Technical Report #3

8th Street Office Building | Richmond, VA

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Acknowledgements

I am very grateful to the firm of Rathgeber/Goss Associates for their sponsorship of the 8th Street Office Building for AE Senior Thesis. Also, this thesis would not be possible without the assistance of Commonwealth Architects, which provided the renderings.

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Executive Summary

In the third technical report regarding the 8th Street Office Building, a detailed lateral analysis that considers torsional effects was performed. The existing lateral system consists of 16 reinforced concrete shear walls of varying heights. The shear walls have a constant thickness of 12" and are arranged around the four main transportation cores of the building.

A computer model of the 8th Street Office Building's lateral system was created using the program ETABS. The shear walls were connected by the rigid diaphragms at each level in order to distribute the lateral loads to the walls according to their relative stiffnesses. The model was initially utilized to calculate the center of mass of each floor. Also, the displacement of each shear wall due to the application of a unit load was obtained from ETABS. The centers of rigidity and centers of pressure were calculated by hand.

Then, direct shear and torsional shear due to wind and seismic lateral loads were calculated separately in both the East-West and North-South directions. This method of analysis was in accordance with the Case 1 wind loading found in ASCE 7-05. The wind and seismic loads are the loads calculated in the first technical report according to ASCE 7-05. The net shears were tabulated by shear wall and floor, and the total shear for each shear wall was accumulated. Finally, load factors of 1.6 for wind and 1.0 for earthquake were considered according to load combinations 4 and 5 in ASCE 7-05. It was discovered that wind controls over earthquake for all of the shear walls except for Shear Walls 1 and 2.

The ETABS computer model was utilized a second time in order to obtain story drifts due to wind and seismic separately. The wind drifts were compared to the ASCE 7-05 recommended allowable drift of H/400, and all of the drifts were concluded to be acceptable. The seismic drifts were adjusted with the appropriate amplification and importance factors and then compared to the ASCE 7-05 mandated allowable drift of $0.015h_{sx}$. The seismic drifts were also concluded to be acceptable.

After the distribution of lateral loads was completed, it was determined that Shear Wall 2 is a critical wall, so a strength check was performed on Shear Wall 2. Since the load combination that controls Shear Wall 2 depends upon the seismic lateral loads, the check was performed according to Chapter 21 of ACI-08. It was determined that the horizontal and vertical reinforcement designed by the engineers is more than adequate, but larger boundary elements than the 12" constant thickness are necessary.

Finally, a wide beam shear check was performed on the mat foundation as a continuation of the Shear Wall 2 strength check. It was concluded that wide beam shear will not cause any mat foundation failures. In addition, overturning was checked by comparing the resisting dead load to the shear induced overturning tension load, and no failures are anticipated.

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Introduction

The new 8th Street Office Building will be located in the bustling Richmond, VA commercial district near the Virginia State Capitol Building. It is intended to be a legacy building that will serve both the needs of the state government and the general public. Initially, the Virginia General Assembly will occupy the 8th Street Office Building for approximately five years while renovations to the Capitol Building are being completed. After that time, it is expected that various Virginia government agencies will move into the new office building.

The 8th Street Office Building will be comprised of 3 $^1/_2$ underground parking garage levels with spaces for 201 cars, ten floors above and a mechanical penthouse. The completed building will stand 176'-5" tall and will enclose approximately 307,000 square feet. Rooftop terraces with planters will be an integral part of the construction on the 3rd, 7th and 10th floors.

A secure main lobby on the first floor will efficiently handle high volume traffic to the large assembly areas. Ground level retail will be located on the corner of East Broad Street and 9th Street. The remainder of the floors will be open office spaces with meeting areas that can be flexibly rearranged to meet the needs of the various tenants. Finally, a six story atrium will connect the building along its southern edge to the existing 9th Street Office Building. The 9th Street Office Building is another Virginia government office building, and the atrium is expected to provide seamless passage between the two buildings. See Figure 1 on the next page for a general site plan.

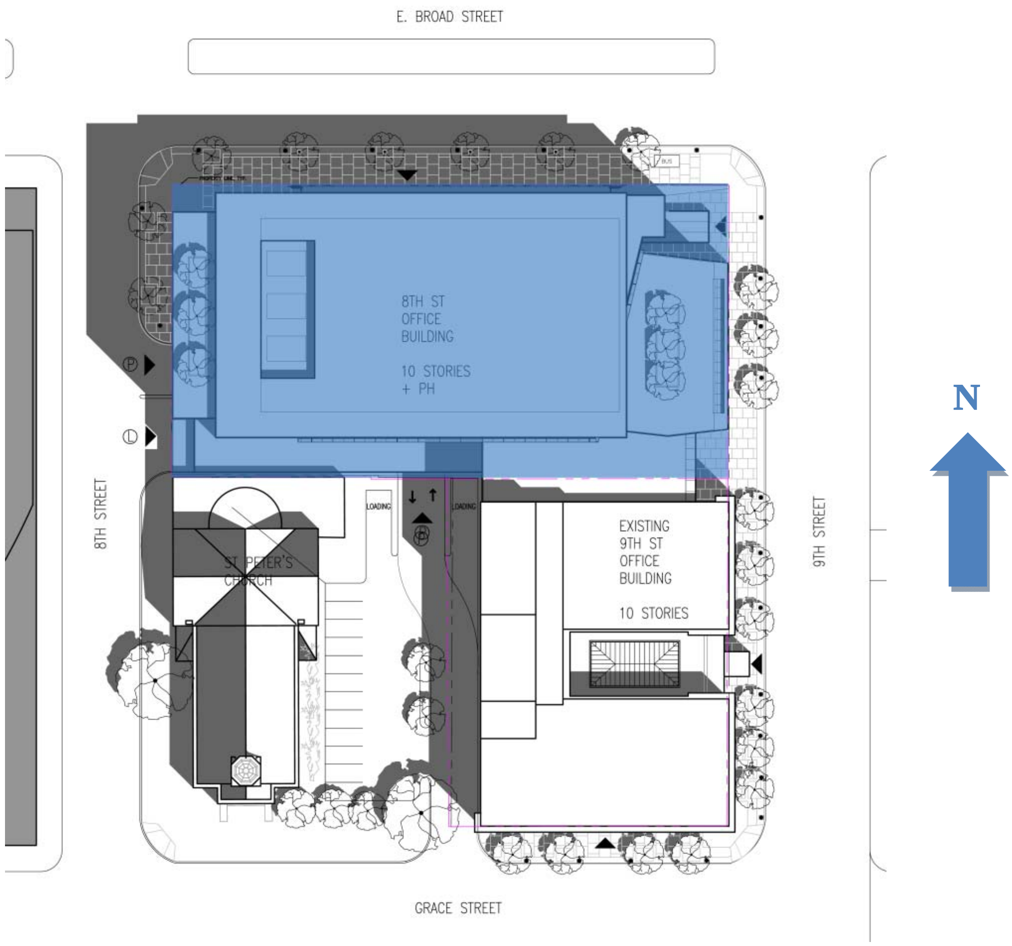

Figure 1 – Site plan

The 8th Street Office Building is designed as a primarily steel structure. However, concrete will play a major role in the construction of the underground parking garage and the shear walls around cores within the building. The façade will consist of several different glass curtain walls and precast concrete panels. Aluminum will be used to frame individual windows and doorways. Finally, a standing seam stainless steel roof will cantilever dramatically over 30'-0" off of the mechanical penthouse. See Figures 2 and 3 for elevations that display façade materials and the cantilevered roof. For a more detailed discussion of the 8th Street Office Building's structural system, please continue to the next section.

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Figure 2 – Broad Street Elevation

Figure 3 – 9th Street Elevation

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Structural System

Foundation

The geotechnical engineering study was conducted by Froehling & Robertson, Inc. of Richmond, VA. A total of nine test borings ranging from 50 to 100 feet were performed in September, 2006 and June-July, 2007. Based on the data from the borings and experience with other buildings located in Richmond, it was recommended in the geotechnical report that the 8th Street Office Building be supported on a mat foundation system. The mat foundation is located at elevations of 130'-0" and 140'-0" since the fourth and lowest level of the underground parking garage is only located on the western half of the site. Based on the elevations, it was recommended that the 4000 pounds per square inch concrete mat foundation be designed for a maximum allowable bearing pressure of 3,500 pounds per square foot. Ultimately, the mat foundation was designed to be 48" thick reinforced with #10 at 12" each way on the top and the bottom throughout the entire foundation.

According to the geotechnical report, the mat foundation system at the proposed elevations will be above the permanent groundwater table. However, the permanent perched water system may cause a substantial flow of water. Therefore, it was recommended that the 12" thick foundation walls be constructed with a minimum of 6" of free-draining granular filter material. Furthermore, the 48" thick mat should be placed on a 12" layer of free-draining aggregate for drainage and to provide uniform bearing pressure.

Parking Garage

The 8th Street Office Building's underground parking garage is comprised of 3 ½ levels and can accommodate 201 vehicles. The concrete columns are sized to be 30"x30" and tend to be reinforced with 16 #10 bars. Typical bay sizes are either 20'-0" by 40'-6" or 20'-0" by 30'-0". The concrete beams are typically sized to be 30"x30" although there are several exceptions. The longest span of the beams is approximately 40'-6". Primary reinforcement for the beams ranges anywhere from #7 to #11 bars. The one way concrete slabs span in the 20'-0" direction, and the majority of the slabs are 8" thick and reinforced with #5 bars spaced at 12".

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Superstructure

The most typical bay sizes for the 8th Street Office Building are either 20'-0" by 40'-6" around the perimeter or 20'0" by 30'-0" through the middle portion of the building. However, there are several variations due to the shape of the building from floor to floor. The composite floor system consists of 3 ¼" of lightweight concrete and 2" deep, 18 gage metal deck for a total depth of 5 ¼". The deck spans W-shape infill beams spaced at 10'-0" on center. The beams tend to be W16x31, W18x35, or W18x40 depending on the length of their span, which most commonly ranges from 30'-0" to 40'-6". Composite action is achieved between the floor system and the beams through ¾" diameter, 4" long headed shear studs. The beams then transfer their loads to W-shape girders whose sizes vary greatly. The girders are connected to W14 columns that range in size from W14x43 to W14x283. The columns are typically spliced every three floors. See Appendix A for typical floor framing plans.

Lateral System

The primary lateral load resisting system for the 8th Street Office Building consists of reinforced concrete shear walls surrounding four cores within the building. The cores are the locations of the main elevators and stairwells for the building. Therefore, openings are provided in the walls for doorways. There are a total of 16 shear walls. Shear Walls 1 thru 4 extend from the 4th floor foundation of the parking garage below grade to the roof. Shear Walls 5 thru 8 extend from the 4th floor foundation of the parking garage below grade to the penthouse mezzanine. Shear Walls 9 thru 12 extend from the 3rd floor foundation of the parking garage below grade to the penthouse mezzanine. Finally, Shear Walls 13 thru 16 extend from the 3rd floor foundation of the parking garage below grade to the penthouse. See Figure 4 for the exact locations of the shear walls in plan. See Appendix B for details of the shear walls in elevation showing their openings. Note that these elevations only extend upwards from the 1st floor in order to simplify the lateral force distribution and analysis in this report.

The shear walls are 12" thick and reinforced horizontally with #6 bars spaced at 12" on each face and vertically with #8 bars spaced at 12" on each face. The shear walls are a constant 12" thickness throughout without larger boundary elements. There is, however, heavier reinforcement of four #10 bars in each of the shear wall corners.

It is assumed that the floor system of the 8th Street Office Building acts as a rigid diaphragm and transfers the lateral loads due to wind and seismic activity completely to the shear walls in relation to their relative stiffness. The shear walls then carry those loads down to the mat foundation.

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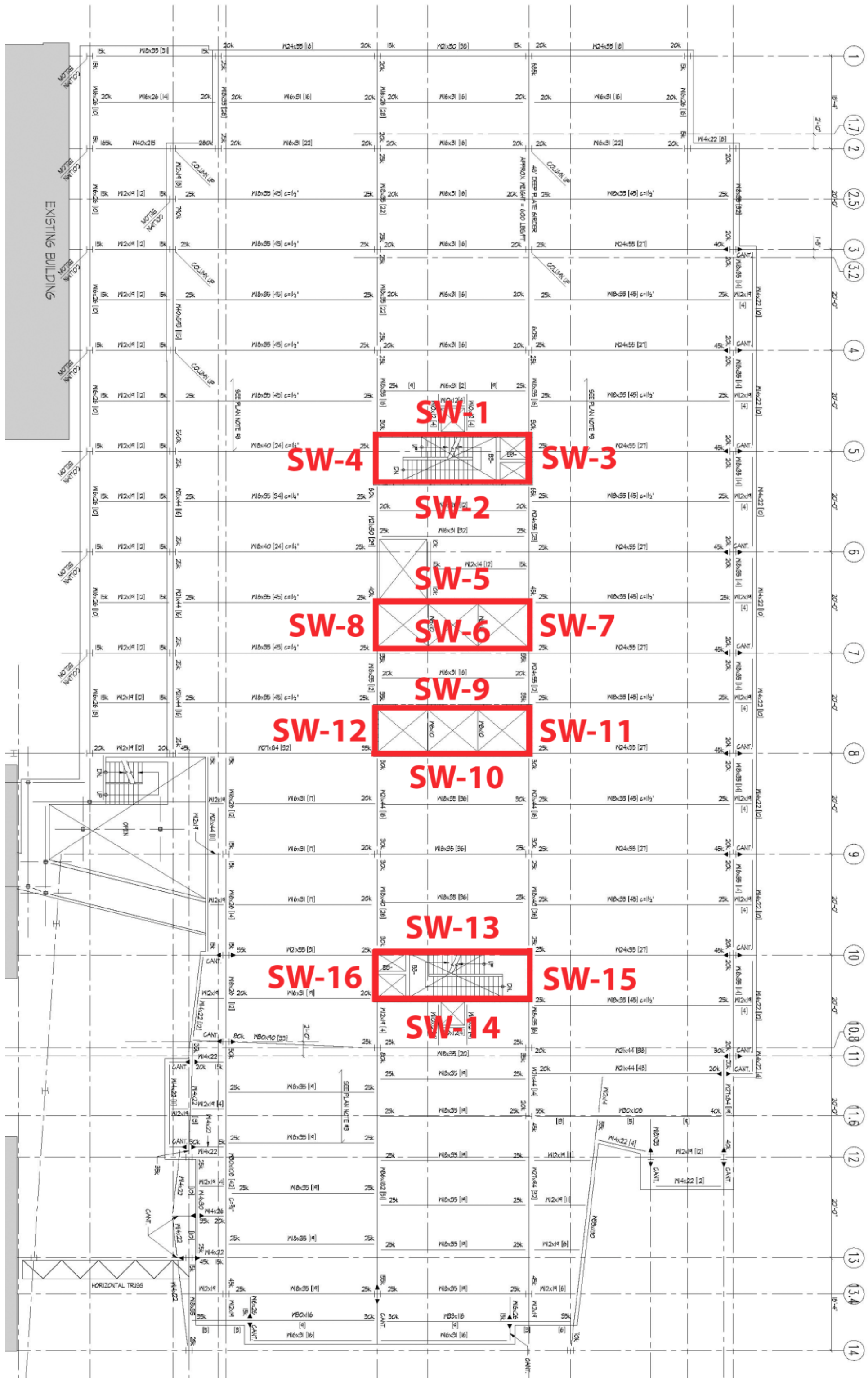

Figure 4 – Locations of Reinforced Concrete Shear Walls

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Codes

Applicable Design Codes:

Model Codes:

Virginia Uniform Statewide Building Code 2003

International Building Code 2003

Structural Standards:

ASCE 7-02, Minimum Design Loads for Buildings and Other Structures

Design Codes:

ACI 318-02, Building Code Requirements for Structural Concrete

AISC Manual of Steel Construction – Allowable Stress Design, 9th Edition

AISC Manual of Steel Construction – Volume II, Connections – ASD, 9th Edition/LRFD, 3rd Edition

Applicable Thesis Codes:

Model Codes:

International Building Code 2006

Structural Standards:

ASCE 7-05, Building Code Requirements for Structural Concrete

Design Codes:

ACI 318-05, Building Code Requirements for Structural Concrete

AISC Steel Construction Manual, 13th Edition

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Building Loads

Gravity Loads

Gravity and lateral loads were determined using ASCE 7-05.

Dead Loads for a Typical Floor:

2" Composite Metal Deck, 18 Gage	2 psf
3 $^1/_4$" Lightweight Concrete Slab (115 pcf)	41 psf
Approximated Self Weight of Steel Framing (Beams, Girders, Columns)	7 psf
Curtain Walls and Precast Concrete Panels	25 psf
Total for Floor System Design	(2 + 41 + 25) → 68 psf
Total	(2 + 41 + 7 + 25) → 75 psf

Superimposed Dead Loads for a Typical Floor:

Fireproofing	2 psf
Finishes	10 psf
Partitions	20 psf
Ceiling	5 psf
MEP	5 psf
Total SDL	42 psf

Penthouse and Penthouse Mezzanine:
Due to large mechanical spaces, a dead load of 100 psf is assumed to account for concrete pads, sloped floors and other miscellaneous loads. This load replaces the superimposed MEP load. Furthermore, partitions are not included.

Terraces/Roofs: A load of 125 psf is assumed to account for self weights of system components and planters and finishes.

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Live Loads for Typical Spaces:

	ASCE 7-05	Design Loads
Lobbies & First Floor Corridors	100 psf	100 psf
Corridors above First Floor	80 psf	100 psf
Stairs	100 psf	100 psf
Walkways & Elevated Platforms	60 psf	not available
Retail – First Floor	100 psf	not available
Assembly Areas with Movable Seats	100 psf	not available
Offices	50 psf	50 psf + 20 psf for partitions
Ordinary Roof	20 psf	30 psf minimum
Roofs used for Roof Gardens or Assembly Purposes	100 psf	not available

A comparison between the live loads from Table 4-1 in ASCE 7-05 and the live loads from Table 4-1 in ASCE 7-02 shows no differences. Thus, only the loads from ASCE 7-05 are tabulated above. The design loads that have been provided by the engineers of record are slightly more conservative than the minimum loads from ASCE 7-05. In addition, the engineers classified the partitions as a live load as opposed to a superimposed dead load, which is not unusual. Finally, a design load of 150 psf was specified by the engineers for mechanical rooms. Since ASCE 7-05 does not provide a live load value for mechanical rooms, a live load of 150 psf will be used.

Lateral Loads

Wind Loads:

The wind loads that were calculated in Technical Report #1 were applied at the center of pressure for each level of the 8th Street Office Building in this report. See Figures 5 and 6 below for the wind loads in the East-West direction and the North-South direction. For detailed calculations, see Appendix D.

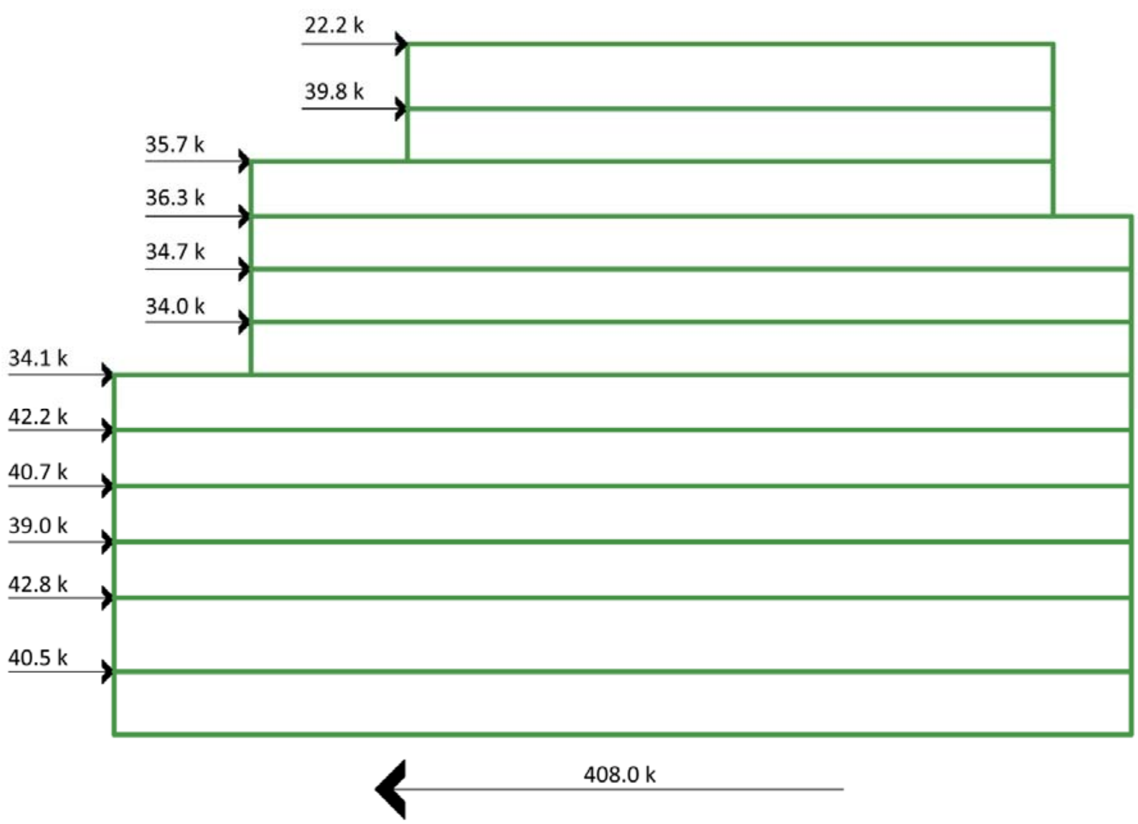

Figure 5 – East-West Wind Load Diagram

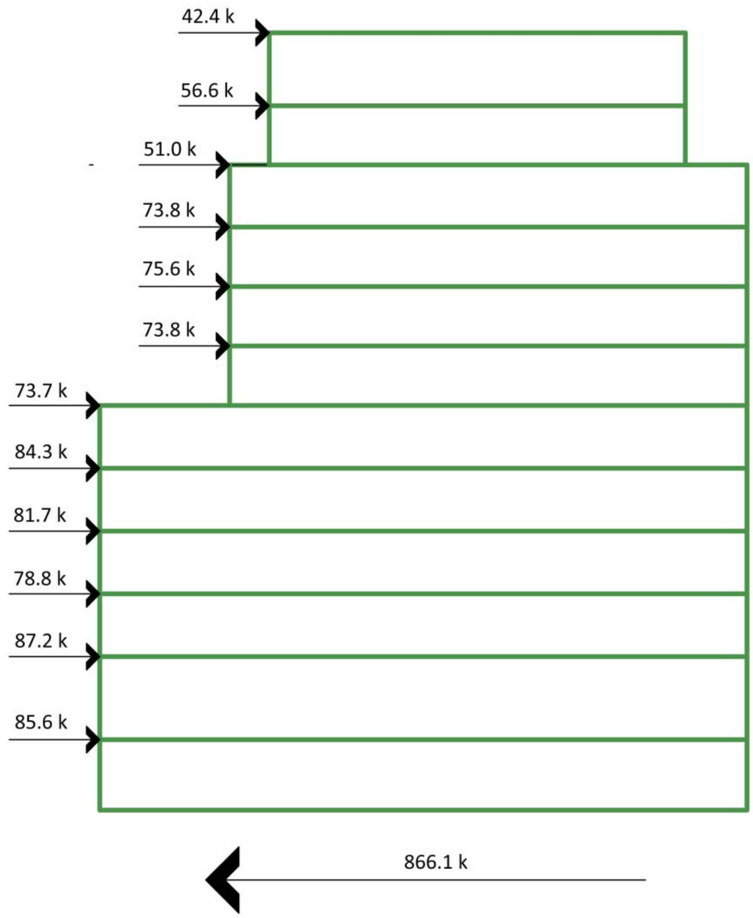

Figure 6 – North-South Wind Load Diagram

Seismic Loads:

The seismic loads that were calculated in Technical Report #1 were applied at the center of mass for each level of the 8th Street Office Building in this report. See Figure 7 below for the seismic loads, which were applied in each direction. For detailed calculations, see Appendix E

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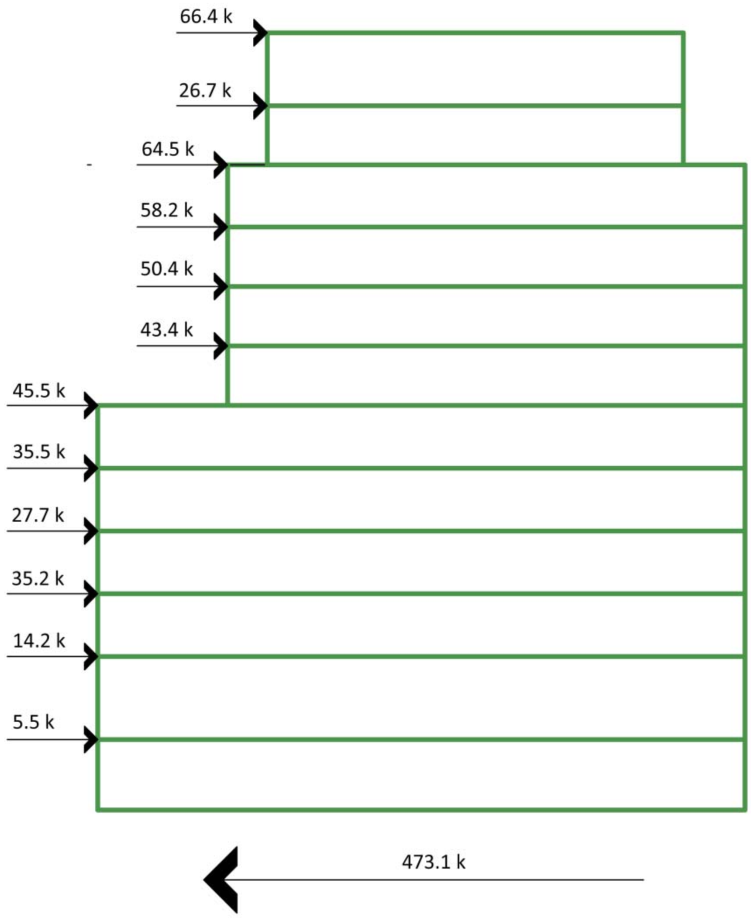

Figure 7 – Seismic Load Diagram

Lateral Force Distribution

A detailed lateral analysis of the 8th Street Office Building was performed with the assistance of the computer program ETABS. In order to simplify the analysis, it was assumed that the shear walls are fixed at the first floor, so the underground parking garage can be neglected. This is a conservative approach since it does not consider the fact that some moment due to shear at the lower levels is dissipated horizontally in reality.

Each floor was modeled as a rigid diaphragm in ETABS, and the center of mass of each floor was obtained directly from the program due to the complex and varied geometry of each floor. See Figure 8 for the floor areas modeled in ETABS, and see Table 1 for the tabulated values of the centers of mass.

Furthermore, each shear wall was modeled independently in ETABS, and a unit force was applied to the top of each shear wall in order to determine the relative stiffness of each wall. Additionally, a couple of shear walls without openings were selected to calculate the stiffness according to the inverse of $\Delta_{flexure}$ since they are considered tall walls. The calculated stiffnesses were identical within three decimal places to the stiffnesses determined in ETABS. Figure 9 provides a visual representation of the shear walls modeled in ETABS. However, see Appendix B for individual details and displacement, stiffness, and distance values. These values were then used to calculate the center of rigidity of each floor, which can be found in Table 2.

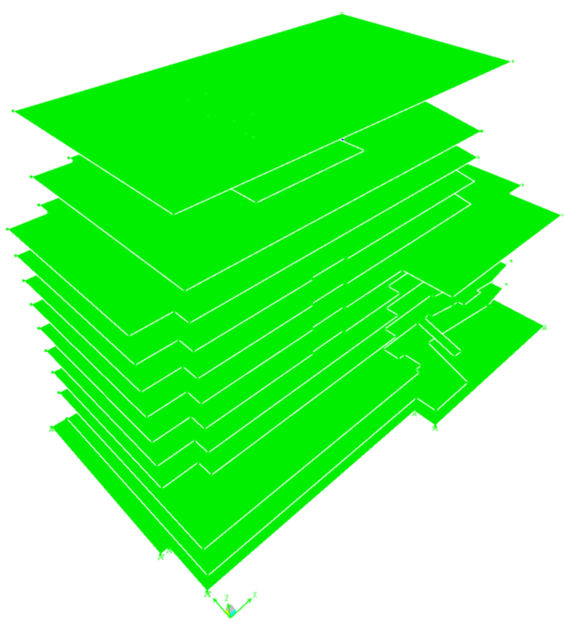

Figure 8 – Floor Areas in ETABS

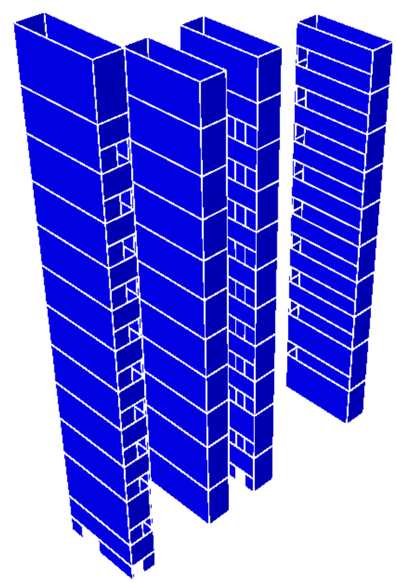

Figure 9 – Shear Walls in ETABS

Center of Mass		
Floor	**x (ft.)**	**y (ft.)**
1st	n/a	n/a
2nd	116.83	73.75
3rd	118.94	80.08
4th	123.42	85.82
5th	123.42	85.82
6th	123.24	86.01
7th	129.55	78.47
8th	105.74	88.26
9th	105.82	88.29
10th	105.97	88.32
PH	113.10	88.42
PH Mezz.	114.85	81.36
Roof	114.58	84.92

Table 1 – Centers of Mass by Floor

Center of Rigidity		
Floor	**x (ft.)**	**y (ft.)**
1st	n/a	n/a
2nd	132.66	84.69
3rd	132.66	84.69
4th	132.66	84.69
5th	132.66	84.69
6th	132.66	84.69
7th	132.66	84.69
8th	132.66	84.69
9th	132.66	84.69
10th	132.66	84.69
PH	132.66	84.69
PH Mezz.	109.68	85.47
Roof	79.90	87.19

Table 2 – Centers of Rigidity by Floor

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The center of pressure of each floor was calculated according to the appropriate dimensions. See Table 3 for the values of the centers of pressure.

Center of Pressure		
Floor	**x (ft.)**	**y (ft.)**
1st	n/a	n/a
2nd	128.33	70.21
3rd	128.33	79.04
4th	128.33	87.13
5th	128.33	87.13
6th	128.33	87.13
7th	125.92	72.59
8th	112.67	87.30
9th	113.17	87.30
10th	114.17	87.30
PH	120.54	87.30
PH Mezz.	118.33	84.92
Roof	114.58	84.92

Table 3 – Centers of Pressure by Floor

Appendix C provides individual floor details with the locations of the center of mass, center of rigidity, and center of pressure of each floor. Finally, it should be noted that the origin for all of the centers is located at the southwest corner of the building.

Since each floor diaphragm is assumed to be infinitely rigid, the distribution of the lateral loads to the shear walls is based on the relative stiffnesses. The wind loads were applied to the center of pressure of each floor, and Case 1 from Figure 6-9 of ASCE 7-05 was utilized. Therefore, 100% of the wind forces were applied in the East-West direction, and 100% of the wind forces were applied in the North-South direction. The remaining load cases may be investigated in more detail in the future. See Appendix D for the calculated direct shear forces and shear forces due to torsion about the center of rigidity by shear wall and floor. See Table 4 below for a summary of the net wind shear by shear wall and floor. Finally, the total wind shear for each shear wall is provided, and a 1.6 load factor is applied in order to determine the most critical shear wall and load combination.

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Wind Shear (kips)														
Shear Wall	Floor												Total	1.6 Total
	2	3	4	5	6	7	8	9	10	PH	PH Mezz.	Roof		
1	12.95	12.47	9.87	10.23	10.62	8.85	9.83	10.20	10.20	6.90	10.68	20.75	133.55	213.68
2	12.84	12.45	10.03	10.40	10.78	9.05	9.90	10.26	10.23	6.94	10.58	21.65	135.09	216.14
3	3.97	4.20	3.82	3.99	4.13	3.35	3.33	3.40	3.56	3.50	5.61	12.75	55.61	88.98
4	2.95	3.11	2.83	2.96	3.06	2.48	2.47	2.52	2.64	2.59	4.16	9.45	41.23	65.97
5	14.95	14.80	12.54	13.00	13.46	11.51	12.10	12.47	12.32	8.42	12.26	n/a	137.83	220.53
6	8.43	8.43	7.31	7.58	7.83	6.75	6.97	7.17	7.06	4.84	7.65	n/a	80.02	128.03
7	5.34	5.64	5.14	5.36	5.56	4.51	4.48	4.57	4.79	4.71	7.54	n/a	57.64	92.22
8	5.34	5.64	5.14	5.36	5.56	4.50	4.48	4.57	4.79	4.71	7.55	n/a	57.64	92.22
9	7.77	7.88	7.05	7.31	7.54	6.57	6.63	6.80	6.65	4.59	8.21	n/a	76.99	123.19
10	13.19	13.33	11.85	12.28	12.68	11.03	11.18	11.47	11.23	7.74	14.44	n/a	130.43	208.68
11	5.34	5.64	5.14	5.36	5.56	4.51	4.48	4.57	4.79	4.71	7.54	n/a	57.64	92.22
12	5.34	5.64	5.14	5.36	5.56	4.50	4.48	4.57	4.79	4.71	7.55	n/a	57.64	92.22
13	13.72	13.33	10.81	11.21	11.62	9.78	10.65	11.02	10.97	7.45	n/a	n/a	110.55	176.88
14	23.09	22.28	17.73	18.39	19.08	15.95	17.62	18.26	18.25	12.35	n/a	n/a	183.00	292.80
15	5.31	5.60	5.10	5.32	5.52	4.47	4.45	4.54	4.75	4.67	n/a	n/a	49.74	79.59
16	6.95	7.34	6.68	6.97	7.23	5.86	5.83	5.95	6.22	6.12	n/a	n/a	65.16	104.26

Table 4 – Wind Shear by Shear Wall

Similarly, the seismic loads were applied to the center of mass of each floor in both the East-West and North-South directions. See Appendix E for the calculated direct shear forces and shear forces due to torsion about the center of rigidity by shear wall and floor. See Table 5 below for a summary of the net seismic shear by shear wall and floor. Finally, the total seismic shear for each shear wall is provided since the load factor of interest is simply 1.0.

Seismic Shear (kips)													
Shear Wall	Floor												Total
	2	3	4	5	6	7	8	9	10	PH	PH Mezz.	Roof	
1	0.93	2.27	5.01	3.94	5.08	5.37	9.22	10.69	12.31	11.82	4.21	195.91	266.75
2	0.91	2.24	5.00	3.94	5.07	5.51	8.82	10.22	11.78	11.47	4.00	196.37	265.32
3	0.54	1.39	3.45	2.72	3.48	4.47	4.26	4.95	5.71	6.33	2.63	7.59	47.52
4	0.40	1.03	2.56	2.01	2.58	3.31	3.16	3.67	4.23	4.69	1.95	5.91	35.51
5	1.02	2.55	5.96	4.69	6.03	7.05	9.16	10.62	12.25	12.49	4.03	n/a	75.84
6	0.56	1.43	3.40	2.67	3.43	4.15	4.86	5.64	6.50	6.81	2.62	n/a	42.07
7	0.73	1.87	4.64	3.65	4.68	6.01	5.73	6.65	7.68	8.51	3.54	n/a	53.69
8	0.73	1.87	4.64	3.65	4.68	6.01	5.73	6.65	7.68	8.51	3.54	n/a	53.69
9	0.50	1.30	3.18	2.50	3.21	4.05	4.08	4.73	5.46	5.96	3.01	n/a	37.99
10	0.86	2.20	5.38	4.23	5.43	6.80	7.06	8.19	9.46	10.22	5.56	n/a	65.39
11	0.73	1.87	4.64	3.65	4.68	6.01	5.73	6.65	7.68	8.51	3.54	n/a	53.69
12	0.73	1.87	4.64	3.65	4.68	6.01	5.73	6.65	7.68	8.51	3.54	n/a	53.69
13	0.96	2.39	5.36	4.22	5.43	5.96	9.30	10.78	12.42	12.16	n/a	n/a	68.98
14	1.64	4.04	8.95	7.05	9.07	9.69	16.24	18.83	21.69	20.92	n/a	n/a	118.13
15	0.72	1.86	4.61	3.63	4.65	5.97	5.69	6.60	7.63	8.45	n/a	n/a	49.80
16	0.95	2.44	6.04	4.75	6.09	7.82	7.45	8.65	9.99	11.07	n/a	n/a	65.23

Table 5 – Seismic Shear by Shear Wall

It can be seen that the wind shear controls for the majority of the shear walls. However, the seismic shear controls for Shear Walls 1 and 2 due to the significant torsional effects at the roof level. Note that the 1.6 and 1.0 factors were obtained from load combinations 4 and 5 in Section 2.3.2 of ASCE 7-05.

Drift Analysis

Due to the complexity of the design of the 8th Street Office Building, the individual floor areas and shear walls were combined into one, cohesive, 3D model in ETABS in order to calculate story drift. See Figure 10 for an image of the ETABS models that was created. Once again, the wind loads were applied at the centers of pressure, and the seismic loads were applied at the centers of mass. Gravity elements were not modeled, but the applied seismic loads take the building's weight into account.

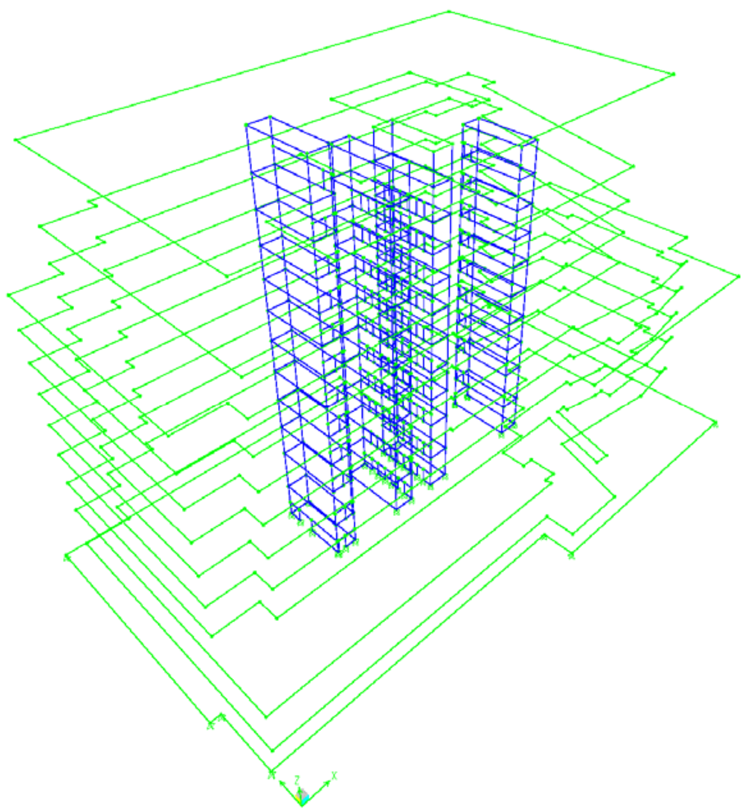

Figure 10 – ETABS Models utilized in Drift Analysis

See Tables 6 and 7 for the East-West and North-South wind drifts obtained from ETABS. The drifts were compared to Δ_{wind} = H/400 according to Section CC.1.2 in the commentary of ASCE 7-05. All wind drifts were found to be acceptable when compared to the allowable drifts. If the wind drifts had been found to be unacceptable, a factor of 0.7 could have been utilized according to Section CC.1.2 of ASCE 7-05.

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East-West Wind Drift									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{wind} = H/400			Total Drift (in)	Allowable Total Drift (in) Δ_{wind} = H/400		
Roof	176.42	0.193	<	0.498	Acceptable	1.54	<	5.29	Acceptable
PH Mezz.	159.83	0.154	<	0.402	Acceptable	1.35	<	4.79	Acceptable
PH	146.42	0.162	<	0.423	Acceptable	1.19	<	4.39	Acceptable
10th	132.33	0.153	<	0.405	Acceptable	1.03	<	3.97	Acceptable
9th	118.83	0.149	<	0.405	Acceptable	0.88	<	3.56	Acceptable
8th	105.33	0.143	<	0.405	Acceptable	0.73	<	3.16	Acceptable
7th	91.83	0.142	<	0.428	Acceptable	0.59	<	2.75	Acceptable
6th	77.58	0.130	<	0.428	Acceptable	0.45	<	2.33	Acceptable
5th	63.33	0.113	<	0.428	Acceptable	0.32	<	1.90	Acceptable
4th	49.08	0.092	<	0.428	Acceptable	0.20	<	1.47	Acceptable
3rd	34.83	0.083	<	0.565	Acceptable	0.11	<	1.04	Acceptable
2nd	16.00	0.029	<	0.480	Acceptable	0.03	<	0.48	Acceptable

Table 6 – East-West Wind Drift

North-South Wind Drift									
Story	Story Height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{wind} = H/400			Total Drift (in)	Allowable Total Drift (in) Δ_{wind} = H/400		
Roof	176.42	0.078	<	0.498	Acceptable	0.453	<	5.29	Acceptable
PH Mezz.	159.83	0.054	<	0.402	Acceptable	0.375	<	4.79	Acceptable
PH	146.42	0.034	<	0.423	Acceptable	0.322	<	4.39	Acceptable
10th	132.33	0.039	<	0.405	Acceptable	0.288	<	3.97	Acceptable
9th	118.83	0.040	<	0.405	Acceptable	0.249	<	3.56	Acceptable
8th	105.33	0.044	<	0.405	Acceptable	0.209	<	3.16	Acceptable
7th	91.83	0.042	<	0.428	Acceptable	0.164	<	2.75	Acceptable
6th	77.58	0.036	<	0.428	Acceptable	0.123	<	2.33	Acceptable
5th	63.33	0.030	<	0.428	Acceptable	0.087	<	1.90	Acceptable
4th	49.08	0.025	<	0.428	Acceptable	0.057	<	1.47	Acceptable
3rd	34.83	0.023	<	0.565	Acceptable	0.032	<	1.04	Acceptable
2nd	16.00	0.009	<	0.480	Acceptable	0.009	<	0.48	Acceptable

Table 7 – North-South Wind Drift

See Tables 8 and 9 for the East-West and North-South seismic drifts obtained from ETABS. These drifts were then modified using a deflection amplification factor of 4.5 from Table 12.2-1 of ASCE 7-05 for ordinary reinforced concrete shear walls as well as an importance factor of 1.25 from Table 11.5-1 of ASCE 7-05. The amplified drifts were compared to $\Delta_{seismic} = 0.015h_{sx}$ according to Table 12.12-1 of ASCE 7-05 for Occupancy Category III and other structures. All seismic drifts were found to be acceptable when compared to the allowable drifts.

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East-West Seismic Drift										
Story	Story Height (ft)	Story Drift (in)	Amplified Story Drift (in)	Allowable Story Drift (in) $\Delta_{seismic} = 0.015h_{sx}$			Amplified Total Drift (in)	Allowable Total Drift (in) $\Delta_{seismic} = 0.015h_{sx}$		
Roof	176.42	0.291	1.047	<	2.986	Acceptable	8.051	<	31.76	Acceptable
PH Mezz.	159.83	0.222	0.798	<	2.414	Acceptable	7.005	<	28.77	Acceptable
PH	146.42	0.241	0.866	<	2.536	Acceptable	6.206	<	26.36	Acceptable
10th	132.33	0.228	0.819	<	2.430	Acceptable	5.340	<	23.82	Acceptable
9th	118.83	0.218	0.785	<	2.430	Acceptable	4.521	<	21.39	Acceptable
8th	105.33	0.210	0.756	<	2.430	Acceptable	3.736	<	18.96	Acceptable
7th	91.83	0.205	0.738	<	2.565	Acceptable	2.980	<	16.53	Acceptable
6th	77.58	0.185	0.667	<	2.565	Acceptable	2.242	<	13.96	Acceptable
5th	63.33	0.160	0.575	<	2.565	Acceptable	1.575	<	11.40	Acceptable
4th	49.08	0.129	0.463	<	2.565	Acceptable	1.000	<	8.83	Acceptable
3rd	34.83	0.112	0.402	<	3.389	Acceptable	0.537	<	6.27	Acceptable
2nd	16.00	0.038	0.135	<	2.880	Acceptable	0.135	<	2.88	Acceptable

Table 8 – East-West Seismic Drift

North-South Seismic Drift										
Story	Story Height (ft)	Story Drift (in)	Amplified Story Drift (in)	Allowable Story Drift (in) $\Delta_{seismic} = 0.015h_{sx}$			Amplified Total Drift (in)	Allowable Total Drift (in) $\Delta_{seismic} = 0.015h_{sx}$		
Roof	176.42	0.053	0.192	<	2.986	Acceptable	1.209	<	31.76	Acceptable
PH Mezz.	159.83	0.033	0.120	<	2.414	Acceptable	1.017	<	28.77	Acceptable
PH	146.42	0.034	0.121	<	2.536	Acceptable	0.897	<	26.36	Acceptable
10th	132.33	0.032	0.115	<	2.430	Acceptable	0.777	<	23.82	Acceptable
9th	118.83	0.031	0.112	<	2.430	Acceptable	0.661	<	21.39	Acceptable
8th	105.33	0.030	0.106	<	2.430	Acceptable	0.549	<	18.96	Acceptable
7th	91.83	0.029	0.104	<	2.565	Acceptable	0.443	<	16.53	Acceptable
6th	77.58	0.026	0.095	<	2.565	Acceptable	0.338	<	13.96	Acceptable
5th	63.33	0.024	0.085	<	2.565	Acceptable	0.243	<	11.40	Acceptable
4th	49.08	0.019	0.070	<	2.565	Acceptable	0.158	<	8.83	Acceptable
3rd	34.83	0.018	0.065	<	3.389	Acceptable	0.088	<	6.27	Acceptable
2nd	16.00	0.007	0.023	<	2.880	Acceptable	0.023	<	2.88	Acceptable

Table 9 – North-South Seismic Drift

Lateral Member Spot Checks for Strength

A shear wall spot check was performed for Shear Wall 2 since it was found to have extremely large total wind shear and seismic shear. The single opening in the shear wall was considered negligible. Load combinations 4 and 5 were considered, and it was determined that the seismic lateral loads controlled. Therefore, Chapter 21 of ACI-08 was utilized in the spot check. Figure 11 shows the applied seismic loads and the factored axial load, shear, and moment that were used in the spot check. See Appendix F for detailed calculations, including a gravity load takedown for the shear wall.

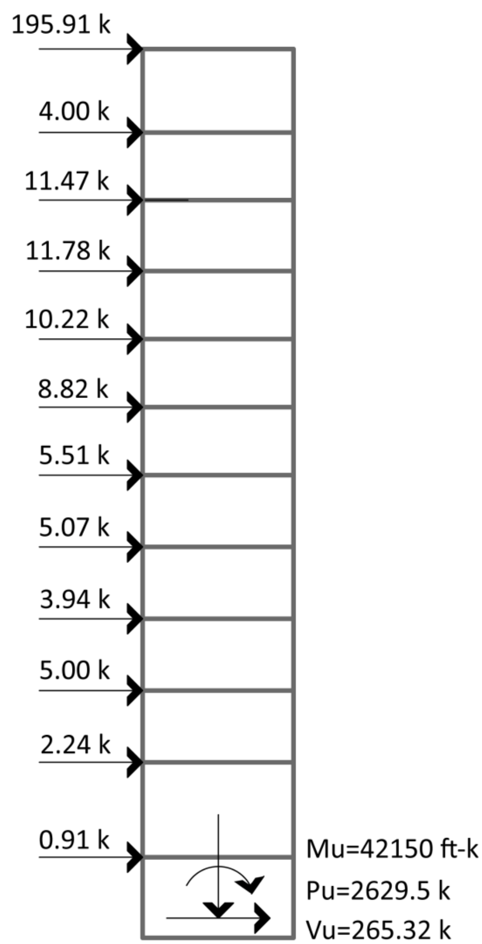

Figure 11 – Shear Wall 2 Controlling Load Combination

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It was determined in the spot check that the horizontal reinforcing of #6 bars spaced at 12" on center for each face and the vertical reinforcement of #8 bars spaced at 12" on center for each face is more than adequate. However, it was discovered that boundary elements with longitudinal reinforcement of 25.9 in^2 are necessary to handle the tension force. Therefore, a larger boundary element than the 12" constant thickness with 4 #10 bars is required, which may have a negative impact on the architecture of the 8th Street Office Building. A suggestion is to use 36" by 48" boundary elements with 18 #11 bars.

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Overturning and Foundation Impact

It was assumed that wide beam shear would be the governing failure condition for the areas of the mat foundation supporting the shear walls. The spot check of Shear Wall 2 was continued to include a check of wide beam shear. The area of interest of the foundation was determined according to a 45° crack pattern around the perimeter of the shear wall. The same loads from the check of the shear wall were used under the assumption that the conservative moment will counteract the gravity load from the parking garage that is neglected. See Appendix F for detailed calculations. It was determined that the mat foundation is adequate to resist wide beam shear.

Finally, a brief check of overturning was performed. Load combination 7 from Section 2.3.2 of ASCE 7-05 was used in order to be conservative. It was found that the dead load is adequate to resist the overturning load due to the seismic induced moment. See Appendix F for calculations.

Conclusion

In the third technical report regarding the 8th Street Office Building, a detailed lateral analysis was performed, and strength and serviceability requirements of the lateral-resisting system were investigated. An ETABS model was created to assist with calculations that would have been too cumbersome to perform by hand due to the complex geometry of the 8th Street Office Building. Centers of mass, shear wall displacements due to a unit load, and story drifts were obtained from the ETABS model. It was assumed that the floor systems act as infinitely rigid diaphragms, and lateral forces are distributed to each shear wall in relation to the wall's relative stiffness.

Wind loads were applied to the centers of pressure separately in the East-West and North-South directions according to Case 1 of ASCE 7-05. Seismic loads were applied to the centers of mass in both the East-West and North-South directions. Direct shear and torsional shear due to eccentricity of the center of rigidity were calculated by shear wall and floor. Net shears were then tabulated and load factors applied for easy comparison of the most critical shear walls. The load combinations of interest were combinations 4 and 5 from ASCE 7-05. Typically, wind was the governing load when assessing the total, factored shears on the shear walls.

The story drifts obtained from the ETABS model were compared to the allowable drifts in ASCE 7-05. Drifts due to wind and earthquake were all deemed to be acceptable. The largest overall building displacements were in the East-West direction due to the geometry of the 8th Street Office Building.

It was decided that Shear Wall 2 was controlled by earthquake, so it was spot checked according to Chapter 21 of ACI-08. The horizontal reinforcement of #6 bars spaced at 12" on center for each face and the vertical reinforcement of #8 bars spaced at 12" on center for each face was found to be sufficient. However, it was discovered that boundary elements with longitudinal reinforcement of 25.9 in^2 are necessary to handle the tension force. Therefore, it was suggested that 36" by 48" boundary elements with 18 #11 bars may be used. In addition to checking the actual shear wall, the spot check was continued to include a wide beam shear check of the appropriate mat foundation area supporting Shear Wall 2. It was concluded that the provided 49.8k is much larger than the needed 11.5k, so wide beam shear is not an issue. Finally, an overturning check was performed, and no problems were discovered as expected.

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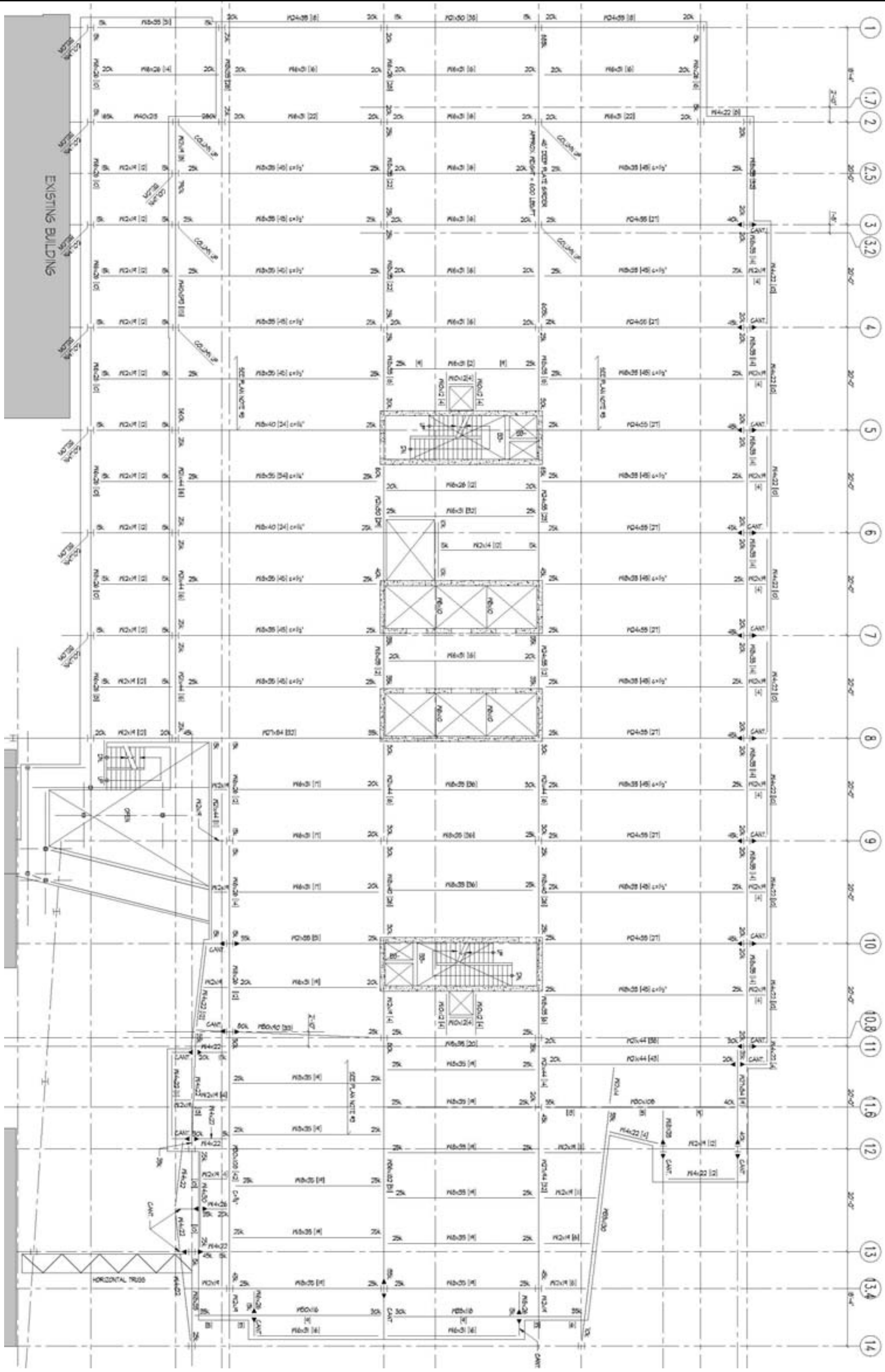
Appendix A – Typical Framing Plans



3rd Floor Framing Plan

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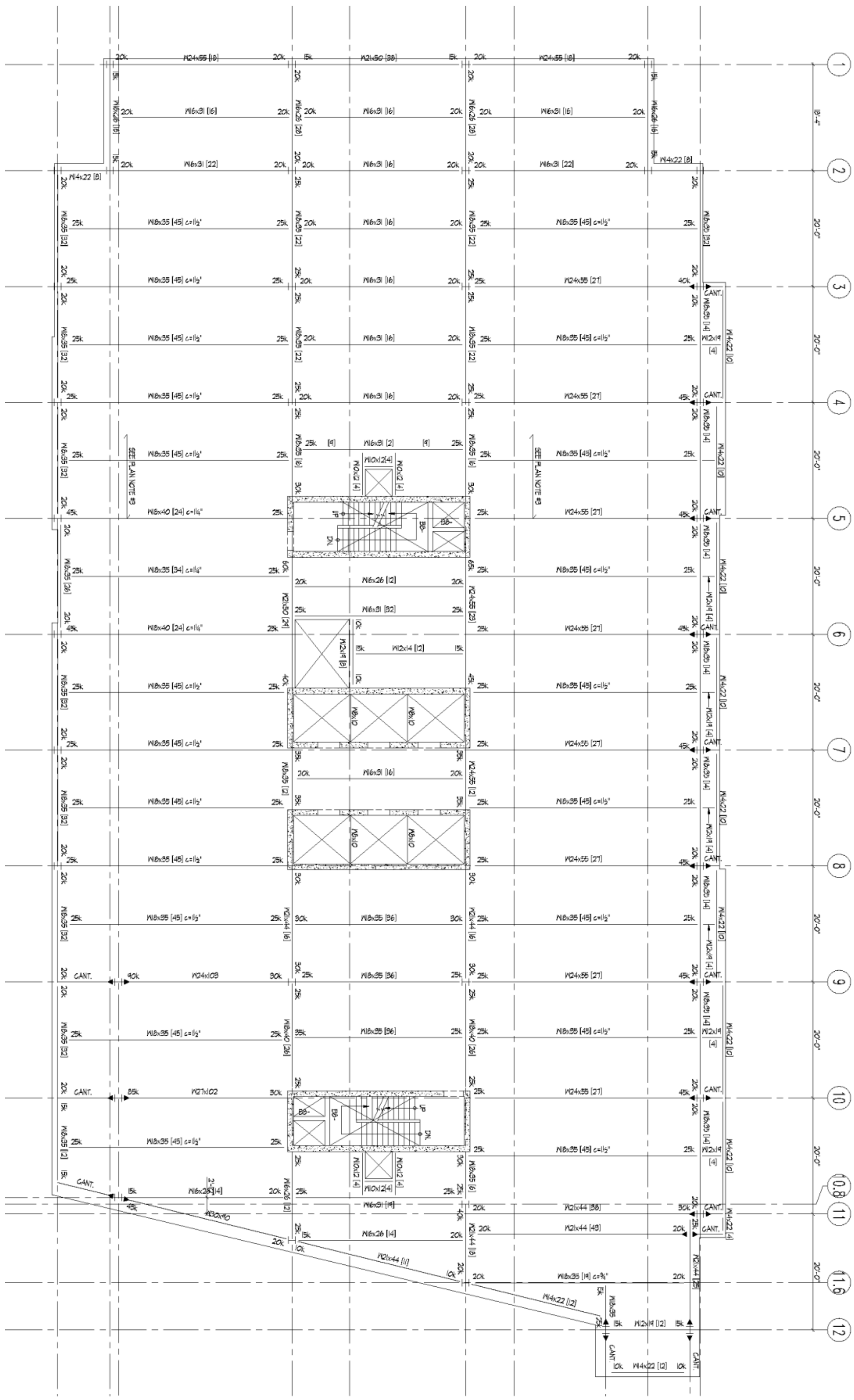
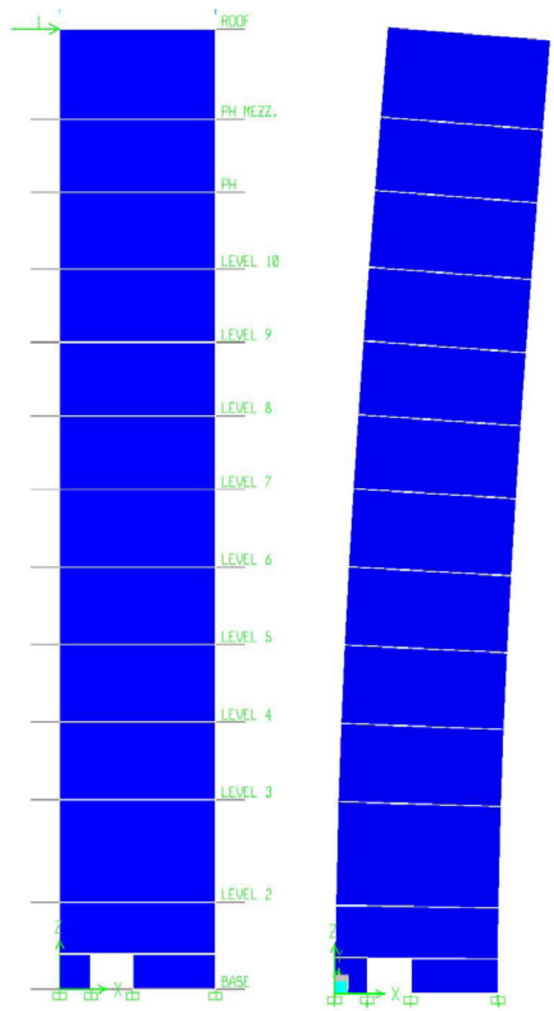


8th Floor Framing Plan

Appendix B – Shear Wall Details and Stiffnesses



Shear Wall 1 Details

Displacement = 0.0194 inches ; Stiffness = 51.55

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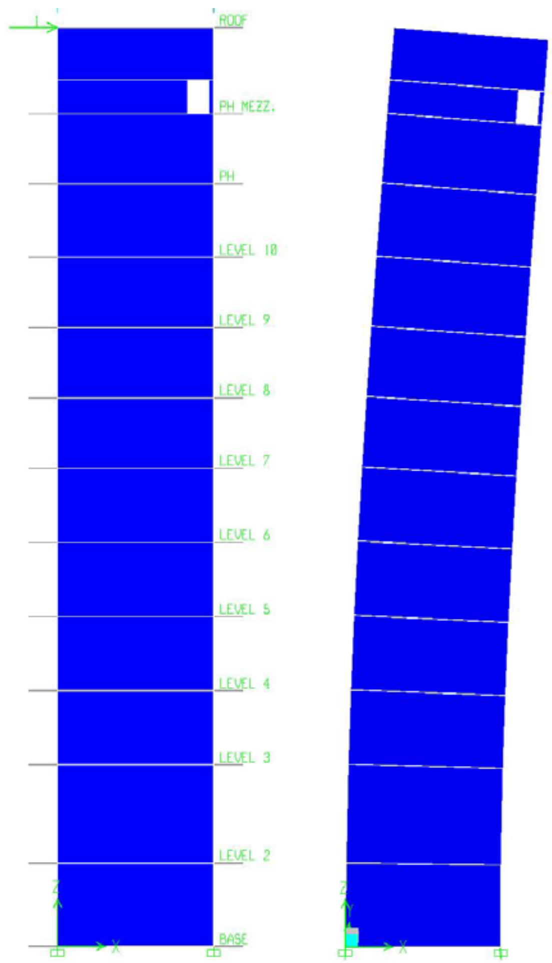

Shear Wall 2 Details

Displacement = 0.0186 inches ; Stiffness = 53.76

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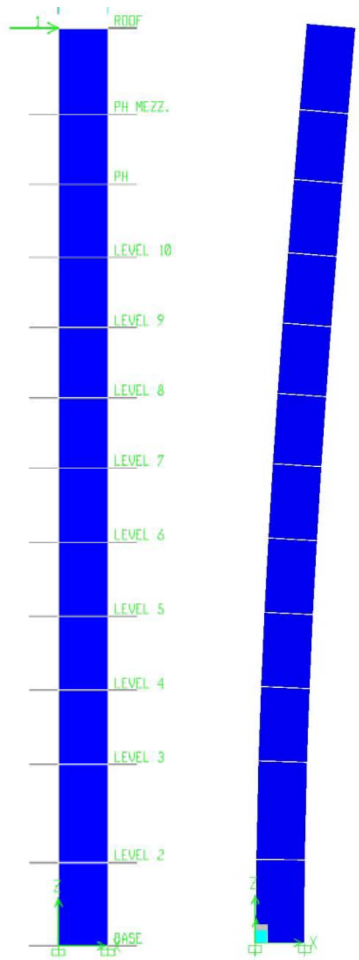
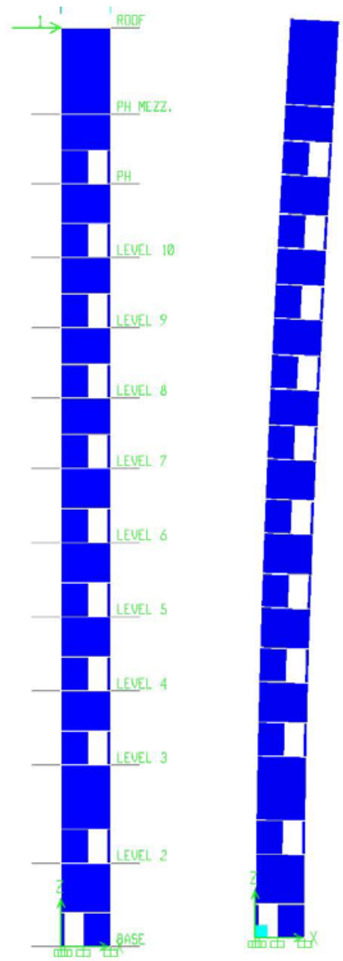


Shear Wall 3 Details

Displacement = 0.5896 inches ; Stiffness = 1.696

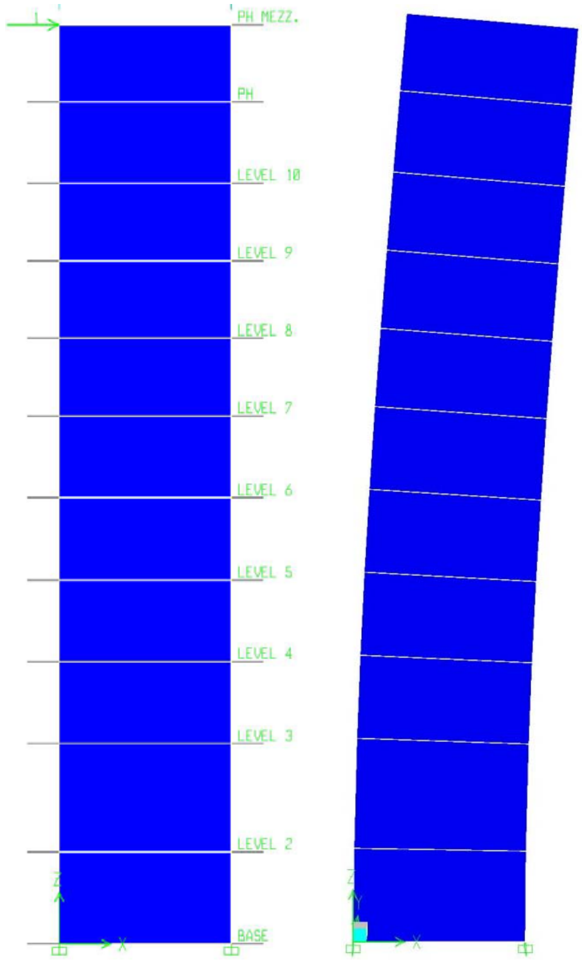
Technical Report #3



Shear Wall 4 Details

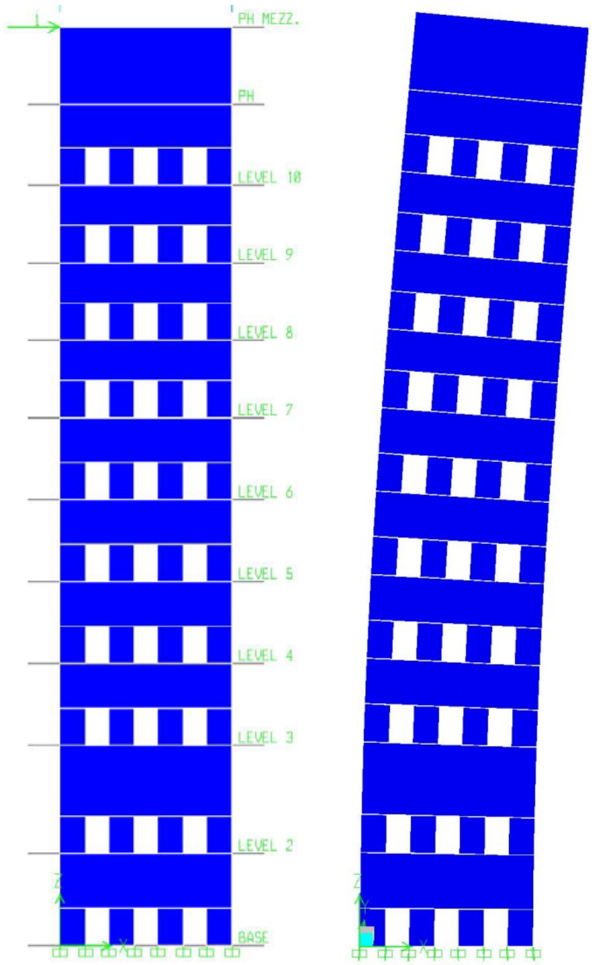
Displacement = 0.7953 inches ; Stiffness = 1.257

Technical Report #3



Shear Walls 5 & 10 Details

Displacement = 0.0139 inches ; Stiffness = 71.94



Shear Walls 6 & 9 Details

Displacement = 0.0232 inches ; Stiffness = 43.103

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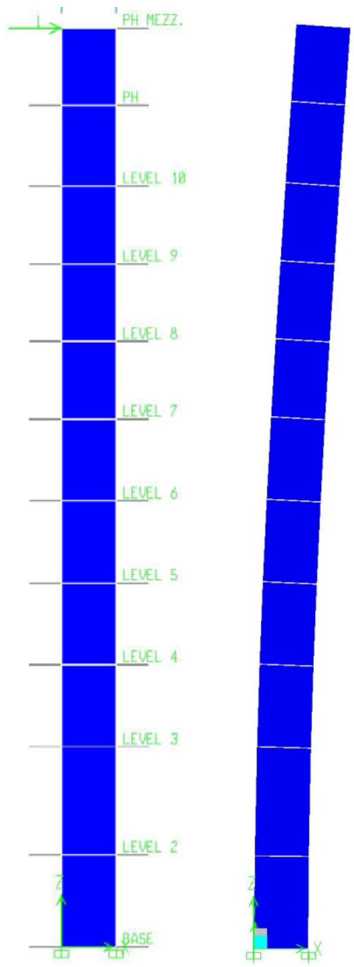
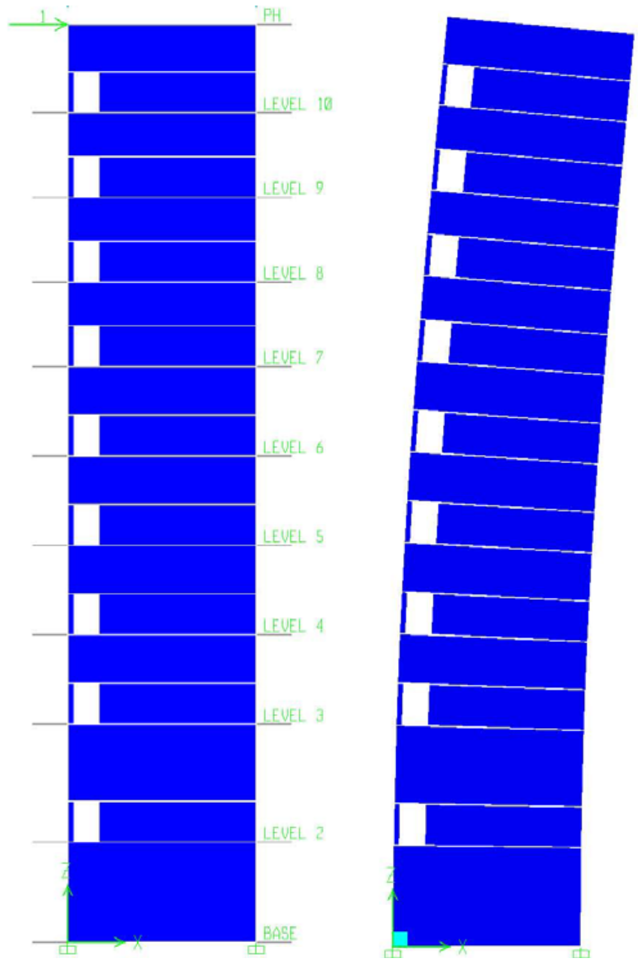


Shear Walls 7, 8, 11 & 12 Details

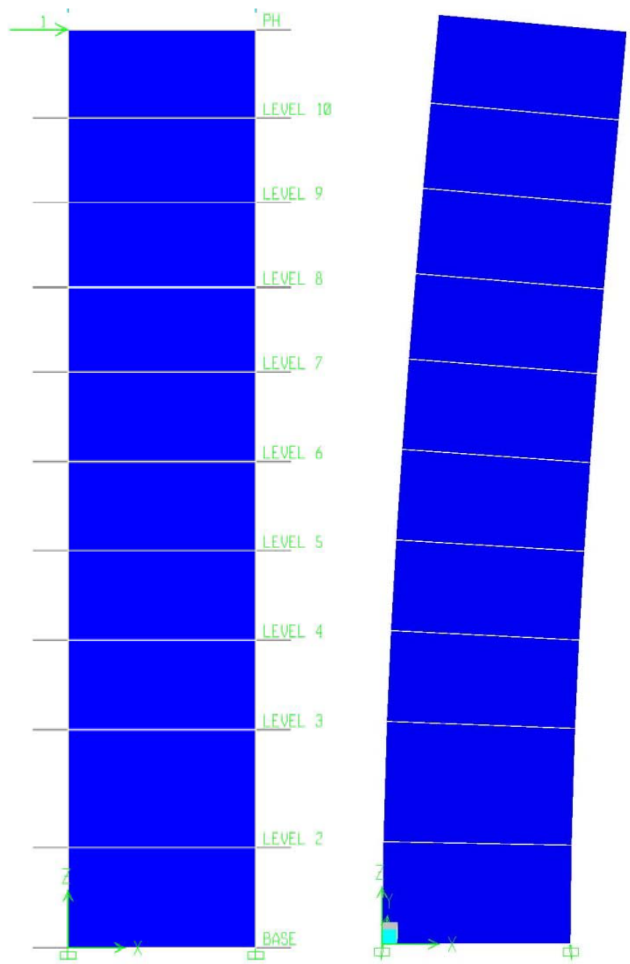
Displacement = 0.4385 inches ; Stiffness = 2.280

Technical Report #3



Shear Wall 13 Details

Displacement = 0.0171 inches ; Stiffness = 58.48



Shear Wall 14 Details

Displacement = 0.0107 inches ; Stiffness = 93.458

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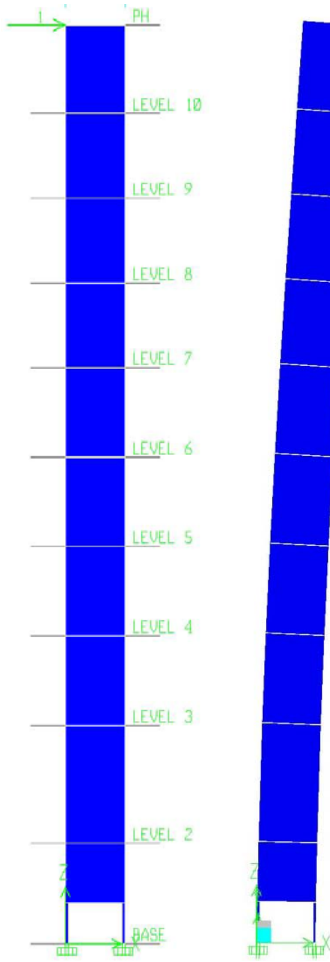
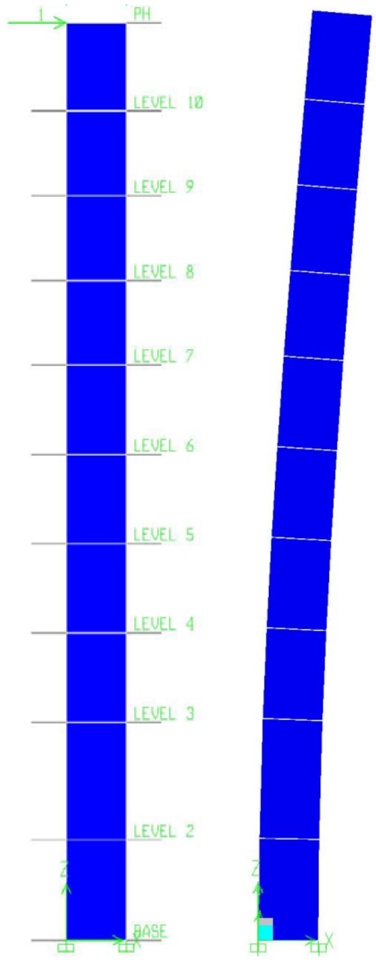


Shear Wall 15 Details

Displacement = 0.4416 inches ; Stiffness = 2.264



Shear Wall 16 Details

Displacement = 0.3371 inches ; Stiffness = 2.966

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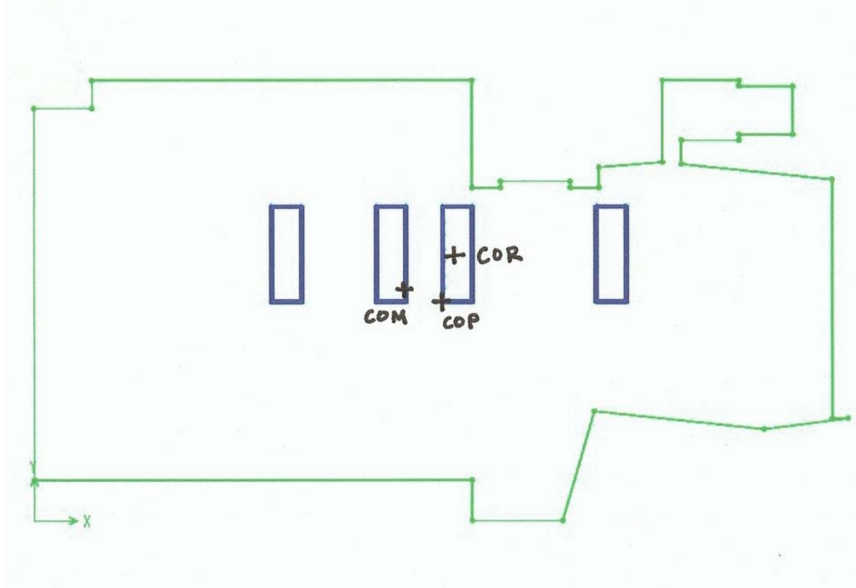
Summary of Stiffnesses and Distances Used to Calculate Centers of Rigidity:

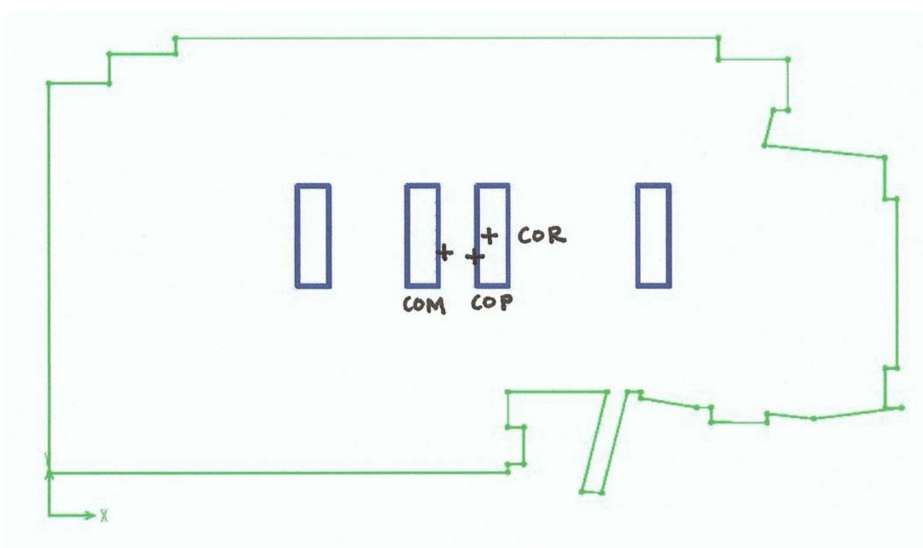
SW	Stiffness	x-distance (ft.)	y-distance (ft.)	Floors
1	51.55	75.052	n/a	1 - roof
2	53.76	84.552	n/a	1 - roof
3	1.70	n/a	100.17	1 - roof
4	1.26	n/a	69.67	1 - roof
5	71.94	108.135	n/a	1 - PH Mezz.
6	43.10	117.47	n/a	1 - PH Mezz.
7	2.28	n/a	100.17	1 - PH Mezz.
8	2.28	n/a	69.67	1 - PH Mezz.
9	43.10	129.135	n/a	1 - PH Mezz.
10	71.94	138.469	n/a	1 - PH Mezz.
11	2.28	n/a	100.17	1 - PH Mezz.
12	2.28	n/a	69.67	1 - PH Mezz.
13	58.48	177.552	n/a	1 - PH
14	93.46	187.052	n/a	1 - PH
15	2.26	n/a	100.17	1 - PH
16	2.97	n/a	69.67	1 - PH

Appendix C – Centers of Mass, Pressure, and Rigidity



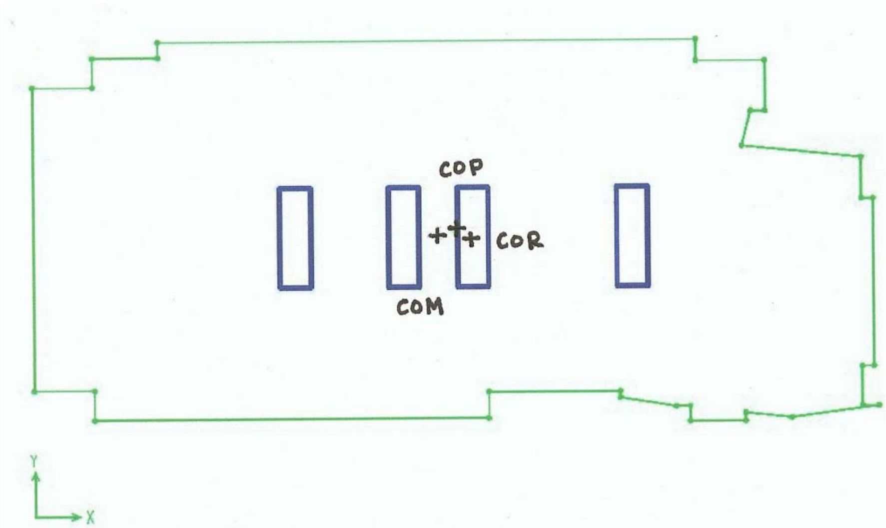
2nd Floor

Center of Mass = (116.83, 73.75) ; Center of Pressure = (128.33, 70.21) ; Center of Rigidity = (132.66, 84.69)



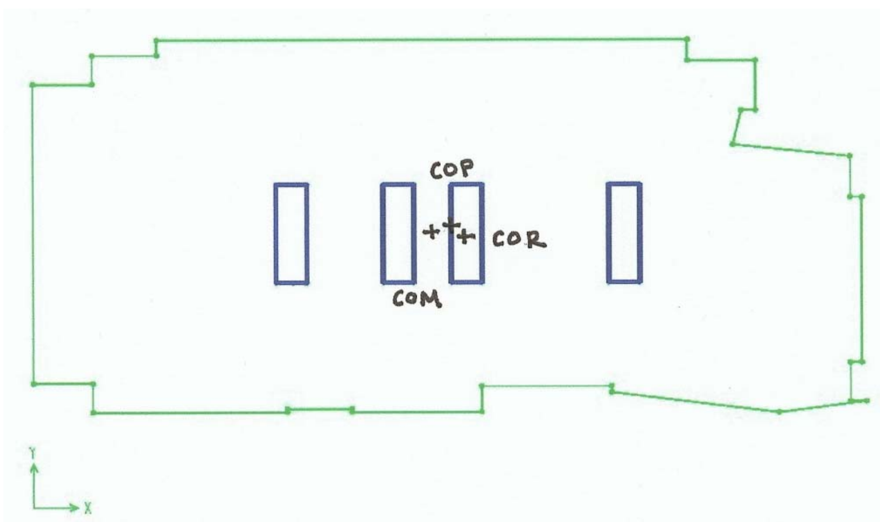
3rd Floor

Center of Mass = (118.94, 80.08) ; Center of Pressure = (128.33, 79.04) ; Center of Rigidity = (132.66, 84.69)



4th & 5th Floors

Center of Mass = (123.42, 85.82) ; Center of Pressure = (128.33, 87.13) ; Center of Rigidity = (132.66, 84.69)



6th Floor

Center of Mass = (123.24, 86.01) ; Center of Pressure = (128.33, 87.13) ; Center of Rigidity = (132.66, 84.69)

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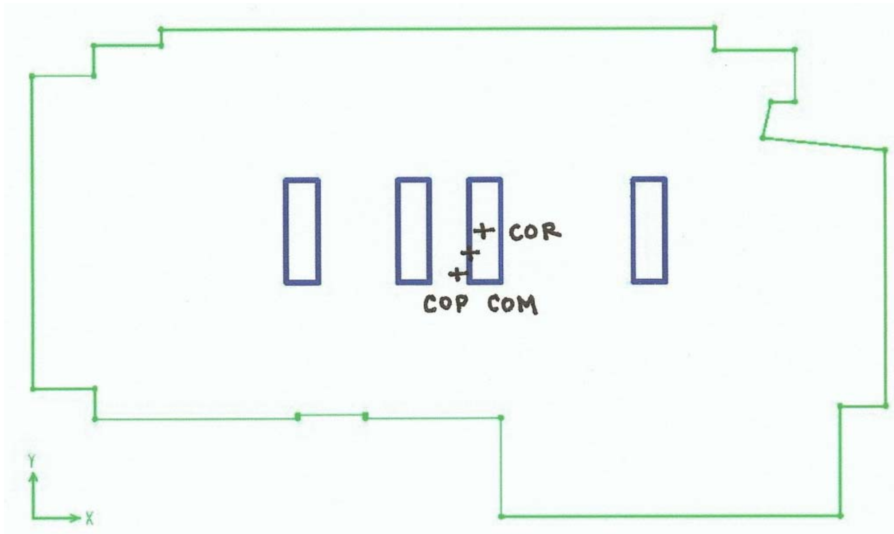
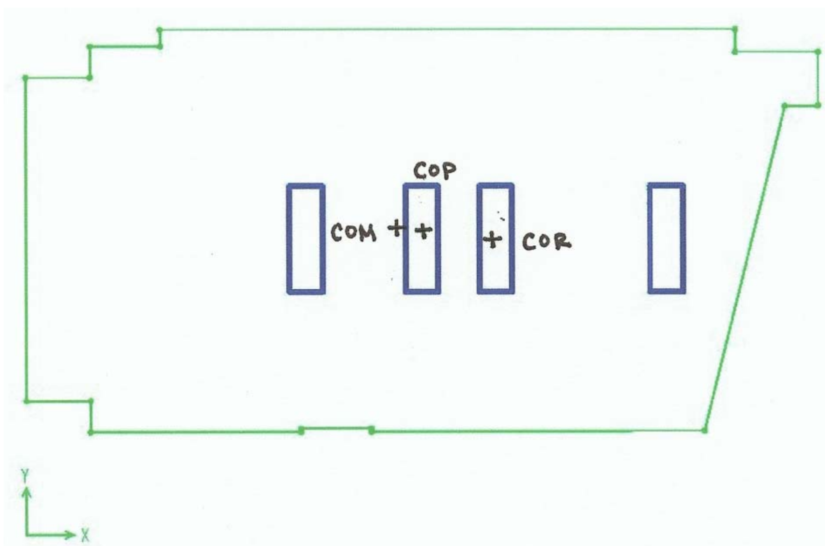


7th Floor

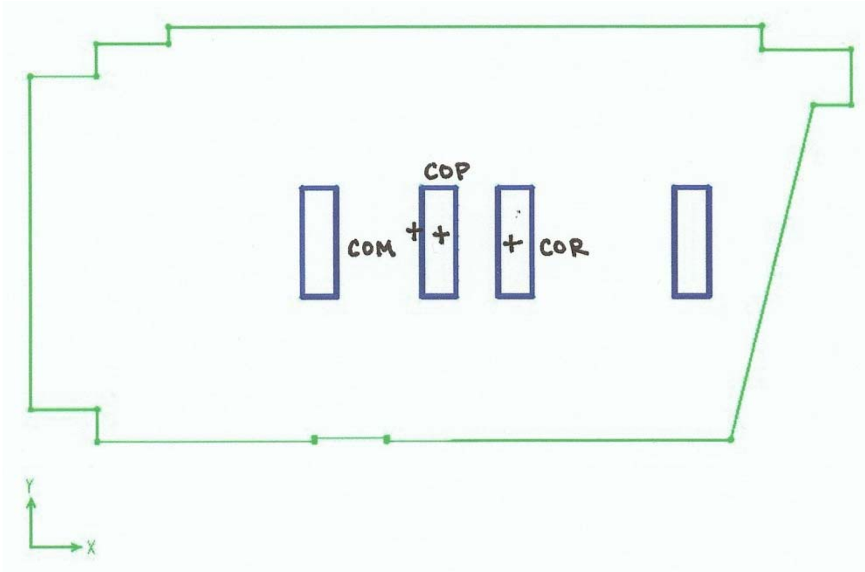
Center of Mass = (129.55, 78.47) ; Center of Pressure = (125.92, 72.59) ; Center of Rigidity = (132.66, 84.69)



8th Floor

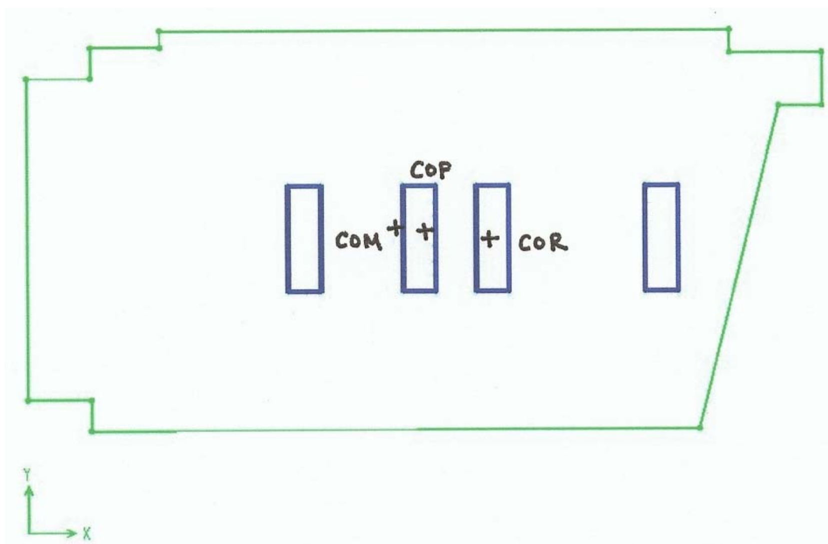
Center of Mass = (105.74, 88.26) ; Center of Pressure = (112.67, 87.30) ; Center of Rigidity = (132.66, 84.69)

Technical Report #3



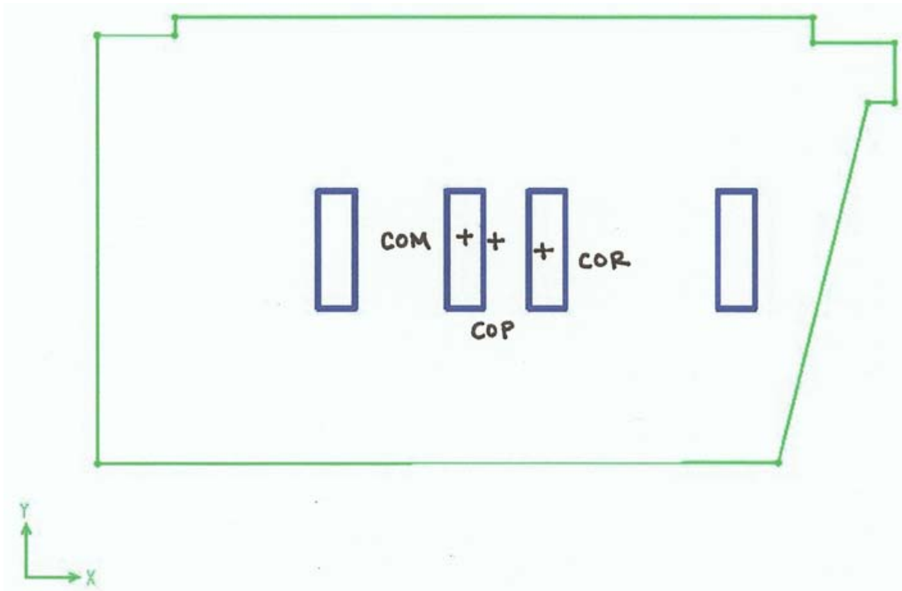
9th Floor

Center of Mass = (105.82, 88.29) ; Center of Pressure = (113.17, 87.30) ; Center of Rigidity = (132.66, 84.69)



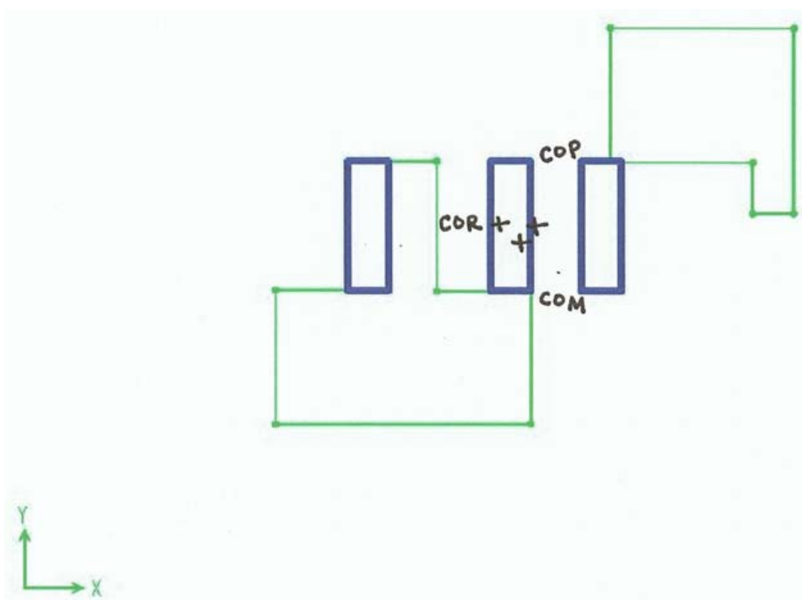
10th Floor

Center of Mass = (105.97, 88.32) ; Center of Pressure = (114.17, 87.30) ; Center of Rigidity = (132.66, 84.69)



Penthouse

Center of Mass = (113.10, 88.42) ; Center of Pressure = (120.54, 87.30) ; Center of Rigidity = (132.66, 84.69)



Penthouse Mezzanine

Center of Mass = (114.85, 81.36) ; Center of Pressure = (118.33, 84.92) ; Center of Rigidity = (109.68, 85.47)

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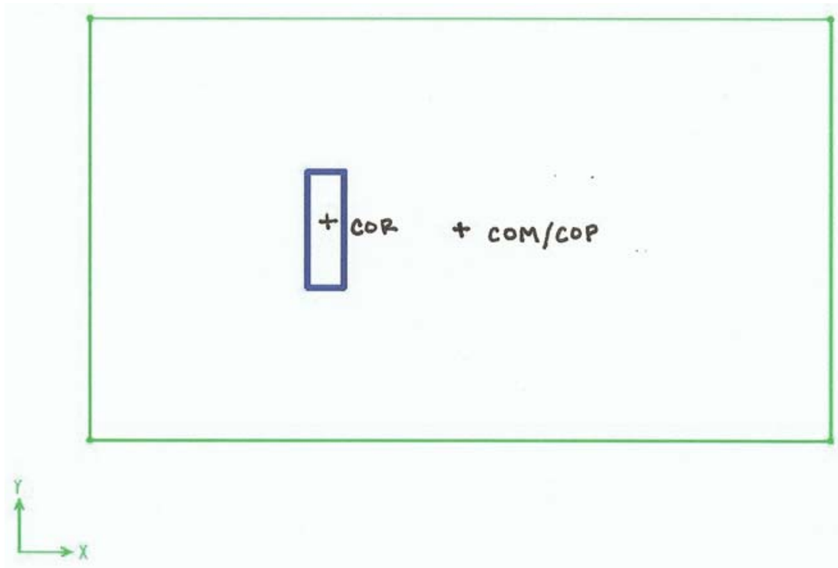

Roof

Center of Mass = (114.58, 84.92) ; Center of Pressure = (114.58, 84.92) ; Center of Rigidity = (79.90, 87.19)

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Appendix D – Wind Loads

Wind Variables		ASCE 7-05 Reference
V	90	(Fig. 6-1)
K_d	0.85	(Table 6-4)
I	1.15	(Table 6-1)
Exposure Category	B	
K_{zt}	1	(Sec. 6.5.7.1)
Enclosure Classification	Enclosed	(Sec. 6.2)
GC_{pi}	± 0.18	(Fig. 6-5)

Wind Analysis 1:

Gust Effect Factor			
	N-S	E-W	ASCE 7-05 Reference
B	260'-8"	145'-3"	(Sec. 6.3)
L	145'-3"	260'-3"	(Sec. 6.3)
h	176'-5"		(Sec. 6.3)
n_1	0.567		(Eq. C6-17)
Structure	Flexible		(Sec. 6.2)
g_r	4.052		(Eq. 6-9)
$\bar{z}$	105.85		(Table 6-2)
$\bar{V}_{\bar{z}}$	79.49		(Eq. 6-14)
$I_{\bar{z}}$	0.247		(Eq. 6-5)
$L_{\bar{z}}$	471.93		(Eq. 6-7)
Q	0.790	0.818	(Eq. 6-6)
R_h	0.158		(Eq. 6-13a)
η=	5.789		
R_B	0.110	0.188	(Eq. 6-13a)
η=	8.553	4.766	
R_L	0.061	0.034	(Eq. 6-13a)
η=	15.955	28.634	
N_1	3.366		(Eq. 6-12)
R_n	0.065		(Eq. 6-11)
β	1.50%		(Sec. C6.5.8)
R	0.205	0.265	(Eq. 6-10)
G_f	0.831	0.858	(Eq. 6-8)

External Pressure Coefficient C_p			
	N-S	E-W	ASCE 7-05 Reference
Windward Wall	0.8	0.8	(Fig. 6-6)
Leeward Wall	-0.5	-0.341	(Fig. 6-6)

	Level	Elevation	Floor-to-Floor Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)					
							N-S			E-W		
							+ 0.18	- 0.18	Net	+ 0.18	- 0.18	Net
Windward	1	172'-0"	16.00	0	-	-	-	-	-	-	-	-
	2	188'-0"	18.83	16.00	0.58	11.76	3.57	12.06	7.82	3.83	12.31	8.07
	3	206'-10"	14.25	34.83	0.73	14.78	5.58	14.07	9.82	5.90	14.38	10.14
	4	221'-1"	14.25	49.08	0.81	16.33	6.61	15.10	10.85	6.96	15.45	11.21
	5	235'-4"	14.25	63.33	0.86	17.50	7.39	15.88	11.63	7.77	16.25	12.01
	6	249'-7"	14.25	77.58	0.92	18.65	8.16	16.64	12.40	8.56	17.05	12.80
	7	263'-10"	13.50	91.83	0.97	19.57	8.77	17.25	13.01	9.19	17.68	13.43
	8	277'-4"	13.50	105.33	1.00	20.34	9.28	17.76	13.52	9.72	18.20	13.96
	9	290'-10"	13.50	118.83	1.04	21.02	9.73	18.22	13.97	10.19	18.67	14.43
	10	304'-4"	14.08	132.33	1.07	21.71	10.19	18.67	14.43	10.66	19.14	14.90
	PH	318'-5"	13.42	146.42	1.10	22.35	10.62	19.10	14.86	11.10	19.59	15.34
	PF Mezz.	331'-10"	16.58	159.83	1.13	22.90	10.98	19.46	15.22	11.47	19.96	15.72
	Roof	348'-5"	-	176.42	1.16	23.57	11.43	19.91	15.67	11.94	20.42	16.18
Leeward	All	348'-5"	-	176.42	1.16	23.57	-14.04	-5.55	-9.79	-11.14	-2.65	-6.90

Wind Analysis 2:

Gust Effect Factor			
	N-S	E-W	ASCE 7-05 Reference
B	260'-8"	145'-3"	(Sec. 6.3)
L	145'-3"	260'-3"	(Sec. 6.3)
h	176'-5"		(Sec. 6.3)
n_1	0.567		(Eq. C6-17)
Structure	Flexible		(Sec. 6.2)
g_r	4.052		(Eq. 6-9)
$\bar{z}$	105.85		(Table 6-2)
$\bar{V}_{\bar{z}}$	79.49		(Eq. 6-14)
$I_{\bar{z}}$	0.247		(Eq. 6-5)
$L_{\bar{z}}$	471.93		(Eq. 6-7)
Q	0.798	0.825	(Eq. 6-6)
R_h	0.158		(Eq. 6-13a)
η=	5.789		
R_B	0.125	0.225	(Eq. 6-13a)
η=	7.481	3.872	
R_L	0.074	0.039	(Eq. 6-13a)
η=	12.962	25.045	
N_1	3.366		(Eq. 6-12)
R_n	0.065		(Eq. 6-11)
β	1.50%		(Sec. C6.5.8)
R	0.22	0.291	(Eq. 6-10)
G_f	0.838	0.868	(Eq. 6-8)

External Pressure Coefficient C_p			
	N-S	E-W	ASCE 7-05 Reference
Windward Wall	0.8	0.8	(Fig. 6-6)
Leeward Wall	-0.5	-0.314	(Fig. 6-6)

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	Level	Elevation	Floor-to-Floor Height (ft)	Height Above Ground (ft)	K_z	q_z	Wind Pressure (psf)					
							N-S			E-W		
							+ 0.18	- 0.18	Net	+ 0.18	- 0.18	Net
Windward	1	172'-0"	16.00	0	-	-	-	-	-	-	-	-
	2	188'-0"	18.83	16.00	0.58	11.76	3.64	12.12	7.88	3.92	12.41	8.16
	3	206'-10"	14.25	34.83	0.73	14.78	5.66	14.15	9.91	6.02	14.50	10.26
	4	221'-1"	14.25	49.08	0.81	16.33	6.70	15.19	10.94	7.09	15.58	11.34
	5	235'-4"	14.25	63.33	0.86	17.50	7.49	15.97	11.73	7.91	16.39	12.15
	6	249'-7"	14.25	77.58	0.92	18.65	8.26	16.75	12.51	8.71	17.20	12.95
	7	263'-10"	13.50	91.83	0.97	19.57	8.88	17.36	13.12	9.35	17.83	13.59
	8	277'-4"	13.50	105.33	1.00	20.34	9.39	17.88	13.63	9.88	18.36	14.12
	9	290'-10"	13.50	118.83	1.04	21.02	9.85	18.34	14.09	10.35	18.84	14.60
	10	304'-4"	14.08	132.33	1.07	21.71	10.31	18.79	14.55	10.83	19.31	15.07
	PH	318'-5"	13.42	146.42	1.10	22.35	10.74	19.23	14.99	11.28	19.77	15.52
	PH Mezz.	331'-10"	16.58	159.83	1.13	22.90	11.11	19.59	15.35	11.66	20.14	15.90
	Roof	348'-5"	-	176.42	1.16	23.57	11.56	20.04	15.80	12.12	20.61	16.37
Leeward	All	348'-5"	-	176.42	1.16	23.57	-14.12	-5.63	-9.88	-10.67	-2.18	-6.42

Combined Results of Wind Analyses:

Level	Floor-to-Floor Height (ft)	Height Above Ground (ft)	Controlling Windward Pressure (psf)		Controlling Leeward Pressure (psf)		Total Controlling Pressure (psf)		Wind Forces					
									Load (kips)		Shear (kips)		Moment (ft-kips)	
			N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W	N-S	E-W
1	16.00	0	-	-	-	-	-	-	0.0	0.0	866.1	408.0	0	0
2	18.83	16.00	7.88	8.16	-9.88	-6.90	17.76	15.06	85.6	40.5	866.1	408.0	1370	648
3	14.25	34.83	9.91	10.26	-9.88	-6.90	19.79	17.16	87.2	42.8	778.8	367.5	3039	1492
4	14.25	49.08	10.94	11.34	-9.88	-6.90	20.82	18.24	78.8	39.0	691.6	324.7	3868	1916
5	14.25	63.33	11.73	12.15	-9.88	-6.90	21.61	19.05	81.7	40.7	612.8	285.6	5176	2579
6	14.25	77.58	12.51	12.95	-9.88	-6.90	22.39	19.85	84.3	42.2	531.1	244.9	6540	3276
7	13.50	91.83	13.12	13.59	-9.88	-6.90	23.00	20.49	73.7	34.1	446.8	236.8	6764	3131
8	13.50	105.33	13.63	14.12	-9.88	-6.90	23.51	21.02	73.8	34.0	373.1	202.7	7776	3580
9	13.50	118.83	14.09	14.60	-9.88	-6.90	23.97	21.50	75.6	34.7	299.3	168.7	8981	4129
10	14.08	132.33	14.55	15.07	-9.88	-6.90	24.43	21.97	73.8	36.3	223.7	134.0	9764	4798
PH	13.42	146.42	14.99	15.52	-9.88	-6.90	24.87	22.42	51.0	35.7	149.9	97.7	7463	5234
PH Mezz.	16.58	159.83	15.35	15.90	-9.88	-6.90	25.23	22.80	56.6	39.8	99.0	62.0	9041	6358
Roof	-	176.42	15.80	16.37	-9.88	-6.90	25.68	23.27	42.4	22.2	42.4	22.2	7482	3914

East – West Wind Direct & Torsional Shears:

East-West Wind Loads

Shear Wall	Level 2		Level 3		Level 4		Level 5		Level 6		Level 7	
	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)
3	3.97	0.01	4.19	0.00	3.82	0.00	3.99	0.00	4.14	0.00	3.34	0.01
4	2.94	0.00	3.11	0.00	2.83	0.00	2.96	0.00	3.07	0.00	2.48	0.01
7	5.34	0.01	5.64	0.00	5.14	0.00	5.36	0.00	5.56	0.00	4.49	0.01
8	5.34	0.01	5.64	0.00	5.14	0.00	5.36	0.00	5.56	0.00	4.49	0.01
11	5.34	0.01	5.64	0.00	5.14	0.00	5.36	0.00	5.56	0.00	4.49	0.01
12	5.34	0.01	5.64	0.00	5.14	0.00	5.36	0.00	5.56	0.00	4.49	0.01
15	5.30	0.01	5.60	0.00	5.10	0.00	5.33	0.00	5.52	0.00	4.46	0.01
16	6.94	0.01	7.34	0.00	6.68	0.00	6.98	0.00	7.23	0.00	5.85	0.01

Shear Wall	Level 8		Level 9		Level 10		Penthouse		PH Mezzanine		Roof	
	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)
3	3.33	0.00	3.40	0.00	3.56	0.00	3.50	0.00	5.59	0.02	12.75	0.00
4	2.47	0.00	2.52	0.00	2.64	0.00	2.59	0.00	4.14	0.02	9.45	0.00
7	4.48	0.00	4.57	0.00	4.78	0.00	4.70	0.00	7.52	0.03	n/a	n/a
8	4.48	0.00	4.57	0.00	4.78	0.00	4.70	0.00	7.52	0.03	n/a	n/a
11	4.48	0.00	4.57	0.00	4.78	0.00	4.70	0.00	7.52	0.03	n/a	n/a
12	4.48	0.00	4.57	0.00	4.78	0.00	4.70	0.00	7.52	0.03	n/a	n/a
15	4.45	0.00	4.54	0.00	4.75	0.00	4.67	0.00	n/a	n/a	n/a	n/a
16	5.83	0.00	5.95	0.00	6.22	0.00	6.12	0.00	n/a	n/a	n/a	n/a

North – South Wind Direct & Torsional Shears:

North-South Wind Loads

Shear Wall	Level 2		Level 3		Level 4		Level 5		Level 6		Level 7	
	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)
1	9.05	3.90	9.22	3.24	8.33	1.53	8.64	1.59	8.92	1.70	7.80	1.06
2	9.44	3.40	9.62	2.83	8.69	1.33	9.01	1.38	9.30	1.48	8.13	0.92
5	12.64	2.32	12.87	1.93	11.63	0.91	12.06	0.94	12.44	1.01	10.88	0.63
6	7.57	0.86	7.71	0.72	6.97	0.34	7.23	0.35	7.46	0.38	6.52	0.23
9	7.57	0.20	7.71	0.17	6.97	0.08	7.23	0.08	7.46	0.09	6.52	0.05
10	12.64	0.55	12.87	0.46	11.63	0.22	12.06	0.22	12.44	0.24	10.88	0.15
13	10.27	3.45	10.46	2.87	9.46	1.35	9.80	1.40	10.12	1.50	8.84	0.94
14	16.42	6.68	16.72	5.55	15.11	2.62	15.67	2.72	16.17	2.91	14.13	1.81

Shear Wall	Level 8		Level 9		Level 10		Penthouse		PH Mezzanine		Roof	
	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)
1	7.81	2.02	8.00	2.20	7.81	2.40	5.39	1.50	8.70	1.99	20.75	0.00
2	8.14	1.76	8.34	1.92	8.14	2.09	5.63	1.31	9.07	1.50	21.65	0.00
5	10.89	1.20	11.16	1.31	10.89	1.42	7.53	0.89	12.14	0.12	n/a	n/a
6	6.53	0.45	6.69	0.48	6.53	0.53	4.51	0.33	7.27	0.37	n/a	n/a
9	6.53	0.10	6.69	0.11	6.53	0.12	4.51	0.08	7.27	0.93	n/a	n/a
10	10.89	0.28	11.16	0.31	10.89	0.34	7.53	0.21	12.14	2.30	n/a	n/a
13	8.86	1.79	9.07	1.94	8.86	2.12	6.12	1.33	n/a	n/a	n/a	n/a
14	14.15	3.46	14.50	3.76	14.15	4.10	9.78	2.57	n/a	n/a	n/a	n/a

Appendix E – Seismic Loads

Seismic Variables		ASCE 7-05 Reference
S_s	0.23	(Fig. 22-1)
S_1	0.06	(Fig. 22-2)
Site Classification	C	(Table 20.3-1)
F_a	1.2	(Table 11.4-1)
F_v	1.7	(Table 11.4-2)
S_{MS}	0.276	(Eq. 11.4-1)
S_{M1}	0.102	(Eq. 11.4-2)
S_{DS}	0.184	(Eq. 11.4-3)
S_{D1}	0.068	(Eq. 11.4-4)
Occupancy Category	III	(Table 1-1)
I	1.25	(Table 11.5-1)
Seismic Design Category	B	(Tables 11.6-1 & 11.6-2)
Equivalent Lateral Force Procedure permitted by (Table 12.6-1)		
T_L	8	(Fig. 22-15)
C_t	0.02	(Table 12.8-2)
x	0.75	(Table 12.8-2)
T_a	0.968	(Eq. 12.8-7)
C_u	1.7	(Table 12.8-1)
T	1.645	(Sec. 12.8.2)
R	5	(Table 12.2-1)
C_s	0.0103	(Eqs. 12.8-2, 12.8-3 & 12.8-5)
W	44481	(Sec. 12.7.2)
V	458	(Eq. 12.8-1)
k	1.234	(Eq. 12.8-12)

Level	Weight w_x (kips)	Height h_x (ft)	$w_x h_x^k$	C_{vx}	Lateral Force Fx (kips)	Story Shear Vx (kips)	Moment Mx (kips)
2	4574	16.00	140017	0.012	5.5	467.6	88
3	4532	34.83	362293	0.031	14.2	453.4	494
4	4215	49.08	514486	0.044	35.2	418.2	1730
5	4226	63.33	706506	0.060	27.7	390.5	1752
6	4218	77.58	905853	0.077	35.5	355.0	2752
7	4395	91.83	1162203	0.099	45.5	309.5	4180
8	3536	105.33	1107494	0.095	43.4	266.1	4569
9	3538	118.83	1285928	0.110	50.4	215.8	5985
10	3582	132.33	1486798	0.127	58.2	157.5	7706
Penthouse	3503	146.42	1647370	0.141	64.5	93.0	9447
Penthouse Mezzanine	1299	159.83	680649	0.058	26.7	66.4	4261
Roof	2863	176.42	1694577	0.145	66.4	0.0	11709
Total	44481	1171.81	11694174	1.000	473.1	473.1	54671

Seismic Direct & Torsional Shears:

Seismic Loads												
Shear Wall	Level 2		Level 3		Level 4		Level 5		Level 6		Level 7	
	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)
1	0.58	0.34	1.50	0.77	3.72	1.29	2.93	1.01	3.75	1.32	4.81	0.56
2	0.61	0.30	1.57	0.67	3.88	1.12	3.06	0.88	3.92	1.15	5.02	0.49
3	0.54	0.00	1.39	0.00	3.45	0.00	2.71	0.00	3.48	0.00	4.46	0.01
4	0.40	0.00	1.03	0.00	2.56	0.00	2.01	0.00	2.58	0.00	3.31	0.01
5	0.81	0.20	2.10	0.46	5.20	0.76	4.09	0.60	5.24	0.79	6.72	0.33
6	0.49	0.08	1.26	0.17	3.11	0.28	2.45	0.22	3.14	0.29	4.02	0.12
7	0.72	0.00	1.87	0.00	4.64	0.00	3.65	0.00	4.68	0.00	6.00	0.01
8	0.72	0.00	1.87	0.00	4.64	0.00	3.65	0.00	4.68	0.00	6.00	0.01
9	0.49	0.02	1.26	0.04	3.11	0.07	2.45	0.05	3.14	0.07	4.02	0.03
10	0.81	0.05	2.10	0.11	5.20	0.18	4.09	0.14	5.24	0.19	6.72	0.08
11	0.72	0.00	1.87	0.00	4.64	0.00	3.65	0.00	4.68	0.00	6.00	0.01
12	0.72	0.00	1.87	0.00	4.64	0.00	3.65	0.00	4.68	0.00	6.00	0.01
13	0.66	0.30	1.70	0.68	4.22	1.14	3.32	0.90	4.26	1.17	5.46	0.50
14	1.05	0.59	2.72	1.32	6.75	2.20	5.31	1.73	6.81	2.27	8.73	0.96
15	0.72	0.00	1.86	0.00	4.61	0.00	3.62	0.00	4.65	0.00	5.95	0.01
16	0.94	0.00	2.43	0.00	6.03	0.00	4.75	0.00	6.09	0.00	7.80	0.02

Shear Wall	Level 8		Level 9		Level 10		Penthouse		PH Mezzanine		Roof	
	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)	Direct Load (k)	Torsional Load (k)
1	4.59	4.63	5.33	5.36	6.16	6.15	6.82	5.00	2.82	1.39	7.02	188.88
2	4.79	4.03	5.56	4.66	6.42	5.36	7.12	4.35	2.95	1.05	7.33	189.04
3	4.25	0.01	4.94	0.01	5.70	0.01	6.32	0.01	2.62	0.02	6.51	1.09
4	3.15	0.00	3.66	0.00	4.23	0.01	4.69	0.01	1.94	0.01	4.82	1.09
5	6.41	2.75	7.44	3.18	8.59	3.65	9.52	2.97	3.94	0.09	n/a	n/a
6	3.84	1.02	4.46	1.18	5.15	1.36	5.70	1.10	2.36	0.26	n/a	n/a
7	5.72	0.01	6.64	0.01	7.67	0.01	8.50	0.01	3.52	0.02	n/a	n/a
8	5.72	0.01	6.64	0.01	7.67	0.01	8.50	0.01	3.52	0.02	n/a	n/a
9	3.84	0.24	4.46	0.27	5.15	0.31	5.70	0.26	2.36	0.65	n/a	n/a
10	6.41	0.65	7.44	0.75	8.59	0.87	9.52	0.70	3.94	1.61	n/a	n/a
11	5.72	0.01	6.64	0.01	7.67	0.01	8.50	0.01	3.52	0.02	n/a	n/a
12	5.72	0.01	6.64	0.01	7.67	0.01	8.50	0.01	3.52	0.02	n/a	n/a
13	5.21	4.09	6.05	4.73	6.98	5.44	7.74	4.42	n/a	n/a	n/a	n/a
14	8.32	7.92	9.67	9.17	11.16	10.53	12.37	8.55	n/a	n/a	n/a	n/a
15	5.68	0.01	6.59	0.01	7.62	0.01	8.44	0.01	n/a	n/a	n/a	n/a
16	7.44	0.01	8.64	0.01	9.98	0.01	11.06	0.01	n/a	n/a	n/a	n/a

Appendix F – Shear Wall Spot Checks

Shear Wall 2 Load Takedown:

Level	Floor-to-Floor Height (ft)	Tributary Area (sq ft)	Shear Wall Length (ft)	Dead Load (psf)	Live Load (psf)	Snow Load (psf)	Shear Wall Weight (pcf)	Dead Load (k)	Live Load (k)	Live Roof Load (k)	Snow Load (k)
2	18.83	600	30	117	100	0	150	154.94	60	0	0
3	14.25	600	30	117	80	0	150	134.33	48	0	0
4	14.25	600	30	117	80	0	150	134.33	48	0	0
5	14.25	600	30	117	80	0	150	134.33	48	0	0
6	14.25	600	30	117	80	0	150	134.33	48	0	0
7	13.5	600	30	117	80	0	150	130.95	48	0	0
8	13.5	600	30	117	80	0	150	130.95	48	0	0
9	13.5	600	30	117	80	0	150	130.95	48	0	0
10	14.08	600	30	117	80	0	150	133.56	48	0	0
PH	13.42	600	30	192	150	0	150	175.59	90	0	0
PH Mezz.	16.58	600	30	192	150	0	150	189.81	90	0	0
Roof	-	600	0	125	20	22	0	75.00	12	12	13.2
							Totals:	1659.05	636	12	13.2

Check Shear Wall 2

Assume the single opening is negligible.

DL = 1659.05 k
LL = 636 k
L_r = 12 k
S = 13.2 k
W = 135.09 k
E = 265.32 k

f'_c = 4000 psi
f_y = 60,000 psi

thickness = t = 12"
l_w = 30 ft
h_w = 176.41 ft

Check Load Combinations 4 and 5

4 $1.2D + 1.6W + L + 0.5S$

$$P_u = 1.2(1659.05) + 636 + 0.5(13.2) = 2633.46\ k$$

$$V_u = 1.6(135.09) = 216.14\ k$$

5 $1.2D + 1.0E + L + 0.2S$

$$P_u = 1.2(1659.05) + 636 + 0.2(13.2) = 2629.5\ k$$

$$V_u = 265.32\ k$$

∴ Use Load Combination 5 and check SW2 according to ACI 2008 Chapter 21. (for seismic)

1.) Check need for Boundary Elements:
(use gross section properties)

$$A_g = \left(\frac{12''}{12}\right)(30') = 30\ ft^2$$

$$I_g = \frac{(12/12)(30)^3}{12} = 2250\ ft^4$$

$$f_c = \frac{P_u}{A_g} + \frac{M_u y}{I_g}$$

M_u was calculated from Seismic shear spreadsheet to be = 42150 ft-k

$$f_c = \frac{2629.5}{30} + \frac{42150(30/2)}{2250} = 369\ k/ft^2$$

$$f_c = 2.56\ k/in^2 > 0.2 f'_c = 0.2(4) = 0.8\ k/in^2$$

Technical Report #3

2.) Check distributed longitudinal & transverse reinforcement:

$2 A_{cv}\sqrt{f'_c} = 2\,(30' \times 12\ \text{in/ft})(12'')\sqrt{4000}\ /1000 = 546.4\ \text{k}$

$546.4\ \text{k} > V_u = 265.32\ \text{k}$ so do not need 2 curtains

However, the design engineer used 2 curtains, so I will too.

$$\rho_\ell,\ \rho_t = \frac{A_{s\ell}}{A_{cv}} \geq 0.0025$$

$A_{cv} = 12'' \times 12'' = 144\ \text{in}^2/\text{ft}$

$A_{s\ell}$ required $= (0.0025)(144) = \boxed{0.36\ \text{in}^2/\text{ft}}$

Assume #6 bars because that is the smallest size used by the design engineer.

$A_{s\ell} = 2(0.44) = 0.88\ \text{in}^2/\text{ft}$

spacing used by design engineer = 12"

$0.88\ \text{in}^2 > 0.36\ \text{in}^2$

⇒ #6 bars @ 12" each face horizontally and
#8 bars @ 12" each face vertically is okay

3.) Check shear capacity:

$V_n = A_{cv}\,(\alpha_c\sqrt{f'_c} + \rho_t f_y)$

$$\frac{h_w}{\ell_w} = \frac{176.41'}{30'} = 5.88 > 2 \Rightarrow \alpha_c = 2.0$$

$A_{cv} = (12'')(30' \times 12\ \text{in/ft}) = 4320\ \text{in}^2$

$$\rho_t = \frac{2\,(0.44)}{(12'')(12'')} = 0.0061$$

$V_n = 4320\,(2\sqrt{4000} + 0.0061 \times 60000)/1000 = 2128\ \text{k}$

$\phi V_n = 0.75(2128) = 1596\ \text{k} > V_u = 265.32\ \text{k}$ okay

4.) Check longitudinal reinforcement for B.E.:

$$T_u = \frac{M_u}{d}$$

From design engineer detail, use B.E. is 12" × 12" (i.e. just corner of shear walls) and 4 #10 bars

$\Rightarrow d = 30' - 12" = 29'$

$$T_u = \frac{42150}{29} = 1453 \text{ k}$$

$$T_u = \phi A_s f_y$$

$$1453 = 0.9 A_s (60) \Rightarrow A_{s\,\text{required}} = \boxed{26.9 \text{ in}^2}$$

$4(1.27) = 5.08 \text{ in}^2 \lll 26.9 \text{ in}^2$ so NOT okay

∴ Should consider larger boundary elements (i.e. not just constant 12" thickness)

for example, could try a 36" × 48" B.E. w/ 18 #11 bars

$$A_{st} = 18(1.56) = 28.1 \text{ in}^2 > 26.9 \text{ in}^2$$

$$\rho_{st} = \frac{28.1 \text{ in}^2}{(36")(48")} = 0.0163 \begin{matrix} > 0.01 = \rho_{min} \\ < 0.06 = \rho_{max} \end{matrix} \quad \text{okay}$$

5.) Check compressive capacity of B.E.:

$$C_u = \frac{P_u}{2} + \frac{M_u}{d} = \frac{2629.5}{2} + \frac{42150}{29} = 2768 \text{ k}$$

$$\phi P_{n(max)} = 0.8\,\phi\,[0.85 f'_c (A_g - A_{st}) + f_y A_{st}]$$

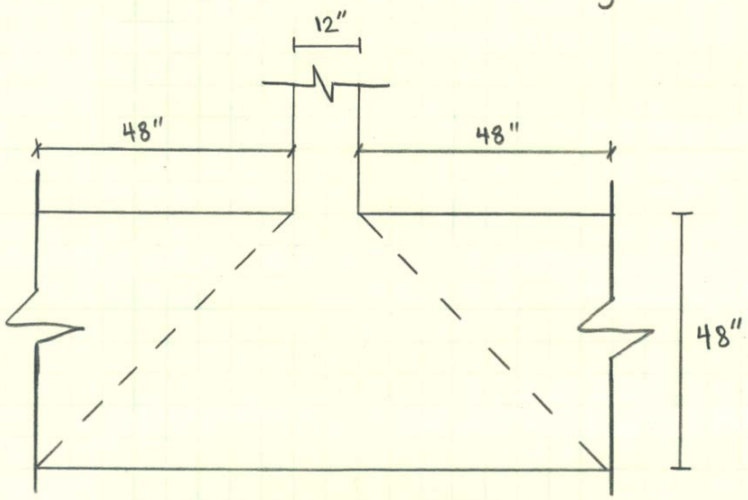
$$\phi P_{n(max)} = 0.8(0.65)[0.85(4)(36 \times 48 - 28.1) + 60 \times 28.1]$$

$$= 3882 \text{ k} > C_u = 2768 \text{ k} \quad \text{okay}$$

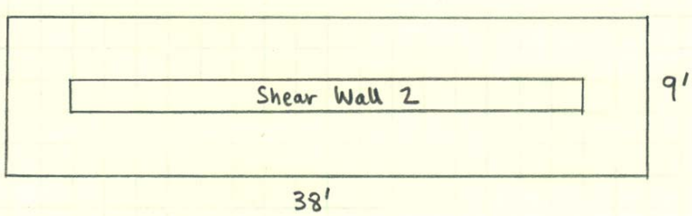
Technical Report #3

Foundation Check Below Shear Wall 2

Assume wide beam shear is the governing failure condition, and use the following area:



Section thru SW2 & foundation



Plan view of SW2 & foundation area of interest

Use $f_c = 369\ k/ft^2$ from SW2 check (under the assumption that the parking garage is not being included)

larger M_u should counteract loss of P_u

$$\phi V_n = 0.75(2)\sqrt{4000}\,(12'')\underbrace{(48'' - 3'' - 1.27'')}_{\substack{h\ -\text{cover}\ -\ d_b \\ d = 43.73''}}/1000$$

$$\phi V_n = 49.8k$$

$f_c = 369\ k/ft^2$ is per the area of the shear wall in plan (i.e. $30\ ft^2$)

$\rightarrow$ use $369\ \frac{k}{ft^2} \times 30 ft^2 \times \frac{1}{9' \times 38'} = 32.4\ ksf$

Technical Report #3

$V_u = (32.4 \text{ ksf})\left(\frac{9'-1'}{2} - \frac{43.73''}{12}\right)(1) = 11.53 \text{ k}$

$V_u = 11.53 \text{ k} < \Phi V_n = 49.8 \text{ k}$ so okay

Quick check of Overturning:

use load combination 7

resisting load $= 0.9D = 0.9(1659.05 \text{ k}) = 1493 \text{ k}$

overturning load $= T_u$ due to $1.0E = 1453 \text{ k}$

$1453 \text{ k} < 1493 \text{ k}$ so okay