Rockville Metro Plaza II

121 Rockville Pike
Rockville, Maryland

Technical Report II

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Executive Summary

Figure 1: 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.

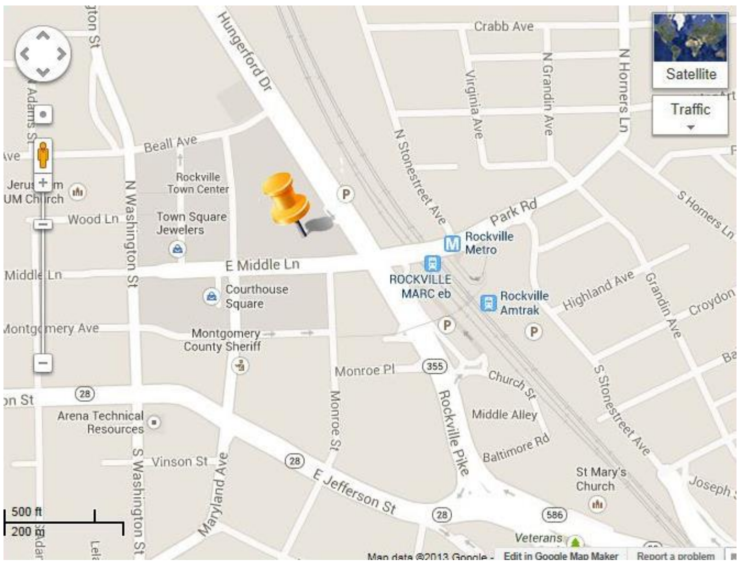
The entire structural system is built using cast-in-place concrete. The lower levels of the structure (parking and retail levels) use flat plate, two-way slabs with mild reinforcing to support the floors. Columns which bear these levels incorporate drop caps in order to better resist punching shear forces. 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.



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.

Figure 2: Map of Site Location – From “maps.google.com”

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.

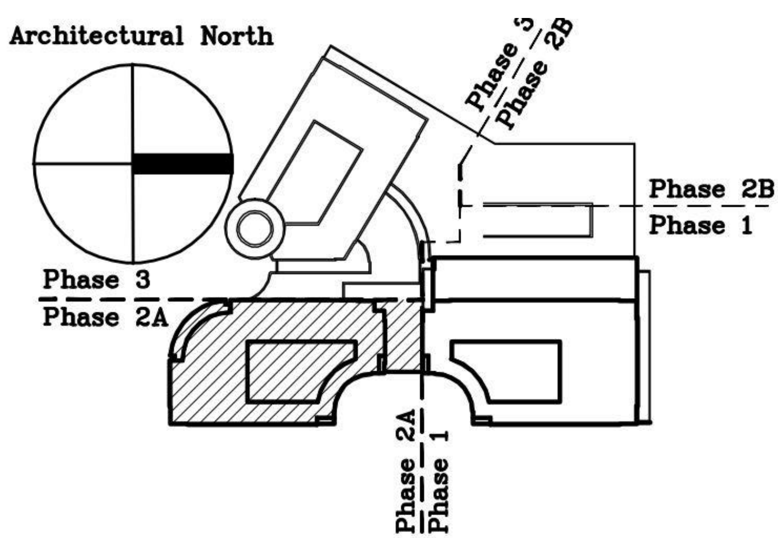


Figure 3: Map of Building Relations – by WDG Arch.

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

Figure 4: Rockville Town Square Obelisk – by JMV

Gravity Loads

Floor Loads

Rockville Metro II utilizes multiple floor systems to comprise its structure. On the office levels, floors are generally comprised of one-way slab systems on a 20' by 40' bay. These slabs are carried by wide, shallow post tension beams which transfer loads to the building's columns. On the parking levels below grade, a two-way slab system is used. These levels are mapped by 26' x 20' bays and thus better suited to be designed as two way slabs.

Garage Slab Loads

Within MET II, the below grade parking garage comprises levels P1, P2, and P3. OF these, 2 and 3 are elevated 8" slabs comprised of normal weight concrete and mild reinforcing.

These lower levels do not have the need for as large of an open space as compared to the office areas. The span here is governed by the diving aisle width that the International Building Code requires. Thus, the slab is designed to the 26' x 20' bay size. Since the aspect ratio is squarer, the section can be designed as a two-way slab system.

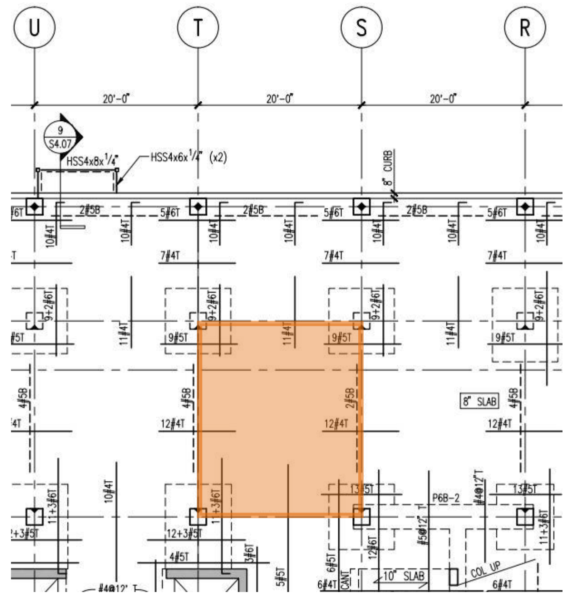


Figure 5: Plan of Garage Bay – by Cagley and Assoc.

In terms of loading, the slab itself once again contributes most of the dead load on the floor system. Such items mechanical and lighting equipment are relatively light and are accounted for in the super imposed dead load. There is no flooring material installed on top of the slab and no hanging ceiling system below. The occupancy live load is defined in the IBC as a garage load of 40 psf (passenger vehicles only). However, the design uses a load of 50 psf which is the minimum load for truck and bus garages.

Table 1: Garage Loads	
Type	**Load Value (psf)**
Slab	100
SDL	5
Live	50

Office Slab Loads

Within MET II, office space comprises the 4th through 11th floors. Due to the consistency in layout for level to level, a typical slab design is used for each level. This is comprised of an 8" normal weight concrete slab with mild reinforcing.

In order to create a larger open space in the layout, the typical bay is designed at 20' x 40' (as seen in figure 6 to the right). This open floor plan allows the tenant of the space to have more flexibility in how they want to organize the space. Due to the uneven aspect ratio of the bay, the slab acts as a one-way system. The slab is reinforced with a bottom mat made of #4 bars at 12" on center.

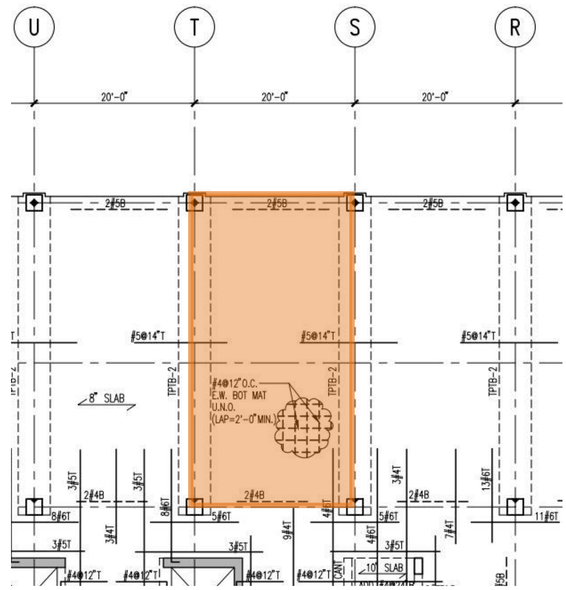


Figure 6: Plan of Office Bay – by Cagley and Assoc.

In terms of loading, the slab itself contributes most of the dead load on 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. 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.

Table 2: Office Loads

Type	Load Value (psf)
Slab	100
SDL	5
Live (Occupant)	80
Live (Partition)	20

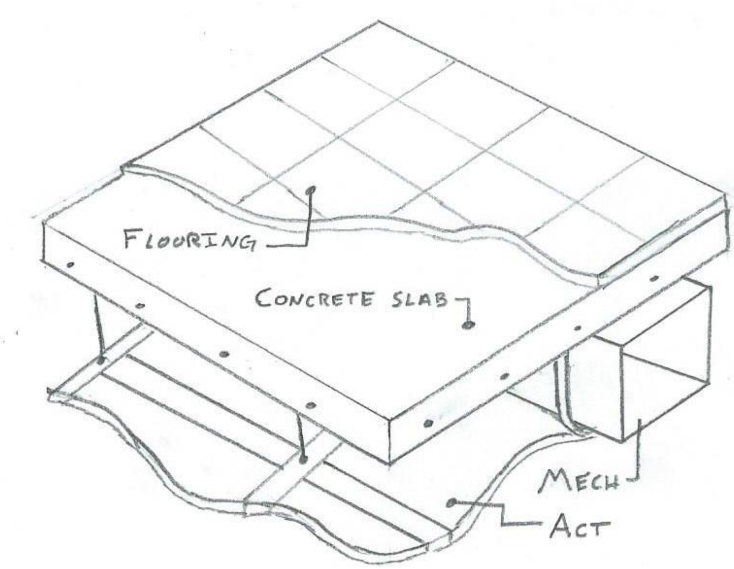


Figure 7: Cut Away of Typical Floor Slab – by JMV

Roof Slab Loads

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.

In order to support the roof, a concrete slab is used in a similar configuration as seen on the office levels: an 8” concrete slab comprised of normal weight concrete and #4 bars as reinforcing. The bays are 40’ x 20’ and the roof slab act as a one-way system and wide, shallow post tension beams are provided to transfer the load to columns.

In terms of loading, the slab itself contributes most of the dead load on the floor system. Hanging loads for the ceiling below are accounted for in the super imposed dead load. The green roof also contributes to the dead load. Live loads are as governed by IBC and ASCE 7. The controlling load is a roof live load of 30 psf for ponding (as the snow load and occupant load were determined to b 17.5 psf and 20 psf respectively).

Table 3: Roof Composition

Item	Design Value (psf)
Vegetation	1
Soil	29
Filter/ Moisture Mat	2
Insulation	3
Roof Membrane	5
Slab	100
SDL	10

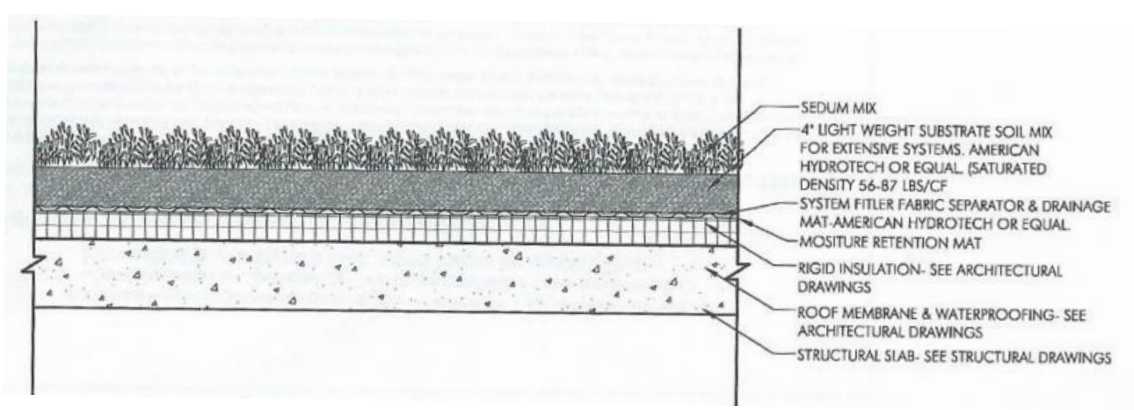


Figure 8: Green Roof Cross Section – by Studio 39

Exterior Wall Load

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.

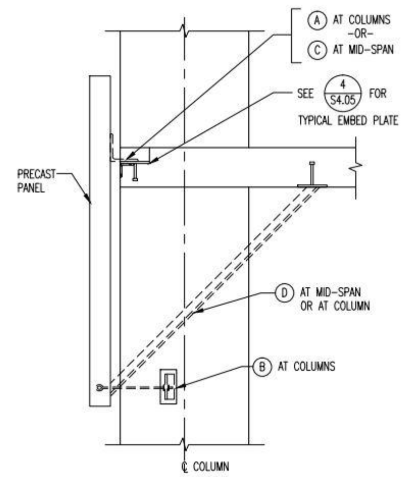


Figure 9: Precast Elevation Detail - by Cagley and Assoc.

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 (as seen in figure 9). 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. Figure 9 depicts the tie back connections and the fact that they occur at two different elevations at each connection point.

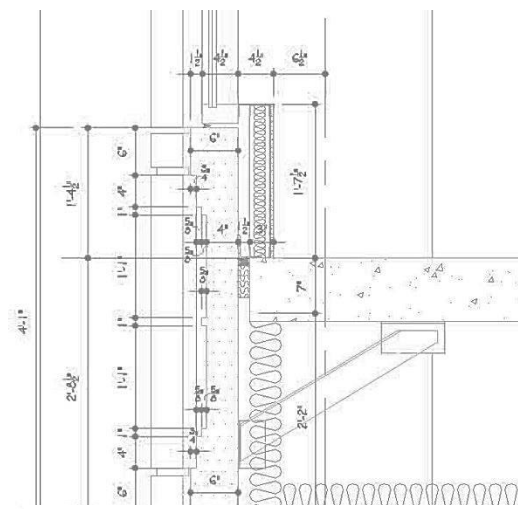


Figure 10: Wall Elevation Section - by Cagley and Assoc.

The aluminum framed window system is set between the precast panels, thus their load bears on the panels. Cold formed steel studs and the remaining wall components such as insulation and dry wall bear directly onto the concrete slab. In designing the structural system of the building, a line load of 500 plf was used by the structural engineer to estimate the load of the wall configuration. During the design stage, this load would be applied to the slab, and would in turn be transferred to the columns. In actuality, the load of the precast concrete panel is directly transferred to the columns. The only load the slab sees comes from lateral loads and from the interior wall components that are set directly on the slab.

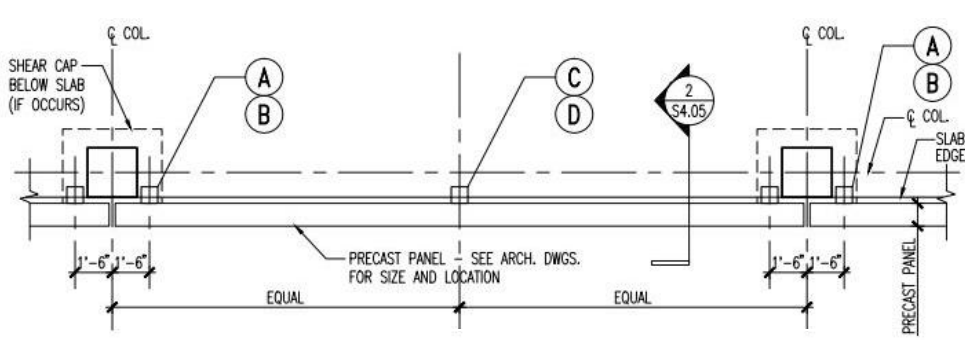


Figure 11: Precast Plan Detail – by Cagley and Assoc.

Gravity Load Summary

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.

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 4: 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 5: Flat Roof Snow Load

Ground Snow Load	P_g=	25 psf
Snow Exposre 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 6: Superimposed Dead Loads

Area	Design Value (psf)
Floor	5
Roof	10

Lateral Analysis – Wind Load

Wind Load

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 A of this document.

Figure 12: Perspective View of Southern Face - JMV

Wind Pressure – East-West

Table 7: 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	142.00	12.71	-7.57	20.27	28.97	28.97	4113.36
	131.42	12.43	-7.57	20.00			
Main Roof	120.83	12.13	-7.57	19.70	59.28	88.24	7162.70
	114.96	11.96	-7.57	19.53			
11th	109.08	11.78	-7.57	19.35	47.52	135.77	5184.07
	103.21	11.60	-7.57	19.17			
10th	97.33	11.41	-7.57	18.97	46.57	182.34	4533.05
	91.46	11.21	-7.57	18.77			
9th	85.58	10.99	-7.57	18.56	45.53	227.87	3896.77
	79.71	10.77	-7.57	18.34			
8th	73.83	10.54	-7.57	18.11	44.38	272.25	3276.68
	67.96	10.29	-7.57	17.86			
7th	62.08	10.03	-7.57	17.60	43.08	315.33	2674.59
	56.21	9.75	-7.57	17.32			
6th	50.33	9.45	-7.57	17.02	41.58	356.91	2092.90
	44.46	9.12	-7.57	16.69			
5th	38.58	8.76	-7.57	16.32	41.54	398.46	1602.80
	32.17	8.31	-7.57	15.88			
4th	25.75	7.80	-7.57	15.37	36.11	434.56	929.74
	20.83	7.34	-7.57	14.91			
P6	15.92	6.80	-7.57	14.37	38.56	473.13	613.81
	7.96	6.63	-7.57	14.20			
Plaza Level	0.00	6.63	-7.57	14.20	23.73	**496.85**	0.00
							36080.47

Base Shear	**496.85 Kips**
Overturning Moment	**36080.47 Kip-ft**

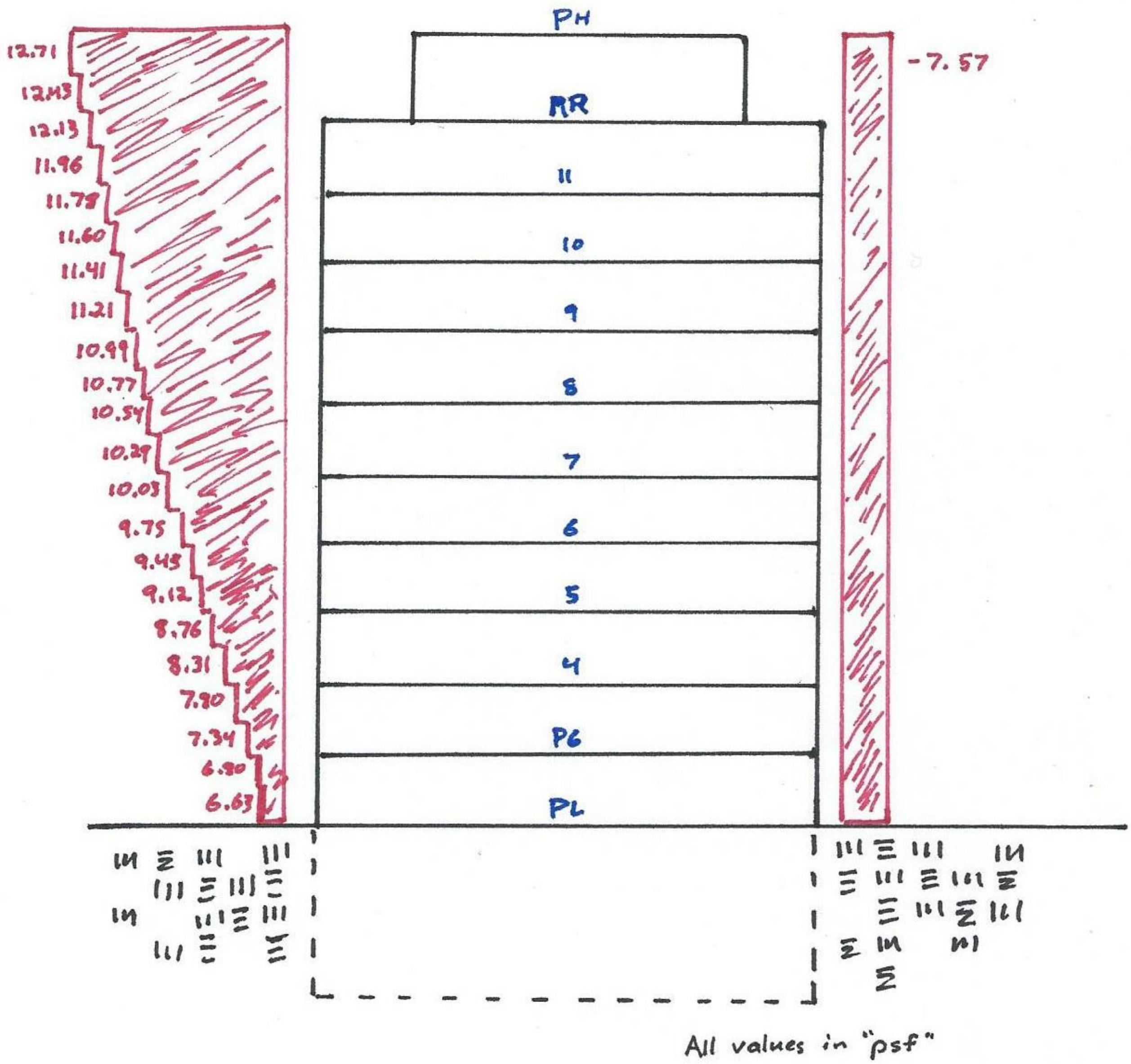


Figure 13: East-West Design Pressure Diagram

Wind Pressure – North-South

Table 8: 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	142.00	13.02	-5.44	18.46	10.16	10.16	1442.90
	131.42	12.74	-5.44	18.18			
Main Roof	120.83	12.44	-5.44	17.88	28.11	38.27	3396.78
	114.96	12.26	-5.44	17.70			
11th	109.08	12.08	-5.44	17.52	24.57	62.84	2679.92
	103.21	11.89	-5.44	17.33			
10th	97.33	11.69	-5.44	17.13	24.01	86.85	2337.01
	91.46	11.48	-5.44	16.93			
9th	85.58	11.27	-5.44	16.71	23.40	110.25	2002.73
	79.71	11.04	-5.44	16.48			
8th	73.83	10.80	-5.44	16.24	22.73	132.98	1677.93
	67.96	10.55	-5.44	15.99			
7th	62.08	10.28	-5.44	15.72	21.97	154.94	1363.68
	56.21	9.99	-5.44	15.43			
6th	50.33	9.68	-5.44	15.12	21.09	176.03	1061.38
	44.46	9.35	-5.44	14.79			
5th	38.58	8.97	-5.44	14.42	20.91	196.94	806.90
	32.17	8.52	-5.44	13.96			
4th	25.75	8.00	-5.44	13.44	18.00	214.94	463.41
	20.83	7.53	-5.44	12.97			
P6	15.92	6.97	-5.44	12.41	19.01	233.95	302.53
	7.96	6.80	-5.44	12.24			
Plaza Level	0.00	6.80	-5.44	12.24	11.69	**245.63**	0.00
							17535.19

Base Shear	**245.63 Kips**
Overturning Moment	**17535.19 Kip-ft**

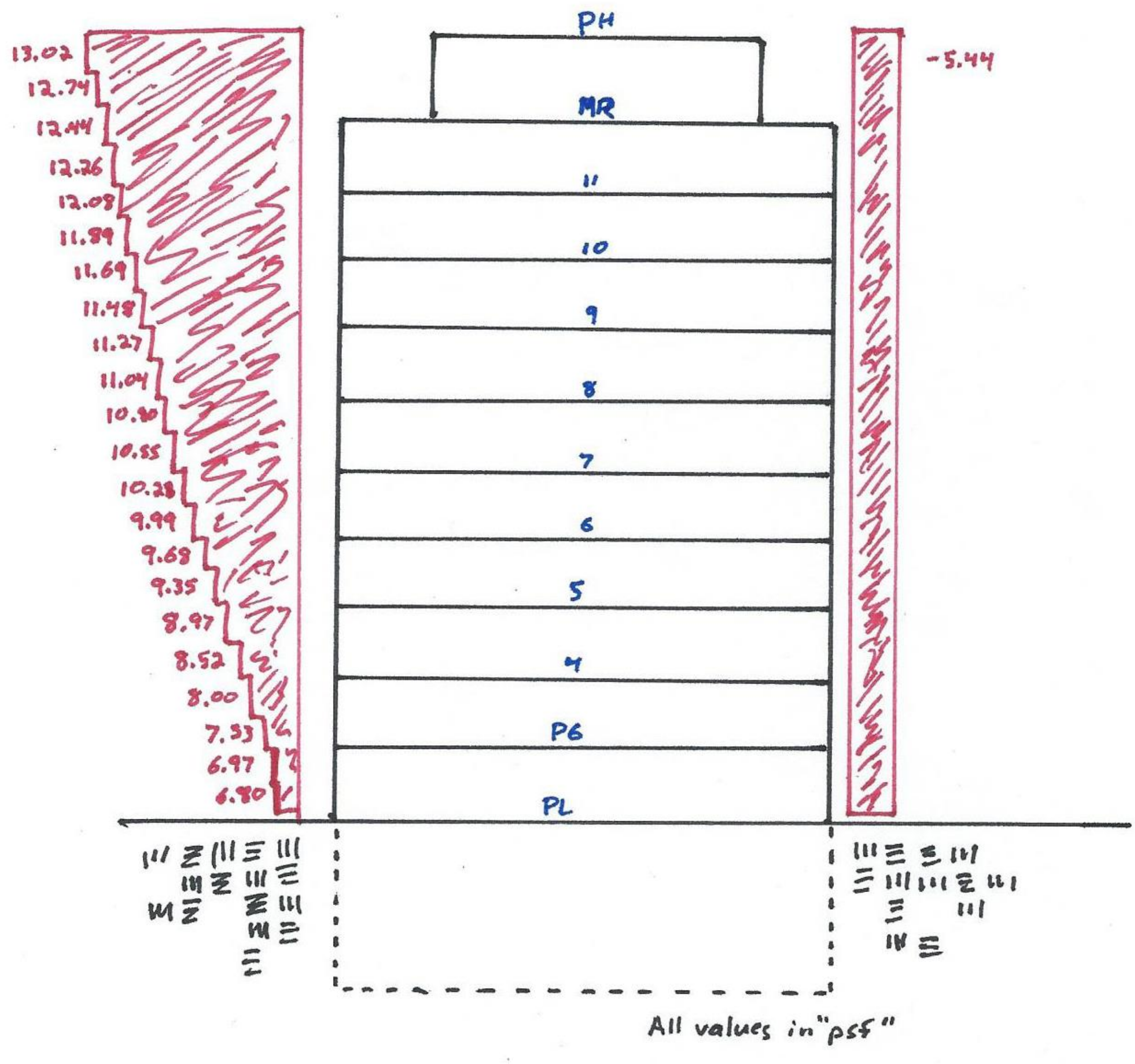


Figure 14: North-South Design Pressure Diagram

Wind Load Summary

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 496.85 kips, compared to the value of 245.63 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. The combination of concrete columns and post tension beams (as well as the rigid slab) form the moment frame systems.

The wind design variables present on the structural documents were consistent with the values determined and used in this analysis. The final design forces used by the structural engineer, however, were not available for direct comparison to the results of this analysis.

Figure 15: Exterior View from Across Rockville Pike – by JMV

Lateral Analysis – Seismic Load

Seismic Load

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.

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 B.

Figure 16: Exterior View from Across Rockville Pike Intersection – by JMV

Table 9: 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	R	4.5	Table 12.2-1
Approximate Period Parameter	C_t	0.02	Table 12.8-2
Building Height	h_n	142’	Arch Dwg.
Approximate Period Parameter	x	0.75	Table 12.8-2
Approx. Fundamental Period	T_a	0.823 s	Eq. 12.8-7
Long Period Transition Period	T_L	8.0 s	Fig. 22-15
Seismic Response Coefficient	C_S	0.0156	Eq.’s 12.8-2,3
Structure Period Exponent	k	1.161	Section 12.8.3

Table 10: Design Values	
Effective Seismic Weight	41163 kips
Base Shear	642.7 kips
Overturning Moment	57708 kips-ft

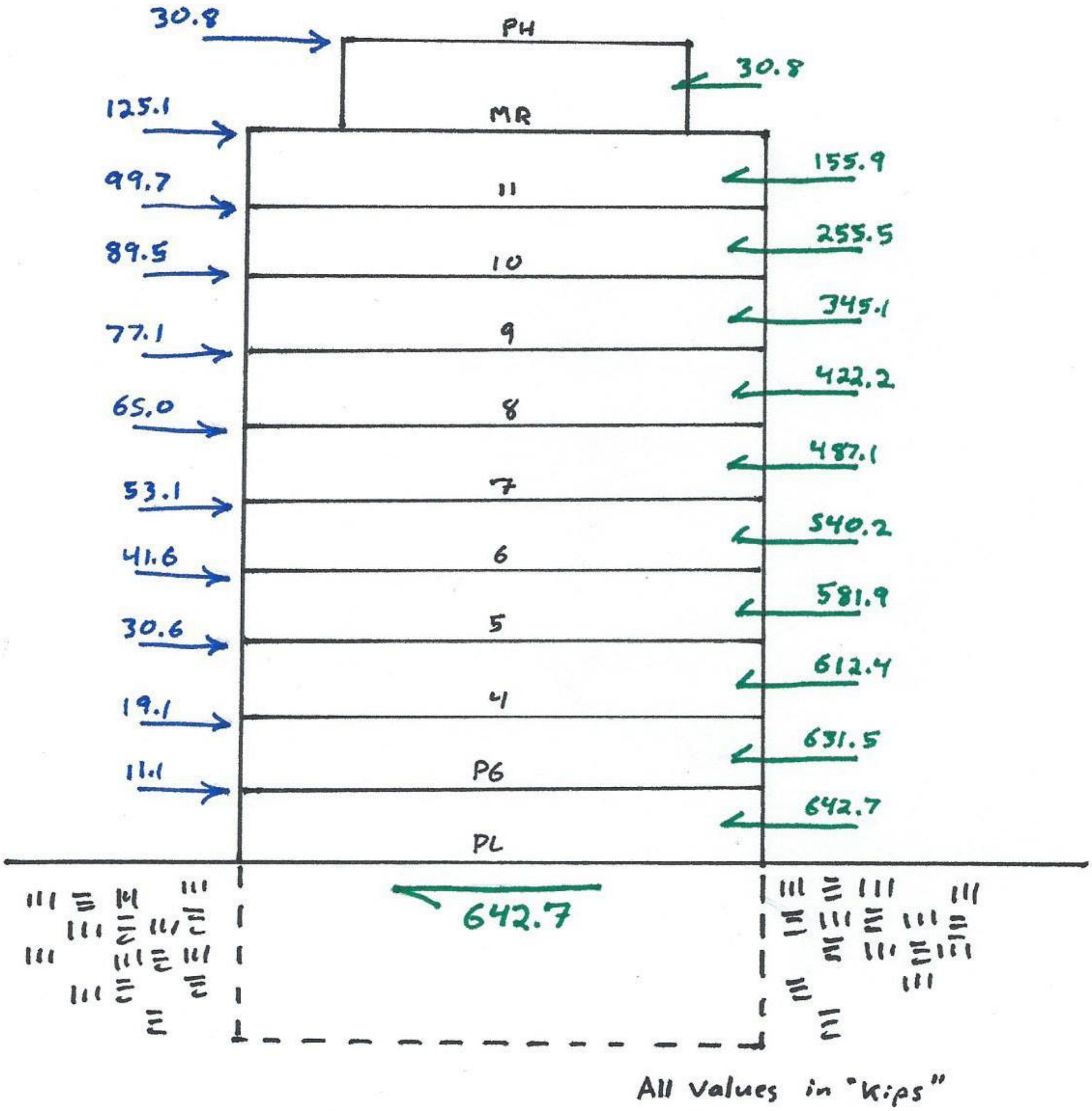


Figure 17: Diagram of Design Values

Table 11: Seismic Calculations					
Level	**Story Weight**	**Height**	**Forces (F_x)**	**Story Shear (V_x)**	**Moments (M_x)**
	(kips)	(ft)	(kips)	(kips)	(k-ft)
Pent Roof	887	142.00	30.8	30.8	4375.638
Main Roof	4342	120.83	125.1	155.9	15111
11th Floor	3897	109.08	99.7	255.5	10871.97
10th Floor	3996	97.33	89.5	345.1	8714.116
9th Floor	3996	85.58	77.1	422.2	6598.774
8th Floor	3996	73.83	65.0	487.1	4795.579
7th Floor	3996	62.08	53.1	540.2	3297.158
6th Floor	3996	50.33	41.6	581.9	2095.07
5th Floor	3996	38.58	30.6	612.4	1179.39
4th Floor	3996	25.75	19.1	631.5	492.1244
P6	4065	15.92	11.1	642.7	176.99
Plaza Level	-	0.00	-	-	-
Total	**41163**	-	**642.7**	-	**57707.81**

Seismic Load Summary

The seismic analysis executed for this document provided a design base shear and overturning moment of 642.7 kips and 57708 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.

The seismic design values determined by the structural engineer of record were not available for direct comparison. However, when comparing the found seismic forces to the results calculated for wind, we find that seismic conditions do control in this case.

Closing

Through this study, a better understanding of Rockville Metro Plaza II's structural systems may be achieved.

In determining the loading and geometry of the structure, the reasoning behind the size of structural components such as columns and slabs becomes clearer. In further studies the loading may be applied to analysis of the various bays of the structure in order to specifically evaluate the roofing and flooring systems of MET II. As bay loads are determined, the loading of the post tension beams, columns, and foundations may be therefore found. The design of these items may be considered as well and their capacity examined.

Figure 18: Exterior Perspective – by JMV

Through the calculation of wind and seismic loading, a similar observation may be viewed over the lateral system. This analysis provides initial supporting evidence as to the choice of lateral system chosen by the structural designer. By comparison of these calculations, it was found that seismic controlled the design of the lateral system as this analysis produced a higher value for the base shear as well as the overturning moment on the structure. To further study the lateral system of MET II, lateral loads could be applied to the shear wall system to verify the size and amount provided.

Appendix A

Wind

J.M.V. | TECH 2 WIND | 1

Calculation For Wind Analysis

ASCE 7-05 →

Method 2 → Building Meets req 6.5.1

Basic Wind Speed

Rockville, MD $V = 90$ mph [Fig. 6-1]

Directionality Factor

$K_d = 0.85$ [Table 6-4]

Importance Factor

$I_w = 1.0$ [Table 6-1]

Exposure Category : B

Topographic Factor

$K_{zt} = 1.0$ [Sect. 6.5.7]

Determine Velocity Pressure Exposure Coefficient

K_z, K_h → See calc tables for values [Table 6-3]

Determine Velocity Pressures

$q_z, q_h = 0.00256 K_z K_{zt} K_d V^2 I$ [Eq. 6-15]

Determine Building Enclosure : Fully Enclosed [Sect. 6.5.9]

$G C_{pn} = +1.5$ windward

$G C_{pn} = -1.0$ Leeward [

Combined Net Design Pressure

$P_p = q_n G C_{pi}$ → see calc tables for values [

Determine Pressure coefficients

$C_p = 0.8$ (windward), -0.5 (leeward) [Fig 6-6]

$G C_{pi} = \pm 0.18$ [Fig 6-5]

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Determine Gust Effect Factor [Sect 6.5.8]

$$G = 0.925 \left(\frac{(1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2})}{(1 + 1.7 g_v I_z)} \right)$$ [Eq 6-8]

$$I_z = c\,(33/\bar{z})^{1/6}$$ [Eq 6-5]

$$Q = \sqrt{\frac{1}{1 + 0.63 \left(\frac{B+h}{L_z}\right)^{0.63}}}$$ [Eq 6-6]

$$L_z = \ell\,(\bar{z}/10)^{\bar{\epsilon}}$$ [Eq 6-7]

$$\bar{V}_z = \bar{b}\,(\bar{z}/33)^{\bar{\alpha}}\, V(88/60)$$ [Eq 6-14]

$$N_1 = (n_1 L_{\bar{z}})/(\bar{V}_{\bar{z}})$$ [Eq 6-12]

$$R_n = 7.47 N_1 / (1 + 10.3 N_1)^{5/3}$$ [Eq 6-11]

$$R_\ell = 1/\eta - 1/2\eta^2\,(1 - e^{-2\eta})$$ [Eq 6-13a]

$R_\ell = R_h$ for $\eta = 4.6 n_1 h / \bar{V}_z$

$R_\ell = R_B$ for $\eta = 4.6 n_1 B / \bar{V}_z$

$R_\ell = R_L$ for $\eta = 15.4 n_1 L / \bar{V}_z$

$$R = \sqrt{(1/\beta) R_n R_h R_B (0.53 + 0.47 R_L)}$$ [Eq 6-10]

n_1 (approx) $= 100/H$ [Eq

$$g_R = \sqrt{2\ln(3{,}600 n_1)} + 0.577/(\sqrt{2\ln(3{,}600 n_1)})$$ [Eq 6-9]

$g_Q = g_v = 3.4$ [Sect 6.5.8.2]

Note building is considered flexible by sect 6.2

Determine Design Wind Pressures

Windward : $P_z = q_z G C_p - q_h (G C_{pi})$

Leeward : $P_h = q_h G C_p - q_h (G C_{pi})$

See calc tables for results

Wind: East-West Direction

Table 12: East-West Design Factors	
Exposure B	
Case 2	
L	120 ft
B	210 ft
L/B	0.571
Natural Period (approx.) (n_1)	0.833
Damping Coeff. (approx.) (β)	0.02
Basic Wind Speed (V)	90 mph
Wind Directionality Factor (K_d)	0.85
Importance Factor (I)	1.0
Exposure Category	B
Topographical Factor (K_{zt})	1.0
Gust Effect Factor (G)	0.825
C_p Windward	0.8
C_p Leeward	-0.5
G_{cpi} Windward	0.18
G_{cpi} Leeward	-0.18
G_{pn} Windward	1.5
G_{pn} Leeward	-1.0

Table 13: East-West Calculation of Design Pressures								
	Height	**K_z, K_h**	**q_z, q_h**	**External Pressure**	**Internal Pressure**	**Net Positive**	**Net Negative**	**Total Pressure**
	(ft)			(psf)	(psf)	(psf)	(psf)	(psf)
Penthouse	142.00	1.09	19.25	12.71	3.47	9.24	16.17	20.27
	131.42	1.07	18.83	12.43	3.47	8.96	15.89	20.00
Main Roof	120.83	1.04	18.39	12.13	3.47	8.67	15.60	19.70
	114.96	1.03	18.13	11.96	3.47	8.50	15.43	19.53
11th	109.08	1.01	17.86	11.78	3.47	8.32	15.25	19.35
	103.21	1.00	17.58	11.60	3.47	8.13	15.06	19.17
10th	97.33	0.98	17.28	11.41	3.47	7.94	14.87	18.97
	91.46	0.96	16.98	11.21	3.47	7.74	14.67	18.77
9th	85.58	0.95	16.66	10.99	3.47	7.53	14.46	18.56
	79.71	0.93	16.33	10.77	3.47	7.31	14.24	18.34
8th	73.83	0.91	15.97	10.54	3.47	7.07	14.01	18.11
	67.96	0.88	15.60	10.29	3.47	6.83	13.76	17.86
7th	62.08	0.86	15.20	10.03	3.47	6.57	13.50	17.60
	56.21	0.84	14.77	9.75	3.47	6.28	13.22	17.32
6th	50.33	0.81	14.32	9.45	3.47	5.98	12.91	17.02
	44.46	0.78	13.82	9.12	3.47	5.65	12.58	16.69
5th	38.58	0.75	13.27	8.76	3.47	5.29	12.22	16.32
	32.17	0.71	12.60	8.31	3.47	4.85	11.78	15.88
4th	25.75	0.67	11.82	7.80	3.47	4.34	11.27	15.37
	20.83	0.63	11.13	7.34	3.47	3.88	10.81	14.91
P6	15.92	0.58	10.30	6.80	3.47	3.33	10.26	14.37
	7.96	0.57	10.05	6.63	3.47	3.16	10.10	14.20
Plaza Level	0.00	0.57	10.05	6.63	3.47	3.16	10.10	14.20
Leeward	120	1.04	18.35	-7.57	3.47	-11.03	-4.10	-

Table 14: East-West Design Pressures							
	Height	**Windward Pressure**	**Leeward Pressure**	**Total Pressure**	**Total Force**	**Story Shear**	**Moment Windward**
	(ft)	(psf)	(psf)	(psf)	(kips)	(kips)	(k-ft)
Penthouse	142.00	12.71	-7.57	20.27	28.97	28.97	4113.36
	131.42	12.43	-7.57	20.00			
Main Roof	120.83	12.13	-7.57	19.70	59.28	88.24	7162.70
	114.96	11.96	-7.57	19.53			
11th	109.08	11.78	-7.57	19.35	47.52	135.77	5184.07
	103.21	11.60	-7.57	19.17			
10th	97.33	11.41	-7.57	18.97	46.57	182.34	4533.05
	91.46	11.21	-7.57	18.77			
9th	85.58	10.99	-7.57	18.56	45.53	227.87	3896.77
	79.71	10.77	-7.57	18.34			
8th	73.83	10.54	-7.57	18.11	44.38	272.25	3276.68
	67.96	10.29	-7.57	17.86			
7th	62.08	10.03	-7.57	17.60	43.08	315.33	2674.59
	56.21	9.75	-7.57	17.32			
6th	50.33	9.45	-7.57	17.02	41.58	356.91	2092.90
	44.46	9.12	-7.57	16.69			
5th	38.58	8.76	-7.57	16.32	41.54	398.46	1602.80
	32.17	8.31	-7.57	15.88			
4th	25.75	7.80	-7.57	15.37	36.11	434.56	929.74
	20.83	7.34	-7.57	14.91			
P6	15.92	6.80	-7.57	14.37	38.56	473.13	613.81
	7.96	6.63	-7.57	14.20			
Plaza Level	0.00	6.63	-7.57	14.20	23.73	**496.85**	0.00
							36080.47

Base Shear	**496.85 Kips**
Overturning Moment	**36080.47 Kip-ft**

Wind: North-South Direction

Table 15: North-South Design Factors	
Exposure B	
Case 2	
L	210 ft
B	120 ft
L/B	1.75
Natural Period (approx.) (n_1)	0.833
Damping Coeff. (approx.) (β)	0.02
Basic Wind Speed (V)	90 mph
Wind Directionality Factor (K_d)	0.85
Importance Factor (I)	1.0
Exposure Category	B
Topographical Factor (K_{zt})	1.0
Gust Effect Factor (G)	0.845
C_p Windward	0.8
C_p Leeward	-0.5
G_{cpi} Windward	0.18
G_{cpi} Leeward	-0.18
G_{pn} Windward	1.5
G_{pn} Leeward	-1.0

Table 16: North-South Calculation of Design Pressures								
	Height	**K_z, K_h**	**q_z, q_h**	**External Pressure**	**Internal Pressure**	**Net Positive**	**Net Negative**	**Total Pressure**
	(ft)			(psf)	(psf)	(psf)	(psf)	(psf)
Penthouse	142.00	1.09	19.25	13.02	3.47	9.56	16.49	18.46
	131.42	1.07	18.83	12.74	3.47	9.27	16.20	18.18
Main Roof	120.83	1.04	18.39	12.44	3.47	8.97	15.90	17.88
	114.96	1.03	18.13	12.26	3.47	8.79	15.73	17.70
11th	109.08	1.01	17.86	12.08	3.47	8.61	15.54	17.52
	103.21	1.00	17.58	11.89	3.47	8.42	15.35	17.33
10th	97.33	0.98	17.28	11.69	3.47	8.23	15.16	17.13
	91.46	0.96	16.98	11.48	3.47	8.02	14.95	16.93
9th	85.58	0.95	16.66	11.27	3.47	7.80	14.73	16.71
	79.71	0.93	16.33	11.04	3.47	7.58	14.51	16.48
8th	73.83	0.91	15.97	10.80	3.47	7.34	14.27	16.24
	67.96	0.88	15.60	10.55	3.47	7.08	14.02	15.99
7th	62.08	0.86	15.20	10.28	3.47	6.82	13.75	15.72
	56.21	0.84	14.77	9.99	3.47	6.53	13.46	15.43
6th	50.33	0.81	14.32	9.68	3.47	6.22	13.15	15.12
	44.46	0.78	13.82	9.35	3.47	5.88	12.81	14.79
5th	38.58	0.75	13.27	8.97	3.47	5.51	12.44	14.42
	32.17	0.71	12.60	8.52	3.47	5.05	11.99	13.96
4th	25.75	0.67	11.82	8.00	3.47	4.53	11.46	13.44
	20.83	0.63	11.13	7.53	3.47	4.06	10.99	12.97
P6	15.92	0.58	10.30	6.97	3.47	3.50	10.43	12.41
	7.96	0.57	10.05	6.80	3.47	3.33	10.26	12.24
Plaza Level	0.00	0.57	10.05	6.80	3.47	3.33	10.26	12.24
Leeward	120	1.04	18.39	-5.44	3.47	-8.91	-1.98	-

Table 17: North-South Design Pressures							
	Height	**Windward Pressure**	**Leeward Pressure**	**Total Pressure**	**Total Force**	**Story Shear**	**Moment Windward**
	(ft)	(psf)	(psf)	(psf)	(kips)	(kips)	(kip-ft)
Penthouse	142.00	13.02	-5.44	18.46	10.16	10.16	1442.90
	131.42	12.74	-5.44	18.18			
Main Roof	120.83	12.44	-5.44	17.88	28.11	38.27	3396.78
	114.96	12.26	-5.44	17.70			
11th	109.08	12.08	-5.44	17.52	24.57	62.84	2679.92
	103.21	11.89	-5.44	17.33			
10th	97.33	11.69	-5.44	17.13	24.01	86.85	2337.01
	91.46	11.48	-5.44	16.93			
9th	85.58	11.27	-5.44	16.71	23.40	110.25	2002.73
	79.71	11.04	-5.44	16.48			
8th	73.83	10.80	-5.44	16.24	22.73	132.98	1677.93
	67.96	10.55	-5.44	15.99			
7th	62.08	10.28	-5.44	15.72	21.97	154.94	1363.68
	56.21	9.99	-5.44	15.43			
6th	50.33	9.68	-5.44	15.12	21.09	176.03	1061.38
	44.46	9.35	-5.44	14.79			
5th	38.58	8.97	-5.44	14.42	20.91	196.94	806.90
	32.17	8.52	-5.44	13.96			
4th	25.75	8.00	-5.44	13.44	18.00	214.94	463.41
	20.83	7.53	-5.44	12.97			
P6	15.92	6.97	-5.44	12.41	19.01	233.95	302.53
	7.96	6.80	-5.44	12.24			
Plaza Level	0.00	6.80	-5.44	12.24	11.69	**245.63**	0.00
							17535.19

Base Shear	**245.63 Kips**
Overturning Moment	**17535.19 Kip-ft**

Appendix B

Seismic

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Calculation for Seismic Analysis

Not detached 1 or 2 Family Dwelling
Not Agricultural Storage
Not Special Considerations
} ∴ Not Exempt [Sect 11.1.2]

Seismic Ground Motion Values

$S_S = 0.156\ g$ [Fig 22-1]

$S_1 = 0.051\ g$ [Fig 22-4]

$S_1 > 0.04$ & $S_S > 0.15$ [Sect. 11.4.1]

Determine Soil site Class → C

$S_{MS} = F_a S_S = (1.2)(0.156)$ [Eq. 11.4-1]

$S_{M1} = F_v S_1 = (1.7)(0.051)$ [Eq. 11.4-2]

$S_{DS} = 2/3\ S_{MS} = 0.1248$ [Eq. 11.4-3]

$S_{D1} = 2/3\ S_{D1} = 0.0578$ [Eq. 11.4-4]

Seismic Design Category

$S_{DS} < 0.167$ → A [TABLE 11.6-1]

$S_{D1} < 0.067$ → A [Table 11.6-2]

Determine occupancy Category → II

∴ Importance Factor = 1.0 [Table 1-1]

- Section 11.6 requirements for simplified design

- I, II or III → Yes
- $S_1 < 0.75$ → Yes
- $h < 40'$ → No

∴ Simplified does not apply

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Permitted Analytical Procedures → SDC B [Table 12.6-1]

- Equivalent Lateral Force Analysis
- Modal Response Spectrum Analysis
- Seismic Response History Procedures

Use Equivalent Lateral Force Analysis

Determine Response Modification Factor

F. - Shearwall Frame Interactive System with ordinary reinforced concrete moment frames & ordinary rein. concrete shear walls [Table 12.2-1]

$R = 4.5$

Determine Approx. Fundamental Period

$T_a = C_t h_n^x = 0.02(142)^{0.75}$ [Eq. 12.8-7]

$T_a = 0.8227$ sec

$T_L = 8$ sec $> T_a$ [Fig 22-15]

$C_s = S_{DS}/R/I = \frac{0.1248}{4.5/1.0} = 0.02773$ [Eq 12.8-2]

not to exceed $C_s = S_{D1}/T(R/I) = \frac{0.0578}{0.82(4.5/1)} = 0.0156$ [Eq 12.8-3]

must be greater than 0.01 , $S_1 = 0.051 < 0.6$ [Eq 12.8-5]

$\therefore C_s = 0.0156$

$K = 1.161$ (by interpolation) [Sect 12.8-3]

Determine Story Force

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$, $V = C_s W$ [Eq 12.8-12] [Eq 12.8-1]

$F_x = C_{vx} V$ [Eq. 12.8-11]

see figures and tables provided for building weights and force calculations

Level Self Weight

Table 18: Penthouse Weight	
Item	**Design Weight (kips)**
Beams	77.9
Slab	390
Roofing	156
SDL	39
Equipment	120
Façade	103.5
Total	886.4

Table 19: Main Roof Weight	
Item	**Design Weight (kips)**
Beams	557.5
Slab	2269.1
Columns	150.4
Roofing	728.1
Shear Wall	196
Equipment	52.8
SDL	221
Façade	167.6
Total	886.4

Table 20: Office (11th) Weight	
Item	**Design Weight (kips)**
Beams	557.3
Slab	2269.1
Columns	391.4
Shear Wall	12.6
Partitions	194.6
Equipment	23.7
SDL	110.5
Façade	223.5
Total	3896.1

Table 21: Office (Typ.) Weight	
Item	**Design Weight (kips)**
Beams	538.4
Slab	2364.7
Columns	399.6
Shear Wall	12.6
Partitions	204.2
Equipment	23.7
SDL	115.3
Façade	223.5
Total	3995.4

Table 22: P6 Level Weight	
Item	**Design Weight (kips)**
Beams	483.6
Slab	2548.2
Columns	322.0
Drops	158.0
Shear Wall	12.6
Equipment	2.2
SDL	124.5
Façade	300.0
Total	4064.4

Seismic Calculations

Table 23: Seismic Calculations							
Level	**Story Weight**	**Height**	**$w_x h_x^k$**	**C_{vx}**	**Forces (F_x)**	**Story Shear (V_x)**	**Moments (M_x)**
	(kips)	(ft)			(kips)	(kips)	(k-ft)
Pent Roof	887	142.00	280216.3	0.05	30.8	30.8	4375.638
Main Roof	4342	120.83	1137226.0	0.19	125.1	155.9	15111
11th Floor	3897	109.08	906338.2	0.16	99.7	255.5	10871.97
10th Floor	3996	97.33	814145.5	0.14	89.5	345.1	8714.116
9th Floor	3996	85.58	701155.6	0.12	77.1	422.2	6598.774
8th Floor	3996	73.83	590648.2	0.10	65.0	487.1	4795.579
7th Floor	3996	62.08	482953.3	0.08	53.1	540.2	3297.158
6th Floor	3996	50.33	378515.1	0.06	41.6	581.9	2095.07
5th Floor	3996	38.58	277970.0	0.05	30.6	612.4	1179.39
4th Floor	3996	25.75	173795.3	0.03	19.1	631.5	492.1244
P6	4065	15.92	101120.0	0.02	11.1	642.7	176.99
Plaza Level	-	0.00	-	-	-	-	-
Total	41163	-	5844083.56	1.00	642.7	-	57707.81

Table 24: Design Values	
Effective Seismic Weight	41163 kips
Base Shear	642.7 kips
Overturning Moment	57708 kips-ft

Appendix C

Building Plans and Elevations

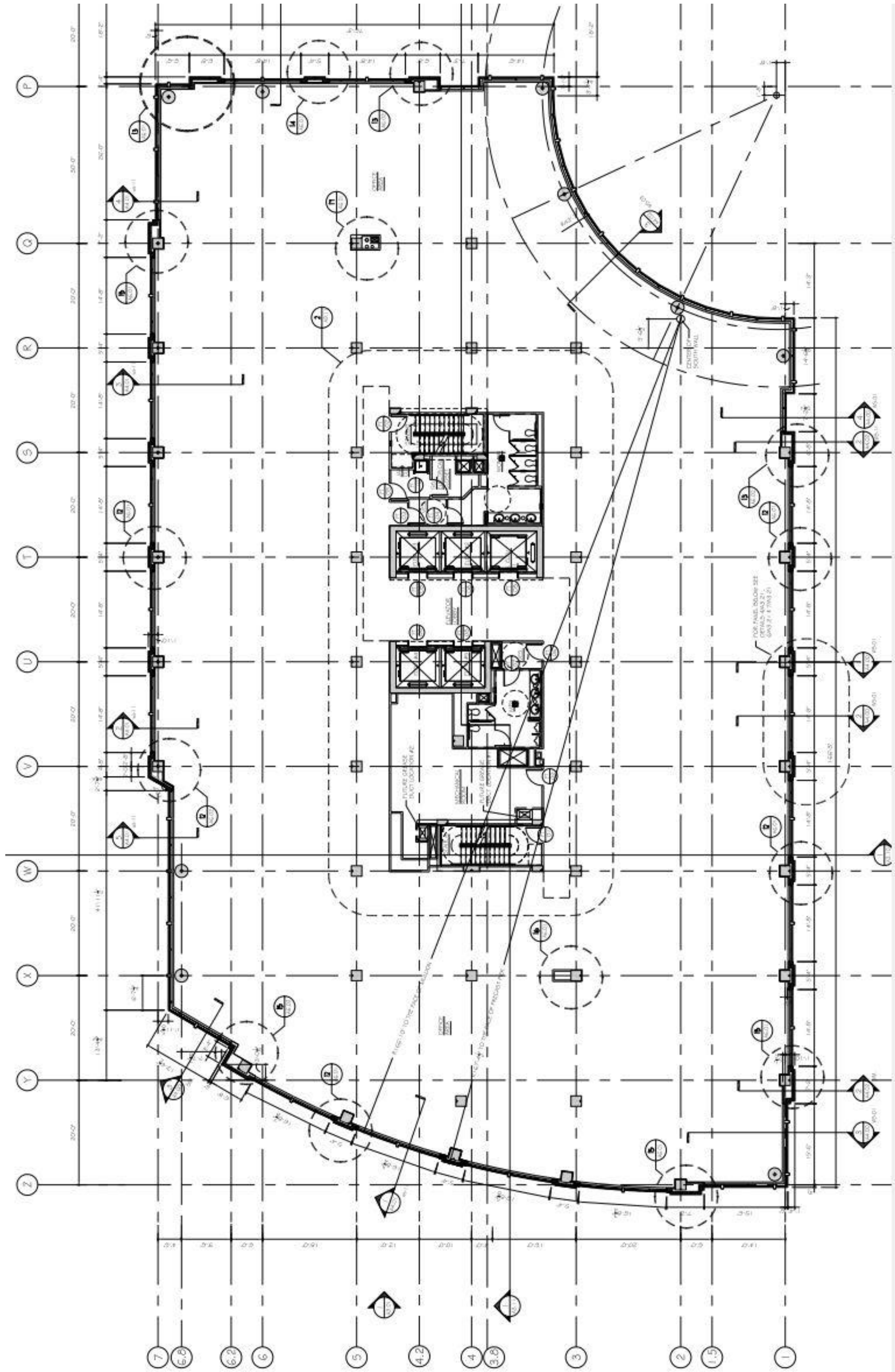

Figure 19: Typical Office Floor Plan – A2.19 of Construction Documents

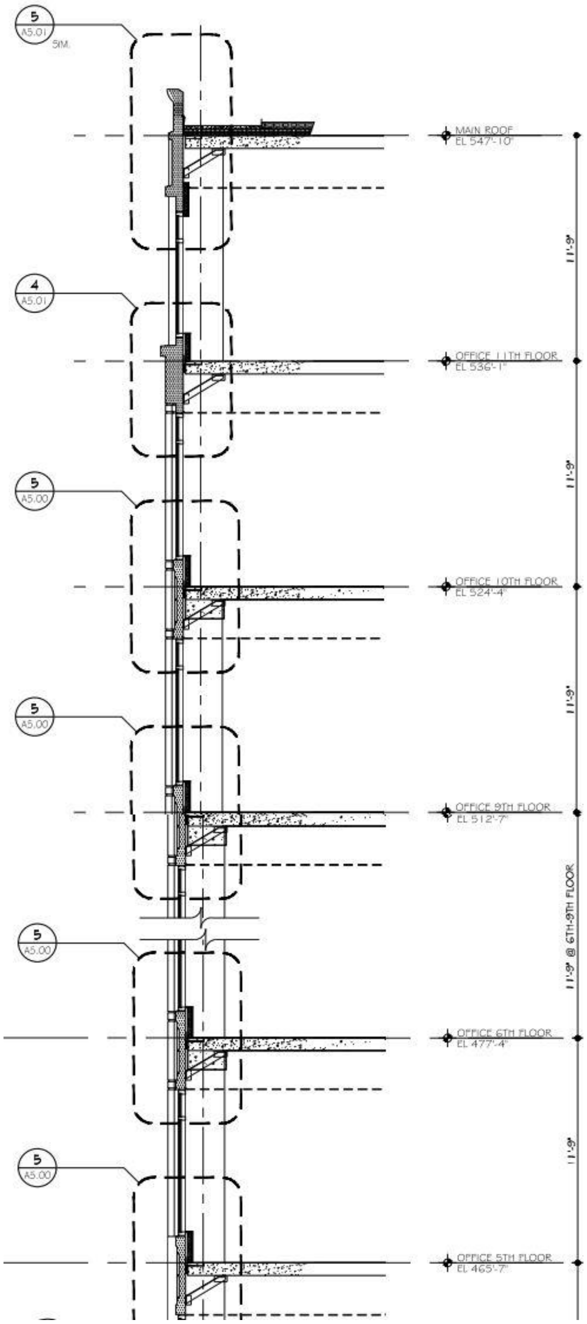

Figure 20: Wall Section – A4.05 of Construction Documents

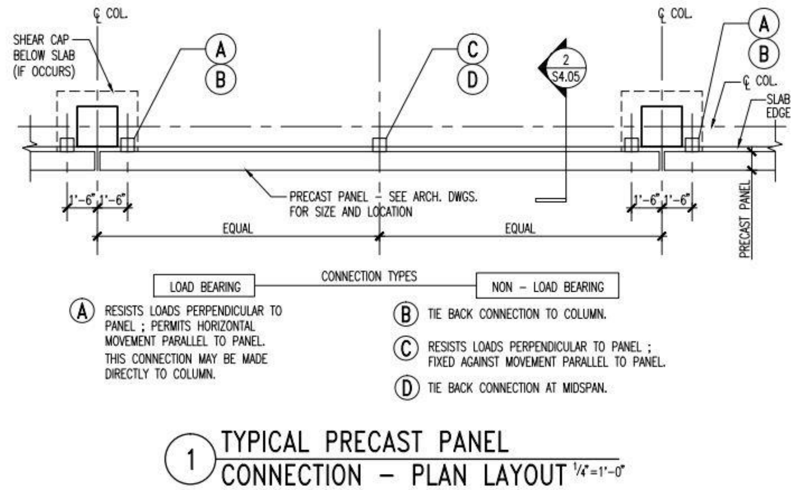

Figure 21: Precast Connection Plan – S4.01 of CD's

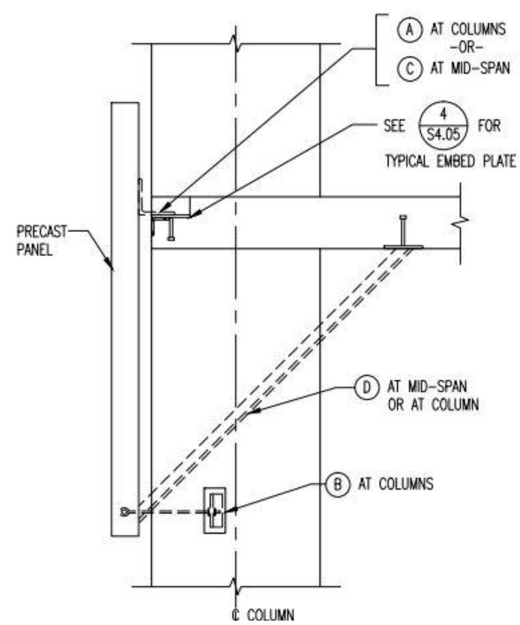

Figure 22: Precast Connection Detail – S4.01 of CD's

Appendix D

Photos

Figure 23: Decorative Precast Panel – by JMV

Figure 24: North East Curtain Wall – by JMV

Figure 25: Unfinished Retail Space – by JMV

Figure 26: South West Corner – by JMV

Figure 27: Projection of Post Tension Beam – by JMV