were added to each level. It was reasoned that this added height would ensure sufficient space for the mechanical elements. This changed the plenum space from a total depth of 2’-9” to 3’-7” overall. This additional depth was made to the overall height rather than taken from the floor to ceiling height of the office space. This was done in order to keep the open and airy feel of the office which was architecturally sought. It was also reasoned that that the tall ceiling height added to the value of the rental space and that this dimension should therefore not to be abridged. The new overall height also elicited the requisite for recalculated lateral loads.

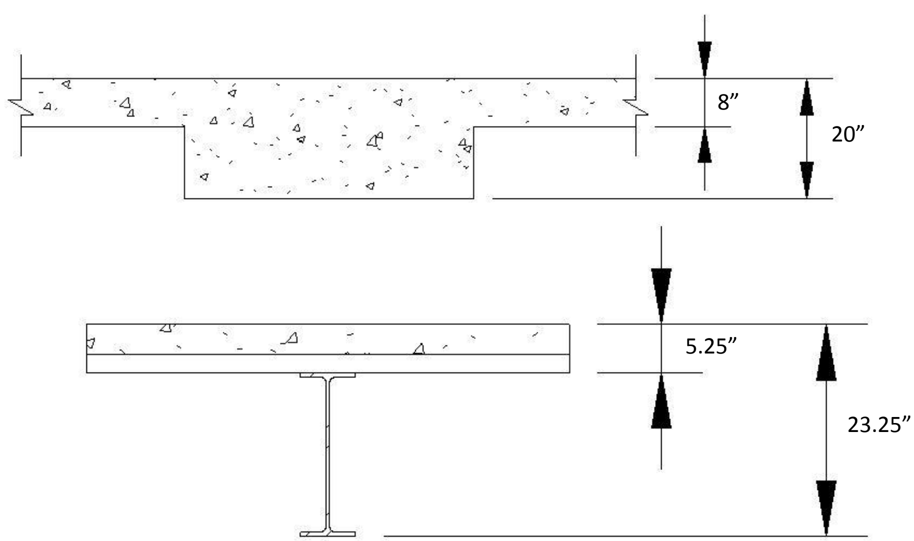


Figure 17: Beam Depth Comparison

Wind Loads

In order to determine the wind load on the structure of the building, ASCE 7-05's Method 2 was implemented (as described in Chapter 6 of the document). Wind loads in each the North-South and East-West directions were analyzed. Based on geographical information and building characteristics, uniform pressures were determined for each face of the structure. These pressures were converted into forces on each story level and used to calculate base shears and overturning moments. Roof uplift forces were not considered at this time. Results and loading diagrams are presented below and on the following pages. Detailed calculations of this analysis may be located in Appendix B of this document.

The wind loads were recalculated for the redesigned steel structure. This was deemed necessary due to the height increase required in the redesigned building, which inevitably alters the lateral loading on the structure. The following tables display the recalculated wind pressures applied to the structure in each respective direction. Load pressure diagrams also included display the distribution of pressures on the face of the structure.

Figure 18: Perspective View of Southern Face - JMV

Wind Pressure – East-West

Table 4: East-West Design Pressures							
	Height	**Windward Pressure**	**Leeward Pressure**	**Total Pressure**	**Total Force**	**Story Shear**	**Overturning Moment**
	(ft)	(psf)	(psf)	(psf)	(kips)	(kips)	(k-ft)
Penthouse	150.33	12.92	-7.73	20.64	29.49	29.49	4433.68
	139.75	12.65	-7.73	20.38			
Main Roof	129.17	12.37	-7.73	20.09	61.58	91.07	7953.77
	122.88	12.19	-7.73	19.92			
11th	116.58	12.01	-7.73	19.74	51.91	142.98	6051.35
	110.29	11.82	-7.73	19.55			
10th	104.00	11.62	-7.73	19.35	50.86	193.84	5289.95
	97.71	11.42	-7.73	19.15			
9th	91.42	11.20	-7.73	18.93	49.73	243.57	4545.84
	85.13	10.98	-7.73	18.71			
8th	78.83	10.74	-7.73	18.47	48.47	292.03	3820.68
	72.54	10.49	-7.73	18.21			
7th	66.25	10.22	-7.73	17.95	47.04	339.08	3116.62
	59.96	9.93	-7.73	17.66			
6th	53.67	9.62	-7.73	17.35	45.40	384.48	2436.49
	47.38	9.29	-7.73	17.01			
5th	41.08	8.92	-7.73	16.64	45.22	429.70	1857.86
	34.25	8.46	-7.73	16.19			
4th	27.42	7.94	-7.73	15.67	39.50	469.20	1083.00
	22.08	7.47	-7.73	15.19			
P6	16.75	6.90	-7.73	14.63	41.63	510.83	697.33
	8.38	6.63	-7.73	14.36			
Plaza Level	0.00	6.63	-7.73	14.36	25.25	**536.08**	0.00
							41286.57

Base Shear	**536.08 Kips**
Overturning Moment	**41286.57 Kip-ft**

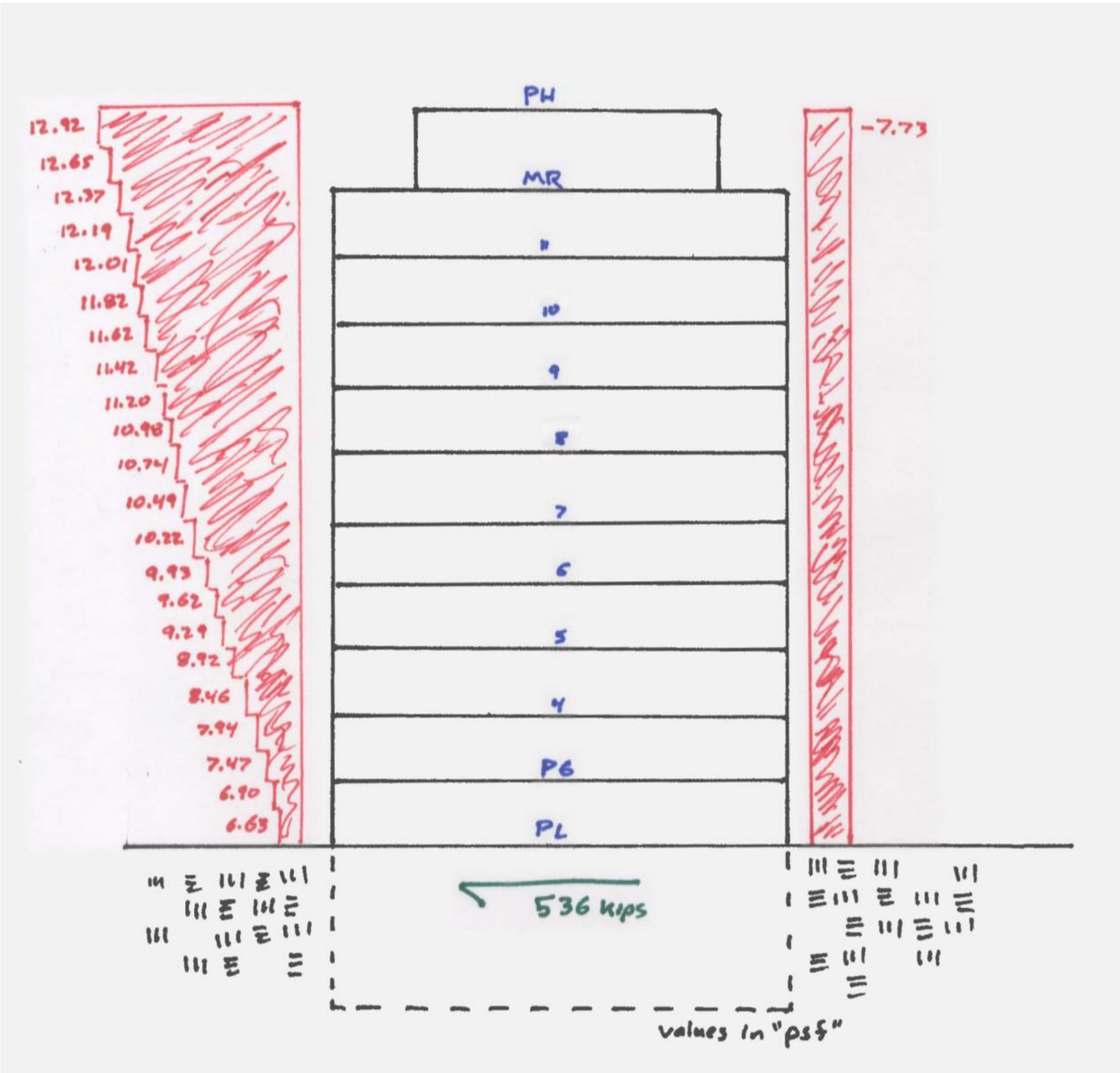
Wind Pressure – North-South

Table 5: North-South Design Pressures

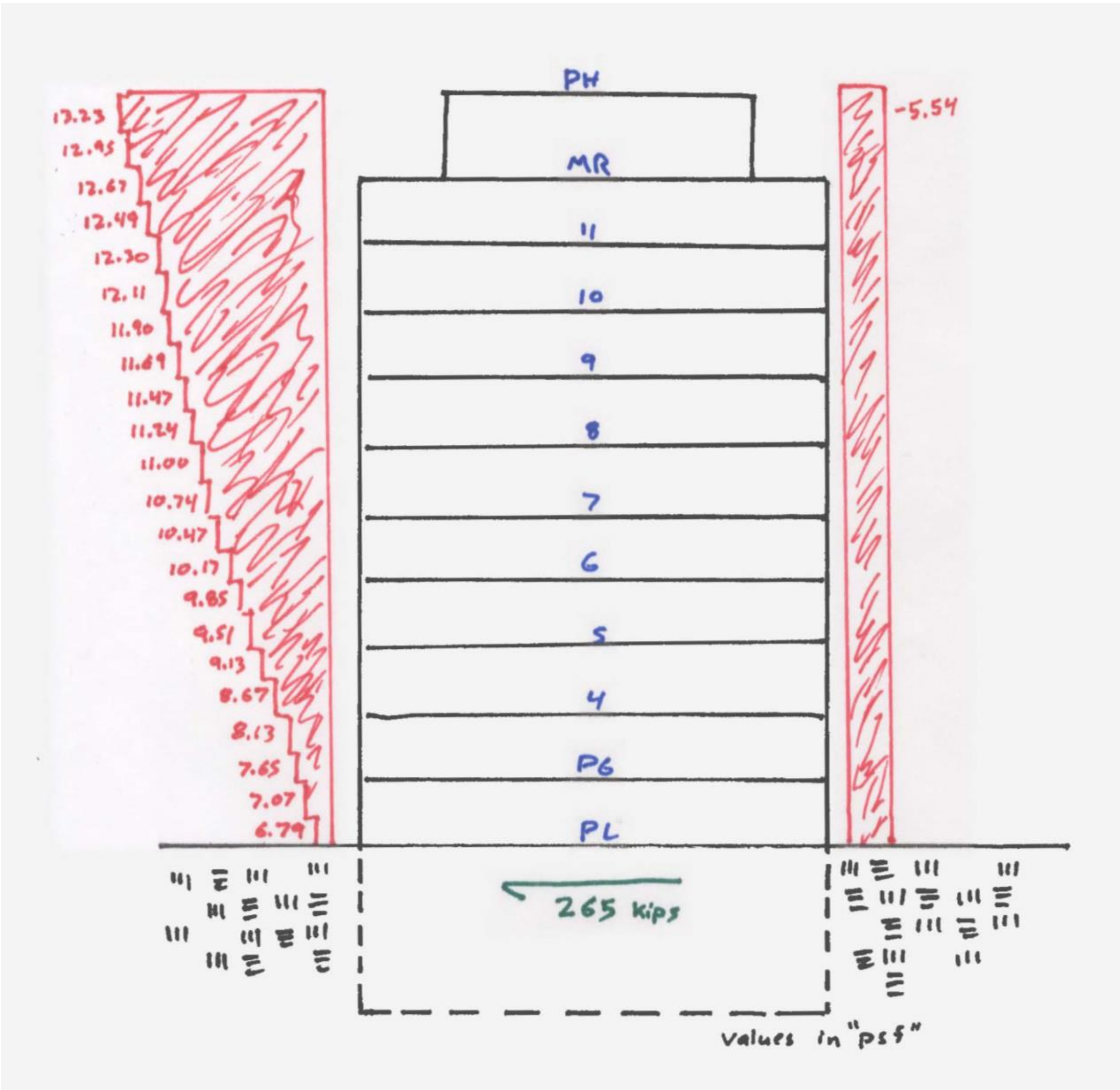
	Height	Windward Pressure	Leeward Pressure	Total Pressure	Total Force	Story Shear	Overturning Moment
	(ft)	(psf)	(psf)	(psf)	(kips)	(kips)	(kip-ft)
Penthouse	150.33	13.23	-5.54	18.77	10.33	10.33	1552.67
	139.75	12.95	-5.54	18.49			
Main Roof	129.17	12.67	-5.54	18.21	29.02	39.35	3748.32
	122.88	12.49	-5.54	18.03			
11th	116.58	12.30	-5.54	17.84	26.79	66.14	3123.62
	110.29	12.11	-5.54	17.65			
10th	104.00	11.90	-5.54	17.45	26.18	92.32	2723.13
	97.71	11.69	-5.54	17.24			
9th	91.42	11.47	-5.54	17.01	25.52	117.84	2332.75
	85.13	11.24	-5.54	16.78			
8th	78.83	11.00	-5.54	16.54	24.78	142.62	1953.47
	72.54	10.74	-5.54	16.28			
7th	66.25	10.47	-5.54	16.01	23.95	166.57	1586.53
	59.96	10.17	-5.54	15.71			
6th	53.67	9.85	-5.54	15.40	22.99	189.56	1233.60
	47.38	9.51	-5.54	15.05			
5th	41.08	9.13	-5.54	14.67	22.73	212.28	933.72
	34.25	8.67	-5.54	14.21			
4th	27.42	8.13	-5.54	13.67	19.65	231.94	538.81
	22.08	7.65	-5.54	13.19			
P6	16.75	7.07	-5.54	12.61	20.46	252.40	342.72
	8.38	6.79	-5.54	12.33			
Plaza Level	0.00	6.79	-5.54	12.33	12.39	**264.79**	0.00
							20069.35

Base Shear	**264.79 Kips**
Overturning Moment	**20069.35 Kip-ft**

East – West Pressure Diagram



North – South Pressure Diagram



Wind Load Summary

The additional height increase of the redesigned steel structure provides slight increases in the wind loading on the structure as anticipated. Each direction experiences a 7.9% increase in base shear values. Base shear increased from 246 kips to 265 kips and 497 kips to 536 kips in the North-South and East-West directions respectively.

Through calculating the wind pressures on the structure, it becomes evident that the wind load in the East-West direction is the most critical. This can be seen by comparing the calculated base shear and overturning moment in each direction. The base shear in the East-West direction is 536.08 kips, compared to the value of 264.79 kips in the North-South direction. The overturning moment follows this relationship as well, with a value in the East-West direction nearly twice as large as that of the North-South direction.

This result was well anticipated when considering the length of each side of the structure. The East and West sides are measured to be 210’ in length while the North and South faces are only 120’ in length. A larger surface area would in turn face more pressure from the wind which translates to a larger force on the structure in said direction. This observation is in agreement with the results obtained from the calculations and analysis.

The benefit in using ASCE 7-05 is that it aids the designer in translating wind speed to a wind pressure which may be applied to the face of the structure. This pressure is then calculated into a resultant force (based on tributary area) which may be assumed to act at each story. This follows the actual load path of the wind force. In order for the floor to transfer the lateral load to shear walls and moment frames, it must be assumed to be a rigid diaphragm. Within MET II, the shear walls are at the core of the structure and also act to create the elevator shaft. Specifically designed steel columns and beams form the moment frame systems.

Figure 19: Exterior View from Across Rockville Pike – by JMV

Seismic Loads

The City of Rockville is not known for high seismic activity. Still it is part of good practice to design a building to withstand such ground motion as the load case may control the design of the lateral system. For this analysis, chapters 11 and 12 of ASCE 7-05 were employed. Using site features and building characteristics (such as seismic ground moth ion values and the weight of the dead load on the structure), forces could be derived based on the building's expected response. This method allows for the base shear and overturning moment of the structure to be determined. These results may then be compared to values calculated in other loading scenarios in order to determine the design value for the structure's lateral system. Note it was once again necessary to recalculate this load in the redesigned steel structure due to the fact that building height as well as floor mass was altered.

The Plaza Level and parking levels below grade did not contribute to the calculations as they were considered to be at or below the seismic base. The weight of the building that was calculated included all dead loads (i.e. concrete structure, superimposed, etc.) plus 50% of the live load for partitions and the full operating weight of equipment.

The equivalent lateral force method was determined to be applicable to this analysis. The main calculations and results of this analysis may be found on the pages that follow. Detailed calculations of other variables (such as building weights) are available in Appendix C.

Figure 20: Exterior View from Across Rockville Pike Intersection – by JMV

Table 6: Seismic Design Variables			
			ASCE Reference
Soil Classification		C	
Occupancy Category		II	Table 1-1
Importance Factor	I_e	1.0	Table 11.5-1
Structural System		F	Table 12.2-1
Spectral Response Acceleration, Short	S_s	0.156g	USGC Website
Spectral Response Acceleration, 1 s	S_1	0.051g	USGC Website
Site Coefficient	F_a	1.2	Table 11.4-1
Site Coefficient	F_v	1.7	Table 11.4-2
MCE Spectral Response Accel., Short	S_{MS}	0.188	Eq. 11.4-1
MCE Spectral Response Accel., 1 s	S_{M1}	0.086	Eq. 11.4-2
Design Spectral Acceleration, Short	S_{DS}	0.1248	Eq. 11.4-3
Design Spectral Acceleration, 1 s	S_{D1}	0.0578	Eq. 11.4-4
Seismic Design Category	S_{DC}	A	Tables 11.6-1,2
Response Modification Coefficient (E-W)	R	3.0	Table 12.2-1
Response Modification Coefficient (N-S)	R	3.25	Table 12.2-1
Approximate Period Parameter	C_t	0.02	Table 12.8-2
Building Height	h_n	149’	Arch Dwg.
Approximate Period Parameter	x	0.75	Table 12.8-2
Approx. Fundamental Period	T_a	0.853 s	Eq. 12.8-7
Long Period Transition Period	T_L	8.0 s	Fig. 22-15
Seismic Response Coefficient (E-W)	C_S	0.0226	Eq.’s 12.8-2,3
Seismic Response Coefficient (N-S)	C_S	0.0209	Eq.’s 12.8-2,3
Structure Period Exponent	k	1.176	Section 12.8.3

Table 7: Design Values		
	East-West	North-South
Effective Seismic Weight	21205 kips	21205 kips
Base Shear	479 kips	442 kips
Overturning Moment	47025 kips-ft	43408 kips-ft

Table 8: Seismic Calculations East-West					
Level	**Story Weight**	**Height**	**Forces (F_x)**	**Story Shear (V_x)**	**Moments (M_x)**
	(kips)	(ft)	(kips)	(kips)	(k-ft)
Pent Roof	551	150.33	26.8	26.8	4024.104
Main Roof	2683	129.17	109.1	135.9	14096.42
11th Floor	1999	116.58	72.1	208.0	8401.099
10th Floor	1999	104.00	63.0	271.0	6552.037
9th Floor	1999	91.42	54.1	325.1	4948.535
8th Floor	2010	78.83	45.7	370.8	3605.862
7th Floor	2010	66.25	37.3	408.1	2469.638
6th Floor	2010	53.67	29.1	437.2	1561.446
5th Floor	2035	41.08	21.5	458.7	883.4656
4th Floor	2041	27.42	13.4	472.1	367.5514
P6	1869	16.75	6.9	479.0	115.1233
Plaza Level	-	0.00	-	-	-
Total	**21205**	-	**479.0**	-	**47025.28**

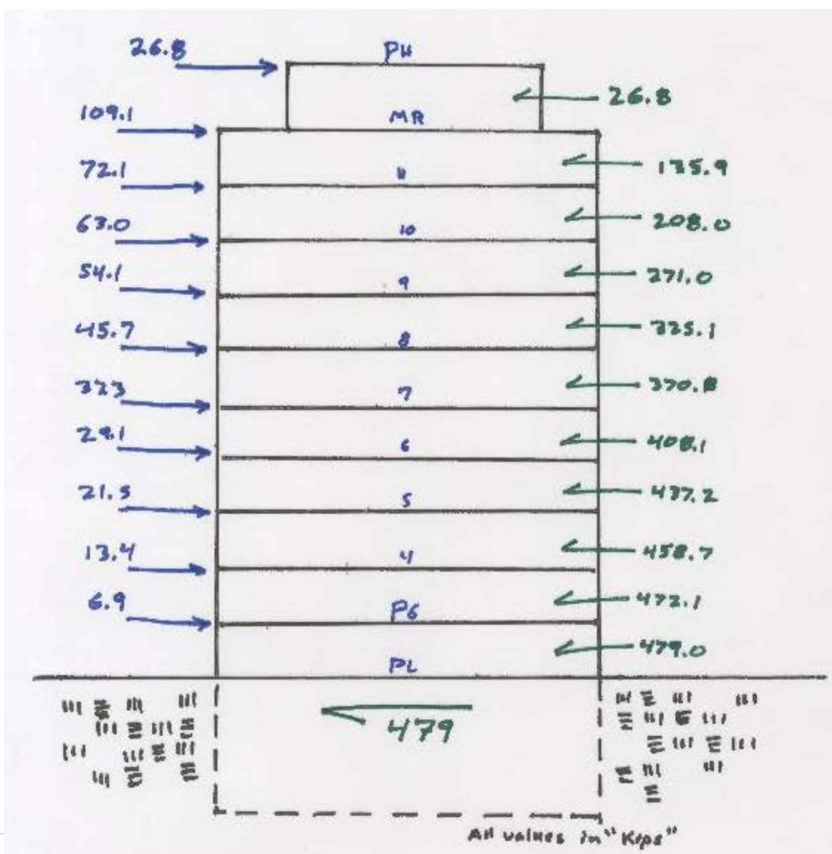
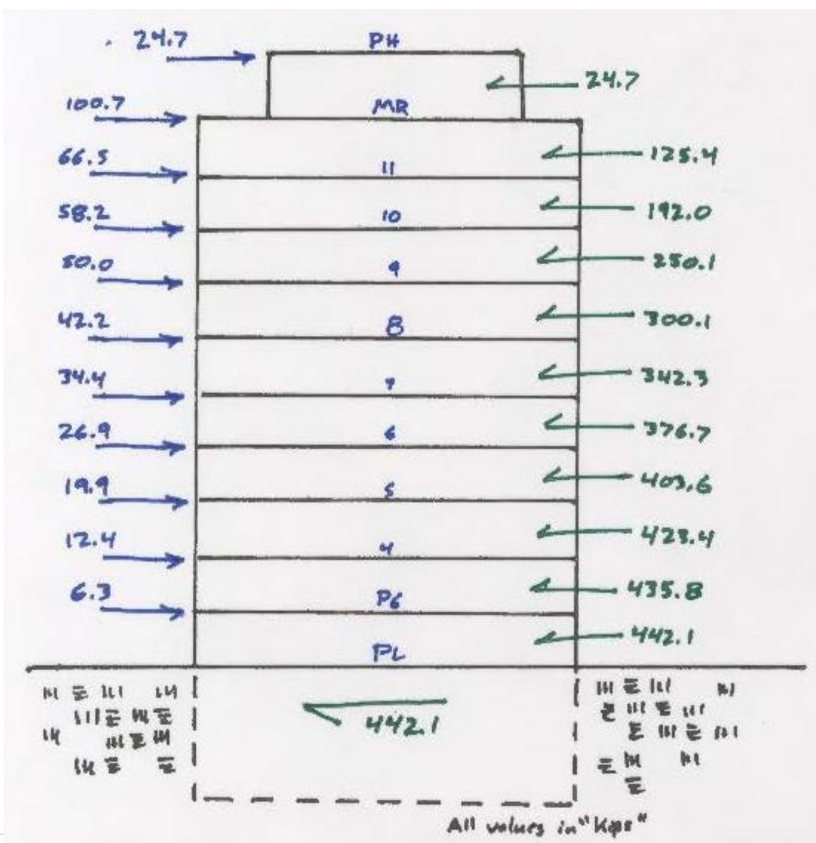


Table 9: Seismic Calculations North-South					
Level	Story Weight	Height	Forces (F_x)	Story Shear (V_x)	Moments (M_x)
	(kips)	(ft)	(kips)	(kips)	(k-ft)
Pent Roof	551	150.33	24.7	24.7	3714.558
Main Roof	2683	129.17	100.7	125.4	13012.08
11th Floor	1999	116.58	66.5	192.0	7754.861
10th Floor	1999	104.00	58.2	250.1	6048.034
9th Floor	1999	91.42	50.0	300.1	4567.878
8th Floor	2010	78.83	42.2	342.3	3328.488
7th Floor	2010	66.25	34.4	376.7	2279.666
6th Floor	2010	53.67	26.9	403.6	1441.335
5th Floor	2035	41.08	19.9	423.4	815.5067
4th Floor	2041	27.42	12.4	435.8	339.2782
P6	1869	16.75	6.3	442.1	106.2676
Plaza Level	-	0.00	-	-	-
Total	**21205**	-	**442.1**	-	**43407.95**



Seismic Load Summary

The seismic analysis executed provides a design base shear and overturning moment for each orthogonal direction of the structure. This is necessary as each direction will have a slightly different lateral force resisting system. In the East West direction, the design base shear is 479 kips and the overturning moment is 47025 kip-ft. In the East-West direction, the base shear and overturning moment have been determined to be 442 kips and 43408 kip-ft respectively. These values were computed using the equivalent lateral force method as defined in ASCE 7-05. This method allows the designer to interpret the expected ground motion and characteristics of the structure into the design forces shown.

As previously stated, it was necessary to recalculate these forces as not only did the building height change, but the entire structural system did as well. This amendment presented a new building weight and a new lateral force resisting system. The building weight was significantly decreased relative to the original concrete structure. This lighter structure therefore produces less seismic forces as less mass is present. However, the Response Modification Coefficient decreased as well. The change of this value (which is a direct result of the lateral force resisting systems employed) offset some of the force reductions that came from the reduced weight. In comparison to the original design, the overall seismic forces were reduced. The new steel structural system experiences approximately 25-30% less seismic force (relative to each direction). (Note that the seismic base shear was calculated to be 643 kips in each direction of the original concrete structure.)

Figure 21: Exterior Perspective – by JMV

When comparing the found seismic forces to the results calculated for wind, we find that seismic conditions do control in this case. Therefore, as with the original concrete design, the new steel structure's lateral system's design is too controlled by the wind load.

Lateral System Redesign

In the original design of the structure, concrete shear walls and concrete moment frames compose the lateral force resisting system in each principle direction. The moment frames were comprised of the columns, slabs, and post-tensioned beams. This essentially meant that the entire building participated in resisting the lateral load.

The redesigned steel system retained the core shear walls of the original design. The possibility of changing the core to braced frames was considered but was rejected for architectural reasons. The remainder of the system is primarily comprised of steel moment frames. Also, one eccentrically braced frame was included in the design. This element was introduced in effort to realign the center of mass with the center of rigidity, with will be elaborated on in the Torsion section. For further explanations of the lateral system selection, see the Architectural Breadth section.

In each principle direction, the floor diaphragm is assumed to be rigid and it therefore is allowed to transfer the lateral load to the lateral force resisting system at each respective level. The braced frames work based on rigid frame action, as they develop shear forces and bending moments in the frame elements and joints of the configuration. The shear walls resist the lateral force by primarily employing shear and axial forces. Finally, the braced frame acts as a truss type element, using axial loads in members to redirect the lateral load to the ground.

The image below depicts the steel redesign lateral system of Rockville Metro Plaza II. In the N-S direction, the shear walls are shown in red and the moment frames in blue. In the E-W direction, the shear walls are shown in purple, the moment frames in green, and the concentrically braced frame in orange.

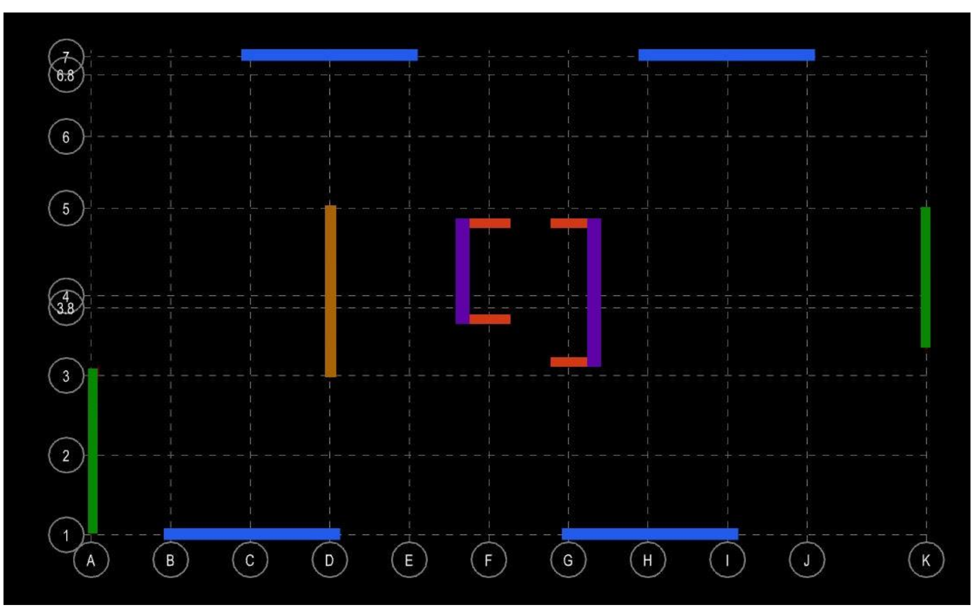


Figure 22: Lateral System Component Locations

Load Combinations

In order to determine the maximum design load on the structure, various load combinations were considered. The minimum combinations that must be considered when designing for strength are defined in section 2.3.2 of ASCE 7-05. Here, seven load combinations are defined as follows:

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

In considering the lateral wind force, ASCE 7-05 cites four different wind combinations that must be considered. These cases are defined in chapter six of the document in Figure 6-9 (shown below). After assessing all possible combinations, Case 2 was found to be the most critical. In considering seismic forces on the structure, ASCE 7-05 cites in section 12.8.4.2 that a minimum of 5 percent accidental must be considered on the structure.

After analyzing the forces and deflections of the required minimum load combinations shown above, it was found that the N-S direction and the E-W direction were both predominantly controlled by the load combination of 0.9D + 1.6W. Considering the location's low seismic activity, it is expected that wind will control the design. It is also reasonable that this load combination controls over 1.2D + 1.6W + L + 0.5(L_r or S or R). This makes sense considering that the relatively lighter steel structure would inevitably have less resistance to uplift.

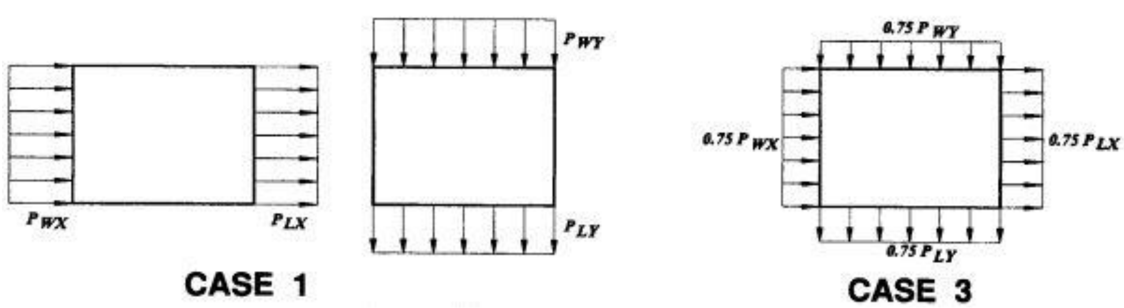


Figure 23: Select ASCE 7-05 Design Wind Load Cases

Computer Modeling

For the design and analysis of each the lateral system and gravity system, RAM Structural Systems were employed. The layout was reproduced in the Modeler module. Here gravity and lateral members were assigned and the building's geometry was established. RAM Steel Beam was then used for the design of gravity members. Once the gravity members of ach floor were configured, the model continued to RAM Steel Column where the gravity columns were designed. In each design module, member designs generated by the software were checked. In many cases, sizes were changed either for economy, size restrictions, or in favor of a more appropriate stud configuration.

The lateral force resisting system was designed using the Frame module of RAM. Here, load cases were initially defined as they applied to drift criteria. Wind and seismic load cases were considered, as well as the possible effects of P-Delta forces. Diaphragms were considered to be rigid in this analysis. Reduced steel sections were not used as the initial focus was to design for drift and to obtain the building's natural periods. The model was run and after iterations of member size adjustments, viable results were observed. Drifts were checked against the accepted industry standard of h/400 and P-Delta effects were satisfied through reviewing that proper Stability Coefficients were obtained.

Next, the members of the lateral system were checked for strength requirements. The reduced stiffness for steel members was employed and Tb=1.0 for an initial starting point. Wind and seismic load cases were created specific for strength design. For these new load cases, the building's natural period (which was achieved in the step prior using the member unreduced stiffness values) was incorporated as per the direct design method.

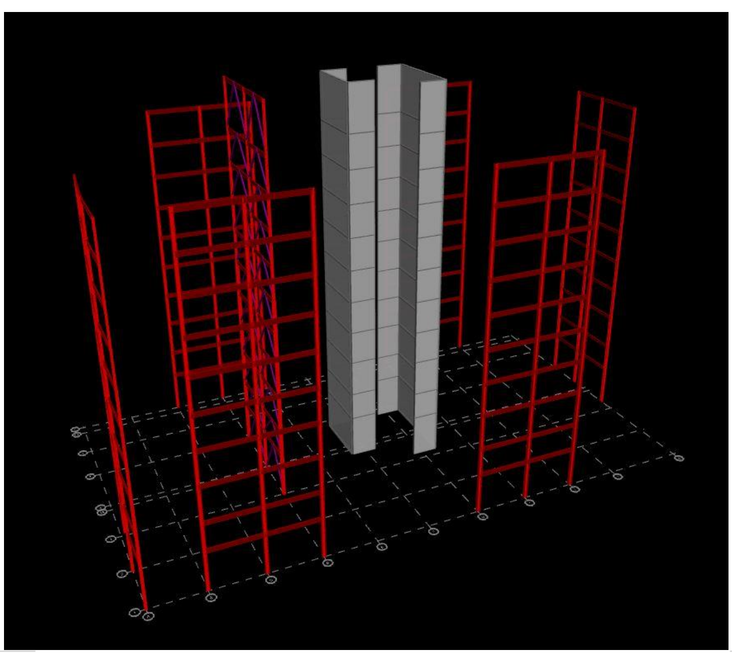


Figure 24: RAM Lateral System Model

Evaluation of whether notional loads needed to be considered in all load cases was conducted. The model was run with and without the effect of P-Delta. The results of the drift ratios were compared to assess the effect of P-Delta effects and it was determined that they need only be considered with gravity loads. B1 factors were engaged to account for small P-Delta effects not accounted for in the analysis. B2 factors were not used as P-delta effects were employed in the analysis. After analysis, it was determined that the reduced stiffness value needed to be modified to Tb=0.986. Although this modification penalizes all members, it was found justifiable relative to other options due to the fact that the change was so insignificant. This adjustment was made and the model was run again. The results were found to pass all criteria of the steel code check relative to AISC-360.

Mode shapes of the structure as well as the deflected shapes were prominently used throughout this process in order to quickly view the building's performance as well as assure that the model was properly functioning.

Initial models of individual bay segments were created in order to determine sizing options and optimal geometric configurations of the floor system. This allowed all feasible options to be weighed and the most suitable one to be selected. This study built upon the results of Tech III. Note that the base of the moment frames will be set top concrete columns of the garage level. Since these columns will be built integrally with the foundation wall, the reaction at the base of the steel columns has been modeled as fixed. When test models were run with the substitution of pinned bases, the resulting displacements differed by 5.2%, thus it was deemed that the fixed assumption was reasonable.

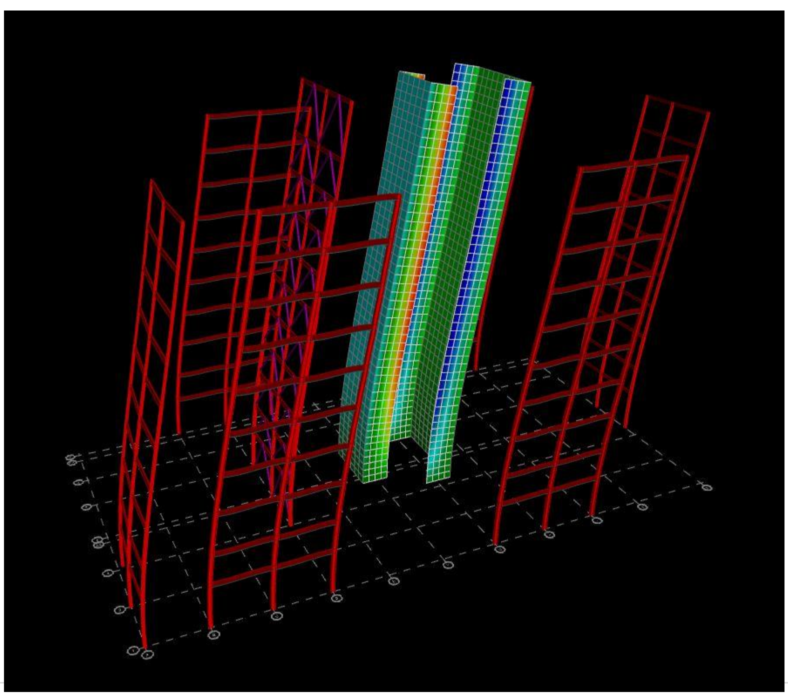


Figure 25: RAM Model – X-Wind Displacement

Wind Drift

In order to obtain the building's story drift values that are incurred due to wind, serviceability wind loads were applied to the computer model. For this calculation, critical locations were selected and assessed (i.e. locations that are farthest from the center of rigidity as they proved to yield the greatest drifts). Industry standards limit the overall building drift to 1/400th of the building's height. For this, the drift of the main roof level is limited as follows:

$$\Delta_{MAX} = (129.17' \times 12''/1') / 400 = 3.87''$$

After analyzing the loads in the computer model for unfactored (serviceability) wind forces, the following results were obtained:

Table 10: Wind Drifts (N-S)		
Level	**Story Drift (in)**	**Total Drift (in)**
Roof	0.306	3.332
11th	0.318	2.899
10th	0.330	2.592
9th	0.341	2.274
8th	0.343	1.944
7th	0.337	1.603
6th	0.316	1.260
5th	0.296	0.923
4th	0.176	0.607
P6	0.134	0.310

Table 11: Wind Drifts (E-W)		
Level	**Story Drift (in)**	**Total Drift (in)**
Roof	0.173	1.496
11th	0.176	1.323
10th	0.177	1.147
9th	0.176	0.969
8th	0.172	0.793
7th	0.163	0.621
6th	0.149	0.458
5th	0.140	0.308
4th	0.087	0.169
P6	0.082	0.082

The above tables prove that the structure's deflection due to wind forces is well within the industry's standard tolerance. It is found that the building will deflect more in the North-South direction. Even though this direction has a small load, there is less stiffness/redundancy in the lateral system of this direction. Therefore it is reasonable that this be the case.

The drift values above satisfy individual story drift limitations for all typical levels (values are less than 12.58x 12 / 400 = 0.377"). Further calculations regarding the values above may be found in the tables of Appendix D. Note that the modeling assumption of fixed bases was employed. As the steel columns will be attached to concrete columns built integrally with the garage wall, this assumption is valid. Comparing models of fixed versus pinned connections in this situation further validated the assumption as approximately only a 5% difference in drifts was observed.

Seismic Drift

In order to obtain the building's story drift values that are incurred due to seismic forces, seismic loads were applied to the computer models. For this calculation, critical locations were selected and assessed (i.e. locations that are farthest from the center of rigidity as they proved to yield the greatest drifts). For this criterion, Chapter 12 of ASCE 7-05 limits story drift to two percent of the building's height. Thus the total drift of the main roof level is limited as follows:

$$\Delta_{MAX} = (129.17' \times 12''/1') \times 0.02 = 31''$$

After analyzing the loads in the computer model for factored (strength) seismic forces, the following results were obtained:

Table 12: Seismic Drifts (N-S)		
Level	**Story Drift (in)**	**Total Drift (in)**
Roof	0.708	5.989
11th	0.724	5.281
10th	0.735	4.556
9th	0.735	3.822
8th	0.716	3.086
7th	0.677	2.370
6th	0.610	1.693
5th	0.547	1.083
4th	0.312	0.536
P6	0.224	0.224

Table 13: Seismic Drifts (E-W)		
Level	**Story Drift (in)**	**Total Drift (in)**
Roof	0.356	1.412
11th	0.177	1.056
10th	0.174	0.879
9th	0.166	0.705
8th	0.153	0.539
7th	0.135	0.386
6th	0.121	0.251
5th	0.071	0.130
4th	0.001	0.060
P6	0.059	0.059

The above drift values have been adjusted as per ASCE 7-05 where:

$$\delta_x = C_d \times \delta_{xe} / I$$

The resulting amplified drifts were calculated using a C_d value of 3 (this value being controlled by the steel moment frames which have been classified as "steel ordinary moment frames" in this scenario). The importance factor was considered as 1.0. It is clear that the total drifts do not exceed the allowable drift for the structure. This is expected given the low seismicity of the geography as well as the reduced weight of the steel structure redesign. This warrants that seismic drifts will not become large enough to result in unfavorable secondary effects.

Torsion

Torsional forces result from a number of different contributing factors. The most common torsion inducing factor is having an eccentricity between the center of rigidity and the applied load. In the case of seismic forces, loads are applied at the center of mass and in the case of wind forces, loads are applied at the center of pressure. The torsional moment on a given level is defined as the applied force multiplied by the perpendicular distance from where it is applied to the center of rigidity. The farther these points are from the center of rigidity, the larger the resulting torsional moment.

Torsional moments are also induced by various load cases as defined in ASCE 7-05. Regarding wind, load patterns 2 and 4 of Figure 6-9 of the document require that a minimum eccentricity equal to 15% of the building width be considered. In the case of seismic forces, the prevision requires a minimal accidental eccentricity of 5% to be considered.

Due to the building's geometry, the centers of mass and pressure do not coincide with the center of rigidity in the models of Rockville Metro Plaza II (as depicted in Figure 24). Thus torsion from eccentricities is created. These torsional moments must be considered in addition to the torsional moments listed in ASCE 7-05. In the original concrete structure, torsion played a significant role in the design, and thus this issue was deeply considered in the redesign of the steel structure. In order to mitigate the effects of torsion, a set of braced frames was added to the lateral force resisting system in effort to return the center of rigidity closer to the centers of mass and pressure, thus reducing the eccentricity. In the original concrete design, the controlling lateral case in this direction was wind load case 2, which incorporates a torsional element. In the steel redesign, wind load case 1 (full wind pressure of a single orthogonal pressure) is the controlling lateral case. Therefore, torsion was successfully mitigated in the redesigned steel structure and it no longer plays as significant a role as it did in the original concrete structure.

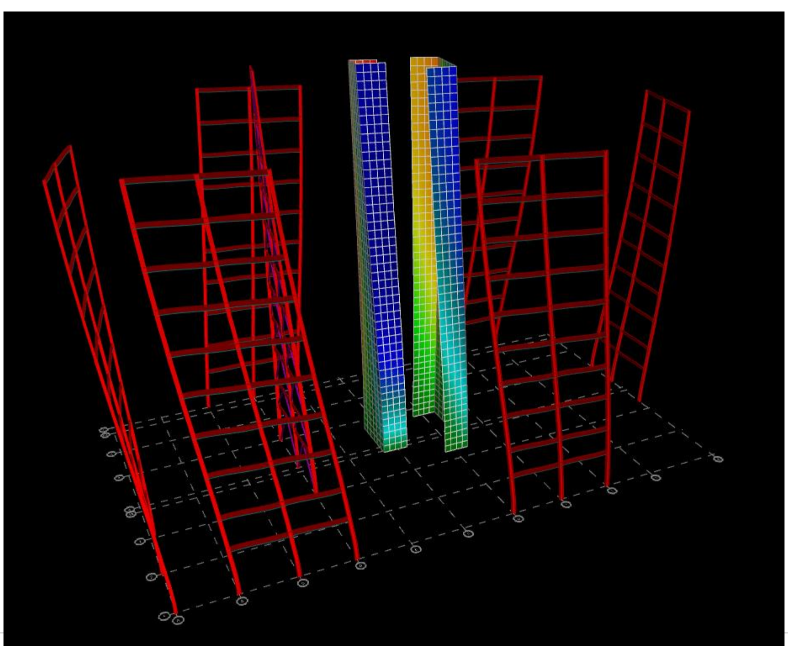


Figure 26: RAM Model – Amplified Displacement of a Case 2 Wind Load

Overturning Moment

Overturning moment is induced by the lateral forces that act on the structure. This item may impact several building components, but their effect is most commonly viewed upon the foundation. While individual footings may be isolated for analysis in order to see how overturning moment will affect them, it is also reasonable to view this issue on a more global scale. By comparing the full overturning moment caused by the lateral load to the resisting moment available from the dead load, it can be quickly assessed as to whether the structure will have a stability issue or not.

In considering individual columns, the moment is transferred via a coupled force. One column within a frame will receive a compressive load while the other receives a tensile load. It is important to ensure that an individual column is not seeing any net tension since uplift should be minimized if not eliminated. It should also be ensure that nominal compressive loads are not exceeded. It is also possible that moment may be accumulated in a single column. This effect must be taken into account as well.

The following data is calculated based on the story shears at each level. Once appropriate load factors are applied, (1.6 to wind and 1.0 to seismic), it becomes evident that wind is controlling this design factor in the E-W direction with a (factored) moment of 66,059 kip-ft (1.6 x 41,287) and seismic is controlling in the N-S direction with a (factored) moment of 43,408 kip-ft (1.0 x 43,408). This is less than the (factored) moment due to the building weight 2,003,872 kip-ft in the N-S direction and 1,145,070 kip-ft in the E-W direction. See Appendix D for further calculations.

From this comparison, it is evident that the structure will not experience overall building overturning. However, elements such as the steel moment frames could potentially see a net uplift force. In such a situation, it becomes necessary to design the connections accordingly, especially the connections at the base of the frame. It should be ensured that if a net uplift force is present at the steel to concrete connection, that the weight of the garage levels are substantial enough to eliminate it by the time it comes to the footing (as typical footings cannot resist this tensile force).

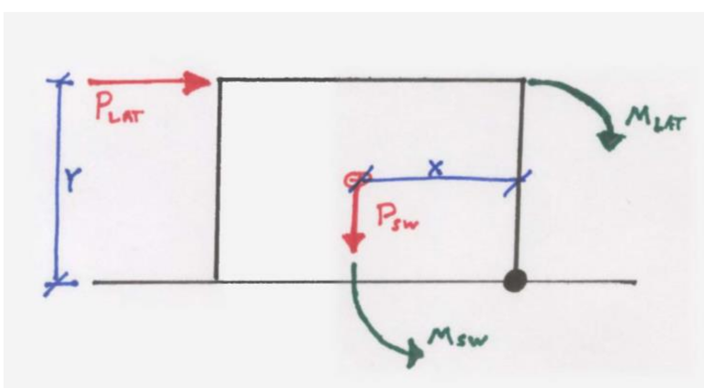


Figure 27: Depiction of Global Overturning Moment

Table 14: Wind Overturning Moment (E-W)			
Level	**Height (ft)**	**Story Force (k)**	**Overturning Moment (k-ft)**
Pent	150.33	29.49	4433.68
Roof	129.17	61.58	7953.77
11th	116.58	51.91	6051.35
10th	104.00	50.86	5289.95
9th	91.42	49.73	4545.84
8th	78.83	48.47	3820.68
7th	66.25	47.04	3116.62
6th	53.67	45.40	2436.49
5th	41.08	45.22	1857.86
4th	27.42	39.50	1083.00
P6	16.75	41.63	697.33
	Totals	**510.83**	**41286.57**

Table 15: Wind Overturning Moment (N-S)			
Level	**Height (ft)**	**Story Force (k)**	**Overturning Moment (k-ft)**
Pent	150.33	10.33	1552.67
Roof	129.17	29.02	3748.32
11th	116.58	26.79	3123.62
10th	104.00	26.18	2723.13
9th	91.42	25.52	2332.75
8th	78.83	24.78	1953.47
7th	66.25	23.95	1586.53
6th	53.67	22.99	1233.60
5th	41.08	22.73	933.72
4th	27.42	19.65	538.81
P6	16.75	20.46	342.72
	Totals	**252.40**	**20069.35**

Table 16: Seismic Overturning Moment (E-W)			
Level	**Height (ft)**	**Story Force (k)**	**Overturning Moment (k-ft)**
Pent	150.33	26.77	4024.10
Roof	129.17	109.13	14096.42
11th	116.58	72.06	8401.10
10th	104.00	63.00	6552.04
9th	91.42	54.13	4948.53
8th	78.83	45.74	3605.86
7th	66.25	37.28	2469.64
6th	53.67	29.10	1561.45
5th	41.08	21.50	883.47
4th	27.42	13.41	367.55
P6	16.75	6.87	115.12
	Totals	**478.99**	**47025.28**

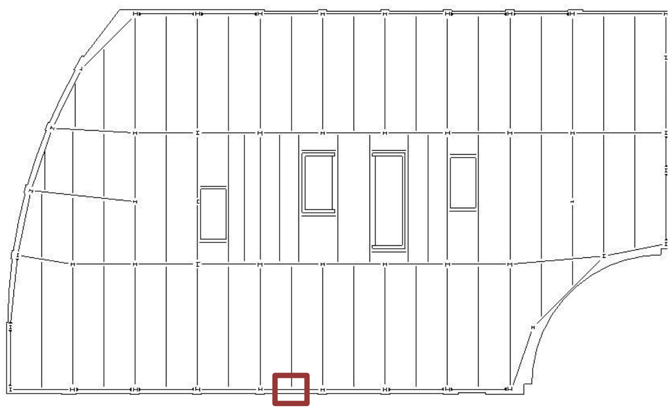
Table 17: Seismic Overturning Moment (N-S)			
Level	**Height (ft)**	**Story Force (k)**	**Overturning Moment (k-ft)**
Pent	150.33	24.71	3714.56
Roof	129.17	100.74	13012.08
11th	116.58	66.52	7754.86
10th	104.00	58.15	6048.03
9th	91.42	49.97	4567.88
8th	78.83	42.22	3328.49
7th	66.25	34.41	2279.67
6th	53.67	26.86	1441.33
5th	41.08	19.85	815.51
4th	27.42	12.37	339.28
P6	16.75	6.34	106.27
	Totals	**442.15**	**43407.95**

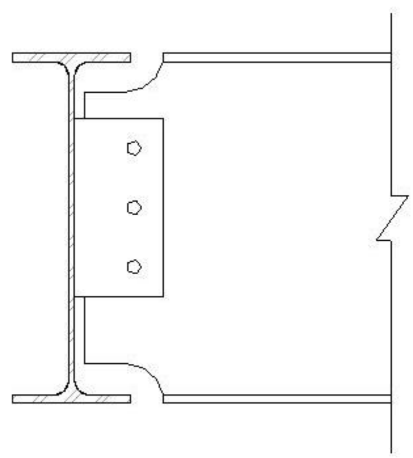
Connection Design

The design of the new steel system requires the investigation of member connections. In designing this aspect, constructability was of great concern. The following depicts some of the typical connections that were designed for the redesigned steel structure. Detailed calculation of these instances may be found in Appendix E.

▪ Typical Beam-to-Girder Connection

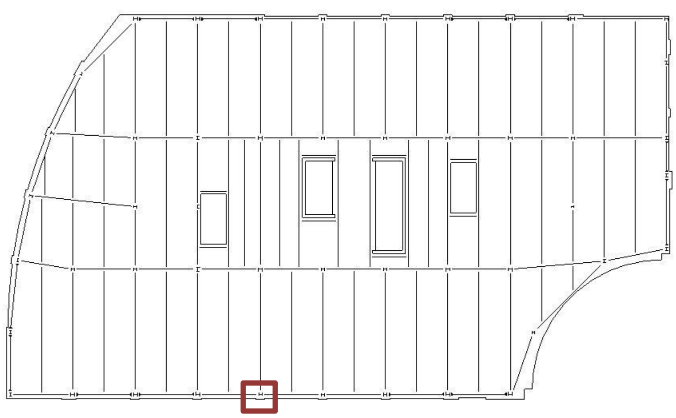
For this connection, a shear tab is employed. The simplicity of the connection and the option for shop welding of the tab will allow for ease in construction.

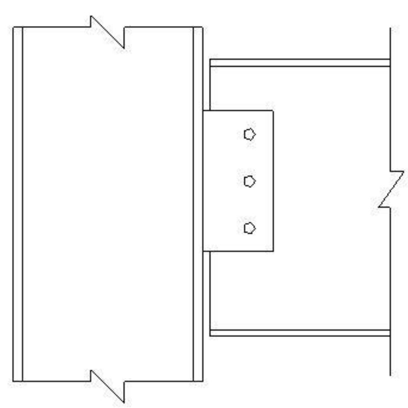




▪ Typical Girder-to-Column Connection

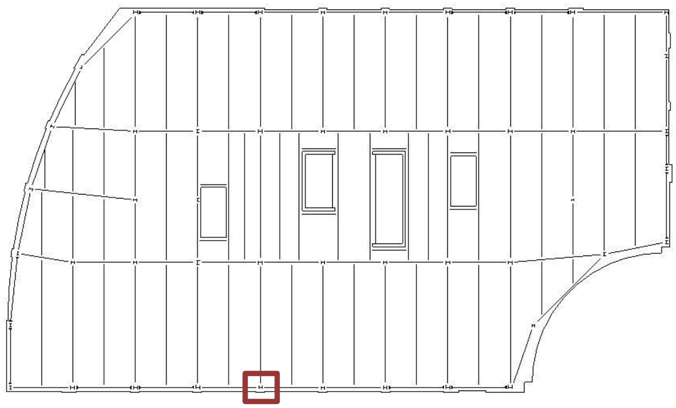
This connection employs a shear tab which will be welded to the flange of the column. Once again, the simplicity of the connection and the option for shop welding of the tab will allow for ease in construction.

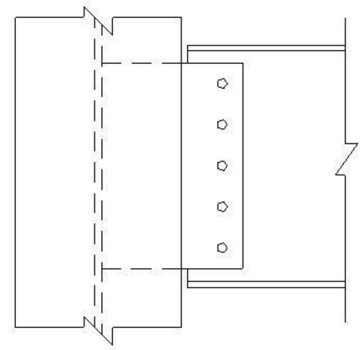




▪ Typical Beam-to-Column Connection (Upper Levels)

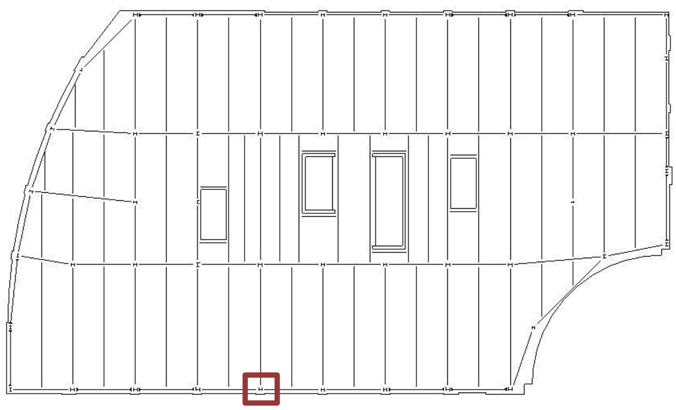
This connection uses an extended shear tab to transfer loading. On the upper levels, the depth of the typical column section is not sufficient for the beam to frame closer to the column's web. Thus the extended tab allows for a connection that may bridge this confined space.

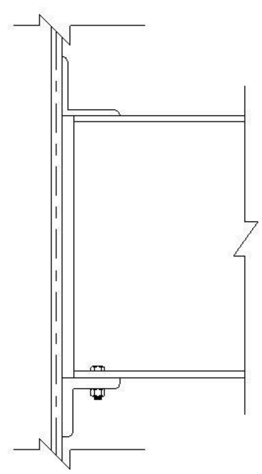




▪ Typical Beam-to-Column Connection (Lower Levels)

This connection uses an unstiffened seated connection to transfer loading. On the lower levels, the depth of the typical column section is sufficient for the beam to frame in close to the column's web. Thus the unstiffened seated connection becomes the preferred connection type due to its relatively simpler constructability.





▪ Other Connections

Moment connections and base plate connections are also prevalent in this design. Detailed designs of these connections may be found in Appendix E of this document.

Architectural Study

Redesigning Rockville Metro Plaza II's structural system from concrete to steel obvious poses many architectural concerns. The original system possessed many advantages. The employment of post tensioned concrete in the gravity system allowed for wide and open floor plans. The use of post tensioning also allowed for a shallow floor depth with sufficient room to accommodate mechanical and electrical elements. The integration of concrete moment frames and shear walls in the lateral system allowed for a very efficient and economical outcome.

The architectural focus of the gravity system's redesign was to preserve the wide open floor plan of the office space. This aspect allows for a versatile area and thus may attract a wide array of potential tenants. In turn, the column layout was preserved and beam depths were minimized. The overall height of the building was still increased by approximately seven feet in order to maintain the original ceiling heights. Note this new height exceeds zoning regulations.

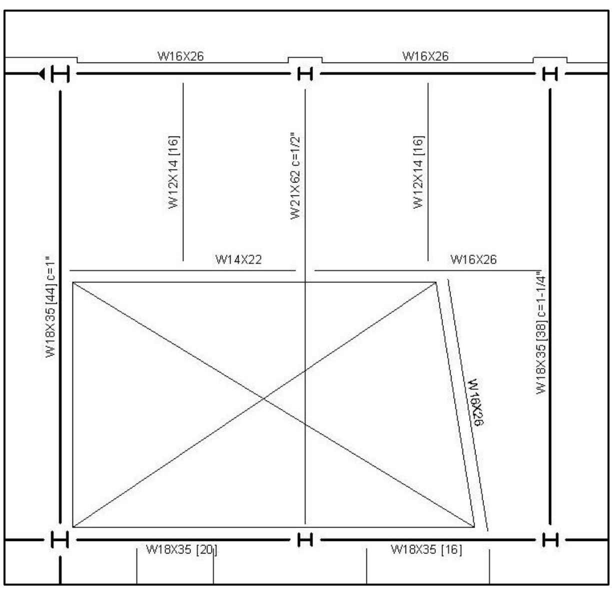
The architectural focus of the lateral system's redesign was to preserve the uninterrupted windows of the façade. This feature allows for an abundance of daylight to illuminate the office space as well as provides great views of the surrounding areas. The goal of retaining these elements eliminated the initial design of braced framed. Originally, it was sought to integrate the braces with the glass curtain walls which intermittently occur on the structure. This option would however inevitably prove expensive as the glass would have angles that would require special orders on an individual level. The next alternative investigated was the use of steel moment frames. This option allowed for the uninterrupted window pattern which was sought. A comparison of a corner office with and without the bracing may be viewed below. The redesigned system also retained to use of concrete shear walls at the core of the structure. Replacing these elements with braced frames was considered, but due to the location of the elevator core and its distance from the column grid pattern, it was reasoned otherwise. Since walls did not typically fall between column lines, the use of braced frames became further unsuitable.

Figure 28: Braced Frame Rendering

Figure 29: Moment Frame Rendering

Another architectural aspect investigated was the option of an opening in the floor plan of a lower office level for the construction of an architectural staircase. Such an aspect would more closely join two levels of the structure and create a more open and inviting feel if the space were to be used as a reception area. This feature could be sought by a tenant who would like the space of multiple floors to be shared in a more intimate manner than just the connection of the elevator core. In this architectural study, the lower level was furnished as a reception area and the upper area was arranged as a lounge/reception type space. The renderings on the following page display the architectural possibilities that accompany this layout option.

The architectural possibility of an opening was thoroughly investigated in the structural design of the building. For the purpose of this investigation, the opening was placed on the "5th floor" level. This essentially connects the 4th and 5th floor offices. Two options were developed for the opening: to either leave the center beam in its current span, or to redirect it as in the figure below. Due to architectural reasons and potential safety concerns of the chosen layout, the latter option was selected. Even though this requires deeper beams, it is postulated that large mechanical ductwork will not be placed in this section as determined from the MEP drawings. Both options are shown below. Also note that the option for the opening to be installed at a later time was also factored into the design of this feature and columns were sized accordingly.



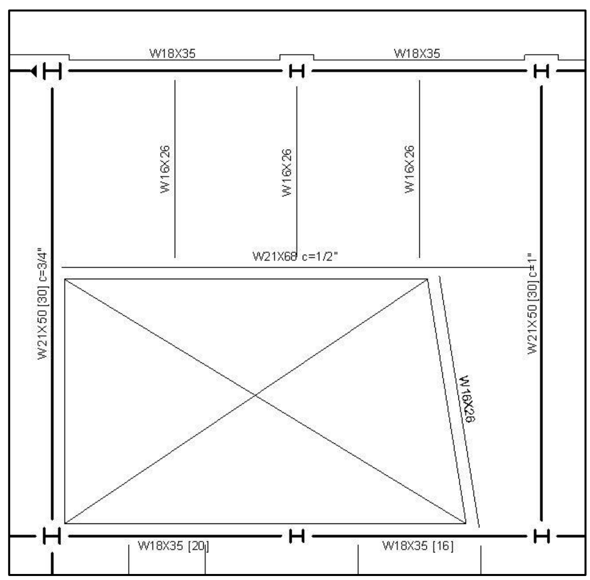


Figure 30:
Upper left – Option 1 Opening
Upper Right – Option 2 Opening
Left – Key locating Bays in Consideration

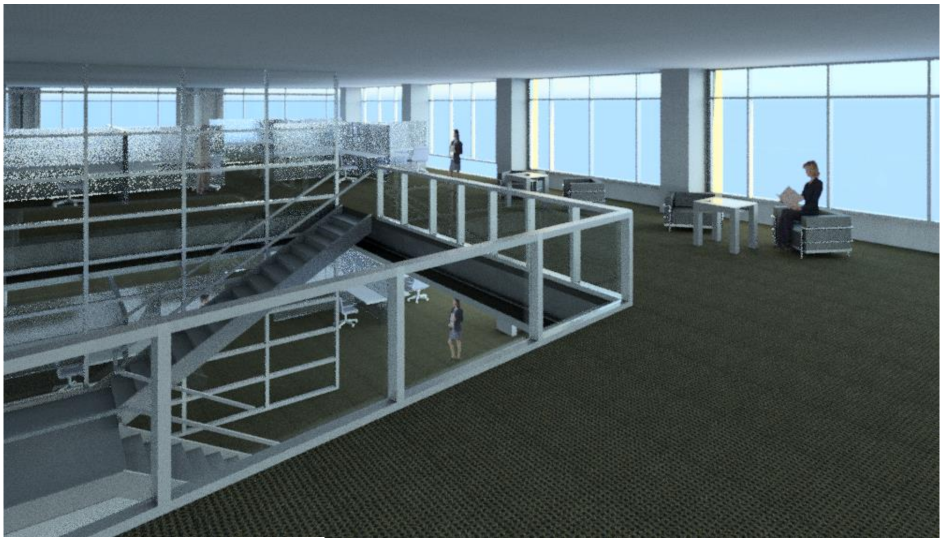


Figure 31: Rendering of Opening and Staircase from the 5th Floor

Figure 32: Rendering of Opening and Staircase from the 4th Floor

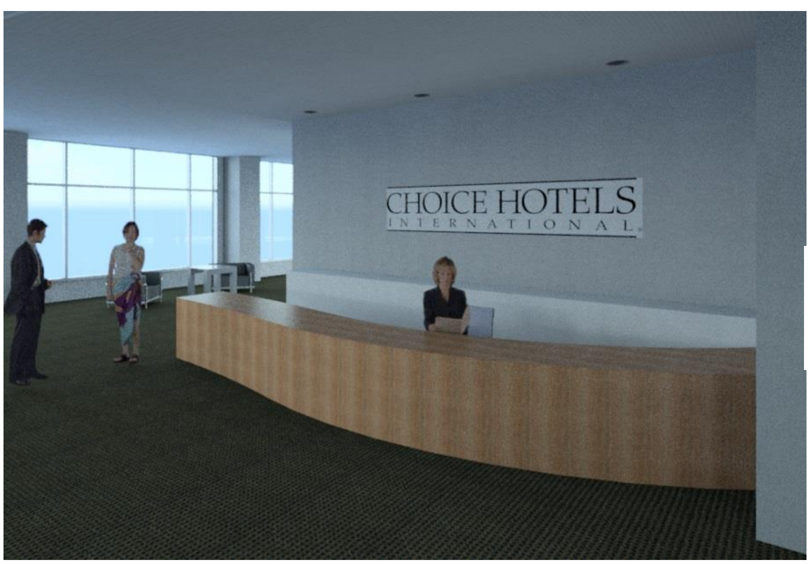


Figure 33: Rendering of Reception Area

Cost/Schedule Study

In order to compare the steel redesign with the original structure, a detailed cost estimate was conducted for each structural system. The items included in each system option are outlined in the tables below. It was concluded that the steel option would result in a slightly lower cost relative to the concrete option (approximately 5.5% less). Note that this only takes into account the structure above the seismic base (as the substructure was not included in the redesign).

The change in structural system also drastically effects the scheduling of construction. The steel system is projected to reduce the construction of the superstructure by 11 months. Summaries of these schedules are provided in Appendix F of this document. This is a logical reduction in time considering that steel is generally faster to erect and that concrete requires extra time for the construction of forms and rebar cages as well as time for curing. While the potential cost savings of the amendment were not fully investigated, it remains evident that at minimum, the steel alternate would potentially allow for the building to be constructed in a short time period.

Steel Option Summary					
	Material	Labor	Equipment	Total	Tot. Incl O&P
Steel Deck	$480,731.15	$107,871.38	$9,380.12	$597,982.65	$736,339.42
Welded Wire Fabric	$34,002.94	$53,935.69	$0.00	$87,938.63	$126,631.62
Placing Concrete	$0.00	$55,342.71	$17,064.00	$72,406.71	$107,610.82
Finishing Concrete	$0.00	$136,011.74	$7,035.09	$143,046.83	$225,122.88
Concrete Topping	$290,783.72	$206,362.64	$63,315.81	$560,462.17	$724,614.27
Steel Beams	$1,635,720.87	$270,638.34	$77,920.05	$1,984,279.25	$2,379,232.78
Steel Columns	$781,144.07	$127,805.41	$36,796.72	$945,746.19	$1,133,117.02
Shear Studs	$8,441.44	$13,265.12	$7,537.00	$29,243.56	$41,754.98
Fireproofing Beams	$84,381.30	$98,710.20	$14,328.90	$197,420.40	$267,472.80
Fireproofing Columns	$45,199.24	$49,719.16	$7,156.55	$102,074.95	$138,234.34
Total	**$3,360,404.72**	**$1,119,662.38**	**$240,534.24**	**$4,720,601.34**	**$5,880,130.93**

Concrete Option Summary					
	Material	Labor	Equipment	Total	Tot. Incl O&P
Concrete Formwork	$502,426.15	$1,634,629.69	$0.00	$2,137,055.84	$3,237,060.29
Structural Concrete	$900,131.59	$0.00	$0.00	$900,131.59	$985,143.36
Placing Concrete	$0.00	$230,177.47	$87,713.87	$317,891.34	$470,352.93
Finishing Concrete	$3,182.12	$185,401.23	$6,915.09	$195,498.45	$306,404.66
Reinforcing	$493,131.92	$414,983.46	$1,718.58	$909,833.96	$1,226,880.19
Total	**$1,898,871.79**	**$2,465,191.86**	**$96,347.54**	**$4,460,411.18**	**$6,225,841.43**

Redesign Summary

The structural focus for this investigation was founded on the academic question of whether Rockville Metro Plaza could be built as a steel framed structure rather than a concrete structure. To begin, the gravity system was redesigned, responding to the needs of architectural and mechanical concern. This proved the need to increase the height of the structure in order to retain the as designed ceiling heights and MEP space clearances. New moment frames, braced frames and shear walls were effectively designed as the lateral system of Rockville Metro Plaza II. While such a combination may not be realistic, this choice was made as an educational opportunity to investigate different configurations. A comprehensive study of the new steel design proved its viability as an alternative structural system.

As the structural focus was in progress, two auxiliary elements were studied. An architectural study provided the necessary background information used in assessing the impacts various lateral systems would potentially have on the building. When considering braced frames, the study yielded several concerns regarding the flow of internal space, views to the exterior, and building entrances. Thus, steel moment frames were employed in order to ameliorate these concerns. The pursuit of retaining the architectural intentions is accompanied by caveats. In retaining the designed ceiling height, the overall building height was increased by approximately eight feet. This places the top of the structure over the zoning restrictive limit. If a steel design such as this were to be competed in Rockville, this aspect would have to be amended.

A construction management study was also completed in order to assess the new structural design on the criteria of schedule and cost. The analysis showed a dramatic decrease in erection time of the reigned structural system relative to the original. In comparing the costs of each system, the steel system proved more efficient though the change was not as drastic as only a five percent decrease was calculated.

After these analyses were completed, it was determined that while there is potential for Rockville Metro Plaza II to be constructed in steel, real world concerns favor the concrete system. Regardless, the educational value of this project to those involved has been immeasurable.

Resources

International Building Code 2009

Minimum Design Loads for Buildings and Other Structures: ASCE 7-05

AISC Steel Construction Manual, Fourteenth Edition

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

R.S. Means Building Construction Data, 2014

Unified Design of Steel Structures, Second Edition

Figure 34: Perspective of Rockville Metro Plaza II