Park Potomac Office Building "E"

Technical Assignment #3

Potomac, MD

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Executive Summary

The purpose of this report is to examine the lateral system of Park Potomac Office Building "E" under wind and seismic loading. Office Building "E" is a seven story, roughly 100 foot tall office building located in Potomac, MD. The seven office levels are each roughly 25,000 square feet and sit on top of two large levels of mostly underground parking. For this analysis, the underground levels were assumed to not contribute to either the wind or seismic forces, resulting in an analysis of only the office levels.

Office Building "E" uses two main systems to resist lateral forces. As a post tensioned concrete structure, concrete moment frames are used in both the N-S and E-W directions. Shear walls are also used at the building core to provide resistance in the N-S direction.

Wind and seismic forces were updated and confirmed from Technical Report #1. Several overly conservative assumptions were reconsidered and more accurate values were found for use in this report. A computer model was utilized to analyze the results of these loads on the structure. Seven load cases were considered from ASCE7-05, along with four main wind scenarios as well. Every likely combination was considered. It was determined that 0.9D + 1.6 W principally controlled, with Wind Case 1 being the critical wind scenario.

Consideration of the building's center of mass and center of rigidity revealed that in a structure that is symmetrical about its x and y axes, in both size and stiffness, it was determined that both of these points lie at the geometric center of the structure. This, along with the controlling load case being Wind Case 1, resulted in negligible overall building torsion.

A 1000K load was applied to the top of the structure and was used to determine the relative stiffness of each element at each level. In general, it was found that the centrally located, longer moment frames took more force that the shorter outside frames. It was also found that the shear walls have a larger relative stiffness at the base, where shear is more critical than flexure.

Overall building drift and story drift were considered for wind and were found to be well within the limit of L/400. Seismic drift also fell within the allowable limits from ASCE7.

Overturning was also considered in this analysis. The most critical shear values at each level were determined and the overall moments due to the applied forces were found. When compared with the building dead load moments, foundation uplift did not occur.

Introduction

Park Potomac Office Building “E” is located prominently off I-270 at Seven Locks and Montrose Roads. It is just one of several planned office buildings that are part of an “urban village” which mixes stunning town homes, Class A office space, and a wide range of amenities including dining and shopping.

Office Building “E” is a central part of the Park Potomac Master Plan. Its central location, at the end of Cadbury Avenue, makes it a focal point for this small community (Figure 1). It also puts it right at the main courtyard that will be a retail gathering point as well.

Figure 1: View from Cadbury Ave.

Material Strength Summary

Concrete:

Footings	3000 psi
Foundation Walls	4000 psi
Columns	Varies
Slab-on-Grade	3500 psi
Reinforced Slabs & Beams	5000 psi
Parking Structure	5000 psi
P.T. Concrete	5000 psi

Structural Steel:

Wide Flanges & Tees	ASTM A992, Fy = 50 ksi
Square/Rectangular Hollow Shapes	ASTM A500, Grade B, Fy = 46 ksi

Masonry:

Compressive Strength	1500 psi

Codes & Design Standards

Original Design:

a. "The International Building Code – 2003", International Code Council

b. "Minimum Design Loads for Buildings and Other Structures" (ASCE7-02), American Society of Civil Engineers

c. "Building Code Requirements for Structural Concrete, ACE 318-02", American Concrete Institute

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

e. "Manual of Standard Practice", Concrete Reinforcing Steel Institute

f. "Post Tensioning Manual", Post Tensioning Institute

g. "Manual of Steel Construction- Allowable Stress Design", Ninth Edition, 1989, American Institute of Steel Construction (Including specifications for structural steel buildings, specifications for structural joints using ASTM A325 of A490 bolts and AISC Code of Standard Practice)

Substituted for thesis analysis:

a. "The International Building Code – 2006", International Code Council

b. "Minimum Design Loads for Buildings and Other Structures" (ASCE7-05), American Society of Civil Engineers

c. "Building Code Requirements for Structural Concrete, ACI 318-08", American Concrete Institute

Existing Structural System

Foundations:

Park Potomac Office Building "E" consists of a seven story office building (Approx. 100' high) that sits above two levels of underground parking. The parking structure levels have a footprint of over 103,000 sq. ft. This is much larger than the office structure, which has a footprint of just more than 25,000 sq. ft.

This relationship has a large impact on the design of the foundation as well. The net allowable bearing pressures for the site are 4000 psi for undisturbed soil and 3,000 psi for foundations place on compacted structural fill. Over 150 spread footings are used throughout the project (Figure 2). All footings are 3000 psi concrete, and foundation walls are 4000 psi concrete. Spread footings, mostly ranging from 10' x 10' to 12' x 12', are used beneath the two levels of parking with no office building above. The majority of these footings are between 28" and 34" deep.

Larger mat footings are used in the center of the project, taking load from the two parking levels and also from the office building above. These larger foundations are up to 52' x 64' in size and can be up to 62" deep.

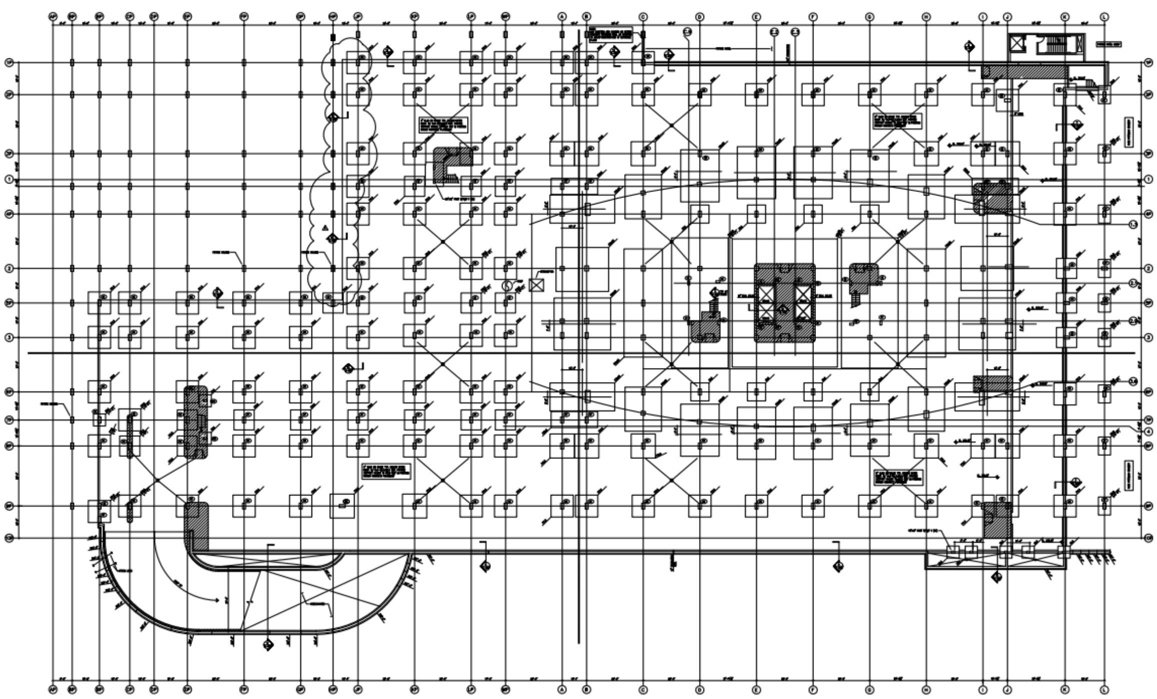

Figure 2: Foundation Plan

Floor System:

The slab on grade at the P2 Parking Level is a 5" thick, 3500 psi concrete slab. It is reinforced with 6x6 – W2.0 x W2.0 welded wire fabric. All other slabs contain 5000psi concrete. Two-way flat slabs are used at the P1 Parking level and the Plaza/First Floor Level as well. The slab is 8" thick at the P1 Level and 12" thick at the Plaza/First Floor Level. These slabs are reinforced as needed to resist negative moment at the columns and positive moments at midspan. Post-tensioning is not used on the parking levels. Tying a post-tensioned slab into foundation walls or other fixed structure does not allow the post-tensioned slab to shrink when stressed. This would result in cracking of the slab if post-tensioning was used below grade. Using this method for the parking garage would also lead to difficulty in stressing the tendons as well. The designers of Office Building "E" use mild reinforcing below grade, and post-tensioning for the slabs above grade.

Above the Plaza Level, Office Building "E" has seven levels of office floors. These floors are 7" thick post-tensioned slabs. The post-tensioning cables induce forces in the slab ranging from 12.5 k/ft up to 35 k/ft. The post-tensioning system uses grouped tendons in the 20" beams in the E-W direction, and a one way slab with uniform tendon layout in the N-S direction. This design allows for ease of construction when laying out the tendons. The post-tensioned slab also allows for cantilevers that exist at the North and South ends of the structure. The load from a 12' cantilever on each end is taken by the uniformly spaced tendons that run through the slab.

Post-tensioning is key to achieving several main goals on this project. The first main goal is that it allows for large spans in the floor layout. The design of this project requires that columns be placed around the exterior walls of the building and the interior core as well. This requires the beams and slab to span long distances over the floor. Post-tensioning achieves these span requirements while maintaining a slab thickness of just 7 inches. Deflection over these spans is controlled effectively, while cracking is reduced as well.

Several steel shapes are utilized on the second floor slab to frame out the canopies above the East and West building entrances. This framing consists of TS5x2 shapes that are welded to ¾" plates and hung from the bottom of the slab by L4x4 angles. Steel shapes (W8x10) are also utilized as elevator rail supports throughout all floors.

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Gravity System:

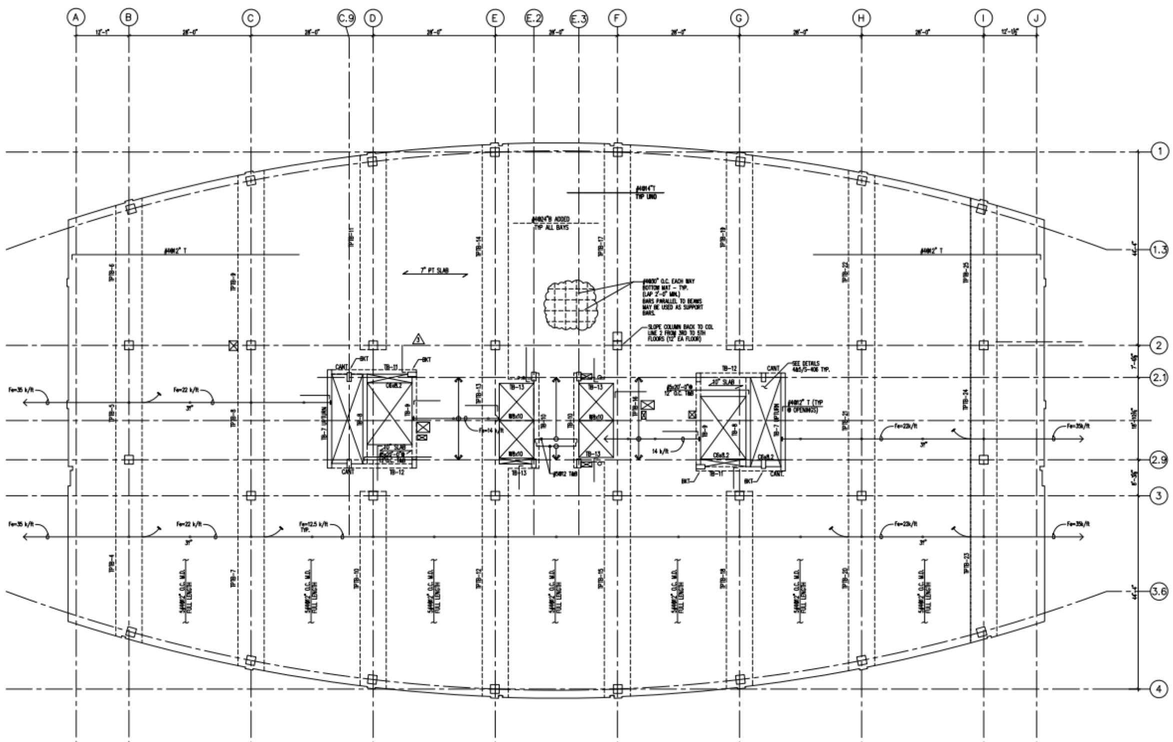

Figure 3: Typical Framing Plan

The majority of the columns in the two levels of parking are 18" x 36" columns reinforced with 10 #9 bars. These columns are typically spaced between 15' and 30' apart. Columns supporting only the two parking levels consist of 4000 psi concrete, while 6000 psi concrete is utilized where load from the office building portion above is carried. Columns in the parking levels utilize drop panels to spread the load and resist punching shear.

In the office portion of the project, a relatively repetitive column layout is achieved. Excluding the central building core, 32 columns are used to transfer the load down through all seven levels. Long span post-tensioned beams are used to transfer load from the floor to the columns. At typically 20" x 72" in size, these shallow, wide beams span in the E-W direction and continue the entire building width. In order to minimize the amount of columns in the tenant spaces and promote flexible space planning, large spans up to nearly 45' exist on each floor.

Columns on the office levels are 24" x 24" at every level and the concrete strength is varied throughout the levels to support an increased load as required. The plaza level through the fourth floor use 5000 psi concrete, while 4000 psi concrete is used above the fourth floor.

Lateral System:

Park Potomac Office Building "E" uses concrete moment frames, as well as shear walls to resist lateral forces. In the E-W direction, the wide post-tensioned beams on each floor create a series of parallel frames that run up through all seven floors. These frames resist any lateral forces on the building in the parallel direction.

Similarly, forces in the N-S direction are resisted by concrete moment frames as well as by four shear walls. The concrete columns and the 7" slab, which is post-tensioned in the N-S direction, combine to create a frame that resists later forces in this direction as well.

The overall lateral system and load distribution of lateral forces will be described in further detail later in this report.

Roof System:

The main roof system consists of a 7" to 8" structural slab. This slab varies in order to create the required roof slopes throughout. The roof contains a Penthouse/Mechanical space, as well as an elevator machine room. The penthouse roof is an 8" two way flat plate system, while the elevator machine room utilizes a 12" thick slab.

TS8x8 posts and TS 6x6 supports are used to frame a 16' tall screen-wall on the roof level to isolate the mechanical spaces from view.

- The remainder of this report will provide loads, a description of load path, and a discussion of load combinations. Lateral force distribution, drift, torsion, and overturning will also be addressed.

Gravity Loads

Floor live loads were determined using ASCE 7-05. These loads were then compared to the design loads used in the original design. The design loads were largely the same as those from ASCE 7-05. A few of the loads used exceeded the required loadings from ASCE 7-05. These loads can be found below.

Table 1: Floor Live Loads

Area	Design Load (psf)	ASCE 7-05 Load (psf)
Assembly Areas	100	100
Corridors	100	100
Corridors Above First Floor	80	80
Lobbies	100	100
Marquees & Canopies	75	75
Mechanical Rooms	150	125
Offices	80 + 20 psf Partitions	50 + 20 psf Partitions
Parking Garages	50	40
Plaza, Top Floor Parking	Fire Truck Load or 250 psf	250
Retail- First Floor	100	100
Stairs and Exitways	100	100
Storage (Light)	125	125

The following superimposed dead loads were also considered in the design of the structure.

Table 2: Superimposed Dead Loads

Area	Design Load (psf)
Floors	5
Roof	10

A flat roof snow load was calculated for this report as well. Beginning with a 30 psf ground snow load for Montgomery County, a flat roof snow load of 21 psf was calculated using the variables shown below from ASCE 7-05. This snow load of 21 psf was identical to the design snow load used by Cagley & Associates. Snow drift loads will occur on the roof level around the screen walls; however, this drift loading was not examined in this report.

Table 3: Flat Roof Snow Load

Ground Snow Load	P_g=	30	psf
Snow Exposure Factor	C_e=	1.0	
(Terrain Category B)			
Thermal Factor	C_t=	1.0	
Importance Factor	I=	1.0	
Flat Roof Snow Load	**p_f**	**21**	**psf**

Wind Loads

Method 2, detailed in Chapter 6 of ASCE 7-05, was used to determine the wind loading for the building. Wind loadings in the N-S and the E-W directions were both analyzed. Detailed calculations can be found in Appendix A of this report. The analysis revealed the uniform pressures that occurred due to wind, which allowed the base shears and overturning moments to be determined as well.

In the E-W direction, the parking levels are completely below grade. The entrance is at the Plaza Level on the East and West sides of the building. This is reflected in the analysis. Roof uplift forces were not considered in this analysis. Unfactored wind forces and loading diagrams can be found below.

Table 4: East- West Design Pressures

Level	Height (ft above Plaza)	Design Pressure Windward (psf)	Design Pressure Leeward (psf)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)
Plaza Level	0	6.83	-7.26	14.09	28.38	415.90
	9	6.83	-7.26	14.09		
2nd Floor	18	7.26	-7.26	14.52	49.54	387.52
	24.25	7.90	-7.26	15.16		
3rd Floor	30.5	8.44	-7.26	15.70	43.91	337.98
	36.75	8.90	-7.26	16.16		
4th Floor	43	9.31	-7.26	16.57	46.34	294.07
	49.25	9.67	-7.26	16.94		
5th Floor	55.5	10.01	-7.26	17.27	48.31	247.73
	61.75	10.32	-7.26	17.58		
6th Floor	68	10.61	-7.26	17.87	49.98	199.43
	74.25	10.88	-7.26	18.14		
7th Floor	80.5	11.13	-7.26	18.39	52.48	149.44
	87	11.38	-7.26	18.64		
Main Roof	93.5	11.62	-7.26	18.88	27.46	96.97
Penthouse	109.5	12.16	-7.26	19.42	69.51	69.51

Base Shear	**416**	**K**

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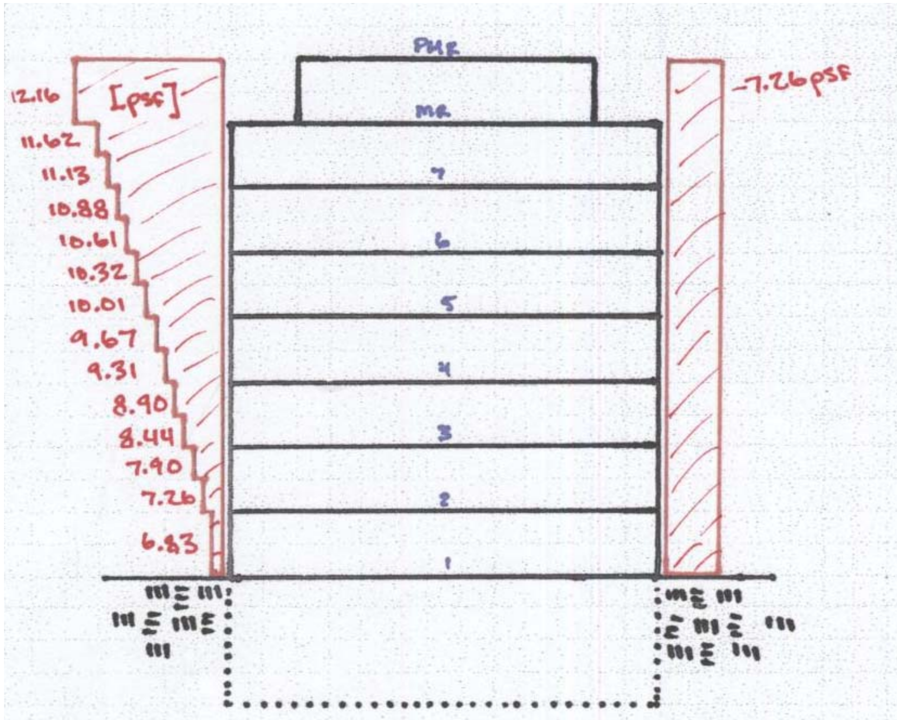

Figure 4: East – West Design Pressures

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Analysis results for the N-S wind direction can be found below. The parking level was not considered in this analysis because it was assumed that the wind would be negligible due to the existing grade and the site layout at these locations. Unfactored results and loading diagrams can be found below for the N-S wind direction.

Table 5: North-South Design Pressures

Level	Height (ft above Plaza)	Design Pressure Windward (psf)	Design Pressure Leeward (psf)	Total Pressure (psf)	Force of Total Pressure (k)	Story Shear Total (k)
Plaza Level	0	6.83	-5.08	11.92	13.67	206.58
	9	6.83	-5.08	11.92		
2nd Floor	18	7.26	-5.08	12.34	23.99	192.91
	24.25	7.90	-5.08	12.98		
3rd Floor	30.5	8.44	-5.08	13.52	21.55	168.92
	36.75	8.90	-5.08	13.98		
4th Floor	43	9.31	-5.08	14.39	22.93	147.37
	49.25	9.67	-5.08	14.76		
5th Floor	55.5	10.01	-5.08	15.09	24.06	124.43
	61.75	10.32	-5.08	15.40		
6th Floor	68	10.61	-5.08	15.69	25.01	100.38
	74.25	10.88	-5.08	15.96		
7th Floor	80.5	11.13	-5.08	16.22	26.36	75.37
	87	11.38	-5.08	16.47		
Main Roof	93.5	11.62	-5.08	16.70	13.84	49.01
Penthouse	109.5	12.16	-5.08	17.24	35.17	35.17

Base Shear	**207**	**K**

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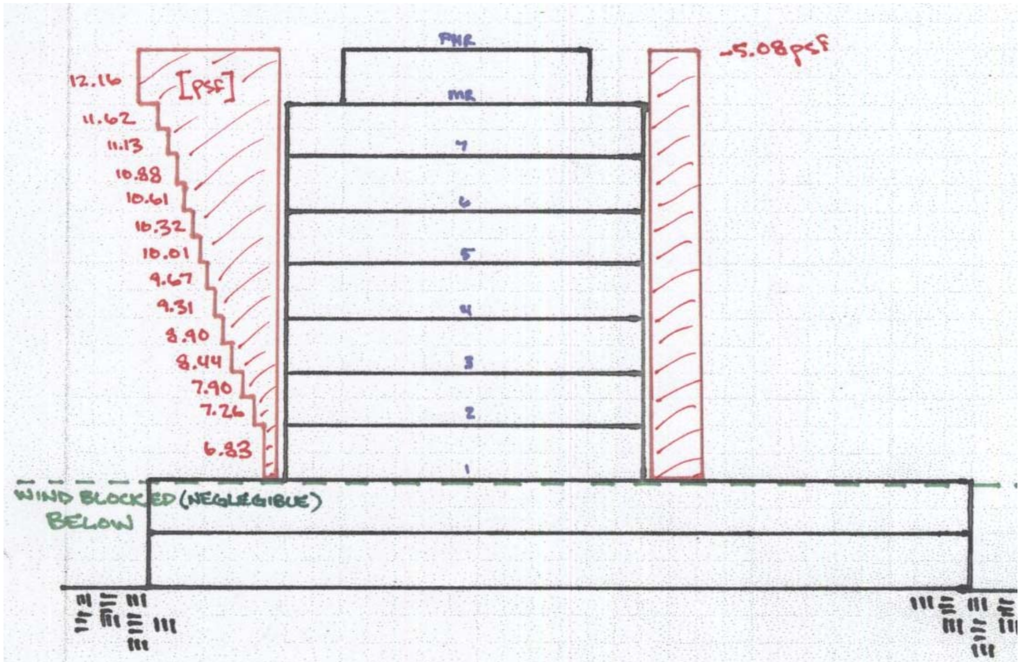

Figure 5: North – South Wind Pressures

Seismic Loads

The layout of the parking levels and the surrounding ground created unique seismic considerations for Office Building "E". The two levels of underground parking were mostly below grade, except on the North side of the structure. This scenario can be seen below.

Figure 6: View from North

Although it is evident that the parking levels are partially exposed on the North side, it will be assumed for this analysis that the seismic base level will be at the plaza level for the structure. This is due to the fact that the parking levels are largely below grade and will act as being mostly fixed. This assumption was confirmed by results obtained in Technical Report #1. For this report, only the office levels will be considered for seismic in both directions.

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The seismic analysis in this report was completed using Chapters 11 and 12 from ASCE 7-05. The equivalent lateral force procedure was determined to be valid for this analysis. Detailed calculations, including building self weights and other variables, are available in Appendix B. The main variables used in the analysis are shown below.

Table 6: Seismic Design Variables

			ASCE Reference
Soil Classification		D	Table 20.3-1
Occupancy		II	Table 1-1
Importance Factor		1.0	Table 11.5-1
Structural System		Ordinary Moment Frames	Table 12.2-1
Spectral Response Acceleration, Short	S_s	0.156	USGS Website
Spectral Response Acceleration, 1 s	S_1	0.051	USGS Website
Site Coefficient	F_a	1.6	Table 11.4-1
Site Coefficient	F_v	2.4	Table 11.4-2
MCE Spectral Response Acceleration, Short	S_{MS}	0.2496	Eq. 11.4-1
MCE Spectral Response Acceleration, 1 s	S_{M1}	0.1224	Eq. 11.4-2
Design Spectral Acceleration, Short	S_{DS}	0.166	Eq. 11.4-3
Design Spectral Acceleration, 1 s	S_{D1}	0.081	Eq. 11.4-4
Seismic Design Category	S_{DC}	B	Table 11.6-2
Response Modification Coefficient	R	3	Table 12.2-1
Approximate Period Parameter	C_t	0.016	Table 12.8-2
Building Height (E-W)	h_n	109.5'	
Building Height (N-S)	h_n	130.5'	
Approximate Period Parameter	x	0.9	Table 12.8-2
Fundamental Period (E-W)	T	1.9745 s	Eq. 12.8-7
Fundamental Period (N-S)	T	2.2705 s	Eq. 12.8-7
Long Period Transition Period	T_L	8.0 s	Fig. 22-15
Seismic Response Coefficient (E-W)	C_s	0.0137	Eq. 12.8-2
Seismic Response Coefficient (N-S)	C_s	0.0119	Eq. 12.8-2
Structure Period Exponent (E-W)	k	1.298	Sec 12.8.3
Structure Period Exponent (N-S)	k	1.392	Sec 12.8.4

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After calculation of the overall building self weight (See Appendix B), base shears can be calculated in order to calculate the forces on the structure. These base shears are shown below in Table 7. The base shears obtained were similar in magnitude to the value of 300K calculated by the design engineer. The values calculated in this report will be used for further analysis.

Table 7: Base Shears			
	Effective Seismic Weight	Seismic Response Coefficient	Base Shear (K)
N-S	W = 26896 K	C_s = 0.0119	322
E-W	W = 26896 K	C_s = 0.0137	371

After the calculation of the base shear values for each direction, the forces can be distributed throughout the building to determine forces at each level and story shear values. These are all unfactored values.

Table 8: Seismic Calculations							
Level	Story Weight (K)	N-S Height (ft)	E-W Height (ft)	N-S Forces (K) Fx	E-W Forces (K) Fx	N-S Story Shear Vx	E-W Story Shear Vx
Penthouse Roof	557	109.5	109.5	8	10	0	0
Main Roof	3837	93.5	93.5	90	100	8	10
7th Floor	3751	80.5	80.5	71	80	97	110
6th Floor	3737	68	68	56	64	168	189
5th Floor	3737	55.5	55.5	42	49	224	253
4th Floor	3737	43	43	29	35	266	302
3rd Floor	3737	30.5	30.5	18	22	295	337
2nd Floor	3800	18	18	9	12	313	359
Plaza/First Floor	19014	0	0	0	0	322	371
Total:	**45910**	**130.5**	**109.5**	**322**	**371**		

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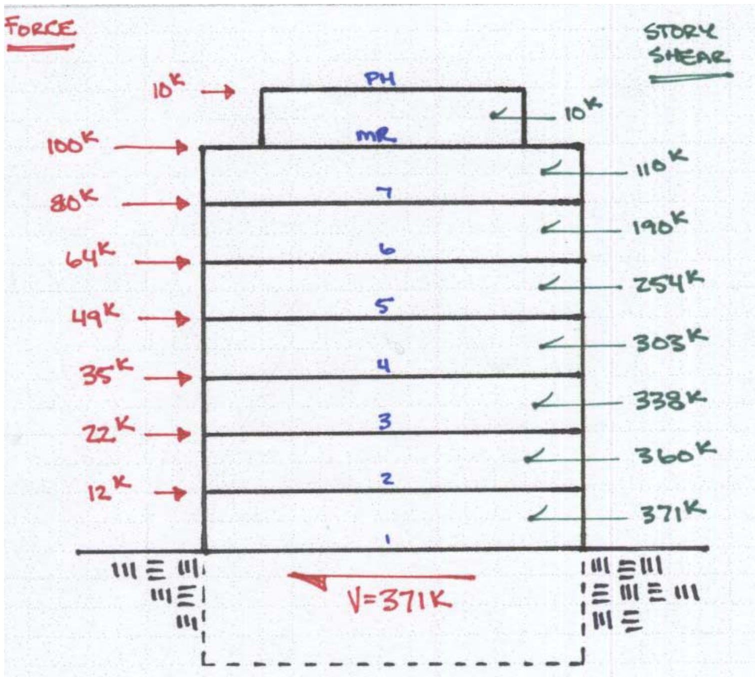

Figure 7: East- West Seismic Forces

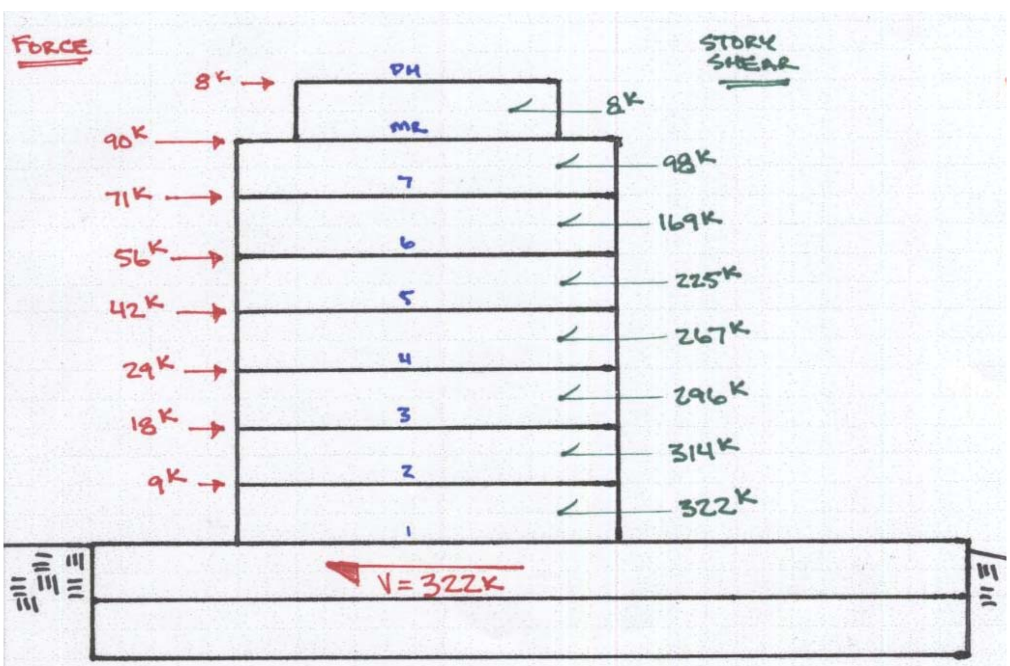

Figure 8: North- South Seismic Forces

Load Path

In the N-S direction, lateral forces are resisted largely by concrete moment frames and by four shear walls at the building core. The moment frames are made up of concrete columns and a 7" thick post tensioned slab that runs one way in the N-S direction.

In the E-W direction, concrete moment frames resist all of the lateral force. These frames are made up of concrete columns, as well as the shallow 72" wide beams that span along the E-W direction.

In both directions, the floor diaphragm transfers lateral forces to the moment frames at each level. The building columns transfer these loads down the building through shear and axial column forces. This process continues throughout the building and down to the foundations, where the forces are transferred to the soil.

A basic plan of the lateral system is shown below in Figure 9. Frames in the N-S direction are shown in red. Frames in the E-W direction are shown in green. Shear walls are shown in blue and the four miscellaneous columns are represented in orange.

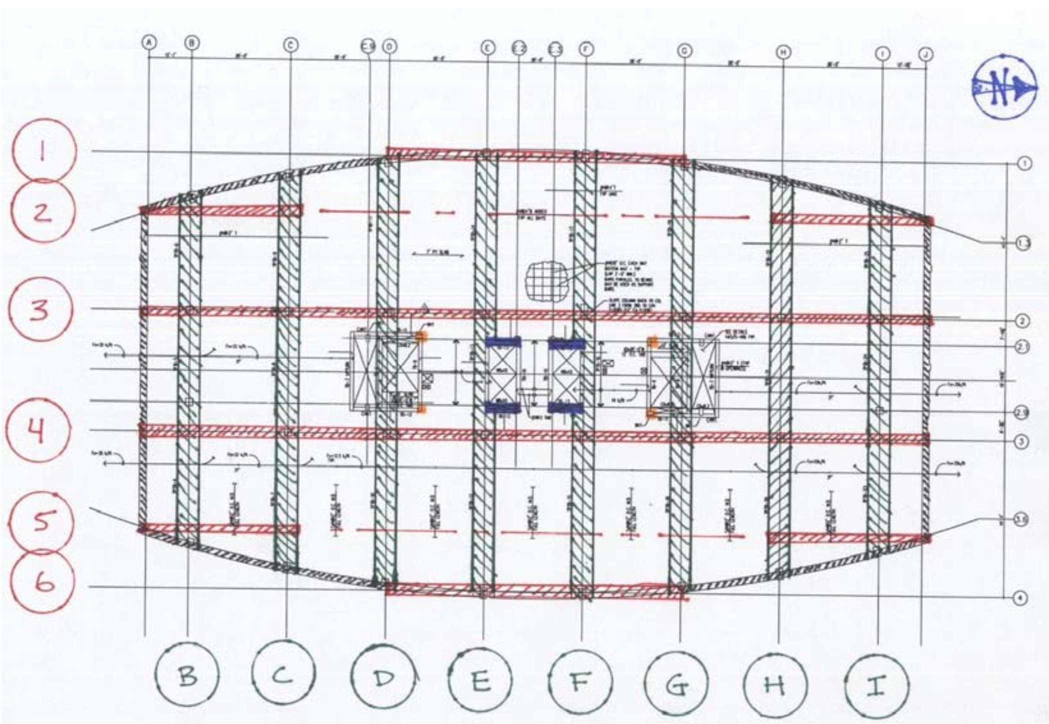

Figure 9: Lateral System Components

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Load Combinations

Per ASCE 7-05 Section 2.3.2, seven load combinations must be considered when dealing with strength design. They are outlined below:

1. 1.4(D + F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5(Lr 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

The following four wind cases were also considered from ASCE7-05 Figure 6-9 shown below. Case 1 proved to be the most critical case after analyzing all combinations.

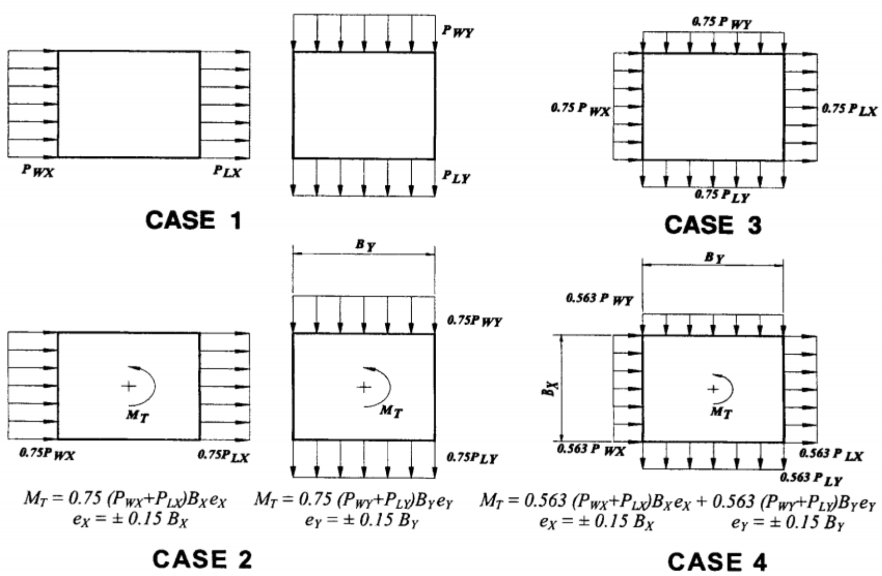
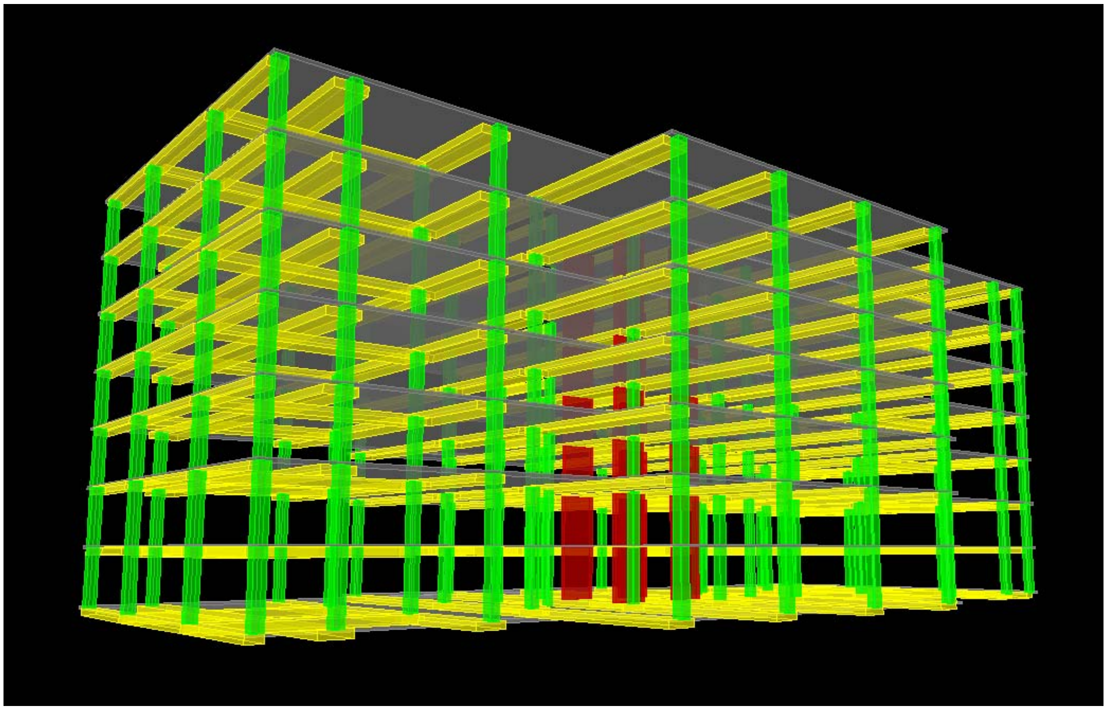


Figure 10: ASCE 7-05 Wind Cases

After analyzing the required load combinations using ETABS and checking the forces and deflections in the different load combinations, it is apparent that for both the N-S and the E-W directions, 0.9D + 1.6 W predominantly controls. This is expected due to the relatively low seismic location. It is also expected that this combination would control over load combination four, due to the fact that a smaller building weight would have less resistance to wind forces, making it more critical.

ETABS Model



A computer model of the structure was used to analyze the lateral system and the forces acting on the structure. ETABS, a computer modeling program from Computers & Structures, Inc. was used for the analysis. In most cases, only the lateral resisting elements would need to be modeled to gain an accurate representation of a building's performance under lateral loading. In this case, nearly all of the building elements needed to be modeled due to the full building participation in resisting lateral forces.

For this model, the overall shape of the building was simplified by squaring off the curved sides, allowing for simpler modeling and analyzing of results. Moment frames were considered linearly, as shown in Figure 9 (Lateral System Components). This simplification will have negligible results on the data output for the lateral system. The model largely consisted of columns, beams, and slabs; however, the small shear walls and coupling beams near the elevator shafts were also included. Modifiers of 0.35 and 0.70 were used for the moment of inertia for beams and columns respectively to account for cracking in the concrete members. All load cases and combinations considered were manually added to the model. The most critical of each was used in the calculations for this report.

Distribution of Lateral Forces

The lateral system design, as well as the overall building shape and floor plans are fairly basic for this structure. The building is symmetrical in shape about its x and y axes. This results in a center of mass located directly in the center of the structure. Similarly, the lateral system is symmetrical as well, both in location and in stiffness of the frames. This creates a center of rigidity located at the building's center, at the same point as the center of mass. These two centrally located points result in negligible eccentricities caused by seismic and concentric wind forces, which eliminates overall building torsion due to these loadings. Building torsion will need to be considered for the eccentrically loaded wind cases, as well as the accidental moment caused by eccentric seismic forces.

Lateral loads were assumed to be distributed throughout the floor by way of a rigid floor diaphragm. This means that at each level the deflections at each point will be the same due to the support of an infinitely rigid floor. This means that determining the relative stiffness of each frame must be done using the stiffness of each frame, rather than the use of tributary floor widths. The stiffer frames will resist more force than less stiff frames. This basic theory was used to determine the relative stiffness of each frame in the N-S and E-W directions.

In order to determine the relative stiffness of each frame, a 1000K load was applied to the top building level in each direction. Section cuts were used in ETABS to determine the shear forces in the columns at each frame. It was confirmed that the sum of all shears at every level was equal to the story shear, or 1000K. This confirmed that all resistive forces were accounted for on all levels. From these forces, the relative stiffnesses were determined for each frame by examining the percentage of the total 1000K that the frame resisted. This basic method was completed in both directions. The results can be found in the following tables:

Table 9: Resisting Forces (X/ N-S)

Level	Frame @ 1	Frame @ 2	Frame @ 3	Walls @ 3	Walls @ 4	Frame @ 4	Frame @ 5	Frame @ 6	4 Misc Col.	Total Force (K)
1	8.34	6.89	-32.25	-475.80	-475.58	-32.14	7.18	8.60	-15.29	-1000
2	-39.40	-72.04	-150.78	-214.51	-219.54	-150.51	-71.88	-39.27	-41.61	-1000
3	-33.37	-66.78	-168.68	-203.20	-207.98	-168.45	-66.64	-33.27	-51.16	-1000
4	-37.22	-74.11	-185.97	-171.91	-176.61	-185.73	-73.99	-37.13	-57.27	-1000
5	-38.37	-76.84	-193.39	-159.52	-163.76	-193.13	-76.73	-38.28	-59.91	-1000
6	-35.10	-69.88	-184.02	-180.70	-184.25	-183.76	-69.75	-35.00	-57.22	-1000
7	-48.09	-96.84	-226.61	-92.08	-94.92	-226.27	-96.77	-48.02	-69.93	-1000

Table 10: Relative Stiffness (X/ N-S)										
Level	Frame @ 1	Frame @ 2	Frame @ 3	Walls @ 3	Walls @ 4	Frame @ 4	Frame @ 5	Frame @ 6	4 Misc Col.	Total Percent
1	-0.8	-0.7	3.2	47.6	47.6	3.2	-0.7	-0.9	1.5	100
2	3.9	7.2	15.1	21.5	22.0	15.1	7.2	3.9	4.2	100
3	3.3	6.7	16.9	20.3	20.8	16.9	6.7	3.3	5.1	100
4	3.7	7.4	18.6	17.2	17.7	18.6	7.4	3.7	5.7	100
5	3.8	7.7	19.3	16.0	16.4	19.3	7.7	3.8	6.0	100
6	3.5	7.0	18.4	18.1	18.4	18.4	7.0	3.5	5.7	100
7	4.8	9.7	22.7	9.2	9.5	22.6	9.7	4.8	7.0	100

Table 11: Resisting Forces (Y/ E-W)										
Level	Frame @ B	Frame @ C	Frame @ D	Frame @ E	Frame @ F	Frame @ G	Frame @ H	Frame @ I	Misc. Col/Wall	Total Force (K)
1	-117.93	-113.99	-100.48	-115.28	-115.29	-100.16	-114.11	-118.16	-104.61	-1000
2	-129.33	-123.68	-88.70	-120.07	-120.09	-88.80	-123.71	-129.36	-76.25	-1000
3	-127.28	-125.13	-91.67	-116.43	-116.44	-91.71	-125.16	-127.34	-78.83	-1000
4	-128.57	-126.25	-91.98	-114.25	-114.26	-92.03	-126.28	-128.63	-77.74	-1000
5	-129.04	-126.91	-92.71	-112.21	-112.22	-92.77	-126.94	-129.10	-78.08	-1000
6	-130.53	-125.23	-92.98	-110.39	-110.40	-93.04	-125.26	-130.59	-81.58	-1000
7	-131.54	-129.04	-95.67	-104.24	-104.25	-95.72	-129.08	-131.61	-78.85	-1000

Table 12: Relative Stiffness (Y/ E-W)										
Level	Frame @ B	Frame @ C	Frame @ D	Frame @ E	Frame @ F	Frame @ G	Frame @ H	Frame @ I	Misc. Col/Walls	Total Percent
1	11.8	11.4	10.0	11.5	11.5	10.0	11.4	11.8	10.5	100
2	12.9	12.4	8.9	12.0	12.0	8.9	12.4	12.9	7.6	100
3	12.7	12.5	9.2	11.6	11.6	9.2	12.5	12.7	7.9	100
4	12.9	12.6	9.2	11.4	11.4	9.2	12.6	12.9	7.8	100
5	12.9	12.7	9.3	11.2	11.2	9.3	12.7	12.9	7.8	100
6	13.1	12.5	9.3	11.0	11.0	9.3	12.5	13.1	8.2	100
7	13.2	12.9	9.6	10.4	10.4	9.6	12.9	13.2	7.9	100

Wind Drift

Wind forces were examined to determine if the overall building drift and the individual story drifts were acceptable. In general, drift should be limited as much as possible; however, a limit of 1/400th of the overall building height was used in this case. For this overall structure, the drift is limited to:

$$\Delta_{MAX} = (111.5' \text{ X } 12)/400 = 3.35''$$

After running the ETABS model for unfactored (serviceability consideration) wind forces in both directions, the following results were obtained:

Table 13: Wind Drift (X/ N-S)		
Level	**Story Drift (in)**	**Total Drift (in)**
1	0.0447	0.0447
2	0.0897	0.1344
3	0.1043	0.2387
4	0.1031	0.3418
5	0.0936	0.4354
6	0.0804	0.5158
7	0.0682	0.5840

Table 14: Wind Drift (Y/ E-W)		
Level	**Story Drift (in)**	**Total Drift (in)**
1	0.2549	0.2549
2	0.2715	0.5264
3	0.2454	0.7718
4	0.2100	0.9818
5	0.1705	1.1523
6	0.1290	1.2813
7	0.0871	1.3684

It is clear that the E-W direction drift is larger than the N-S drift, which seems logical due to the larger wind force in that direction, as well as a smaller building width. From the data, it is clear that the maximum building drift in both directions is acceptable as it is less than the allowable value of 3.35". When considering individual story drift, it is conservative to look at the smallest story height to find the allowable story drift. For the 12.5' high floor-to-floor height, the maximum allowable story drift value is 0.375". We see that this value is not exceeded in either direction. All drift values are acceptable.

Seismic Drift

Seismic forces were examined to determine if the overall building drift was acceptable. Drift due to seismic forces is a strength consideration, due to the PΔ effects that result. For this overall structure, based on ASCE7-05 Chapter 12, the drift is limited to:

$$\Delta_{MAX} = 0.020 \text{ X } (111.5' \text{ X } 12) = 26.76''$$

After running the ETABS model for factored (strength consideration) seismic forces in both directions, the following results were obtained:

Table 15: Seismic Drift (X/ N-S)		
Level	Story Drift (in)	Total Drift (in)
1	0.0503	0.0503
2	0.1065	0.1568
3	0.1299	0.2867
4	0.1331	0.4198
5	0.1236	0.5434
6	0.1070	0.6504
7	0.0901	0.7405

Table 16: Seismic Drift (Y/ E-W)		
Level	Story Drift (in)	Total Drift (in)
1	0.1537	0.1537
2	0.1823	0.3360
3	0.1794	0.5154
4	0.1635	0.6789
5	0.1384	0.8173
6	0.1058	0.9231
7	0.0695	0.9926

These drift values were adjusted based equation 12.8-15 of ASCE 7-05:

$$\delta_x = \frac{C_d\, \delta_{xe}}{I}$$

This resulted in respective amplified drifts of 1.85" and 2.48" for the N-S and E-W directions. These amplified drifts were found using a C_d factor of 2.5 for reinforced concrete moment frames and an importance factor of 1.0. It is clear that these values will not exceed the allowable value for the structure. This ensures that the deflections that occur from seismic forces will not result in detrimental secondary effects.

Torsion

Overall building torsion results from several scenarios. The largest and most common case of building torsion results from a center of mass that differs in location from the building's center of rigidity. This creates a case where the loads are applied at an eccentricity on the building. This eccentricity times the force results in a moment on the overall building. Torsion also can result from the accidental eccentricity caused by seismic forces as described in ASCE 7-05 Section 12.8.4.2. Additionally, Cases 2 and 4 from the previously considered wind cases can also result in an additional eccentricity causing torsion. In both of these wind cases, the eccentricity is equal to 15% of the building width.

As previously mentioned, due to the building's symmetrical geometric shape, as well as the symmetrical frame stiffnesses about the x and y axes, the center of mass and center of rigidity are both at the same location. This creates no torsion from eccentricity. In addition to this, it has been shown that seismic does not control and that Wind Case 1 is the controlling wind case. Taking all of this into account, it is clear that the overall torsion on the building due to these forces is negligible, resulting in negligible overall building torsion.

Overturning

Overturning issues can have an impact on a variety of building components, probably the most common of which is the building's foundations. Overturning occurs when the lateral forces on a building are not offset by the moment created by the building's self weight. This creates a scenario where uplift must be considered for the foundations. Foundations must utilize friction from the soil and be used in tension, rather than in compression.

Overturning moments can also have an effect on the columns in a building as well. Overall building moments are transferred through axial forces in the columns. These moments put some columns in compression, and others in tension. This is something that must be taken into account as well.

The following overturning moments were determined from taking the critical factored story shear from ETABS at each level and assuming that force acted at the midpoint of each story. The height and force were used to determine the moments, which were summed to determine the overturning moment in that direction.

Table 17: Seismic Overturning Moment (X/ N-S)			
Level	**Height**	**Story Shear (K)**	**Overturning Moment (ft - k)**
1	18	323	2907
2	30.5	314	7615
3	43	296	10878
4	55.5	267	13150
5	68	225	13894
6	80.5	169	12548
7	93.5	98	8526
		Total Moment:	69517

Table 18: Seismic Overturning Moment (Y/ E-W)			
Level	**Height**	**Story Shear (K)**	**Overturning Moment (ft - k)**
1	18	372	3348
2	30.5	360	8730
3	43	338	12422
4	55.5	303	14923
5	68	254	15685
6	80.5	190	14108
7	93.5	110	9570
		Total Moment:	78784

Table 19: Wind Overturning Moment (X / N-S)			
Level	**Height**	**Story Shear (K)**	**Overturning Moment (ft - k)**
1	18	330.7	2976
2	30.5	270.4	6557
3	43	235.8	8667
4	55.5	199.2	9811
5	68	160.6	9920
6	80.5	120.6	8958
7	93.5	78.4	6821
		Total Moment:	53709

Table 20: Wind Overturning Moment (Y/ E-W)			
Level	**Height**	**Story Shear (K)**	**Overturning Moment (ft - k)**
1	18	661.4	5953
2	30.5	523.7	12699
3	43	454.1	16687
4	55.5	380.8	18754
5	68	304.2	18782
6	80.5	224.8	16691
7	93.5	143.0	12444
		Total Moment:	102012

After calculation of the moment resulting from the building's self weight, it has been determined that overturning will not cause uplift in any areas of the foundation design. It was initially expected that overturning or foundation uplift would not occur due to the large building self weight. This expectation was confirmed by the dead load moments of 2708090 ft-k for the N-S direction and 1543158 ft-k for the E-W directions. These moment calculations can be seen in more detail in Appendix C.

Conclusion

After analyzing the lateral loads from wind and seismic forces using the computer model which were confirmed by hand calculations, the following conclusions were determined:

- The primary controlling load case from ASCE7-05 was 0.9D + 1.6W.
- The controlling wind case was Wind Case 1.
- The center of mass and center of rigidity were both found to be at the geometric center of the structure.
- Overall building torsion was negligible.
- Overall wind drift and story drift were found to be well within the L/400 limit.
- Drift due to seismic forces was found to be acceptable.
- Overturning moment was found to not cause uplift in the foundations.
- Building shear walls and columns were determined to be adequate (Calculations located in Appendix C).

Appendix A: Wind

Technical Assignment #3

KYLE WAGNER | WIND ANALYSIS | TECH 1 | 1

WIND IN EAST-WEST DIRECTION

DETERMINE VELOCITY PRESSURES, q_z q_h

BASIC WIND SPEED

POTOMAC, MD $V = 90$ mph [FIG 6-1]

WIND DIRECTIONALITY FACTOR

$K_d = 0.85$ [TABLE 6-4]

IMPORTANCE FACTOR

$I = 1.0$

EXPOSURE CATEGORY: B

TOPOGRAPHIC FACTOR

$K_{zt} = 1.0$

DETERMINE VEL PRESSURE EXPOSURE COEFF K_z, K_h

SEE FIGURE FOR DATA

DETERMINE VEL PRESSURES

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

$q_h = 0.00256 K_h K_{zt} K_d V^2 I$

SEE FIGURE FOR DATA

GUST EFFECT FACTORS

ASSUME: MOST LIKELY RIGID

$n_1 = 100/H = 100/93.5 = 1.07 > 1$ ∴ RIGID

$G = 0.85$ (6.5.8.1)

Technical Assignment #3

KYLE WAGNER | WIND ANALYSIS | TECH 1 | 2

BUILDING FULLY ENCLOSED W/ PARAPET (16')

$GC_{pn} = +1.5$ WINDWARD
$GC_{pn} = -1.0$ LEEWARD

COMBINED NET DESIGN PRESSURE ON PARAPET

$p_p = q_p GC_{pn}$

SEE FIGURE FOR DATA

BUILDING NOT LOW RISE , RIGID BUILDING → YES

DETERMINE q_z, q_h, C_p, GC_{pi}

SEE FIGURES

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 FIGURE

Technical Assignment #3

Table 21: E-W Basic Wind Analysis Factors		
Exposure B		
Case 2		
L (Most conservative) =	127.5	ft
B=	223.75	ft
L/B=	0.570	
Basic Wind Speed	V=	90
Wind Directionality Factor	K_d=	0.85
Importance Factor	I=	1.0
Exposure Category	Category	B
Topographical Factor	K_{zt}=	1.0
Gust Effect Factor	G=	0.85
Cp Windward	C_p=	0.8
Cp Leeward	C_p=	-0.5
G_{cpi} Windward		0.18
G_{cpi} Leeward		-0.18
GC_{pn} Windward		1.5
GC_{pn} Leeward		-1

Table 22: E-W Velocity Pressure Exposure Coefficients, Kh and Kz

Level	Height (ft above Plaza)	Kz, Kh, Kp	qz, qh, qp Windward	qz, qh Leeward
Plaza Level	0	0.570	10.047	17.087
	9	0.570	10.047	17.087
2nd Floor	18	0.605	10.671	17.087
	24.25	0.659	11.620	17.087
3rd Floor	30.5	0.704	12.407	17.087
	36.75	0.742	13.085	17.087
4th Floor	43	0.776	13.686	17.087
	49.25	0.807	14.227	17.087
5th Floor	55.5	0.835	14.721	17.087
	61.75	0.861	15.177	17.087
6th Floor	68	0.885	15.601	17.087
	74.25	0.908	15.998	17.087
7th Floor	80.5	0.929	16.371	17.087
	87	0.950	16.739	17.087
Main Roof	93.5	0.969	17.087	17.087
Penthouse Level	109.5	1.014	17.876	17.087

Technical Assignment #3

Table 23: E-W Calculation of Design Wind Pressures								
					Windward		Leeward	
Level	**Height (ft above Plaza)**	**External Pressure Windward (psf)**	**External Pressure Leeward (psf)**	**Internal Pressure (psf)**	**Net Pressure P Pos (psf)**	**Net Pressure P Neg (psf)**	**Net Pressure P Pos (psf)**	**Net Pressure P Neg (psf)**
Plaza Level	0	6.83	-7.26	3.22	3.61	10.05	-10.48	-4.04
	9	6.83	-7.26	3.22	3.61	10.05	-10.48	-4.04
2nd Floor	18	7.26	-7.26	3.22	4.04	10.47	-10.48	-4.04
	24.25	7.90	-7.26	3.22	4.68	11.12	-10.48	-4.04
3rd Floor	30.5	8.44	-7.26	3.22	5.22	11.65	-10.48	-4.04
	36.75	8.90	-7.26	3.22	5.68	12.12	-10.48	-4.04
4th Floor	43	9.31	-7.26	3.22	6.09	12.52	-10.48	-4.04
	49.25	9.67	-7.26	3.22	6.46	12.89	-10.48	-4.04
5th Floor	55.5	10.01	-7.26	3.22	6.79	13.23	-10.48	-4.04
	61.75	10.32	-7.26	3.22	7.10	13.54	-10.48	-4.04
6th Floor	68	10.61	-7.26	3.22	7.39	13.83	-10.48	-4.04
	74.25	10.88	-7.26	3.22	7.66	14.10	-10.48	-4.04
7th Floor	80.5	11.13	-7.26	3.22	7.91	14.35	-10.48	-4.04
	87	11.38	-7.26	3.22	8.16	14.60	-10.48	-4.04
Main Roof	93.5	11.62	-7.26	3.22	8.40	14.84	-10.48	-4.04

Technical Assignment #3

Table 24: E-W Design Pressures							
Level	**Height (ft above Plaza)**	**Design Pressure Windward (psf)**	**Design Pressure Leeward (psf)**	**Total Pressure (psf)**	**Force of Total Pressure (k)**	**Story Shear Total (k)**	**Moment Windward (ft-k)**
Plaza Level	0	6.83	-7.26	14.09	28.38	415.90	0
	9	6.83	-7.26	14.09			
2nd Floor	18	7.26	-7.26	14.52	49.54	387.52	892
	24.25	7.90	-7.26	15.16			
3rd Floor	30.5	8.44	-7.26	15.70	43.91	337.98	1339
	36.75	8.90	-7.26	16.16			
4th Floor	43	9.31	-7.26	16.57	46.34	294.07	1993
	49.25	9.67	-7.26	16.94			
5th Floor	55.5	10.01	-7.26	17.27	48.31	247.73	2681
	61.75	10.32	-7.26	17.58			
6th Floor	68	10.61	-7.26	17.87	49.98	199.43	3399
	74.25	10.88	-7.26	18.14			
7th Floor	80.5	11.13	-7.26	18.39	52.48	149.44	4224
	87	11.38	-7.26	18.64			
Main Roof	93.5	11.62	-7.26	18.88	27.46	96.97	2568
Penthouse	109.5	12.16	-7.26	19.42	69.51	69.51	7611
							24706

East- West Direction

Base Shear	**416**	**K**
Overturning Moment	**24706**	**ft-k**

Table 25: N-S Basic Wind Analysis Factors		
Exposure B		
Case 2		
L (Most Conservative)=	223.75	ft
B=	127.5	ft
L/B=	1.754901961	
Basic Wind Speed	V=	90
Wind Directionality Factor	K_d=	0.85
Importance Factor	I=	1
Exposure Category	Category	B
Topographical Factor	K_{zt}=	1
Gust Effect Factor	G=	0.85
Cp Windward	C_p=	0.8
Cp Leeward	C_p=	-0.35
G_{cpi} Windward		0.18
G_{cpi} Leeward		-0.18
GC_{pn} Windward		1.5
GC_{pn} Leeward		-1

Table 26: N-S Velocity Pressure Exposure Coefficients, Kh and Kz				
Level	**Height (ft above Plaza)**	**Kz, Kh, Kp**	**qz, qh, qp Windward**	**qz, qh Leeward**
Plaza Level	0	0.570	10.047	17.087
	9	0.570	10.047	17.087
2nd Floor	18	0.605	10.671	17.087
	24.25	0.659	11.620	17.087
3rd Floor	30.5	0.704	12.407	17.087
	36.75	0.742	13.085	17.087
4th Floor	43	0.776	13.686	17.087
	49.25	0.807	14.227	17.087
5th Floor	55.5	0.835	14.721	17.087
	61.75	0.861	15.177	17.087
6th Floor	68	0.885	15.601	17.087
	74.25	0.908	15.998	17.087
7th Floor	80.5	0.929	16.371	17.087
	87	0.950	16.739	17.087
Main Roof	93.5	0.969	17.087	17.087
Penthouse Level	109.5	1.014	17.876	17.087

Technical Assignment #3

Table 27: N-S Calculation of Design Wind Pressures								
					Windward		Leeward	
Level	Height (ft above Plaza)	External Pressure Windward (psf)	External Pressure Leeward (psf)	Internal Pressure (psf)	Net Pressure P Pos (psf)	Net Pressure P Neg (psf)	Net Pressure P Pos (psf)	Net Pressure P Neg (psf)
Plaza Level	0	6.83	-5.08	3.22	3.61	10.05	-8.30	-1.87
	9	6.83	-5.08	3.22	3.61	10.05	-8.30	-1.87
2nd Floor	18	7.26	-5.08	3.22	4.04	10.47	-8.30	-1.87
	24.25	7.90	-5.08	3.22	4.68	11.12	-8.30	-1.87
3rd Floor	30.5	8.44	-5.08	3.22	5.22	11.65	-8.30	-1.87
	36.75	8.90	-5.08	3.22	5.68	12.12	-8.30	-1.87
4th Floor	43	9.31	-5.08	3.22	6.09	12.52	-8.30	-1.87
	49.25	9.67	-5.08	3.22	6.46	12.89	-8.30	-1.87
5th Floor	55.5	10.01	-5.08	3.22	6.79	13.23	-8.30	-1.87
	61.75	10.32	-5.08	3.22	7.10	13.54	-8.30	-1.87
6th Floor	68	10.61	-5.08	3.22	7.39	13.83	-8.30	-1.87
	74.25	10.88	-5.08	3.22	7.66	14.10	-8.30	-1.87
7th Floor	80.5	11.13	-5.08	3.22	7.92	14.35	-8.30	-1.87
	87	11.38	-5.08	3.22	8.17	14.60	-8.30	-1.87
Main Roof	93.5	11.62	-5.08	3.22	8.40	14.84	-8.30	-1.87

Technical Assignment #3

Table 28: N-S Design Pressures							
Level	**Height (ft above Plaza)**	**Design Pressure Windward (psf)**	**Design Pressure Leeward (psf)**	**Total Pressure (psf)**	**Force of Total Pressure (k)**	**Story Shear Total (k)**	**Moment Windward (ft-k)**
Plaza Level	0	6.83	-5.08	11.92	13.67	206.58	0
	9	6.83	-5.08	11.92			
2nd Floor	18	7.26	-5.08	12.34	23.99	192.91	432
	24.25	7.90	-5.08	12.98			
3rd Floor	30.5	8.44	-5.08	13.52	21.55	168.92	657
	36.75	8.90	-5.08	13.98			
4th Floor	43	9.31	-5.08	14.39	22.93	147.37	986
	49.25	9.67	-5.08	14.76			
5th Floor	55.5	10.01	-5.08	15.09	24.06	124.43	1335
	61.75	10.32	-5.08	15.40			
6th Floor	68	10.61	-5.08	15.69	25.01	100.38	1701
	74.25	10.88	-5.08	15.96			
7th Floor	80.5	11.13	-5.08	16.22	26.36	75.37	2122
	87	11.38	-5.08	16.47			
Main Roof	93.5	11.62	-5.08	16.70	13.84	49.01	1294
Penthouse	109.5	12.16	-5.08	17.24	35.17	35.17	3851
							12378

North- South Direction

Base Shear	**207**	**K**
Overturning Moment	**12378**	**ft-k**

Appendix B: Seismic

Technical Assignment #3

KYLE WAGNER | TECH 3 SEISMIC | 11/20/09 | 1

NOT DETACHED 1 OR 2 FAMILY DWELLING
NOT ON AG STORAGE
NO SPECIAL CONSIDERATIONS

SEISMIC GROUND MOTION VALUES

DETERMINE S_S AND S_1 [FIG 22-1 AND 22-14]

$S_S = 0.156$

$S_1 = 0.051$

$S_S > 0.15$ AND $S_1 > 0.04$

DETERMINE SOIL SITE CLASS

SOIL SITE CLASS D

DETERMINE S_{MS} AND S_{M1}

$S_{MS} = F_a S_S = 1.6\,(0.156) = 0.2496$

$S_{M1} = F_V S_1 = 2.4\,(0.051) = 0.1224$

DETERMINE S_{DS} AND S_{D1}

$S_{DS} = 2\,S_{MS}/3 = 2(0.2496)/3 = 0.166$

$S_{D1} = 2\,S_{M1}/3 = 2\,(0.1224)/3 = 0.081$

SEISMIC DESIGN CATEGORY

$S_S = 0.156 > 0.15$ AND $S_1 = 0.051 > 0.04$

DETERMINE OCCUPANCY CATEGORY IBC TABLE 1604.5 (II)

I, II OR III YES

$S_1 < 0.75$

SIMPLIFIED DESIGN? → NO > 3 STORIES

FIND SDC:

$S_{D1} = 0.081$ $\quad 0.067 \le S_{D1} \le 0.133$ SEISMIC DESIGN CAT B

Technical Assignment #3

KYLE WAGNER | TECH 3 SEISMIC | 11/20/09 | 2

PERMITTED ANALYTICAL PROCEDURES

SDC: B

∴ USE
- EQUIVALENT LATERAL FORCE PROCEDURE
- MODAL RESPONSE SPECTRUM ANALYSIS
- SEISMIC RESPONSE HISTORY PROCEDURES

USE ELFP

DETERMINE RESPONSE MODIFICATION COEFF R

ORDINARY MOMENT FRAMES → $R = 3$

IMPORTANCE FACTOR $I = 1.0$

DETERMINE APPROXIMATE FUNDAMENTAL PERIOD

$$T_a = C_t h_n^x$$

- FOR E-W FORCE

$h_n = 109.5'$
TO PENTHOUSE ROOF

- FOR N-S FORCE

$h_n = 130.5'$
TO PENTHOUSE ROOF

FOR CONC. MOMENT RESISTING FRAMES $C_t = 0.016$ $x = 0.9$

$T_x = 1.9745$
↑ FROM ETABS MOD

$T_y = 2.2705$
↑ ETABS

$T_L = 8 > T_a$

$T_L = 8 > T_a$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \le \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$

$$C_s = \frac{0.081}{1.9745(3)} = 0.0137 \le \frac{0.166}{3} = 0.055 ✓$$

$$C_s = \frac{0.081}{2.2705(3)} = 0.0119 \le \frac{0.166}{3} = 0.055 ✓$$

$C_s = 0.0119$

$C_s = 0.0137$

$S_1 = 0.051 < 0.6$

$K_{EW} = 1.2975$
$K_{N\text{-}S} = 1.3915$ BY INTERPOLATION

PLEASE SEE FIGURES FOR BUILDING WEIGHTS AND FORCE CALCULATIONS

Technical Assignment #3

Table 29: Basic Building Information				
Level	**Height Above Plaza (in)**	**Floor-Floor Distance (ft)**	**Area (SF)**	**Slab Thickness (in)**
Penthouse Roof	109.5	16	2000	8
Main Roof	93.5	13	26350	7
7th Floor	80.5	12.5	26350	7
6th Floor	68	12.5	26276	7
5th Floor	55.5	12.5	26276	7
4th Floor	43	12.5	26276	7
3rd Floor	30.5	12.5	26276	7
2nd Floor	18	18	26276	7
Plaza/First Floor	0	11	108989	12
P1 Level*	-11	10	108989	8
P2 Level/Foundation*	-21	0	103561	5
* Parking Ramp Excluded				

Table 30: Penthouse Level Self Weight				
(Assume Elevator Room at 16': Conservative)				
Slabs:	Penthouse		200	K
	Elevator Machine Room		95	K
Superimposed Dead	10 psf		263	K
Penthouse Total:			**557**	**K**

Technical Assignment #3

Table 31: Roof Self Weight				
Slabs:	Main Roof Slab		2306	K
PT Beam Weight:			879	K
Primary Column Weight:	(32) 24" x 24" Columns		125	K
Superimposed Dead:	10 psf		264	K
Building Core:				
	(12) 12x24 Columns		23	K
	Core Beams (12" x 30")		17	K
	Core Beams (12" x 24")		24	K
	Core Beams (12" x 20")		22	K
Building Envelope:				
	10 psf Assumed		42	K
Mechanical:				
	AHU (2 units)		127	K
	Cooling Tower		6	K
	AC Unit		4	K
Roof Total:			**3837**	**K**

Table 32: Level 7 Self Weight				
Slab Weight:	7" Thick Slab		2306	K
PT Beam Weight:			879	K
Primary Column Weight:	(32) 24" x 24" Columns		245	K
Superimposed Dead:	5 psf		131	K
Building Core:				
	(12) 12" x 24" Columns		46	K
	Core Beams (12" x 30")		17	K
	Core Beams (12" x 24")		24	K
	Core Beams (12" x 20")		22	K
Building Envelope:				
	10 psf Assumed		82	K
Level 7 Total:			**3751**	**K**

Technical Assignment #3

Table 33: Levels 3-6 Self Weight				
Slab Weight:	7" Thick Slab		2299	K
PT Beam Weight:			879	K
Primary Column Weight:	(32) 24" x 24" Columns		240	K
Superimposed Dead:	5 psf		131	K
Building Core:				
	(12) 12" x 24" Columns		45	K
	Core Beams (12" x 30")		17	K
	Core Beams (12" x 24")		24	K
	Core Beams (12" x 20")		22	K
Building Envelope:				
	10 psf Assumed		80	K
Level 3-6 Total:			**3737**	**K - Per Floor**

Table 34: Level 2 Self Weight				
Slab Weight:	7" Thick Slab		2299	K
PT Beam Weight:			879	K
Primary Column Weight:	(32) 24" x 24" Columns		293	K
Superimposed Dead:	5 psf		131	K
Building Core:				
	(12) 12" x 24" Columns		55	K
	Core Beams (12" x 30")		17	K
	Core Beams (12" x 24")		24	K
	Core Beams (12" x 20")		22	K
Building Envelope:				
	10 psf Assumed		80	K
Level 2 Total:			**3800**	**K**

Technical Assignment #3

Table 35: Plaza/Ground Level Self Weight				
Slab Weight:	12" Thick Slab		16348	K
Columns Above:	(32) 24" x 24"		173	K
Columns Below:				
	(16) 12" x 24"		26	K
	(163) 18" x 36"		605	K
	(19) 24" x 24"		63	K
	(4) 24" x 30"		17	K
	(5) 24" x 42"		29	K
	(4) 28" x 45"		29	K
	(4) 30" x 72"		50	K
	(1) 39" x 36"		8	K
Drop Panels:	(225) 10' x 10' x 5.5"		1547	K
Building Core:				
	Core Beams (12" x 30")		17	K
	Core Beams (12" x 24")		24	K
	Core Beams (12" x 20")		22	K
Building Envelope:				
	10 psf Assumed		58	K
Plaza/Ground Total:			**19014**	**K**

Technical Assignment #3

Table 36: P1 Parking Level Self Weight				
Slab Weight:	8" Thick Slab		10899	K
Columns:				
	(16) 12" x 24"		50	K
	(163) 18" x 36"		1155	K
	(19) 24" x 24"		120	K
	(4) 24" x 30"		32	K
	(5) 24" x 42"		55	K
	(4) 28" x 45"		55	K
	(4) 30" x 72"		95	K
	(1) 39" x 36"		15	K
Drop Panels:	(225) 10' x 10' x 5.5"		1547	K
Building Core:				
	Core Beams (12" x 30")		17	K
	Core Beams (12" x 24")		24	K
	Core Beams (12" x 20")		22	K
Level P1 Total:			**14085**	**K**

Table 37: P2 Parking Level Self Weight				
Columns:				
	(16) 12" x 24"		24	K
	(163) 18" x 36"		550	K
	(19) 24" x 24"		57	K
	(4) 24" x 30"		15	K
	(5) 24" x 42"		26	K
	(4) 28" x 45"		26	K
	(4) 30" x 72"		45	K
	(1) 39" x 36"		7	K
Level P2 Total:			**751**	**K**

Total Building Weight	26896 K

Table 38: Base Shears			
	Effective Seismic Weight	Seismic Response Coefficient	Base Shear (K)
N-S	W = 26896 K	Cs = 0.0119	322
E-W	W = 26896 K	Cs = 0.0137	371

Table 39: Seismic Calculations									
Level	Story Weight (K)	N-S Height (ft)	E-W Height (ft)	N-S Forces (K) Fx	E-W Forces (K) Fx	N-S Story Shear Vx	E-W Story Shear Vx	N-S Moments (ft-k) Mx	E-W Moments (ft-k) Mx
Penthouse Roof	557	109.5	109.5	8	10	0	0	835	1094
Main Roof	3837	93.5	93.5	90	100	8	10	8399	9307
7th Floor	3751	80.5	80.5	71	80	97	110	5688	6407
6th Floor	3737	68	68	56	64	168	189	3780	4327
5th Floor	3737	55.5	55.5	42	49	224	253	2325	2714
4th Floor	3737	43	43	29	35	266	302	1263	1510
3rd Floor	3737	30.5	30.5	18	22	295	337	555	686
2nd Floor	3800	18	18	9	12	313	359	161	209
Plaza/First Floor	19014	0	0	0	0	322	371	0	0
Total:	**45910**	**130.5**	**109.5**	**322**	**371**			**23007**	**26254**

$\Sigma\, w_i h_i^k$ N-S	193617378
$\Sigma\, w_i h_i^k$ E-W	60060701

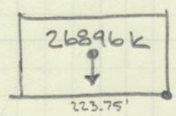
Appendix C: Lateral Analysis

Technical Assignment #3

KYLE WAGNER | TECH 3 | 12/01/09

OVERTURNING

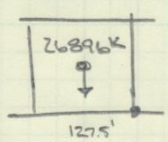
N-S DEAD LOAD MOMENT



$M_R = 0.9(26896^k) \cdot (223.75'/2) = 2708090\ ^{'k}$

$> 69517\ ^{'k}$ ∴ OK

E-W DEAD LOAD MOMENT



$M_R = 0.9(26896^k) \cdot (127.5'/2) = 1543158\ ^{'k}$

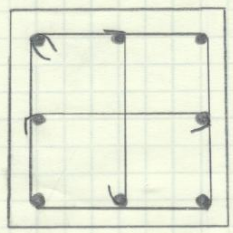
> 78784 ∴ OK

∴ NO UPLIFT IN FOUNDATIONS IN EITHER DIRECTION

Technical Assignment #3

KYLE WAGNER | TECH 3 | SPOT CHECKS | 1

CHECK COLUMN F-1.3

24"

24"

24" x 24" (8) #8 , #3 TIES @ 16" o.c.

$f'_c = 5000$ psi

$f_y = 60$ ksi

CHECK AT LEVEL 2

$A_s = 8(0.79 \text{ in}^2) = 6.32 \text{ in}^2$

PURE AXIAL, P_o

$$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29000} = 0.00207$$

$$P_o = 0.85 f'_c A_c + A_s f_y = 0.85(5)[(24 \times 24) - 6.32] + 6.32(60)$$

$$P_o = 2800 \text{K}$$

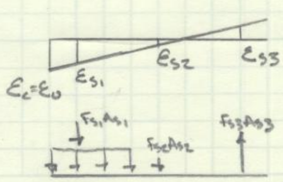
BALANCED STRAIN CONDITION (M_b, P_b)

$$c = \frac{0.003}{0.003 + \varepsilon_y} d_2$$

$$d_2 = 24 - 1.5 - 3/8 - {}^{8}/_{8}/2 = 21.625 \text{ in}$$

$$c = \frac{0.003}{0.003 + 0.00207}(21.625) = 12.796''$$

$$a = \beta_1 c = 0.80(12.796'') = 10.24''$$



$$\varepsilon_{s1} = \frac{0.003}{10.24}(10.24 - 2.375)$$

$$= 0.00230 < \varepsilon_y$$

$$0.0023(29000) = 66.7$$

$$f_{s1} = 60 \text{ ksi}$$

Technical Assignment #3

KYLE WAGNER | TECH 3 | SPOT CHECKS | 2

$$\varepsilon_{s2} = \frac{0.003}{10.24}(10.24 - 12) = -0.000516$$

$$f_{s2} = -0.000516(29000) = -14.95 \text{ ksi}$$

$$\varepsilon_{s3} = \frac{0.003}{10.24}(10.24 - 21.625) = -0.00334$$

$$-0.00334(29000) = -96.73 \text{ ksi} \rightarrow f_{s3} = -f_y = -60 \text{ ksi}$$

$$P_b = 0.85(5)(24)(0.80)(10.24) + 3(60) + 2(-14.95) + 3(-60)$$

$$P_b = 805.68^{k}$$

$$M_b = 0.85(5)(24)(0.80)(10.24)\left(12 - \frac{0.80(12.796)}{2}\right)$$

$$+3(60)(12-2.375) + 2(-14.95)(12-12) + 3(-60)(12-21.625)$$

$$M_b = 9215.15^{\prime\prime k} = 767.93^{\prime k}$$

PURE BENDING

ASSUME ε_{s1} DOES NOT YIELD, ε_{s2} AND ε_{s3} YIELD

$$f_{s1} = \frac{0.003}{c}(c - 2.375)\,29000$$

$$f_{s2} = -60 \text{ ksi}$$

$$f_{s3} = -60 \text{ ksi}$$

$$\Sigma F = 0 = 0.85(5)(24)(0.80)c + 3f_{s1} + 2f_{s2} + 3f_{s3}$$

$$c = 3.005$$

VERIFY: $\varepsilon_{s1} = \left(\frac{0.003}{3.005}\right)(3.005 - 2.375) = 0.000629 < 0.00207$

$$f_{s1} = 18.24 \text{ ksi}$$

Technical Assignment #3

KYLE WAGNER | TECH 3 | SPOT CHECKS | 3

$$\varepsilon_{s2} = \frac{0.003}{3.005}(3.005-12) = -0.00898 > 0.00207$$

$$\therefore f_{s2} = -60\text{ ksi} \quad f_{s3} = -60\text{ ksi}$$

$$M_o = 0.85(5)(24)(0.80)(3.005)\left(12 - \frac{0.80(3.005)}{2}\right)$$

$$+ 3(18.24) + 2(-60)(12-12) + 3(-60)(12-21.625)$$

$$M_o = 4435''^{k} \qquad M_o = 369.6'^{k}$$

PURE TENSION

$$T_o = \sum A_{si} \cdot f_{si} = 6.32\text{ in}^2 \cdot -60\text{ ksi} = -379.2^{k}$$

POINT BETWEEN P_o AND BALANCED STRAIN

Pick $c = 24''$

$$\varepsilon_{s1} = \frac{0.003}{24}(24-2.375) = 0.0027 > \varepsilon_y \rightarrow f_{s1} = 60\text{ ksi}$$

$$\varepsilon_{s2} = \frac{0.003}{24}(24-12) = 0.0015 < \varepsilon_y \rightarrow f_{s2} = 43.5\text{ ksi}$$

$$\varepsilon_{s3} = \frac{0.003}{24}(24-21.625) = 0.000297 < \varepsilon_y \rightarrow f_{s3} = 8.61\text{ ksi}$$

$$P_n = 0.85(5)(24)(0.80)(24) + 3(60) + 2(43.5) + 3(8.61)$$

$$P_n = 2251^{k}$$

$$M_n = 0.85(5)(24)(0.80)(24)\left(12 - \frac{0.80(24)}{2}\right) + 3(60)(12-2.375)$$

$$+ 2(43.5)(12-12) + 3(8.61)(12-21.625)$$

$$M_n = 6184''^{k} = 515'^{k}$$

Technical Assignment #3

KYLE WAGNER | TECH 3 | SPOT CHECKS | 4

AXIAL LOAD

TRIB AREA 609.3 SF

$P_{U_{SLAB}} = 6\left[609.3\text{ SF} \cdot (7''/12)\right](150)/1000 = 320^K$

$P_{U_{COL}} = 6\left[12.5' \cdot (2' \times 2')\right](150)/1000 = 45^K$

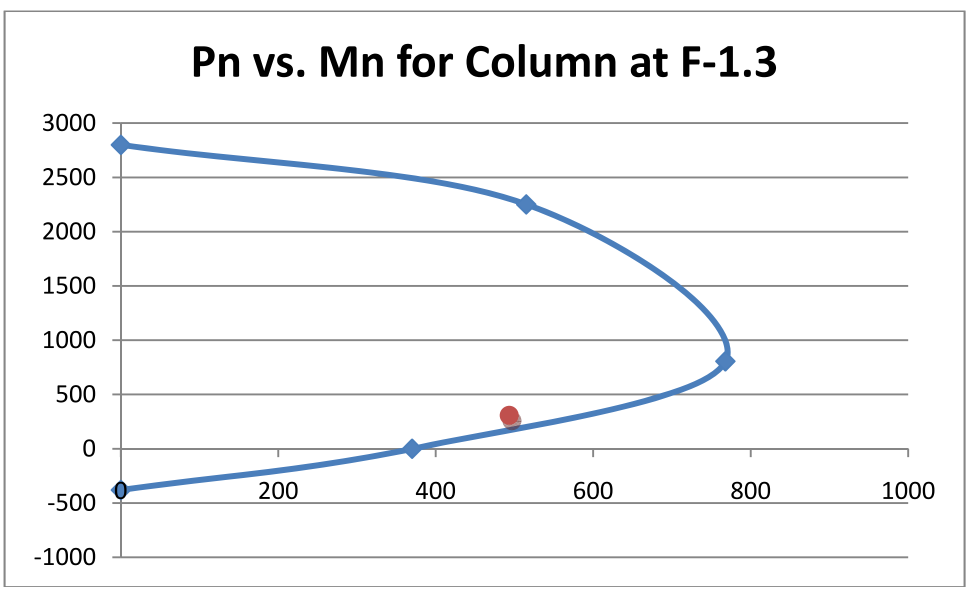
$P_U = 365^K$

MOMENT

COLUMN SHEAR = 39.09 K

$12.5'(39.09^K) \rightarrow 488.6^{'K}$

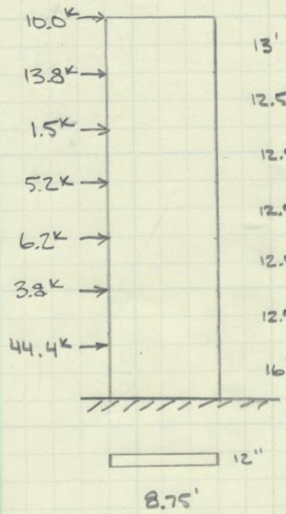
∴ OKAY, POINTS FALL WITHIN INTERACTION DIAGRAM ✓



Technical Assignment #3

KYLE WAGNER | TECH 3 | 12/01/09 | SPOT CHECKS | 1

SHEAR WALL



$f'_c = 5$ KSI

$M_U = 10(91.5) + 13.8(78.5) + 1.5(66) + 5.2(53.5)$
$+ 6.2(41) + 3.8(28.5) + 44.4(16)$
$= 3448^{'K}$

$V_U = 84.9$ K

$P_U = 152$K (SELF WT + TRIB AREA)

MAX PERMITTED SHEAR

$V_U \leq \phi V_{n\,max} = \phi\, 10 \sqrt{f'_c}\, h\, d$

$d = 0.8\, l_w = 0.8(8.75) = 84"$

$\phi V_n = 0.75(10)\sqrt{5000}\,(12")(84")/_{1000} = 534 > V_U = 8$

$\therefore$ OK

SHEAR STRENGTH BY CONCRETE

$V_c = 2\sqrt{f'_c}\, h\, d = 2\sqrt{5000}\,(12")(84)/_{1000} = 142^K$

$\frac{1}{2}\phi V_c = \frac{1}{2}(0.75)(142) = 53.25 < V_U = 85 \therefore V_S$ BASED ON CH 11.9.9

DETERMINE V_S BASED ON $V_U \leq \phi V_n = \phi(V_c + V_s)$

$84.9K = 0.75(53.25 + V_s)$

$V_{S\,REQ} = 59.95^K$

$\frac{A_v}{s} = \frac{V_s}{F_y d} = \frac{59.95}{60(84)} = 0.0119$

TRY (2) #4 @ 12"

$\rho_t = \frac{A_v}{sh} = \frac{2(0.2)}{12 \cdot 12} = 0.00278 > 0.0025 \therefore$ OKAY ✓

SPACING OKAY BY INSPECTION ✓

$\boxed{\text{USE (2) \#4 @ 12" FOR HORIZ REINF.}}$

Technical Assignment #3

KYLE WAGNER | TECH 3 12/01/09 | SPOT CHECKS | 2

REQUIRED SHEAR REINF

$$\rho_\ell = \frac{A_v}{sh} \geq 0.0025 + 0.5\left(2.5 - \frac{93.5}{8.75}\right)(0.00278 - 0.0025)$$

$\therefore$ USE $0.0025 \rightarrow$ MIN

TRY (2) #4

$$s = \frac{A_v}{0.0025h} = \frac{2 \cdot 0.2}{0.0025(12'')} = 13.33''$$

$\boxed{\text{USE (2) \#4 @ 12'' FOR VERT REINF}}$

DESIGN OF FLEXURE

$M_u = 3448^{'k}$ $\quad \ell_w = 105''$

$$M_u = \phi M_n = \phi A_s f_y Jd$$

$$(3448)(12) = 0.9\, A_s (60)(0.9(84))$$

$$A_s = 10.14 \text{ in}^2$$

$$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{10.14(60)}{0.85(5)(12)} = 11.93''$$

CALC Jd

$$Jd = d - a/2 = 84 - 11.93/2 = 78.0''$$

RECALC As

$$(3448)(12) = 0.9\, A_s (60)(78.0)$$

$$A_s = 9.82''$$

ASSUME 8 #9 BARS ($A_s = 8.0 \text{ in}^2$)

$$a = \frac{A_s f_y}{0.85 f'_c h} = \frac{8.0(60)}{0.85(5)(12)} = 9.41''$$

$$c = a/\beta_1 = 9.41/0.80 = 11.76''$$

$$\varepsilon_t = 0.003\left(\frac{102 - 11.76}{11.76}\right) = 0.023 > 0.005$$

$\therefore$ OKAY ✓

$\boxed{\text{USE (8) \#9 FOR FLEXURE}}$ ACTUAL DESIGN IS ADEQUATE