Rockville Metro Plaza II

121 Rockville Pike
Rockville, Maryland

Technical Report IV

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Introduction

The following report evaluates the lateral loads present on Rockville Metro Plaza II. Lateral loads, including wind and seismic are calculated using the approaches outlined in the 2005 version of the American Society of Civil Engineer's prevision entitled Minimum Design Loads for Buildings and Other Structures.

Computer software was implemented in order to distribute the lateral loads to members of the lateral force resisting system. For this investigation, Etabs and SAP2000 were employed. Approaches used to model the lateral system are defined in this document, as well as results.

Using Sap2000, a full 2-D analysis of the building's lateral systems was completed. This approach utilized many simplifications in regards to the building's geometry in order to make the process more efficient. Through this approach, it was found that wind controlled the majority of the design.

Using Etabs, a 3-D model was created. Lateral forces were placed on the modeled structure and it was confirmed that wind mainly controls the design of the lateral system. Results of the Etabs analysis are included in this report as well as an interpretation of the results. This information displays the loads to which the elements of the building's lateral system must be designed to withstand.

Spot checks of certain elements were performed in order to verify the design of the structure. It was concluded that the structure is well designed to withstand the lateral loads to which it will be subjected.

Architectural Rendering of RMP II

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 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.

To respond to the potential for lateral loads on the structure such as seismic and wind, concrete shear walls were incorporated into the structural design. These walls were placed near 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.

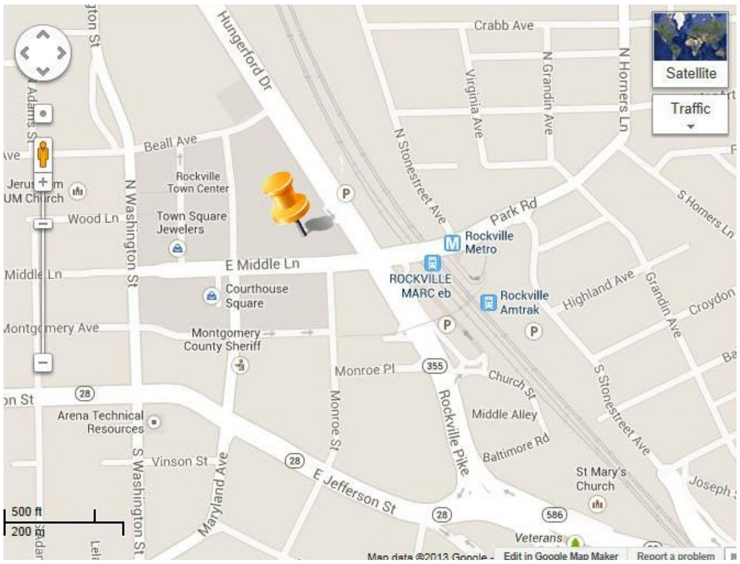

Figure 2: Map of Site Location – From "maps.google.com"

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.

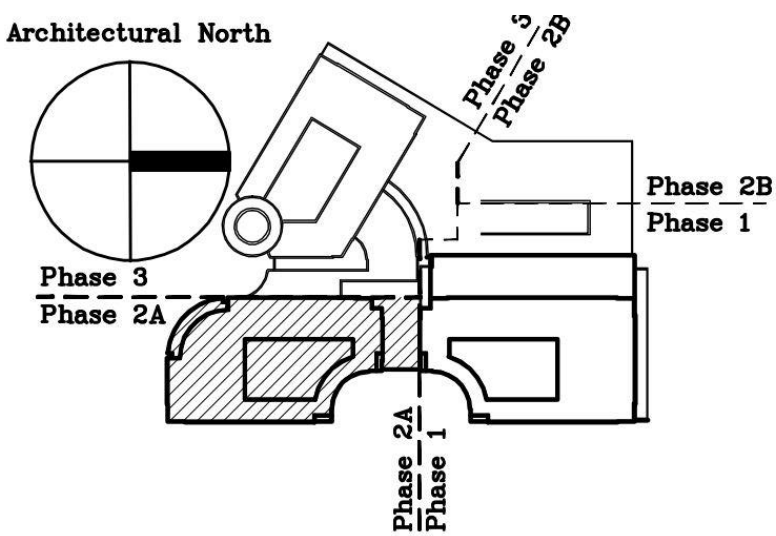

Figure 3: Map of Building Relations – by WDG Arch.

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.

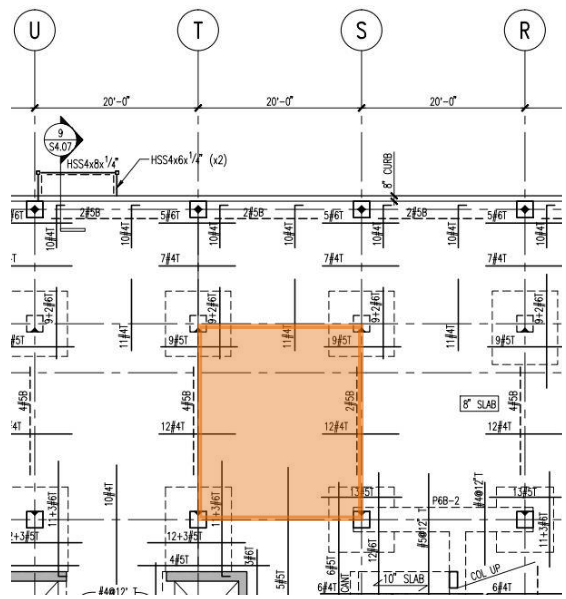

Figure 5: Plan of Garage Bay – by Cagley and Assoc.

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.

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.

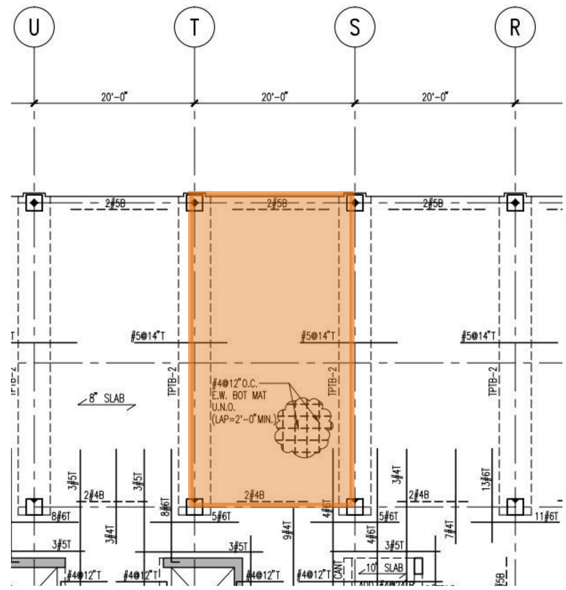

Figure 6: Plan of Office Bay – by Cagley and Assoc.

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

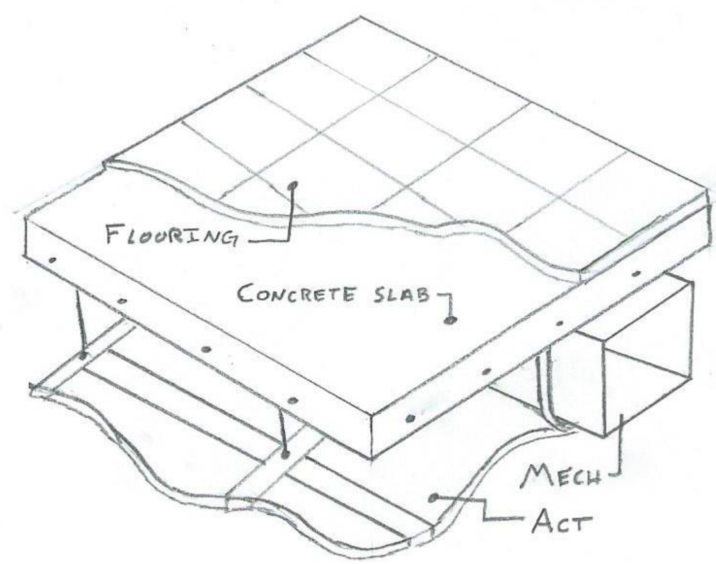

Figure 7: Cut Away of Typical Floor Slab – by JMV

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

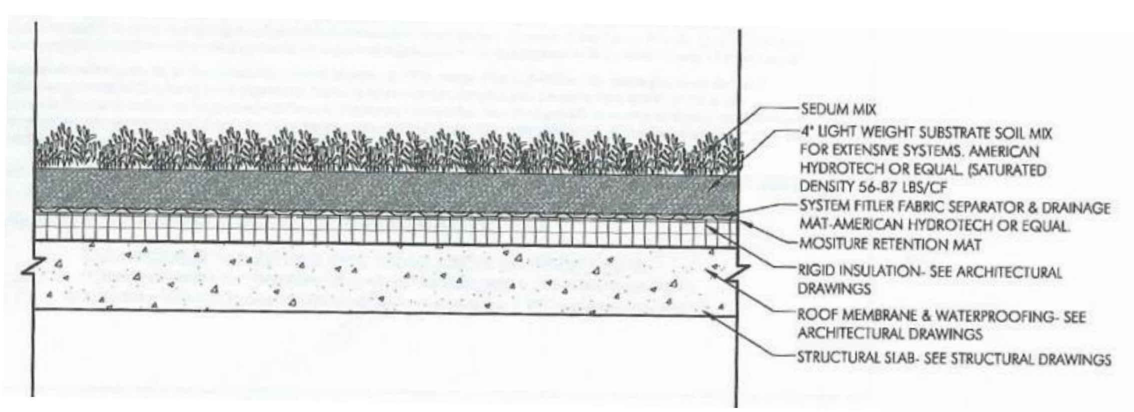

Figure 8: Green Roof Cross Section – by Studio 39

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.

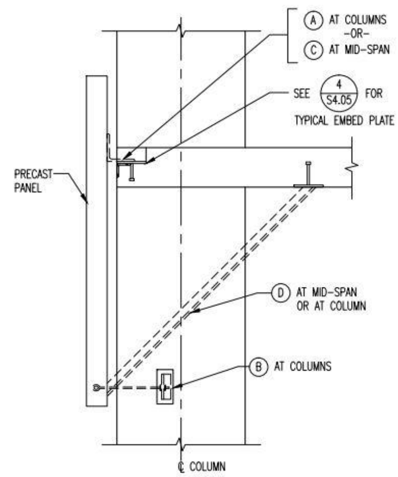

Figure 9: Precast Elevation Detail - by Cagley and Assoc.

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.

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.

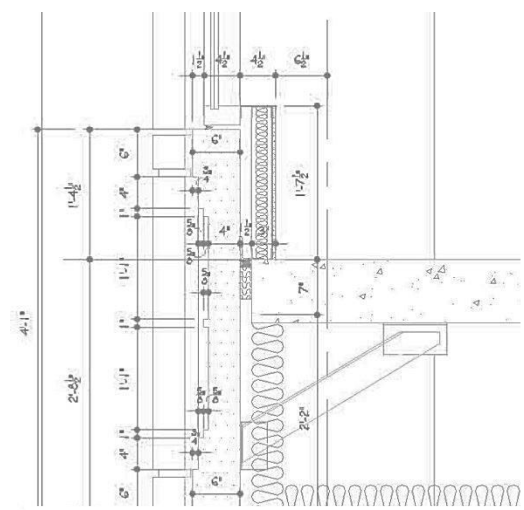

Figure 10: Wall Elevation Section - by Cagley and Assoc.

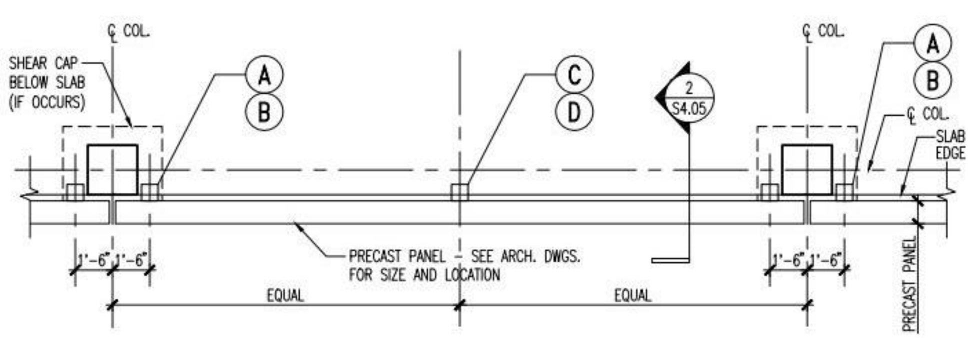

Figure 11: Precast Plan Detail – by Cagley and Assoc.

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**

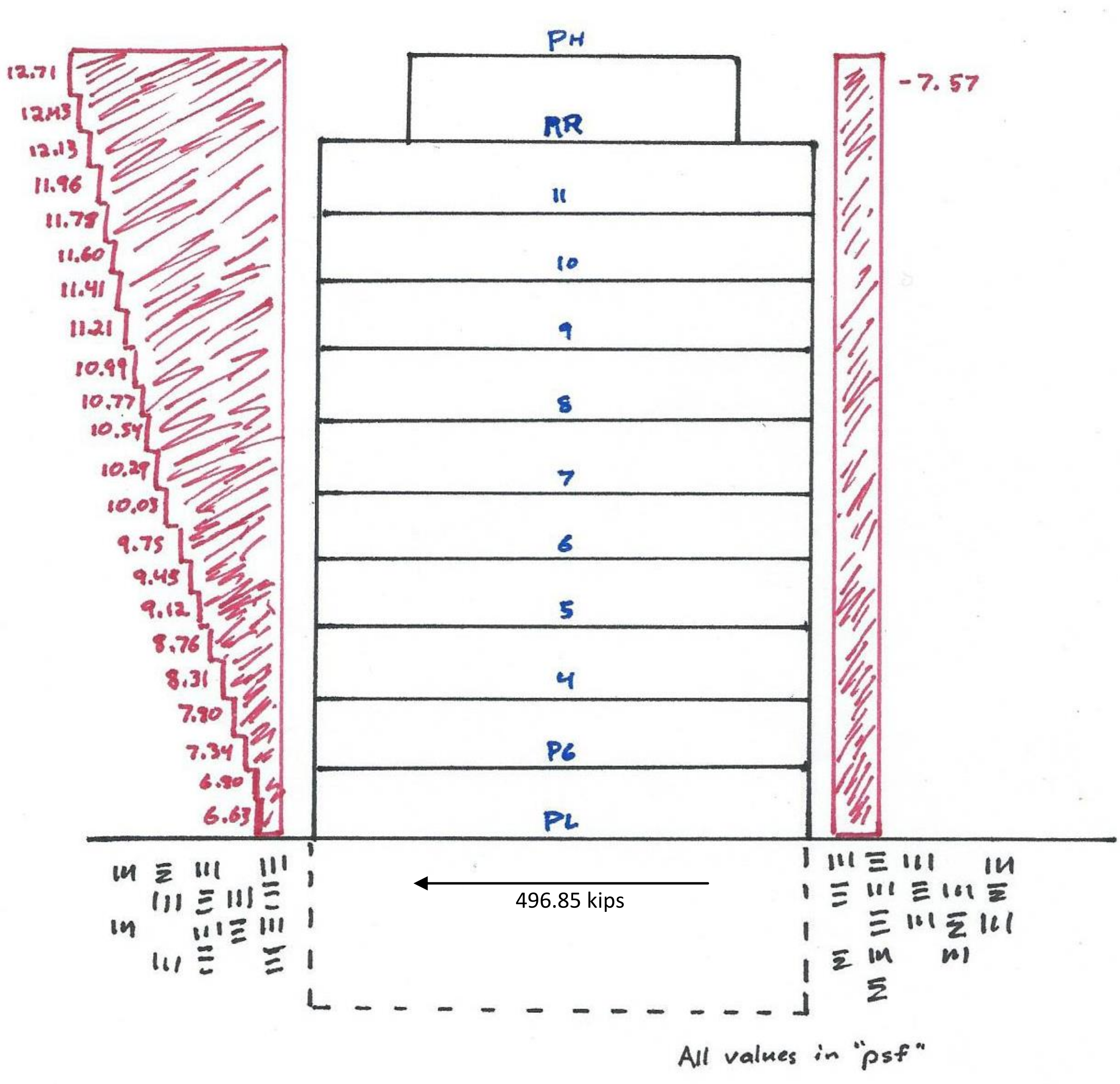

Figure 13: East-West Design Pressure Diagram

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**

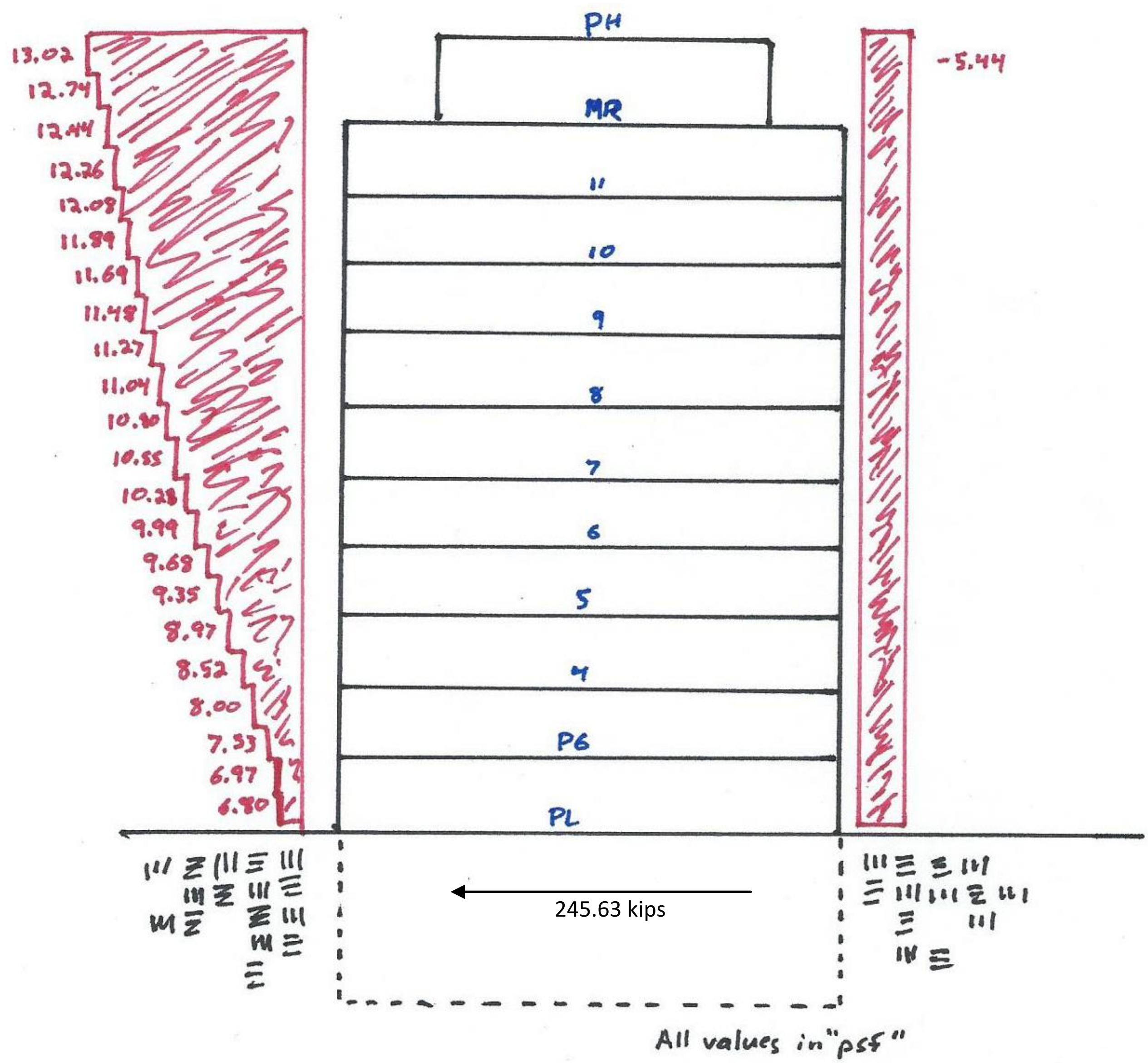

Figure 14: North-South Design Pressure Diagram

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

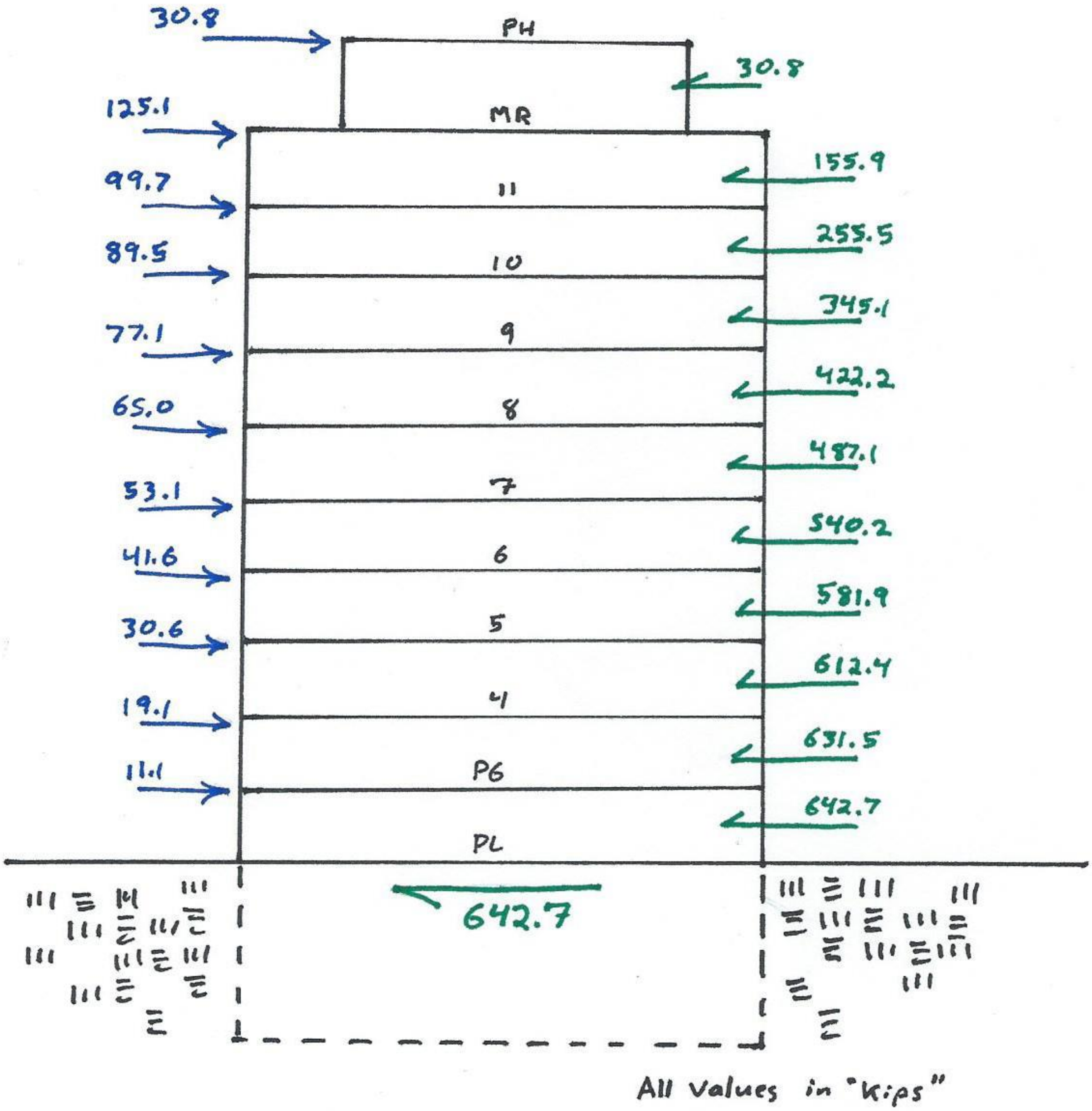

Figure 17: Diagram of Design Values

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.

Load Path

Within Rockville Metro Plaza II, concrete shear walls and concrete moment frames work together to resist the lateral loads on the structure.

In the North-South direction, the four shear walls that participate in resisting lateral force are the 12" thick returns of the elevator core. The concrete moment frames that contribute in this direction are comprised of columns and the 8" thick one way slab.

In the East-West direction, the two 12" thick shear walls that form the back of the elevator core participate in resisting lateral force. The concrete moment frames that contribute in this direction are comprised of columns and the 48" wide post tensioned beams.

Each direction of the structure acts similarly, in that lateral forces are applied to the floor diaphragm of the structure which in turn transfers the load to the concrete moment frames and concrete shear walls. These elements transfer the lateral load down to the foundation via shear and axial forces. At the foundations, shallow footings spread the load to the soil below.

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

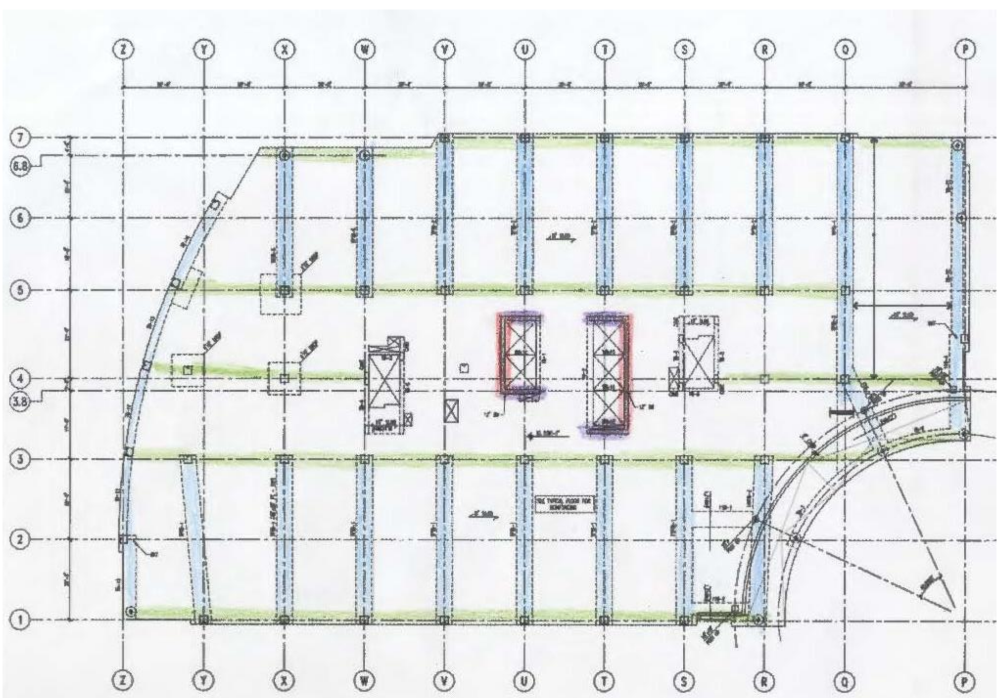

Figure 18: Plan Identification of Lateral System

Load Cases

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). Due to the relatively large mass of the concrete structure, the amplification of dead load in this combination aids in reducing the overturning moment produced by wind.

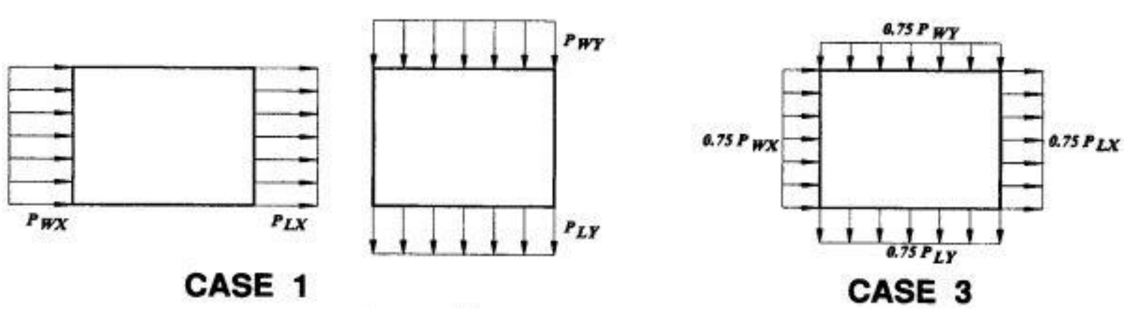

Figure 19: Select ASCE 7-05 Design Wind Load Cases

SAP2000 Model

A computer model of Rockville Metro Plaza II's lateral force resisting elements was created in order to assess how the structure responds to lateral forces. Computers & Structures, Incorporated's structural analysis program, SAP2000, was employed to asses these loads. When only the lateral forces are in consideration, it is typical to model only the elements that contribute to the lateral force resistance system. The structure's lateral force resisting system includes concrete moment frames and shear walls. Thus there is full building participation in the resistance of lateral forces which required the full building to be considered. Figure 20 displays a portion of the elements considered.

In modeling the lateral systems of the structure with this program, several assumptions were made, geometries were idealized, lateral elements were kept to a minimal bank, concrete gradations were kept consistent by level, and rigid diagrams were employed. The model only considers the structure to the point of the seismic base. The effective flange of members incorporating the slab was calculated as per ACI-05. Appropriate modifiers as per ACI-05 were used to adjust the moment of inertia of concrete elements in order to account for cracking. The calculations process is defined on the following page.

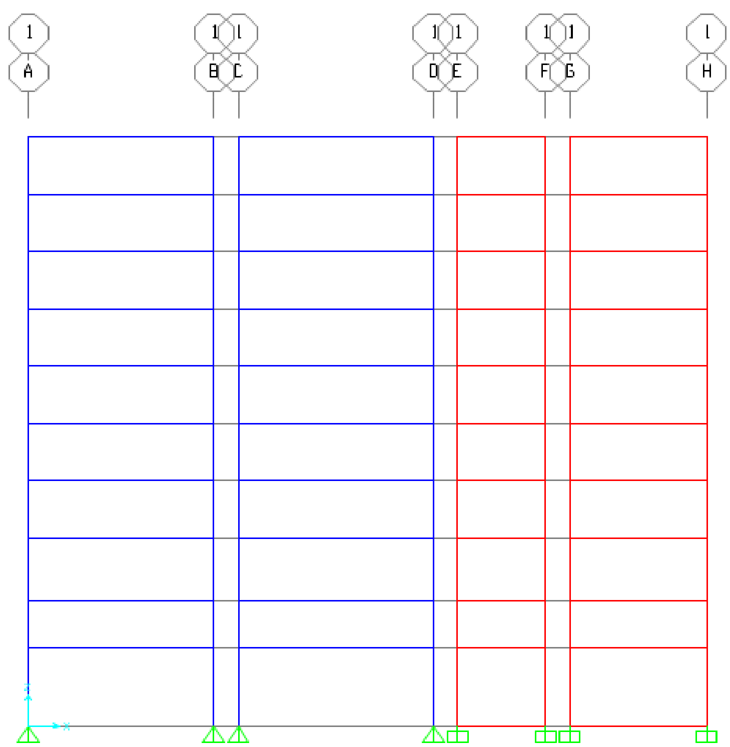

Figure 20: Elevation of E-W LFRS in SAP2000

Lateral Force Distribution in SAP2000

Once the building's geometry was defined, the lateral force resisting elements could be identified. The geometry of the structure was idealized as shown in Figure 21. Typical lateral force resisting elements were assigned and these elements were then modeled in SAP2000 (as shown in Figure 20). In order to find the stiffness of each element, a dummy load of 10 kips was applied as a point load to the top of individual elements. Displacements and drifts were recovered directly from the model, and using the relationship of P=kΔ, the stiffness of each element was found.

Next, the building's plan geometries along with the calculated stiffness values were entered into a spreadsheet per level (see Appendix C for sample spreadsheets). The geometric properties of center of rigidity, center of mass, and torsional moment of inertia were in turn calculated. Force distribution to each element was achieved using the paradigm of load follows stiffness. In other words, the stiffness of one element was divided by the sum of stiffness values on a given level in order to achieve the percent of load that said element will be required to carry. Using this data, direct force, torsional force, and total force were calculated for each element per level. The calculation procedure of the process may be found in Appendix C.

The maximum load for each element was found by subjecting it to the load cases previously described. The maximum load was then compared to the nominal capacity of the element in order to verify the suitability of the design. Design checks of elements considered critical may be found in Appendix D.

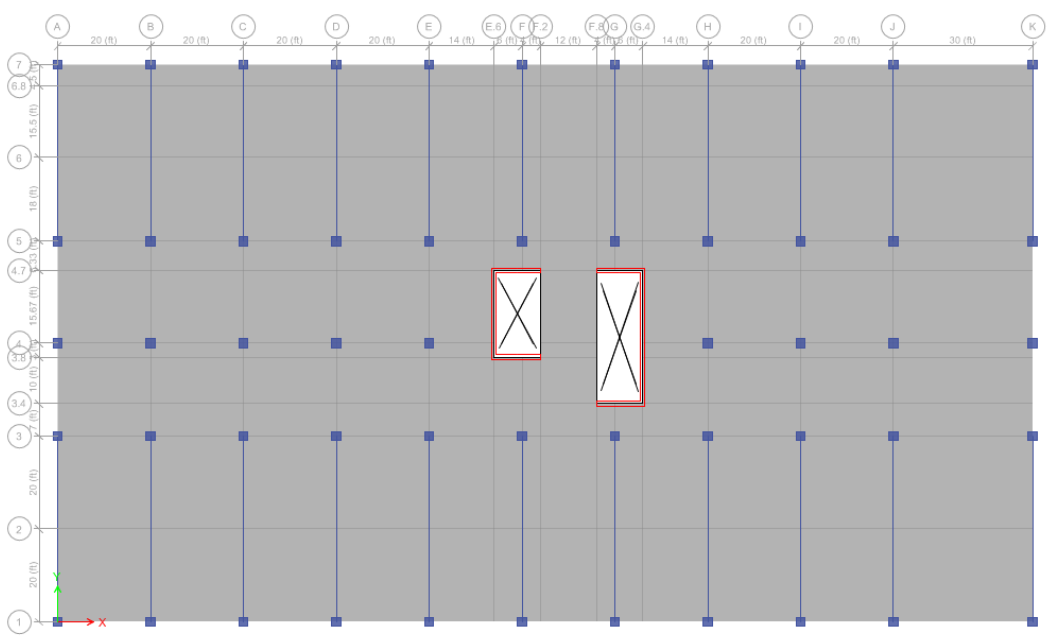

Figure 21: Plan of 2-D Model's Geometries

Etabs Model

A computer model of Rockville Metro Plaza II was created in order to assess how the structure acts under lateral forces. Computers & Structures, Incorporated's structural analysis program, Etabs, was employed to asses these loads. When only the lateral forces are in consideration, it is typical to model only the elements that contribute to the lateral force resistance system. The structure's lateral force resisting system includes concrete moment frames and shear walls. Thus there is full building participation in the resistance of lateral forces which in turn required nearly all building elements to be modeled.

In modeling the structure, several simplifications were made in order to streamline the modeling as well as ease the interpretation of results: Curved geometries of the structure were idealized and squared off (see Figure 22), frame elements were kept to a limited bank of options, levels below the seismic base were discounted, concrete moment frames were kept consistent, and concrete strengths were graded in a uniform fashion be elevation. These amendments will have only minor impacts on the overall results of the structure's performance. The model consists mainly of concrete shear walls, beams, columns, and slabs. Appropriate modifiers as per ACI-05 were used to adjust the moment of inertia of concrete elements in order to account for cracking. All load combinations were entered manually into the model and the most critical was used in calculations included in this report.

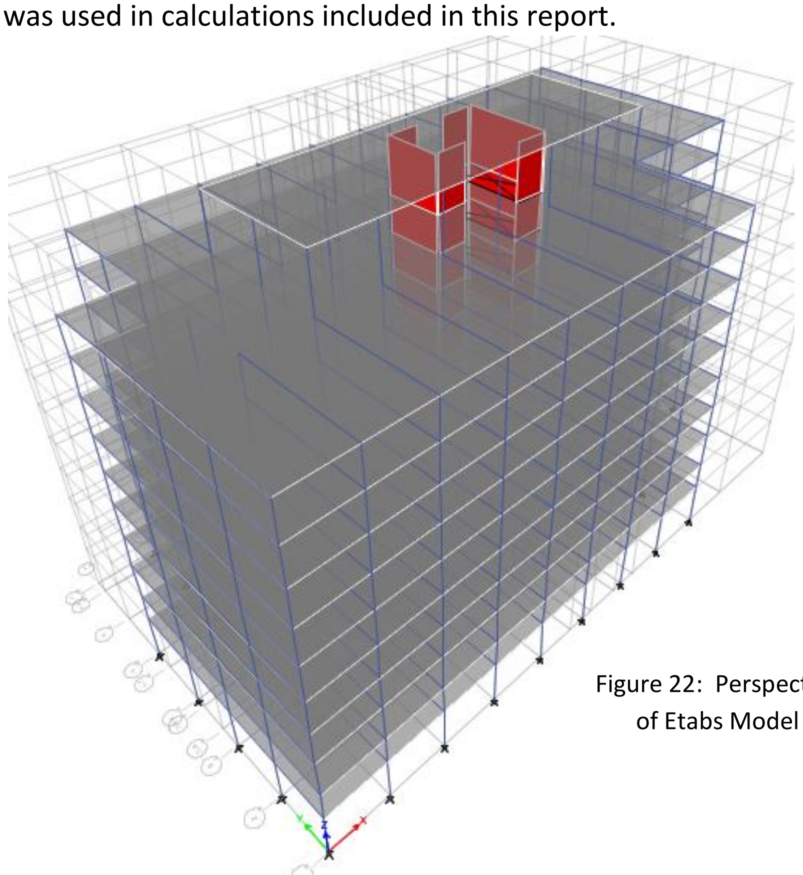

Figure 22: Perspective of Etabs Model

Lateral Force Distribution in Etabs

Within the 3D model, lateral loads are distributed based on elemental stiffness via a rigid floor diaphragm assumption. This assumption essentially glues all point of an elevation together, allowing then to move a one solid unit rather than as individual points. This assumption thus adopts the ideology that the forces will be distributed to lateral force resisting element via relative stiffness values rather than by tributary areas.

Once the building's geometry was defined in the 3D model, a 1000 kip dummy load was applied to the top of the building. The shear forces were then determined in each element per level. The distribution was confirmed by summing the shear forces on each level, which equate to 1000 kips (the applied load). From these forces, the relative stiffness values of the elements were determined by once again dividing the shear force in an element by the sum of shear forces on that level. This basic calculation was employed in each direction of the structure and the results may be found in Appendix C.

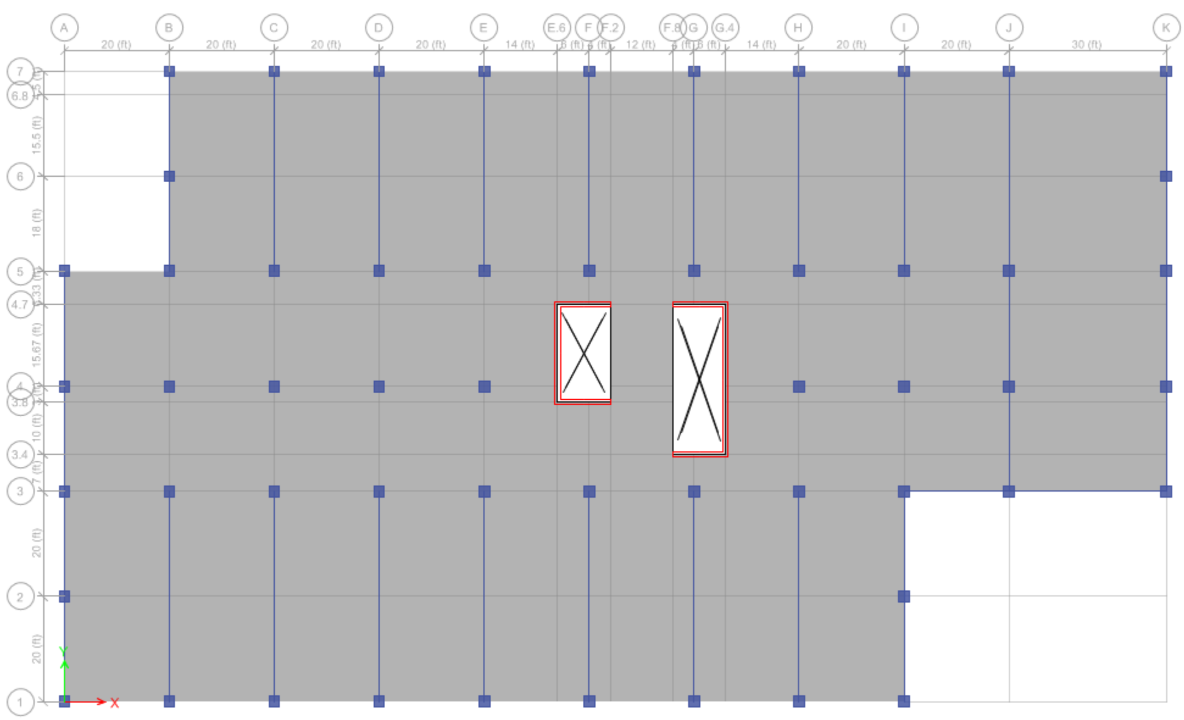

Figure 23: Plan of 3-D Model's Geometries

Comparison of Models

In comparing the two types of models, I found that the 3-D model assessed the structure to be a stiffer building than was found in via the SAP2000 results. This is evident upon comparing the wind drift data for the two models (see Appendix A). The 2-D analysis provided higher wind drift than did the 3-D version.

The 2-D model incorporates more assumptions regarding the geometry of the structure. This surely leads to some differences in the stiffness values of specific elements which in turn would affect the final results.

In order to maintain a simple model, garage levels below the structure were not modeled as these levels would neither directly see the wind nor seismic force. Therefore the support reactions on each model are not entirely accurate. The reactions would be somewhere between a pin type and a fixed connection. In order to see the results of this sort of modification, the fixed vs. pinned results of the Etabs model may be compared (see results in Appendix C).

The models display two different methods of analyzing the same structure. Overall, the results between the two correspond to one another. Through assessing general trends as to how each distributes applied loads, this fact may be seen. Still, there are some significant differences that require more investigation. For instance, in the Etabs model, the center of rigidity changes only a few feet whereas the 2-D analysis found this property to shift nearly nine feet from base to roof.

There were also numerous modeling issues that occurred while using Etabs. Modeling assumptions played a significant role in the results that ensued from this prototype. For example, the program's assumption of modeling the core shear walls as a c-channel when it was intended to act as three separate walls.

To conclude, the models confirm one another on most levels. However, caution must be taken when selecting which model's results are more reliable. Therefore, this report will mainly use data collected from the 2-D analysis.

Wind Drift

In order to obtain the building's story drift values that are incurred due to wind, wind loads were applied to the 2-D models. For this calculation, critical locations were selected and assessed (i.e. locations that are farthest from the center of rigidity as they will 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:

$$\Delta_{MAX} = (120.83' \times 12''/1') / 400 = 3.62''$$

After analyzing the loads in the 2-D model for unfactored (serviceability) wind forces, the following results were obtained:

Table 12: Wind Drifts (N-S)		
Level	**Story Drift (in)**	**Total Drift (in)**
Roof	0.0937	2.7954
11th	0.1321	2.7017
10th	0.1721	2.5696
9th	0.2078	2.3975
8th	0.2427	2.1897
7th	0.2758	1.947
6th	0.3076	1.6712
5th	0.3695	1.3636
4th	0.3052	0.9941
P6	0.6889	0.6889

Table 13: Wind Drifts (E-W)		
Level	**Story Drift (in)**	**Total Drift (in)**
Roof	0.1327	2.5448
11th	0.1613	2.4121
10th	0.1953	2.2508
9th	0.2269	2.0555
8th	0.2540	1.8286
7th	0.2756	1.5746
6th	0.2900	1.299
5th	0.3242	1.009
4th	0.2493	0.6848
P6	0.4355	0.4355

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 11.75 x 12 / 400 = 0.3523"), however in the North South direction the two lowest levels exceed the allowable (see tables of Appendix A). This is most likely due to the modeling assumption that the bases are pinned, which drastically reduces the stiffness of the frame element on these lower levels. In reality, the support would have some fraction of moment restraint which would increase the member stiffness at this level, and lessen the drift value.

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 2-D models. For this calculation, critical locations were selected and assessed (i.e. locations that are farthest from the center of rigidity as they will 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:

$$\Delta_{MAX} = (120.83' \times 12''/1') \times 0.02 = 29''$$

After analyzing the loads in the 2-D model for factored (strength) seismic forces, the following results were obtained:

Table 14: Seismic Drifts (N-S)

Level	Story Drift (in)	Total Drift (in)
Roof	2.3459	18.5504
11th	2.3643	16.2045
10th	2.3535	13.8402
9th	2.2995	11.4867
8th	2.1969	9.1872
7th	2.0309	6.9903
6th	1.7892	4.9595
5th	1.5845	3.1703
4th	0.8735	1.5858
P6	0.7124	0.7124

Table 15: Seismic Drifts (E-W)

Level	Story Drift (in)	Total Drift (in)
Roof	1.3050	21.6938
11th	1.5710	20.3888
10th	1.8643	18.8178
9th	2.1080	16.9535
8th	2.2863	14.8455
7th	2.3905	12.5593
6th	2.4163	10.1688
5th	2.5895	7.7525
4th	1.9180	5.1630
P6	3.2450	3.2450

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 4.5 for shear wall frame interactive system with ordinary reinforced concrete moment frames and reinforced concrete walls. 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 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, they 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. Wind load case 2 was found to control the design of most elements within Rockville Moto Plaza II. Therefore, torsion does play a significant role in the design of this structure.

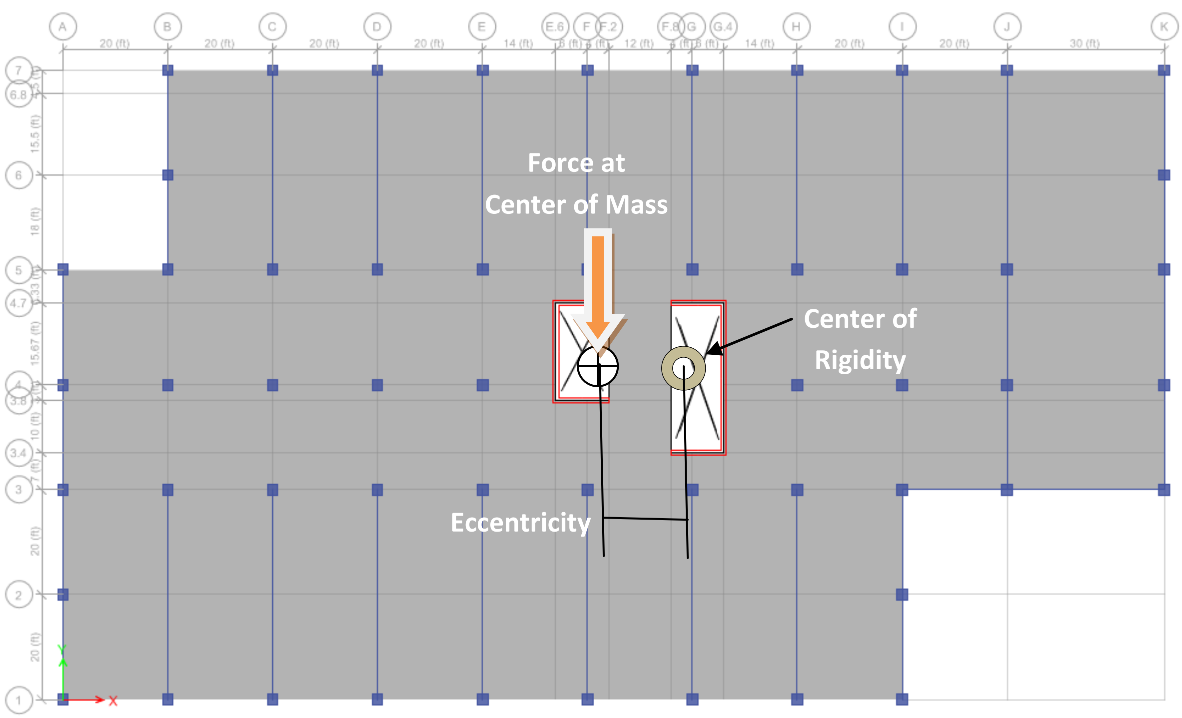

Figure 24: Depiction of Torsion Source

Overturning

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 concrete is not a suitable material for compressive forces. It should also be ensure that nominal compressive loads are not exceeded. It is also possible that moment is 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 with a (factored) moment of 61,676 kip-ft (1.6 x 38,547). This is less than the (factored) moment due to the building weight 3,889,903 kip-ft in the N-S direction and 2,222,802 kip-ft in the E-W direction. See Appendix D for further calculations.

Table 16: Seismic Overturning Moment			
Level	**Height (ft)**	**Story Force (k)**	**Overturning Moment (k-ft)**
Pent	142.00	30.81	4375.6
Roof	120.83	125.06	15110.6
11th	109.08	99.67	10871.6
10th	97.33	89.53	8713.8
9th	85.58	77.10	6598.5
8th	73.83	64.95	4795.4
7th	62.08	53.11	3297.0
6th	50.33	41.62	2094.9
5th	38.58	30.57	1179.3
4th	25.75	19.11	492.1
P6	15.92	11.12	177.0
	Totals	**642.65**	**57705.9**

Table 17: Wind Overturning Moment (E-W)			
Level	**Height (ft)**	**Story Force (k)**	**Overturning Moment (k-ft)**
Pent	142.00	10.16	1442.9
Roof	120.83	28.11	3396.7
11th	109.08	24.57	2679.8
10th	97.33	24.01	2336.9
9th	85.58	23.40	2002.7
8th	73.83	22.73	1677.9
7th	62.08	21.97	1363.6
6th	50.33	21.09	1061.3
5th	38.58	20.91	806.8
4th	25.75	18.00	463.4
P6	15.92	19.01	302.6
	Totals	**233.95**	**17534.6**

Table 18: Wind Overturning Moment (N-S)			
Level	**Height (ft)**	**Story Force (k)**	**Overturning Moment (k-ft)**
Pent	142.00	46.35	6581.4
Roof	120.83	59.28	7162.5
11th	109.08	47.52	5183.9
10th	97.33	46.57	4532.9
9th	85.58	45.53	3896.6
8th	73.83	44.38	3276.5
7th	62.08	43.08	2674.4
6th	50.33	41.58	2092.8
5th	38.58	41.54	1602.7
4th	25.75	36.11	929.7
P6	15.92	38.56	613.9
	Totals	**490.51**	**38547.4**

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, material, and detailing of structural components such as columns and walls becomes clearer. In further studies, the modeling of the structure could be refined in order to obtain a more accurate depiction of the distribution of lateral forces within the building's structural system.

As lateral loads are determined, the loading of the concrete moment frames and concrete shear walls may be therefore found. The design of these items and their corresponding capacities were examined.

Figure 25: Exterior Perspective – by JMV

Through this calculation of wind and seismic loading, it was found that the structure is sound in both strength and serviceability requirements. 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 wind controlled the design of the majority of lateral components. This hypothesis was confirmed as the wind cases in this analysis produced a higher value for the base shear as well as the overturning moment on the structure.

To further study the loading of the structure, a more detailed analysis of the lateral system could be completed by refining the models used here. Also, further investigation could be applied to the review of reinforcing details to ensure their adequacy.

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]

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

Determine Gust Effect Factor [Sect 6.5.8]

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

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

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

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

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

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

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

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

$$g_R = \sqrt{2\ln(3{,}600n_1)} + 0.577/(\sqrt{2\ln(3{,}600n_1)})$$ [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 : $P_z = q_z GC_p - q_h(GC_{pi})$

Leeward : $P_h = q_h GC_p - q_h(GC_{pi})$

See calc tables for results

Wind: East-West Direction

Table A.1: 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 A.2: 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 A.3: 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 A.4: 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 A.5: 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 A.6: 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**

Wind Drift Values - 2-D Analysis Results

Table A.7: Wind Drifts (N-S)			
Level	**Allowable (in)**	**Story Drift (in)**	**Total Drift (in)**
Roof	0.3525	0.0937	2.7954
11th	0.3525	0.1321	2.7017
10th	0.3525	0.1721	2.5696
9th	0.3525	0.2078	2.3975
8th	0.3525	0.2427	2.1897
7th	0.3525	0.2758	1.947
6th	0.3525	0.3076	1.6712
5th	0.3849	0.3695	1.3636
4th	0.2949	0.3052	0.9941
P6	0.4776	0.6889	0.6889

Table A.8: Wind Drifts (E-W)			
Level	**Allowable (in)**	**Story Drift (in)**	**Total Drift (in)**
Roof	0.3525	0.1327	2.5448
11th	0.3525	0.1613	2.4121
10th	0.3525	0.1953	2.2508
9th	0.3525	0.2269	2.0555
8th	0.3525	0.2540	1.8286
7th	0.3525	0.2756	1.5746
6th	0.3525	0.2900	1.299
5th	0.3849	0.3242	1.009
4th	0.2949	0.2493	0.6848
P6	0.4776	0.4355	0.4355

Wind Drift Values - 3-D Analysis Results

Table A.9: Wind Drifts (N-S)			
Level	**Allowable (in)**	**Story Drift (in)**	**Total Drift (in)**
Roof	0.3525	0.2358	1.8606
11th	0.3525	0.2361	1.6248
10th	0.3525	0.2340	1.3887
9th	0.3525	0.2286	1.1547
8th	0.3525	0.2190	0.9261
7th	0.3525	0.2036	0.7071
6th	0.3525	0.1809	0.5035
5th	0.3849	0.1618	0.3226
4th	0.2949	0.0901	0.1608
P6	0.4776	0.0707	0.0707

Table A.10: Wind Drifts (E-W)			
Level	**Allowable (in)**	**Story Drift (in)**	**Total Drift (in)**
Roof	0.3525	0.0723	1.0601
11th	0.3525	0.0836	0.9878
10th	0.3525	0.0935	0.9042
9th	0.3525	0.1008	0.8107
8th	0.3525	0.1082	0.7099
7th	0.3525	0.1138	0.6017
6th	0.3525	0.1164	0.4879
5th	0.3849	0.1238	0.3715
4th	0.2949	0.0859	0.2477
P6	0.4776	0.1618	0.1618

Appendix B

Seismic

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

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

$S_{DS} < 0.167 \rightarrow A$ [TABLE 11.6-1]

$S_{D1} < 0.067 \rightarrow A$ [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 B.1: Penthouse Weight	
Item	**Design Weight (kips)**
Beams	77.9
Slab	390
Roofing	156
SDL	39
Equipment	120
Façade	103.5
Total	886.4

Table B.2: 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 B.3: 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 B.4: 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 B.5: 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 B.6: 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 B.7: Design Values	
Effective Seismic Weight	41163 kips
Base Shear	642.7 kips
Overturning Moment	57708 kips-ft

Seismic Overturning Moment – 2-D Analysis Results

Table B.8: Seismic Overturning Moment			
Level	**Height (ft)**	**Story Force (k)**	**Overturning Moment (k-ft)**
Pent	142.00	30.81	4375.6
Roof	120.83	125.06	15110.6
11th	109.08	99.67	10871.6
10th	97.33	89.53	8713.8
9th	85.58	77.10	6598.5
8th	73.83	64.95	4795.4
7th	62.08	53.11	3297.0
6th	50.33	41.62	2094.9
5th	38.58	30.57	1179.3
4th	25.75	19.11	492.1
P6	15.92	11.12	177.0
	Totals	**642.65**	**57705.9**

Seismic Drifts – 2-D Analysis Results

Table B.9			Seismic Drifts (N-S)			Seismic Drifts (E-W)	
Level	**Allowable Drift (in)**		**Story Drift (in)**	**Total Drift (in)**		**Story Drift (in)**	**Total Drift (in)**
Roof	2.8200		2.3459	18.5504		1.3050	21.6938
11th	2.8200		2.3643	16.2045		1.5710	20.3888
10th	2.8200		2.3535	13.8402		1.8643	18.8178
9th	2.8200		2.2995	11.4867		2.1080	16.9535
8th	2.8200		2.1969	9.1872		2.2863	14.8455
7th	2.8200		2.0309	6.9903		2.3905	12.5593
6th	2.8200		1.7892	4.9595		2.4163	10.1688
5th	3.0792		1.5845	3.1703		2.5895	7.7525
4th	2.3592		0.8735	1.5858		1.9180	5.1630
P6	3.8208		0.7124	0.7124		3.2450	3.2450

Appendix C

Calculations

John Vais | Tech 4

2-D Calculations Procedure

i) Determine Stiffness of Elements

- Model individual elements in SAP 2000
- Apply 10 kip dummy load
- calculate stiffness at each level based on

$$P = k\Delta x$$

ii) Determine Center of mass

- Based on Geometric Floor area

iii) Determine Center of Rigidity

$$x_r = \frac{\Sigma R_i x_i}{\Sigma R_i} \qquad y_r = \frac{\Sigma R_i y_i}{\Sigma R_i}$$

iv) Determine Direct Shears

$$V_{i_d} = (R_i / \Sigma R_i) V$$

iv) Determine Direct Shears

$$V_{i_d} = (R_i / \Sigma R_i) V$$

v) Determine Torsional Shear

$$V_{i_t} = \frac{V e d_i R_i}{J}$$

where $J = \Sigma R_i d_i^2$

vi) Calculate Total shear in each Element

$$V_{Tot} = V_{i,d} + V_{i,t}$$

Excel Load Combinations – Base Reactions

Building Forces as Values at Base

Wind

	Case	Fx (kips)	Fy (kips)	Mx (k-ft)	My (k-ft)	Mt (k-ft)
1	1-NS	245.63	0.00	17535.19	0.00	0.00
2	1-EW	0.00	496.85	0.00	36080.47	0.00
3	2-NS(+)	184.22	0.00	13151.39	0.00	3316.04
4	2-NS(-)	184.22	0.00	13151.39	0.00	-3316.04
5	2-EW(+)	0.00	372.64	0.00	27060.35	11738.20
6	2-EW(-)	0.00	372.64	0.00	27060.35	-11738.20
7	3(+)	184.22	372.64	1315.39	27060.35	0.00
8	3(-)	184.22	-372.64	-1315.39	-27060.35	0.00
9	4 (++)	138.29	279.73	9872.31	20313.31	11300.71
10	4 (-+)	138.29	279.73	9872.31	20313.31	6322.23
11	4(+-)	138.29	279.73	9872.31	20313.31	-6322.23
12	4(--)	138.29	279.73	9872.31	20313.31	-11300.71

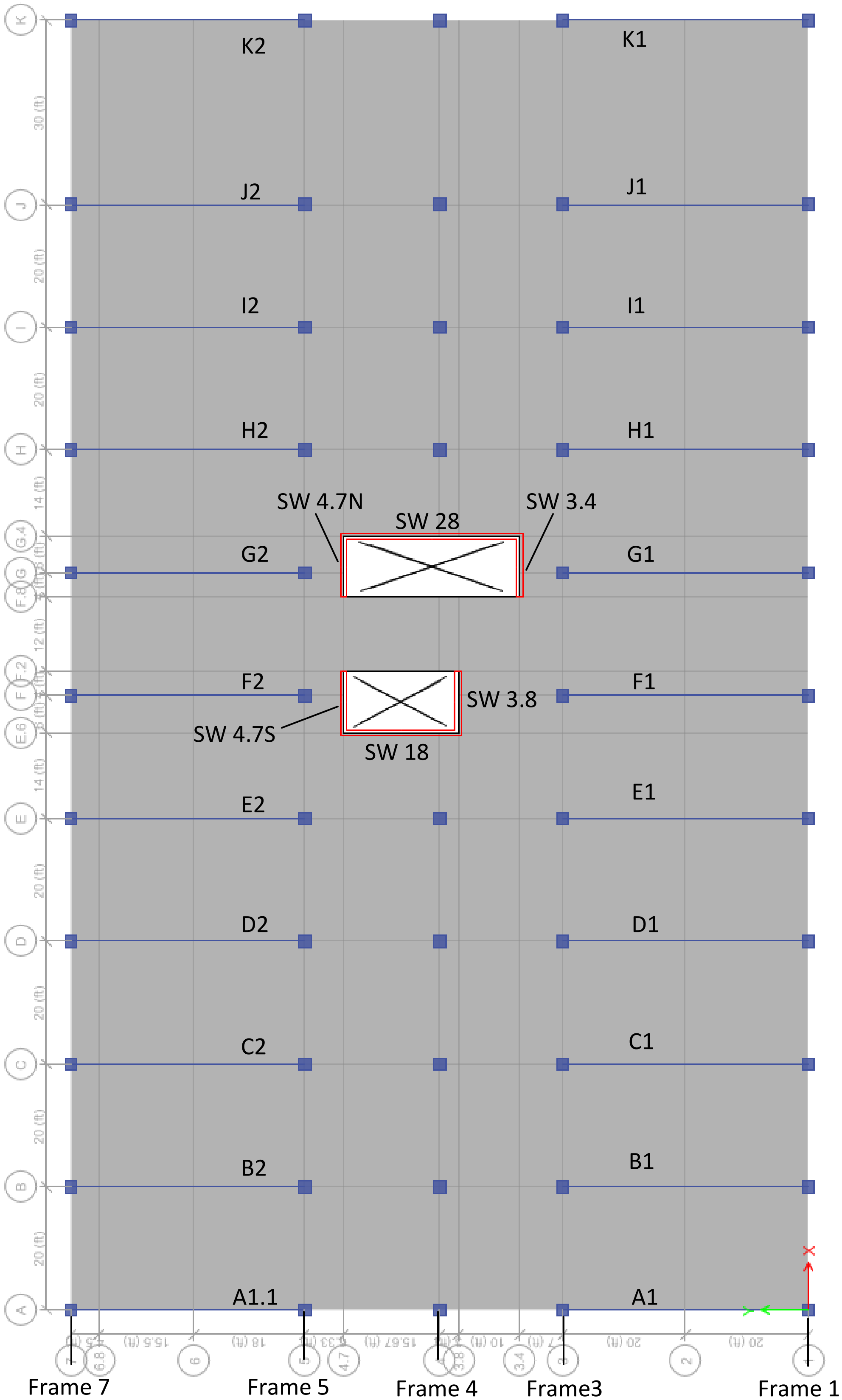
Seismic

	Case	Fx (kips)	Fy (kips)	Mx (k-ft)	My (k-ft)	Mt (k-ft)
1	X	624.70	0.00	57707.81	0.00	0.00
2	X (+5)	624.70	0.00	57707.81	0.00	3855.91
3	X (-5)	624.70	0.00	57707.81	0.00	-3855.91
4	Y	0.00	624.70	0.00	57707.81	0.00
5	Y (+5)	0.00	624.70	0.00	57707.81	6747.85
6	Y (-5)	0.00	624.70	0.00	57707.81	-6747.85

Test Forces

	Case	Fx (kips)	Fy (kips)	Mx (k-ft)	My (k-ft)	Mt (k-ft)
1	1000X	1000.00	0.00			0.00
2	1000Y	0.00	1000.00			0.00

Excel Calculation Plan – Element ID's



Excel Calculation Example – Select Pages from Wind Load 5

Level	Roof	Load	W5
Center of Mass x	105	Fx	0.00 kips
Center of Mass y	60	Fy	66.18 kips
Center Of Rigidity x	108.1892	M	2084.79 kip-ft
Center Of Rigidity y	61.22355		
Eccentricity x	3.189169		
Eccentricity y	1.223551		

Element	Location x (ft)	Location y (ft)	Stiffness (k/in)	k*x (k/in*ft)	K*y (k/in*ft)	Fx Direct (kips)	Fy Direct (kips)	dx (ft)	dy (ft)	J (k/in*ft^2)	Fx-M (kips)	Fy-M (kips)	Force (kips)
E-W Direction													
A1	0	-	58.31	0.0	-	-	1.78	108.19	-	682501	-	2.25	4.02
A1.1	0	-	60.72	0.0	-	-	1.85	108.19	-	710680	-	2.34	4.19
B1	20	-	58.31	1166.2	-	-	1.78	88.19	-	453489	-	1.83	3.61
B2	20	-	60.72	1214.3	-	-	1.85	88.19	-	472212	-	1.91	3.76
C1	40	-	58.31	2332.4	-	-	1.78	68.19	-	271123	-	1.42	3.19
C2	40	-	60.72	2428.7	-	-	1.85	68.19	-	282317	-	1.47	3.33
D1	60	-	58.31	3498.5	-	-	1.78	48.19	-	135405	-	1.00	2.78
D2	60	-	60.72	3643.0	-	-	1.85	48.19	-	140996	-	1.04	2.89
E1	80	-	58.31	4664.7	-	-	1.78	28.19	-	46334	-	0.59	2.36
E2	80	-	60.72	4857.3	-	-	1.85	28.19	-	48247	-	0.61	2.46
SW 18	94	-	182.15	17122.0	-	-	5.55	14.19	-	36673	-	0.92	6.47
F1	100	-	58.31	5830.9	-	-	1.78	8.19	-	3910	-	0.17	1.95
F2	100	-	60.72	6071.6	-	-	1.85	8.19	-	4072	-	0.18	2.03
G1	120	-	58.31	6997.1	-	-	1.78	-11.81	-	8134	-	-0.25	1.53
G2	120	-	60.72	7286.0	-	-	1.85	-11.81	-	8470	-	-0.26	1.59
SW 28	126	-	680.27	85714.3	-	-	20.73	-17.81	-	215800	-	-4.32	16.42
H1	140	-	58.31	8163.3	-	-	1.78	-31.81	-	59005	-	-0.66	1.12
H2	140	-	60.72	8500.3	-	-	1.85	-31.81	-	61441	-	-0.69	1.16
I1	160	-	58.31	9329.4	-	-	1.78	-51.81	-	156523	-	-1.08	0.70
I2	160	-	60.72	9714.6	-	-	1.85	-51.81	-	162985	-	-1.12	0.73
J1	180	-	58.31	10495.6	-	-	1.78	-71.81	-	300688	-	-1.49	0.29
J2	180	-	60.72	10929.0	-	-	1.85	-71.81	-	313102	-	-1.55	0.30
K1	210	-	58.31	12244.9	-	-	1.78	-101.81	-	604399	-	-2.11	-0.34
K2	210	-	60.72	12750.5	-	-	1.85	-101.81	-	629353	-	-2.20	-0.35
		Sum X	2171.7	234954.6	-	-	66.18	-	-	5807857	-	0.00	66.18
N-S Direction													
Frame 1	-	0	76.34	-	0.0	0	-	-	-61.22	286132	-1.66	-	-1.66
Frame 3	-	40	76.34	-	3053.4	0	-	-	-21.22	34385	-0.58	-	-0.58
SW 3.4	-	47	31.10	-	1461.9	0	-	-	-14.22	6293	-0.16	-	-0.16
Sw 3.8	-	57	31.10	-	1772.9	0	-	-	-4.22	555	-0.05	-	-0.05
Frame 4	-	60	76.34	-	4580.2	0	-	-	-1.22	114	-0.03	-	-0.03
SW 4.7S	-	75.5	31.10	-	2348.4	0	-	-	14.28	6340	0.16	-	0.16
SW 4.7N	-	75.5	31.10	-	2348.4	0	-	-	14.28	6340	0.16	-	0.16
Frame 5	-	82	76.34	-	6259.5	0	-	-	20.78	32951	0.56	-	0.56
Frame 7	-	120	76.34	-	9160.3	0	-	-	58.78	263715	1.60	-	1.60
		Sum Y	506.10	-	30985.01	0.00	-	-	-	636824	0.00	-	0.00

Stiffnesss

38' Frame	60.72
40' Frame	58.31
18' SW	182.15
28' SW	680.27
10' Return	31.10
Long Frame	76.34

Note: Force values are in direction of "applied values" not resisting values
Note: Use Center of Pressure for Wind calc in place of CM

CM,x	101.0873
CM,y	62.17647
CP,x	105
CP,y	60

Level	9th	Load	W5
Center of Mass x	105	Fx	0.00 kips
Center of Mass y	60	Fy	170.90 kips
Center Of Rigidity x	109.6697	M	5383.50 kip-ft
Center Of Rigidity y	61.42224		
Eccentricity x	4.669732		
Eccentricity y	1.422243		

Element	Location x (ft)	Location y (ft)	Stiffness (k/in)	k*x (k/in*ft)	K*y (k/in*ft)	Fx Direct (kips)	Fy Direct (kips)	dx (ft)	dy (ft)	J (k/in*ft^2)	Fx-M (kips)	Fy-M (kips)	Force (kips)
E-W Direction													
A1	0	-	48.10	0.0	-	-	3.97	109.67	-	578521	-	6.01	9.98
A1.1	0	-	50.20	0.0	-	-	4.14	109.67	-	603788	-	6.27	10.41
B1	20	-	48.10	962.0	-	-	3.97	89.67	-	386756	-	4.92	8.88
B2	20	-	50.20	1004.0	-	-	4.14	89.67	-	403648	-	5.13	9.27
C1	40	-	48.10	1924.0	-	-	3.97	69.67	-	233471	-	3.82	7.79
C2	40	-	50.20	2008.0	-	-	4.14	69.67	-	243668	-	3.99	8.13
D1	60	-	48.10	2886.0	-	-	3.97	49.67	-	118667	-	2.72	6.69
D2	60	-	50.20	3012.0	-	-	4.14	49.67	-	123850	-	2.84	6.98
E1	80	-	48.10	3848.0	-	-	3.97	29.67	-	42342	-	1.63	5.59
E2	80	-	50.20	4016.1	-	-	4.14	29.67	-	44191	-	1.70	5.84
SW 18	94	-	209.64	19706.5	-	-	17.29	15.67	-	51476	-	3.74	21.03
F1	100	-	48.10	4810.0	-	-	3.97	9.67	-	4498	-	0.53	4.50
F2	100	-	50.20	5020.1	-	-	4.14	9.67	-	4694	-	0.55	4.69
G1	120	-	48.10	5772.0	-	-	3.97	-10.33	-	5133	-	-0.57	3.40
G2	120	-	50.20	6024.1	-	-	4.14	-10.33	-	5357	-	-0.59	3.55
SW 28	126	-	781.25	98437.5	-	-	64.43	-16.33	-	208342	-	-14.54	49.89
H1	140	-	48.10	6734.0	-	-	3.97	-30.33	-	44248	-	-1.66	2.30
H2	140	-	50.20	7028.1	-	-	4.14	-30.33	-	46181	-	-1.74	2.41
I1	160	-	48.10	7696.0	-	-	3.97	-50.33	-	121844	-	-2.76	1.21
I2	160	-	50.20	8032.1	-	-	4.14	-50.33	-	127165	-	-2.88	1.26
J1	180	-	48.10	8658.0	-	-	3.97	-70.33	-	237920	-	-3.86	0.11
J2	180	-	50.20	9036.1	-	-	4.14	-70.33	-	248311	-	-4.02	0.12
K1	210	-	48.10	10101.0	-	-	3.97	-100.33	-	484183	-	-5.50	-1.53
K2	210	-	50.20	10542.2	-	-	4.14	-100.33	-	505329	-	-5.74	-1.60
		Sum X	2072.2	227257.9	-	-	170.90	-	-	4873583	-	0.00	170.90
N-S Direction													
Frame 1	-	0	65.32	-	0.0	0	-	-	-61.42	246420	-4.57	-	-4.57
Frame 3	-	40	65.32	-	2612.7	0	-	-	-21.42	29975	-1.59	-	-1.59
SW 3.4	-	47	35.86	-	1685.2	0	-	-	-14.42	7458	-0.59	-	-0.59
Sw 3.8	-	57	35.86	-	2043.7	0	-	-	-4.42	701	-0.18	-	-0.18
Frame 4	-	60	65.32	-	3919.0	0	-	-	-1.42	132	-0.11	-	-0.11
SW 4.7S	-	75.5	35.86	-	2707.1	0	-	-	14.08	7106	0.58	-	0.58
SW 4.7N	-	75.5	35.86	-	2707.1	0	-	-	14.08	7106	0.58	-	0.58
Frame 5	-	82	65.32	-	5356.0	0	-	-	20.58	27658	1.53	-	1.53
Frame 7	-	120	65.32	-	7838.0	0	-	-	58.58	224125	4.36	-	4.36
		Sum Y	470.00	-	28868.73	0.00	-	-	-	550681	0.00	-	0.00

Stiffnesss

38' Frame	50.20
40' Frame	48.10
18' SW	209.64
28' SW	781.25
10' Return	35.86
Long Frame	65.32

Note: Force values are in direction of "applied values" not resisting values
Note: Use Center of Pressure for Wind calc in place of CM

CM,x	101.0873
CM,y	62.17647
CP,x	105
CP,y	60

Level	6th	Load	W5	
Center of Mass x	105	Fx	0.00	kips
Center of Mass y	60	Fy	267.69	kips
Center Of Rigidity x	111.7334	M	8432.09	kip-ft
Center Of Rigidity y	61.81771			
Eccentricity x	6.733432			
Eccentricity y	1.817713			

Element	Location x (ft)	Location y (ft)	Stiffness (k/in)	k*x (k/in*ft)	K*y (k/in*ft)	Fx Direct (kips)	Fy Direct (kips)	dx (ft)	dy (ft)	J (k/in*ft^2)	Fx-M (kips)	Fy-M (kips)	Force (kips)
E-W Direction													
A1	0	-	45.77	0.0	-	-	4.85	111.73	-	571367	-	9.89	14.74
A1.1	0	-	47.92	0.0	-	-	5.08	111.73	-	598196	-	10.35	15.43
B1	20	-	45.77	915.3	-	-	4.85	91.73	-	385127	-	8.12	12.97
B2	20	-	47.92	958.3	-	-	5.08	91.73	-	403211	-	8.50	13.58
C1	40	-	45.77	1830.7	-	-	4.85	71.73	-	235500	-	6.35	11.20
C2	40	-	47.92	1916.6	-	-	5.08	71.73	-	246559	-	6.65	11.73
D1	60	-	45.77	2746.0	-	-	4.85	51.73	-	122487	-	4.58	9.43
D2	60	-	47.92	2874.9	-	-	5.08	51.73	-	128239	-	4.79	9.87
E1	80	-	45.77	3661.3	-	-	4.85	31.73	-	46087	-	2.81	7.66
E2	80	-	47.92	3833.3	-	-	5.08	31.73	-	48252	-	2.94	8.02
SW 18	94	-	317.46	29841.3	-	-	33.66	17.73	-	99833	-	10.89	44.55
F1	100	-	45.77	4576.7	-	-	4.85	11.73	-	6301	-	1.04	5.89
F2	100	-	47.92	4791.6	-	-	5.08	11.73	-	6597	-	1.09	6.17
G1	120	-	45.77	5492.0	-	-	4.85	-8.27	-	3128	-	-0.73	4.12
G2	120	-	47.92	5749.9	-	-	5.08	-8.27	-	3274	-	-0.77	4.31
SW 28	126	-	1176.47	148235.3	-	-	124.75	-14.27	-	239453	-	-32.46	92.29
H1	140	-	45.77	6407.3	-	-	4.85	-28.27	-	36567	-	-2.50	2.35
H2	140	-	47.92	6708.2	-	-	5.08	-28.27	-	38285	-	-2.62	2.46
I1	160	-	45.77	7322.7	-	-	4.85	-48.27	-	106621	-	-4.27	0.58
I2	160	-	47.92	7666.5	-	-	5.08	-48.27	-	111627	-	-4.47	0.61
J1	180	-	45.77	8238.0	-	-	4.85	-68.27	-	213287	-	-6.04	-1.19
J2	180	-	47.92	8624.8	-	-	5.08	-68.27	-	223303	-	-6.33	-1.24
K1	210	-	45.77	9611.0	-	-	4.85	-98.27	-	441937	-	-8.70	-3.84
K2	210	-	47.92	10062.3	-	-	5.08	-98.27	-	462689	-	-9.10	-4.02
		Sum X	2524.4	282063.9	-	-	267.69	-	-	4777927	-	0.00	267.69
N-S Direction													
Frame 1	-	0	59.45	-	0.0	0	-	-	-61.82	227196	-7.11	-	-7.11
Frame 3	-	40	59.45	-	2378.1	0	-	-	-21.82	28300	-2.51	-	-2.51
SW 3.4	-	47	54.53	-	2562.7	0	-	-	-14.82	11972	-1.56	-	-1.56
Sw 3.8	-	57	54.53	-	3108.0	0	-	-	-4.82	1266	-0.51	-	-0.51
Frame 4	-	60	59.45	-	3567.2	0	-	-	-1.82	196	-0.21	-	-0.21
SW 4.7S	-	75.5	54.53	-	4116.7	0	-	-	13.68	10207	1.44	-	1.44
SW 4.7N	-	75.5	54.53	-	4116.7	0	-	-	13.68	10207	1.44	-	1.44
Frame 5	-	82	59.45	-	4875.1	0	-	-	20.18	24217	2.32	-	2.32
Frame 7	-	120	59.45	-	7134.4	0	-	-	58.18	201259	6.69	-	6.69
		Sum Y	515.37	-	31858.85	0.00	-	-	-	514821	0.00	-	0.00

Stiffnesss

38' Frame	47.92
40' Frame	45.77
18' SW	317.46
28' SW	1176.47
10' Return	54.53
Long Frame	59.45

Note: Force values are in direction of "applied values" not resisting values
Note: Use Center of Pressure for Wind calc in place of CM

CM,x	101.0873
CM,y	62.17647
CP,x	105
CP,y	60

Level	P6		Load	W5
Center of Mass x		105	Fx	0.00 kips
Center of Mass y		60	Fy	354.84 kips
Center Of Rigidity x		117.4467	M	11177.59 kip-ft
Center Of Rigidity y		63.14667		
Eccentricity x		12.44673		
Eccentricity y		3.146671		

Element	Location x (ft)	Location y (ft)	Stiffness (k/in)	k*x (k/in*ft)	K*y (k/in*ft)	Fx Direct (kips)	Fy Direct (kips)	dx (ft)	dy (ft)	J (k/in*ft^2)	Fx-M (kips)	Fy-M (kips)	Force (kips)
E-W Direction													
A1	0	-	19.29	0.0	-	-	1.27	117.45	-	266032	-	11.02	12.29
A1.1	0	-	19.92	0.0	-	-	1.31	117.45	-	274721	-	11.38	12.69
B1	20	-	19.29	385.7	-	-	1.27	97.45	-	183141	-	9.14	10.42
B2	20	-	19.92	398.3	-	-	1.31	97.45	-	189123	-	9.44	10.76
C1	40	-	19.29	771.5	-	-	1.27	77.45	-	115680	-	7.27	8.54
C2	40	-	19.92	796.7	-	-	1.31	77.45	-	119458	-	7.50	8.82
D1	60	-	19.29	1157.2	-	-	1.27	57.45	-	63648	-	5.39	6.66
D2	60	-	19.92	1195.0	-	-	1.31	57.45	-	65726	-	5.57	6.88
E1	80	-	19.29	1542.9	-	-	1.27	37.45	-	27045	-	3.51	4.79
E2	80	-	19.92	1593.3	-	-	1.31	37.45	-	27928	-	3.63	4.94
SW 18	94	-	1098.90	103296.7	-	-	72.53	23.45	-	604120	-	125.35	197.88
F1	100	-	19.29	1928.6	-	-	1.27	17.45	-	5871	-	1.64	2.91
F2	100	-	19.92	1991.6	-	-	1.31	17.45	-	6062	-	1.69	3.00
G1	120	-	19.29	2314.4	-	-	1.27	-2.55	-	126	-	-0.24	1.03
G2	120	-	19.92	2390.0	-	-	1.31	-2.55	-	130	-	-0.25	1.07
SW 28	126	-	3846.15	484615.4	-	-	253.85	-8.55	-	281378	-	-160.04	93.81
H1	140	-	19.29	2700.1	-	-	1.27	-22.55	-	9810	-	-2.12	-0.84
H2	140	-	19.92	2788.3	-	-	1.31	-22.55	-	10130	-	-2.19	-0.87
I1	160	-	19.29	3085.8	-	-	1.27	-42.55	-	34923	-	-3.99	-2.72
I2	160	-	19.92	3186.6	-	-	1.31	-42.55	-	36064	-	-4.12	-2.81
J1	180	-	19.29	3471.6	-	-	1.27	-62.55	-	75466	-	-5.87	-4.60
J2	180	-	19.92	3584.9	-	-	1.31	-62.55	-	77931	-	-6.06	-4.75
K1	210	-	19.29	4050.1	-	-	1.27	-92.55	-	165209	-	-8.68	-7.41
K2	210	-	19.92	4182.4	-	-	1.31	-92.55	-	170606	-	-8.97	-7.65
		Sum X	5376.3	631427.1	-	-	354.84	-	-	2810328	-	0.00	354.84
N-S Direction													
Frame 1	-	0	34.19	-	0.0	0	-	-	-63.15	136325	-10.50	-	-10.50
Frame 3	-	40	34.19	-	1367.5	0	-	-	-23.15	18317	-3.85	-	-3.85
SW 3.4	-	47	194.55	-	9144.0	0	-	-	-16.15	50723	-15.28	-	-15.28
Sw 3.8	-	57	194.55	-	11089.5	0	-	-	-6.15	7350	-5.82	-	-5.82
Frame 4	-	60	34.19	-	2051.3	0	-	-	-3.15	339	-0.52	-	-0.52
SW 4.7S	-	75.5	194.55	-	14688.7	0	-	-	12.35	29690	11.69	-	11.69
SW 4.7N	-	75.5	194.55	-	14688.7	0	-	-	12.35	29690	11.69	-	11.69
Frame 5	-	82	34.19	-	2803.4	0	-	-	18.85	12152	3.14	-	3.14
Frame 7	-	120	34.19	-	4102.6	0	-	-	56.85	110506	9.46	-	9.46
		Sum Y	949.15	-	59935.68	0.00	-	-	-	395091	0.00	-	0.00

Stiffnesss

38' Frame	19.92
40' Frame	19.29
18' SW	1098.90
28' SW	3846.15
10' Return	194.55
Long Frame	34.19

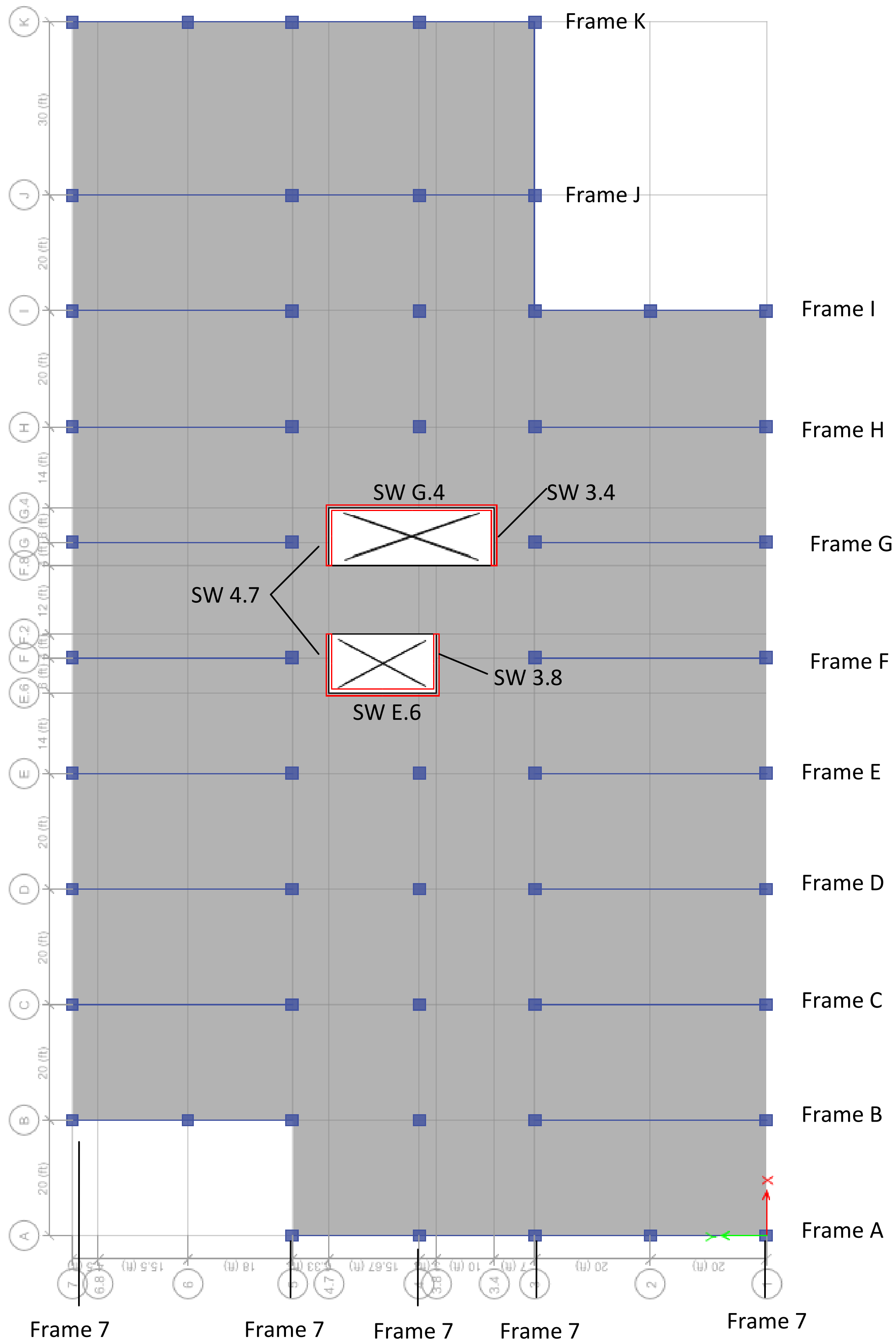
Note: Force values are in direction of "applied values" not resisting values
Note: Use Center of Pressure for Wind calc in place of CM

CM,x	101.0873
CM,y	62.17647
CP,x	105
CP,y	60

Excel Calculation Example - Summary of Wind Load 5

Element	Roof	11th	10th	9th	8th	7th	6th	5th	4th	P6	Total
E-W Direction											
A1	4.02	2.04	1.99	1.93	1.81	1.63	1.33	0.90	0.11	-3.46	12.29
A1.1	4.19	2.11	2.08	2.03	1.89	1.71	1.42	0.96	0.10	-3.80	12.69
B1	3.61	1.81	1.76	1.71	1.58	1.40	1.11	0.68	-0.05	-3.19	10.42
B2	3.76	1.88	1.84	1.79	1.65	1.48	1.18	0.72	-0.06	-3.49	10.76
C1	3.19	1.58	1.53	1.48	1.36	1.18	0.88	0.46	-0.20	-2.92	8.54
C2	3.33	1.64	1.60	1.56	1.42	1.24	0.95	0.49	-0.22	-3.18	8.82
D1	2.78	1.35	1.30	1.26	1.13	0.95	0.66	0.24	-0.36	-2.65	6.66
D2	2.89	1.40	1.37	1.32	1.18	1.00	0.71	0.26	-0.38	-2.87	6.88
E1	2.36	1.12	1.08	1.03	0.90	0.73	0.44	0.02	-0.52	-2.38	4.79
E2	2.46	1.16	1.13	1.08	0.94	0.77	0.47	0.02	-0.55	-2.56	4.94
SW 18	6.47	4.71	4.83	5.02	5.83	7.37	10.32	16.87	26.90	109.56	197.88
F1	1.95	0.89	0.85	0.81	0.68	0.50	0.21	-0.20	-0.67	-2.11	2.91
F2	2.03	0.93	0.89	0.85	0.71	0.53	0.24	-0.21	-0.71	-2.25	3.00
G1	1.53	0.66	0.62	0.58	0.45	0.28	-0.01	-0.42	-0.83	-1.83	1.03
G2	1.59	0.69	0.65	0.61	0.47	0.30	0.00	-0.44	-0.87	-1.93	1.07
SW 28	16.42	11.15	11.16	11.16	12.45	13.79	16.16	19.04	15.55	-33.07	93.81
H1	1.12	0.44	0.40	0.36	0.23	0.05	-0.23	-0.65	-0.99	-1.56	-0.84
H2	1.16	0.45	0.42	0.38	0.24	0.06	-0.24	-0.67	-1.03	-1.62	-0.87
I1	0.70	0.21	0.17	0.13	0.00	-0.17	-0.46	-0.87	-1.14	-1.29	-2.72
I2	0.73	0.21	0.18	0.14	0.00	-0.18	-0.47	-0.91	-1.20	-1.31	-2.81
J1	0.29	-0.02	-0.06	-0.09	-0.23	-0.40	-0.68	-1.09	-1.30	-1.02	-4.60
J2	0.30	-0.02	-0.06	-0.10	-0.24	-0.41	-0.71	-1.14	-1.36	-1.00	-4.75
K1	-0.34	-0.37	-0.40	-0.43	-0.57	-0.73	-1.01	-1.42	-1.54	-0.61	-7.41
K2	-0.35	-0.38	-0.42	-0.45	-0.59	-0.77	-1.07	-1.49	-1.60	-0.54	-7.65
	66.18	35.64	34.93	34.15	33.28	32.31	31.19	31.16	27.08	28.92	354.84
N-S Direction											
Frame 1	-1.66	-1.09	-0.96	-0.85	-0.85	-0.82	-0.86	-1.03	-1.09	-1.28	-10.50
Frame 3	-0.58	-0.38	-0.34	-0.30	-0.30	-0.30	-0.32	-0.39	-0.42	-0.54	-3.85
SW 3.4	-0.16	-0.13	-0.14	-0.16	-0.20	-0.29	-0.48	-0.96	-1.98	-10.78	-15.28
Sw 3.8	-0.05	-0.04	-0.04	-0.05	-0.07	-0.10	-0.17	-0.34	-0.74	-4.23	-5.82
Frame 4	-0.03	-0.02	-0.02	-0.02	-0.03	-0.03	-0.04	-0.07	-0.09	-0.16	-0.52
SW 4.7S	0.16	0.13	0.14	0.15	0.19	0.26	0.42	0.80	1.56	7.89	11.69
SW 4.7N	0.16	0.13	0.14	0.15	0.19	0.26	0.42	0.80	1.56	7.89	11.69
Frame 5	0.56	0.37	0.32	0.28	0.27	0.26	0.26	0.29	0.28	0.25	3.14
Frame 7	1.60	1.04	0.92	0.80	0.79	0.76	0.78	0.90	0.91	0.96	9.46
	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Etabs Model Plan – Element ID's



Etabs Stiffness Tables – 1000 kip load – N-S Direction – Pinned Bases

Pinned Bases

Force Distribution

Level	Frame 1	Frame 3	SW 3.4	SW 3.8	Frame 4	SW 4.7	Frame 5	Frame 7	Misc	Total
Roof										
11	-145.3	-260.6	-34.6	-31.3	-70.5	-90.4	-168.2	-159.5	-39.5	-999.9
10	-100.3	-213.1	-86.6	-80.2	-58.1	-186.8	-136.3	-109.5	-29.0	-999.9
9	-106.5	-216.1	-82.8	-74.5	-58.4	-176.2	-138.9	-116.4	-30.1	-999.9
8	-106.4	-222.5	-81.4	-71.0	-58.2	-173.4	-140.5	-116.4	-30.0	-999.8
7	-100.7	-209.3	-91.9	-80.2	-55.5	-191.1	-132.5	-110.0	-28.7	-999.9
6	-94.5	-196.1	-102.8	-90.1	-52.2	-210.1	-124.2	-103.0	-26.9	-999.9
5	-87.8	-179.5	-115.0	-101.3	-49.9	-230.5	-115.0	-95.6	-25.5	-1000.1
4	-67.5	-135.3	-149.9	-134.3	-39.5	-294.1	-87.2	-72.2	-20.0	-1000.0
P6	-90.4	-175.6	-110.5	-98.0	-58.6	-217.8	-119.2	-101.0	-29.0	-1000.1
Plaza	27.5	28.8	-305.1	-295.2	32.0	-571.0	35.3	33.8	13.9	-1000.0

Percent Distribution

Level	Frame 1	Frame 3	SW 3.4	SW 3.8	Frame 4	SW 4.7	Frame 5	Frame 7	Misc	Total
Roof										
11	14.5	26.1	3.5	3.1	7.1	9.0	16.8	16.0	4.0	100.0
10	10.0	21.3	8.7	8.0	5.8	18.7	13.6	11.0	2.9	100.0
9	10.7	21.6	8.3	7.5	5.8	17.6	13.9	11.6	3.0	100.0
8	10.6	22.3	8.1	7.1	5.8	17.3	14.1	11.6	3.0	100.0
7	10.1	20.9	9.2	8.0	5.6	19.1	13.3	11.0	2.9	100.0
6	9.5	19.6	10.3	9.0	5.2	21.0	12.4	10.3	2.7	100.0
5	8.8	17.9	11.5	10.1	5.0	23.0	11.5	9.6	2.5	100.0
4	6.8	13.5	15.0	13.4	4.0	29.4	8.7	7.2	2.0	100.0
P6	9.0	17.6	11.0	9.8	5.9	21.8	11.9	10.1	2.9	100.0
Plaza	-2.8	-2.9	30.5	29.5	-3.2	57.1	-3.5	-3.4	-1.4	100.0

Fx=1000 kips

Etabs Stiffness Tables – 1000 kip load – E-W Direction – Pinned Bases

Basic Model - Fixed Bases

Force Distribution

Level	Frame A	Frame B	Frame C	Frame D	Frame E	SW E.6	Frame F	Frame G	SW G.4	Frame H	Frame I	Frame J	Frame K	Total Force
Roof														
11	-102.90	-74.35	-36.72	-34.49	-33.23	-117.63	-32.92	-32.12	-363.99	-29.16	-48.64	-41.95	-51.92	-1000.0
10	-83.7	-58.6	-30.8	-28.6	-26.6	-188.6	-25.6	-25.3	-415.0	-20.6	-33.9	-28.4	-34.3	-1000.0
9	-84.15	-59.26	-30.60	-28.43	-26.68	-189.30	-25.79	-25.52	-412.02	-21.21	-34.47	-28.52	-34.07	-1000.0
8	-86.30	-59.60	-29.97	-27.76	-25.95	-182.54	-25.19	-24.85	-422.92	-20.35	-33.67	-27.93	-32.96	-1000.0
7	-81.6	-56.6	-28.5	-26.3	-24.4	-198.7	-23.4	-22.9	-435.0	-18.7	-30.5	-24.9	-28.5	-1000.0
6	-76.9	-53.3	-26.6	-24.4	-22.6	-219.1	-21.5	-20.8	-444.6	-16.9	-27.3	-21.9	-24.1	-1000.0
5	-70.8	-49.1	-24.4	-22.3	-20.5	-246.8	-19.3	-18.4	-452.2	-15.0	-23.7	-18.4	-19.0	-1000.0
4	-56.5	-40.0	-20.3	-18.1	-16.2	-305.3	-14.4	-13.2	-469.1	-10.4	-15.6	-11.3	-9.7	-1000.0
P6	-63.3	-40.9	-17.8	-16.1	-14.6	-311.0	-14.1	-13.1	-459.9	-10.1	-16.7	-12.3	-10.2	-1000.0
Plaza	-29.6	-26.4	-17.1	-15.0	-12.9	-398.8	-9.4	-7.8	-466.1	-6.7	-6.8	-3.3	-0.3	-1000.0

Percent Distribution

Level	Frame A	Frame B	Frame C	Frame D	Frame E	SW E.6	Frame F	Frame G	SW G.4	Frame H	Frame I	Frame J	Frame K	Total Force
Roof														
11	10.3	7.4	3.7	3.4	3.3	11.8	3.3	3.2	36.4	2.9	4.9	4.2	5.2	100.0
10	8.4	5.9	3.1	2.9	2.7	18.9	2.6	2.5	41.5	2.1	3.4	2.8	3.4	100.0
9	8.4	5.9	3.1	2.8	2.7	18.9	2.6	2.6	41.2	2.1	3.4	2.9	3.4	100.0
8	8.6	6.0	3.0	2.8	2.6	18.3	2.5	2.5	42.3	2.0	3.4	2.8	3.3	100.0
7	8.2	5.7	2.8	2.6	2.4	19.9	2.3	2.3	43.5	1.9	3.1	2.5	2.9	100.0
6	7.7	5.3	2.7	2.4	2.3	21.9	2.1	2.1	44.5	1.7	2.7	2.2	2.4	100.0
5	7.1	4.9	2.4	2.2	2.1	24.7	1.9	1.8	45.2	1.5	2.4	1.8	1.9	100.0
4	5.6	4.0	2.0	1.8	1.6	30.5	1.4	1.3	46.9	1.0	1.6	1.1	1.0	100.0
P6	6.3	4.1	1.8	1.6	1.5	31.1	1.4	1.3	46.0	1.0	1.7	1.2	1.0	100.0
Plaza	3.0	2.6	1.7	1.5	1.3	39.9	0.9	0.8	46.6	0.7	0.7	0.3	0.0	100.0

Fy=1000 kips

Etabs Stiffness Tables – 1000 kip load – N-S Direction – Fixed Bases

Basic Model - Fixed Bases

Force Distribution

Level	Frame 1	Frame 3	SW 3.4	SW 3.8	Frame 4	SW 4.7	Frame 5	Frame 7	Misc	Total
Roof										
11	-144.7	-258.32	-35.4	-32.1	-70.5	-93.8	-166.8	-158.9	-39.6	-1000.0
10	-97.7	-208.7	-91.4	-82.4	-56.7	-195.4	-133.1	-106.7	-28.1	-1000.0
9	-104.81	-212.7	-85.7	-75.7	-57.9	-182.1	-136.8	-114.7	-29.7	-1000.0
8	-104.9	-219.25	-83.6	-72.6	-57.9	-178.7	-138.6	-114.9	-29.7	-1000.1
7	-99.0	-205.52	-94.3	-82.7	-54.9	-197.2	-130.2	-108.1	-28.2	-1000.0
6	-92.5	-191.92	-105.6	-92.9	-51.5	-216.7	-121.6	-100.9	-26.4	-1000.0
5	-85.7	-174.97	-118.3	-104.0	-49.2	-237.0	-112.4	-93.5	-25.0	-1000.0
4	-65.8	-131.01	-153.7	-135.8	-39.3	-299.0	-85.0	-70.7	-19.7	-1000.0
P6	-69.4	-145.56	-147.6	-127.3	-39.2	-282.7	-91.9	-76.3	-20.2	-1000.0
Plaza	-32.2	-49.13	-210.9	-198.0	-25.1	-402.2	-38.3	-32.4	-11.7	-1000.0

Percent Distribution

Level	Frame 1	Frame 3	SW 3.4	SW 3.8	Frame 4	SW 4.7	Frame 5	Frame 7	Misc	Total
Roof										
11	14.5	25.8	3.5	3.2	7.1	9.4	16.7	15.9	4.0	100.0
10	9.8	20.9	9.1	8.2	5.7	19.5	13.3	10.7	2.8	100.0
9	10.5	21.3	8.6	7.6	5.8	18.2	13.7	11.5	3.0	100.0
8	10.5	21.9	8.4	7.3	5.8	17.9	13.9	11.5	3.0	100.0
7	9.9	20.6	9.4	8.3	5.5	19.7	13.0	10.8	2.8	100.0
6	9.3	19.2	10.6	9.3	5.2	21.7	12.2	10.1	2.6	100.0
5	8.6	17.5	11.8	10.4	4.9	23.7	11.2	9.4	2.5	100.0
4	6.6	13.1	15.4	13.6	3.9	29.9	8.5	7.1	2.0	100.0
P6	6.9	14.6	14.8	12.7	3.9	28.3	9.2	7.6	2.0	100.0
Plaza	3.2	4.9	21.1	19.8	2.5	40.2	3.8	3.2	1.2	100.0

Fx-1000 kips

Etabs Stiffness Tables – 1000 kip load – E-W Direction – Fixed Bases

Basic Model - Fixed Bases

Force Distribution

Level	Frame A	Frame B	Frame C	Frame D	Frame E	SW E.6	Frame F	Frame G	SW G.4	Frame H	Frame I	Frame J	Frame K	Total Force
Roof														
11	-102.90	-74.35	-36.72	-34.49	-33.23	-117.63	-32.92	-32.12	-363.99	-29.16	-48.64	-41.95	-51.92	-1000.0
10	-83.7	-58.6	-30.8	-28.6	-26.6	-188.6	-25.6	-25.3	-415.0	-20.6	-33.9	-28.4	-34.3	-1000.0
9	-84.15	-59.26	-30.60	-28.43	-26.68	-189.30	-25.79	-25.52	-412.02	-21.21	-34.47	-28.52	-34.07	-1000.0
8	-86.30	-59.60	-29.97	-27.76	-25.95	-182.54	-25.19	-24.85	-422.92	-20.35	-33.67	-27.93	-32.96	-1000.0
7	-81.6	-56.6	-28.5	-26.3	-24.4	-198.7	-23.4	-22.9	-435.0	-18.7	-30.5	-24.9	-28.5	-1000.0
6	-76.9	-53.3	-26.6	-24.4	-22.6	-219.1	-21.5	-20.8	-444.6	-16.9	-27.3	-21.9	-24.1	-1000.0
5	-70.8	-49.1	-24.4	-22.3	-20.5	-246.8	-19.3	-18.4	-452.2	-15.0	-23.7	-18.4	-19.0	-1000.0
4	-56.5	-40.0	-20.3	-18.1	-16.2	-305.3	-14.4	-13.2	-469.1	-10.4	-15.6	-11.3	-9.7	-1000.0
P6	-63.3	-40.9	-17.8	-16.1	-14.6	-311.0	-14.1	-13.1	-459.9	-10.1	-16.7	-12.3	-10.2	-1000.0
Plaza	-29.6	-26.4	-17.1	-15.0	-12.9	-398.8	-9.4	-7.8	-466.1	-6.7	-6.8	-3.3	-0.3	-1000.0

Percent Distribution

Level	Frame A	Frame B	Frame C	Frame D	Frame E	SW E.6	Frame F	Frame G	SW G.4	Frame H	Frame I	Frame J	Frame K	Total Force
Roof														
11	10.3	7.4	3.7	3.4	3.3	11.8	3.3	3.2	36.4	2.9	4.9	4.2	5.2	100.0
10	8.4	5.9	3.1	2.9	2.7	18.9	2.6	2.5	41.5	2.1	3.4	2.8	3.4	100.0
9	8.4	5.9	3.1	2.8	2.7	18.9	2.6	2.6	41.2	2.1	3.4	2.9	3.4	100.0
8	8.6	6.0	3.0	2.8	2.6	18.3	2.5	2.5	42.3	2.0	3.4	2.8	3.3	100.0
7	8.2	5.7	2.8	2.6	2.4	19.9	2.3	2.3	43.5	1.9	3.1	2.5	2.9	100.0
6	7.7	5.3	2.7	2.4	2.3	21.9	2.1	2.1	44.5	1.7	2.7	2.2	2.4	100.0
5	7.1	4.9	2.4	2.2	2.1	24.7	1.9	1.8	45.2	1.5	2.4	1.8	1.9	100.0
4	5.6	4.0	2.0	1.8	1.6	30.5	1.4	1.3	46.9	1.0	1.6	1.1	1.0	100.0
P6	6.3	4.1	1.8	1.6	1.5	31.1	1.4	1.3	46.0	1.0	1.7	1.2	1.0	100.0
Plaza	3.0	2.6	1.7	1.5	1.3	39.9	0.9	0.8	46.6	0.7	0.7	0.3	0.0	100.0

Fy=1000 kips

Appendix D

Spot Checks

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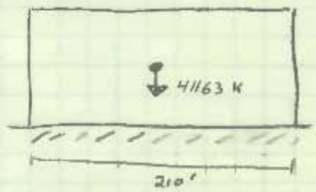
Over Turning Moment

N-S Direction



$M_r = 41163\,(210/2) =$

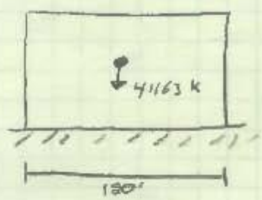
4322115 Kip ft

wind controls → $0.9D > 1.6W$?

$3{,}889{,}903 > 61{,}876$

∴ ok

E-W Direction



$M_r = 41163\,(120/2) =$

2469780 Kip-ft

seismic controls → $0.9D > 1.0E$?

$2{,}222{,}802 > 53{,}706$

∴ ok

- Thus there is no uplift in foundations in either direction.

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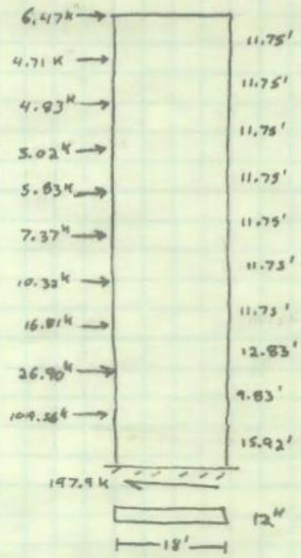
Shear Wall - 18'



$f'_c = 5000 psi$

$M_u = 1.6(6689) + 0.9(378 \times 9) = 7640.6$ kip-ft (OTM - SWM)

$V_u = (1.6)\ 197.9 = 316.6$ kips

$P_u = 1215.2$ kips (selfwt + trib area)

Controlling case - wind : case 2 - EW(+)

$V_u \le \phi V_{n,max} = \phi\, 10\sqrt{f'_c}\, hd$

$d = 0.8 \ell_w = 0.8(18 \times 12) = 172.8"$

$\phi V_n = 0.75(10)\sqrt{5000}\,(12)(172.8)/1000 = 1099.7$ kips

$\phi V_n > V_u \therefore \underline{ok}$

Shear strength of Concrete

$V_c = 2\sqrt{f'_c}\, hd = 2\sqrt{5000}\,(12)(172.8)/1000 = 293.25$ kips

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(293.25) = 109.97$ kips $< V_u \therefore V_s$ required

Determine Reinforcing Needed

$V_u \le \phi V_n = \phi(V_c + V_s) \rightarrow 316.6 = 109.97 + \phi V_s \rightarrow V_s = 275.5$ kips

$A_v/s = \frac{V_s}{f_y d} = \frac{275.5}{(60,000)(172.8)} = 0.027$ in²/in $\rightarrow 0.32$ in²/ft

Try (2)#4 @ 12" $\rightarrow A_s = 0.40$ in²/ft

$\rho_t = \frac{A_v}{sh} = \frac{2(0.2)}{12 \times 12} = 0.00278 > 0.0025 \therefore \underline{ok}$

spacing ok by inspection

$\therefore$ use (2) #4 bars @ 12" o.c. for horizontal reinforcing

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Determine Required shear reinforcing

$\rho_\ell = A_v/sh \geq 0.0025 + 0.5\left(2.5 - \frac{120.83}{18}\right)(0.00278 - 0.0025) < 0.0025$

$\therefore$ use $\rho_\ell \geq 0.0025$ as min.

Try (2) #4

$s = A_v/\rho_\ell h = \frac{2(0.2)}{0.0025(12)} = 13.33'' \rightarrow$ use 12''

$\therefore$ use (2) #4 bars @ 12'' o.c. for Vertical Reinforcing

Design for flexure

$M_u \leq \phi M_n = \phi A_s F_y I_d$

$7641(12) = (0.9) A_s (60) [0.9(172.8)] \rightarrow A_s = 10.92 \text{ in}^2$

$a = \frac{A_s F_y}{0.85(f'_c)b} = \frac{10.92(60)}{0.85(5)(12)} = 12.84''$

calc $I_d = d - a/2 = 172.8 - \frac{12.84}{2} = 166.38''$

recalc $A_s \rightarrow 7641(12) = (0.9) A_s (60)(166.38) \rightarrow A_s = 10.21 \text{ in}^2$

Assume 8 #11 bars $\rightarrow A_s = 8(1.56) = 12.48 \text{ in}^2$

$A_{s\,prov} > A_{s\,req} \therefore$ ok

$c = a/\beta_1 = 12.01/0.80 = 15.01''$

$\varepsilon_t = 0.003\left(\frac{172.8 - 15.01}{15.01}\right) = 0.032 > 0.005 \therefore$ ok

$\therefore$ Actual design is adequate (8 #11 bars)

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Shear Wall - 28'

$M_u = 1.6(16325.63) - 0.9(1050 \times 14) = 12891.04$ k-ft

$V_u = 1.6(323.24) = 517.184$ kips

P_u: $L = (10 \times 33 \times 7)100 = 231$ kips

$D = (10 \times 33 \times 7)105 + (140 \times 1 \times 28)150 = 830.6$ kips

Controling Case: Wind case 2 - EW(-)

$V_u \le \phi V_{n,max} = \phi 10 \sqrt{f'_c}\, hd$

$d = 0.8 l_w = 268.8$ ft

$\phi V_n = (0.75)(10)\sqrt{5000}(12)(268.8)/1000 = 1710.6$ kips

$\phi V_n > V_u$ ∴ ok

Shear Strength of concrete:

$V_c = 2\sqrt{f'_c}\, hd = 2\sqrt{5000}(12)(268.8)/1000 = 456.17$ kips

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(456.17) = 171.06$ kips $< V_u$ ∴ V_s is required

Determine Reinforcing Required:

$V_u \le \phi V_n = \phi(V_c + V_s) \rightarrow 517.18 \le 171.06 + \phi V_s \rightarrow V_s = 461.9$ kips

$A_v/s = \frac{V_s}{F_y d} = \frac{461.9}{(60)(268.8)} = 0.0286$ in²/in $\rightarrow 0.343$ in²/ft

Try (2) #4 @ 12" $\rightarrow A_s = 0.40$ in²/ft

$\rho_t = \frac{A_v}{sh} = \frac{(0.2)(2)}{12 \times 12} = 0.00278 > 0.0025$ ∴ ok

- spacing is ok by inspection

∴ use (2) #4 bars @ 12" o.c. for horizontal reinforcing

John Vais | Tech 4 | Spot Check

Determine Required shear reinforcing

$\rho_\ell = A_v/sh \geq 0.0025 + 0.5\left(2.5 - \frac{120.83}{28}\right)(0.00278 - 0.0025) < 0.0025$

$\therefore$ use $\rho_\ell \geq 0.0025$ as minimum

Try (2) #4 bars

$s = A_v/\rho_\ell h = \frac{2(0.2)}{0.0025(12)} = 13.33'' \rightarrow$ use 12''

$\therefore$ use (2) #4 bars @ 12'' o.c. for vertical reinforcing

Design for Flexure

$M_u \leq \phi M_n = \phi A_s F_y I_d$

$12891(12) \leq (0.9) A_s (60)(0.9)(268.8) \rightarrow A_s = 11.84 in^2$

$a = \frac{A_s F_y}{0.85(f'c)(b)} = \frac{11.84(60)}{0.85(5)(12)} = 13.93''$

calc $I_d = d - a/2 = 268.8 - 13.9/2 = 261.8''$

recalc $A_s \rightarrow 12891(12) \leq (0.9) A_s (60)(261.8) = 10.94 in^2$

Assume 8 #11 bars $\rightarrow A_s = 8(1.56) = 12.48 in^2$

$A_{s\,prov} > A_{s\,req} \therefore$ ok

$c = a/\beta_1 = 13.93/0.80 = 17.41''$

$\varepsilon_t = 0.003\left(\frac{268.8 - 17.41}{17.41}\right) = 0.043 > 0.005 \therefore$ ok

$\therefore$ Actual design is confirmed (8 #11 bars)

John Vais | TECH 4 SpoT CHecks

Return Vall

$f'_c = 5000\ psi$

$M_u = 1.0(6916) - (780 \times 5)\,0.9 = 3406$ Kip-ft (OTM-SWM)

$V_u = 1.0(135.09) = 135.1$ Kips

$P_u = 780$ Kips (Sw + trib area)

controlling case - Quake case 2

$V_u \le \phi V_{n,max} = \phi 10 \sqrt{f'_c}\, hd$

$d = 0.8(120) = 96''$

$\phi V_n = 0.75(10)\sqrt{5000}(12)(96)/1000 = 610.9$ kips

$\phi V_n > V_u \therefore$ ok

shear strength of concrete

$V_c = 2\sqrt{f'_c}\, hd = 2\sqrt{5000}(12)(96)/1000 = 162.9$ kips

$\tfrac{1}{2}\phi V_c = \tfrac{1}{2}(0.75)(162.9) = 61.1$ kips $< V_u \therefore V_s$ required

Determine Reinforcing Needed

$V_u \le \phi V_n = \phi(V_c + V_s) \rightarrow 135.1 = 61.1 + \phi V_s \rightarrow V_s = 98.7$ Kips

$A_v/s = V_s/F_y d = \frac{98.7}{(60000)(96)} = 1.7\times10^{-5}$ in²/in

Try 2 #4 @ 12" $\rightarrow A_s = 0.40$ in²/ft

$\rho_t = \frac{A_v}{sh} = \frac{2(0.2)}{12 \times 12} = 0.00278 > 0.0025 \therefore$ ok

spacing is ok by inspection

$\therefore$ use (2) #4 bars @ 12" o.c. for horizontal reinforcing

John Vaiz | TECH 4 Spot Checks

Determine Required Shear Reinforcing

$$\rho_\ell = A_v/sh \geq 0.0025 + 0.5\left(2.5 - \frac{120.83}{10}\right)(0.00278 - 0.0025) < 0.0025$$

$\therefore$ use $\rho_\ell \geq 0.0025$ as minimum

Try (2) #4 bars

$$s = A_v/\rho_\ell h = \frac{2(0.2)}{0.0025(12)} = 13.33'' \rightarrow \text{use } 12''$$

$\therefore$ use (2) #4 bars @ 12" o.c. for vertical reinforcing

Design for flexure

$$M_u \leq \phi M_n = \phi A_s F_y \, l_d$$

$$3406(12) = (0.9) A_s (60)(0.4)(96) \rightarrow A_s = 8.76 \text{ in}^2$$

$$a = \frac{A_s F_y}{0.85(f'_c) b} = \frac{8.76(60)}{0.85(5)(12)} = 10.31''$$

$$\text{calc } l_d = d - a/2 = 96 - 10.31/2 = 90.85''$$

$$\text{recalc } A_s \rightarrow 3406(12) = (0.9)(A_s)(60)(90.85) \rightarrow A_s = 8.33 \text{ in}^2$$

Assume 8 #11 bars as designed $\rightarrow A_s = 8(1.56) = 12.48 \text{ in}^2$

$A_{s,prov} > A_{s,req}$ $\therefore$ ok

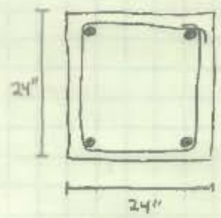
$$c = a/\beta_1 = 10.31/0.80 = 12.9$$

$$\varepsilon_t = 0.003\left(\frac{96 - 12.9}{12.9}\right) = 0.019 > 0.005 \therefore \text{ok}$$

$\therefore$ Actual design is confirmed (8 #11 bars)

J.M.V. | Tech 3 | Sys 1

Column Analysis



24"x24" (4)#11, #4 ties @ 22" oc

$f'_c = 4000$ psi $f_y = 60$ ksi 1.5" CLR

Check at level 6 [applies for typ. interior & exterior]

$A_s = 4(1.56) = 6.24 \text{ in}^2$

Pure Axial (P_o)

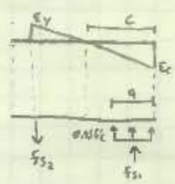
$\varepsilon_y = f_y/E_s = 0.00207$

$P_o = 0.85 f'_c (A_g - A_{st}) + A_s f_y = 0.85(4)(24\times24 - 6.24) + (60)(6.24) = 2311.6$ kips

Balanced Strain (M_b, P_b)

$c = \frac{0.003}{0.003+\varepsilon_y} d_2 = \frac{0.003}{0.003+0.00207}(24 - 1.5 - 1/2 - 1.41/2) = 12.6"$

$a = \beta_1 c = 0.85(12.6) = 10.7"$



$\varepsilon_{s1} = \frac{0.003}{12.6}(12.6 - 2.705) = 0.0023 > \varepsilon_y$ ∴ 60 ksi

$\varepsilon_{s2} = \frac{0.003}{12.6}(12.6 - 23.47) = -0.0025 > \varepsilon_y$ ∴ -60 ksi

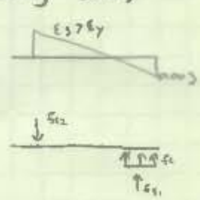
$P_b = 0.85(4)(0.85)(12.6)(24) + 2(1.56)(60) + 2(1.56)(-60)$

$P_b = 873.9$ kips

$M_b = 0.85(4)(0.85)(12.6)(24)\left[12 - \frac{10.7}{2}\right] + (3.12)(60)[12 - 2.7] + (3.12)(-60)[12 - 21.3]$

$M_b = 697.0$ kip-ft

Pure Bending (M_o)



$f_{s1} = \frac{0.003}{c}(c - 2.705)(29000)$ Assume f_{s1} does not yield and ε_{s2} does

$f_{s2} = -60$ ksi

$\Sigma F = 0: 0.85(4)(24)(0.85)c + 2(1.56)f_{s1} + 2(1.56)f_{s2}$

∴ $c = 2.70"$

$f_{s1} = -3.86 < 60$ ∴ ok

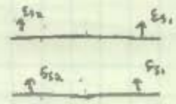
$\varepsilon_{s2} = \varepsilon_{s2} < -0.00207$ ∴ ok

$M_o = 0.85(4)(24)(0.85\times2.70)\left(12 - \frac{0.85\times2.70}{2}\right) + (3.12)(-3.86)(12-2.7) + (3.12)(-60)(12-21.3)$

$M_o = 305.11$ kip-ft

J.M.V. | TECH 3 | SYS 1

Pure Tension (T_o)



$T_o = \Sigma A_{si} f_{si}$ $\quad f_{si} = -60$ ksi

$= 4(1.56)(-60) = -374.4$ kips

Point Between P_o & P_b

choose $c = h = 24''$

$\varepsilon_{s_1} = \frac{0.003}{24}(24 - 2.7) = 0.00266 > \varepsilon_y \therefore f_{s_1} = 60$ ksi

$\varepsilon_{s_2} = \frac{0.003}{24}(24 - 21.7) = 0.0002875 < \varepsilon_y \therefore f_{s_2} = 8.34$ ksi

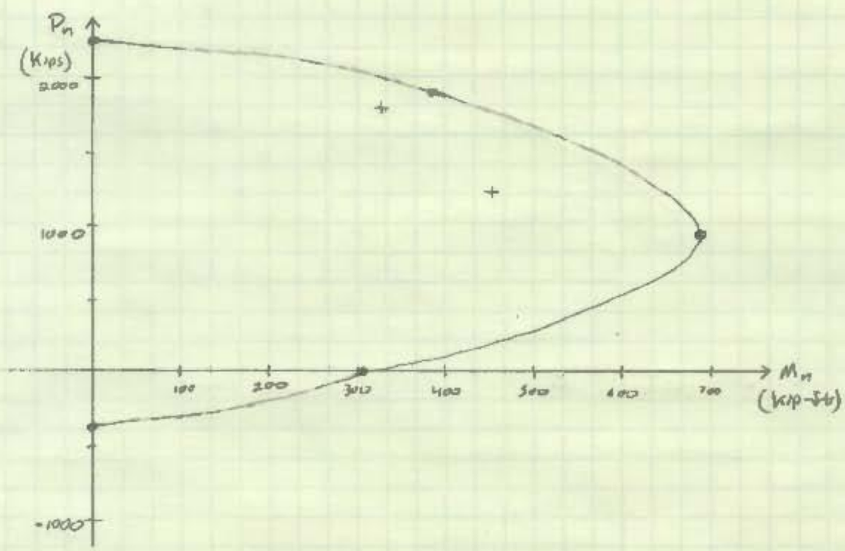
$P_n = 0.85(4)(24)(0.85)(24) + (3.12)(60) + (3.12)(8.34)$

$P_n = 1877.9$ kips

$M_n = 0.85(4)(24)(0.85)(24)\left[12 - \frac{0.85(24)}{2}\right] + (3.12)(60)(12 - 2.7) + (3.12)(8.34)(12 - 21.3)$

$M_n = 374.6$ kip-ft

Interaction Curve



J.M.V. | TECH 3 SYS 1

Column Capacity - Exterior Column (U-7)

Check for Max Combined Axial & Flexural under Gravity Loading

$P_u = 832$ kips [from Column takedown calculation]

$M_u = 283.7$ kip-ft [from PT beam calculation, let $M_u = a_2 M_o$]

Using interaction curve: Let $\phi = 0.65$ (compression controlled)

Let $P_u/\phi = P_n = 1280$ kips

Let $M_u/\phi = M_n = 436.5$ kip-ft

Point is within the curve ∴ ok

Column Capacity - Interior Column (S-3)

Check for Max Combined Axial & Flexure under Gravity Loading

$P_u = 1176.2$ kips [from column takedown calculation]

$M_u \approx 220.0$ kip-ft [estimated from beam & slab calculations]

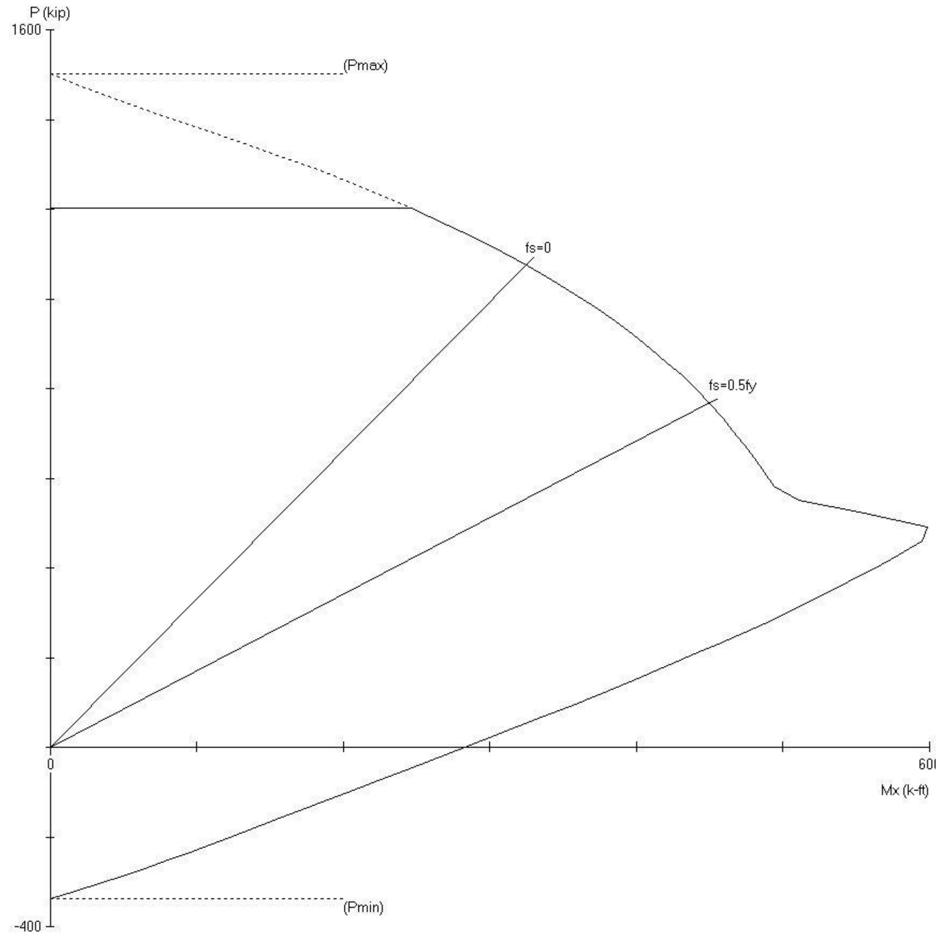
Using interaction curve: Let $\phi = 0.65$ [compression controlled]

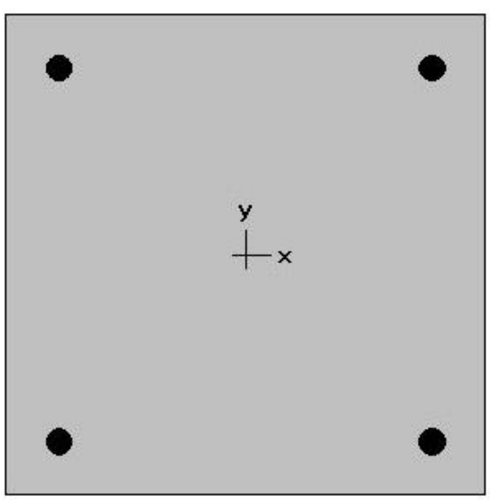
Let $P_u/\phi = P_n = 1809.5$ kips

Let $M_u/\phi = M_n = 338.5$ kip-ft

Point is within the curve ∴ ok

Column Interaction Diagram – Typical Exterior/Interior Column





24 x 24 in
1.08% reinf.

MATERIAL:
=========
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi

SECTION:
========
Ag = 576 in^2
Ix = 27648 in^4
Iy = 27648 in^4
Xo = 0 in
Yo = 0 in

REINFORCEMENT:
===============
4 #11 bars @ 1.083%
As = 6.24 in^2
Confinement: Tied
Clear Cover = 2.00 in
Min Clear Spacing = 17.18 in

```
STRUCTUREPOINT - spColumn v4.81 (TM)                                                    P
Licensed to: Penn State University. License ID: 59919-1033951-4-22545-2CF68             1
untitled.col                                                                            0


General Information:
====================
  File Name: untitled.col
  Project:
  Column:                                  Engineer:
  Code:     ACI 318-11                     Units: English

  Run Option: Investigation                Slenderness: Not considered
  Run Axis:   X-axis                       Column Type: Structural

Material Properties:
====================
  f'c   = 4 ksi                            fy   = 60 ksi
  Ec    = 3605 ksi                         Es   = 29000 ksi
  Ultimate strain = 0.003 in/in
  Beta1 = 0.85

Section:
========
  Rectangular: Width = 24 in               Depth = 24 in

  Gross section area, Ag =  576 in^2
  Ix =  27648 in^4                         Iy =  27648 in^4
  rx =  6.9282 in                          ry =  6.9282 in
  Xo =  0 in                               Yo =  0 in

Reinforcement:
==============
  Bar Set: ASTM A615
  Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)   Size Diam (in) Area (in^2)
  ---- --------- -----------   ---- --------- -----------   ---- --------- -----------
  #  3      0.38        0.11   #  4      0.50        0.20   #  5      0.63        0.31
  #  6      0.75        0.44   #  7      0.88        0.60   #  8      1.00        0.79
  #  9      1.13        1.00   # 10      1.27        1.27   # 11      1.41        1.56
  # 14      1.69        2.25   # 18      2.26        4.00

  Confinement: Tied; #3 ties with #10 bars,  #4 with larger bars.
  phi(a) = 0.8,  phi(b) = 0.9,  phi(c) = 0.65

  Layout: Rectangular
  Pattern: Sides Different  (Cover to longitudinal reinforcement)
  Total steel area: As = 6.24 in^2 at rho = 1.08%
  Minimum clear spacing = 17.18 in

                   Top       Bottom         Left        Right
                 --------   --------     --------     --------
    Bars         2  #11      2  #11        0  # 3       0  # 3
    Cover(in)         2           2            2            2

Control Points:
===============
                           Axial Load P      X-Moment      Y-Moment NA depth Dt depth    eps_t    Phi
  Bending about                     kip          k-ft          k-ft       in       in
  --------------------- ------------ ------------- ------------- -------- -------- -------- ------
   X @ Max compression         1502.5          0.00          0.00    68.62    21.30 -0.00207  0.650
     @ Allowable comp.         1202.0        246.08          0.00    23.72    21.30 -0.00031  0.650
     @ fs = 0.0                1074.8        324.90          0.00    21.30    21.30  0.00000  0.650
     @ fs = 0.5*fy              767.8        449.57          0.00    15.83    21.30  0.00103  0.650
     @ Balanced point           561.3        497.74          0.00    12.60    21.30  0.00207  0.650
     @ Tension control          482.0        605.74          0.00     7.99    21.30  0.00500  0.900
     @ Pure bending              -0.0        282.88          0.00     2.70    21.30  0.02064  0.900
     @ Max tension             -337.0         -0.00         -0.00     0.00    21.30  9.99999  0.900

  -X @ Max compression         1502.5          0.00          0.00    68.62    21.30 -0.00207  0.650
     @ Allowable comp.         1202.0       -246.08         -0.00    23.72    21.30 -0.00031  0.650
     @ fs = 0.0                1074.8       -324.90         -0.00    21.30    21.30  0.00000  0.650
     @ fs = 0.5*fy              767.8       -449.57          0.00    15.83    21.30  0.00103  0.650
     @ Balanced point           561.3       -497.74          0.00    12.60    21.30  0.00207  0.650
     @ Tension control          482.0       -605.74          0.00     7.99    21.30  0.00500  0.900
     @ Pure bending              -0.0       -282.88         -0.00     2.70    21.30  0.02064  0.900
     @ Max tension             -337.0         -0.00         -0.00     0.00    21.30  9.99999  0.900


  *** End of output ***
```

John Vais | Tech 4

Column Interaction

- Consider Columns on level 5 of structure @ Grid line A

  - M_{wind} = 34.5 kip-ft (max value)
  - P_{wind} = 16.4 kip
  - M_{Dead} ≈ 120 kip-ft [from Beam Analysis]
  - P_{Dead} ≈ 606 kips [from column TD's]

  $M_u = 1.6(34.5) + 0.9(120) = 163.2$ kip-ft

  $P_u = 1.6(16.4) + 0.9(606) = 571.64$ kips

- By inspection of column interaction curve, this column passes for this load, as the plotted point of (163 kip-ft, 572 kips) falls within the curve.

Note: Columns along Grid line A were chosen as they recieve the highest torsional load / deflect the most. This is due to their being farthest from the center of rigidity. Columns on an elevated floor were chosen as their values in the Etabs models were considered more relyable relative to lower floors (where the pin assumption incorrectly influences results).

Appendix E

Building Plans and Elevations

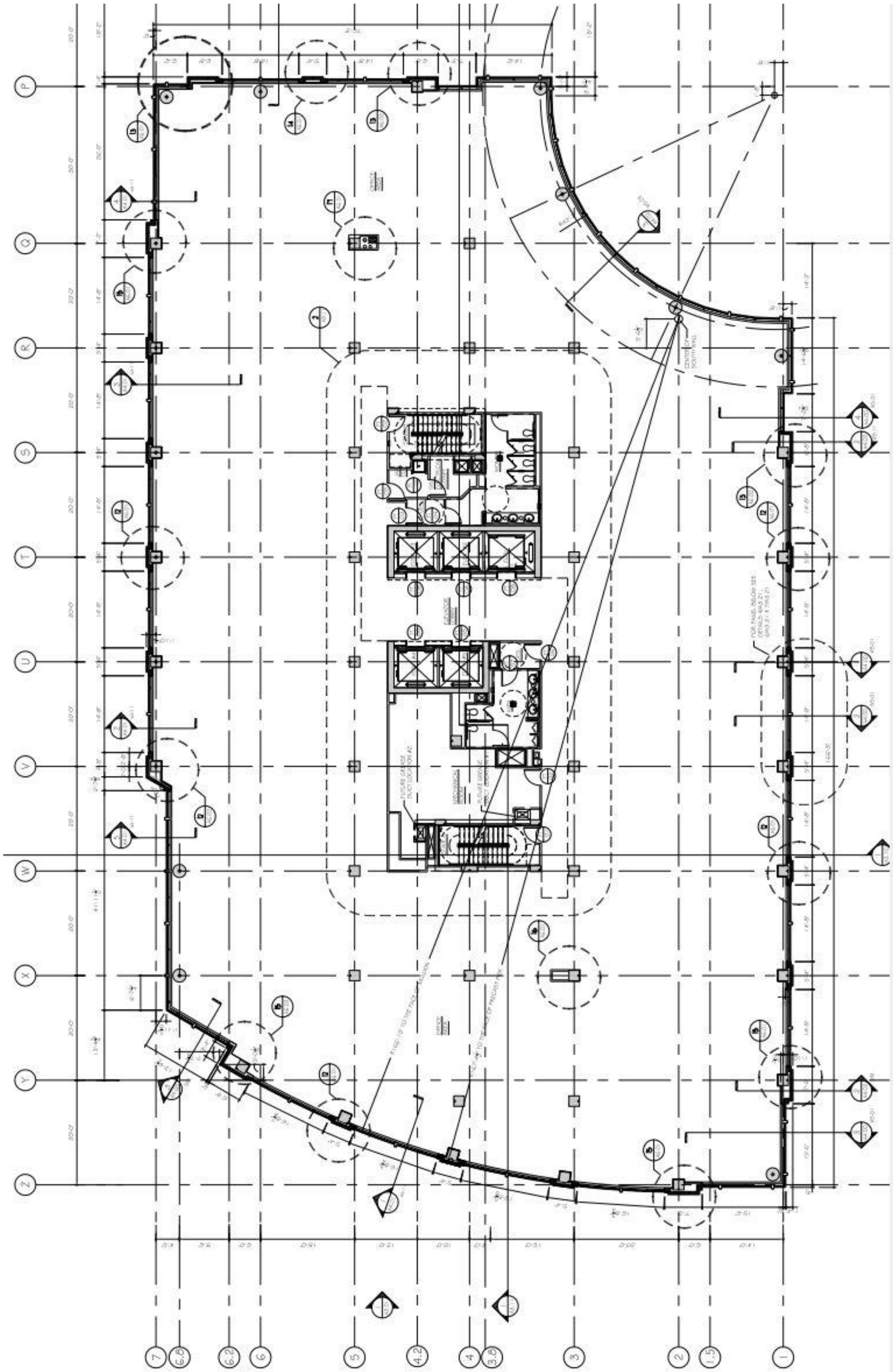


Figure E.1: Typical Office Floor Plan – A2.19 of Construction Documents

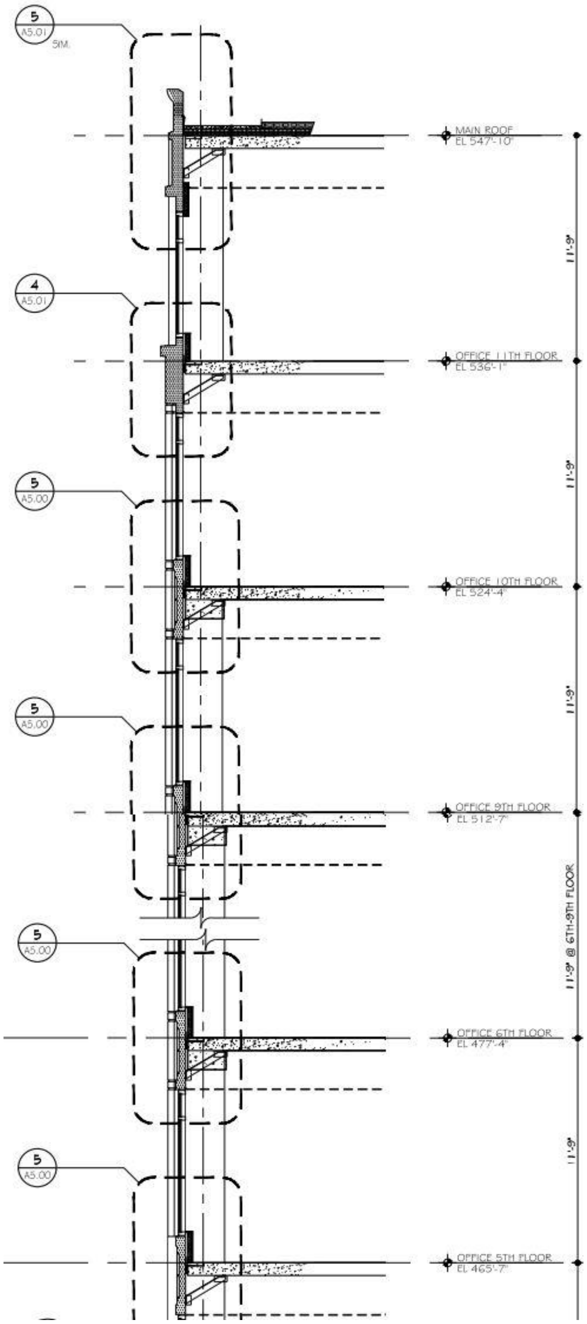


Figure E.2: Wall Section – A4.05 of Construction Documents

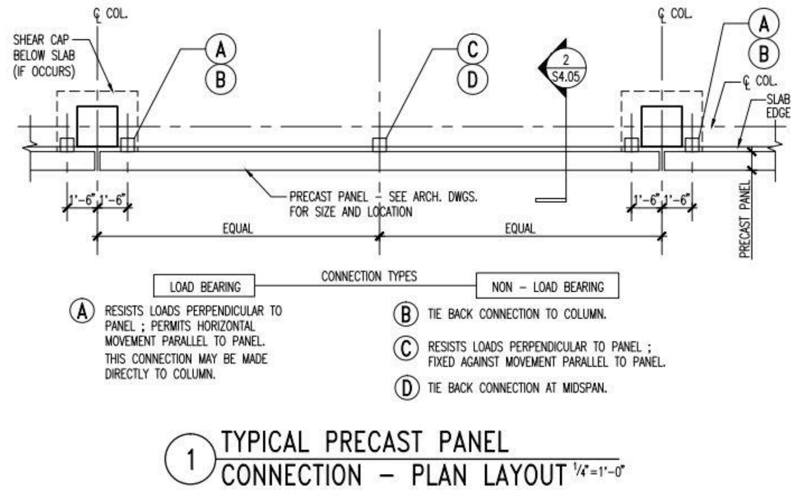


Figure E.3: Precast Connection Plan – S4.01 of CD's

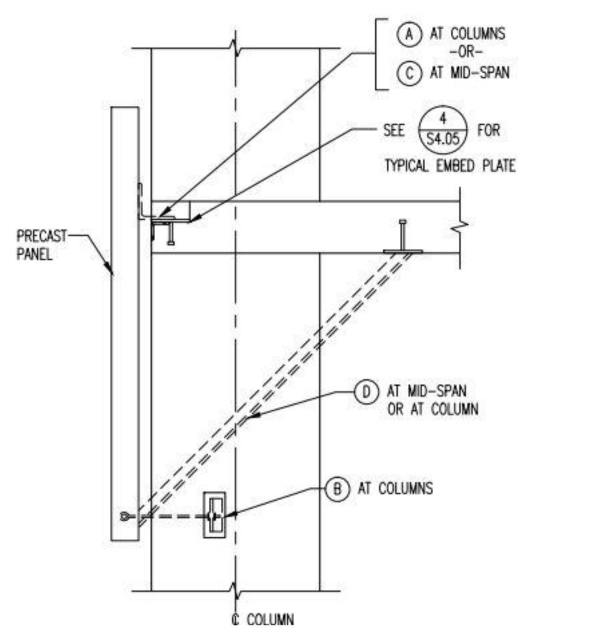


Figure E.4: Precast Connection Detail – S4.01 of CD's

Appendix F

Photos

Figure F.1: Decorative Precast Panel – by JMV

Figure F.2: North East Curtain Wall – by JMV

Figure F.3: Unfinished Retail Space – by JMV

Figure F.5: Projection of Post Tension Beam – by JMV

Figure F.4: South West Corner – by JMV