Technical Report #1

8^{th} Street Office Building | Richmond, VA

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October 5, 2009

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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 first technical report regarding the 8th Street Office Building, the existing structural conditions and concepts are investigated. The building is first introduced with an explanation of its various functions and a detailed description of the structural system including the mat foundation, steel framing and concrete shear wall lateral system. Then, the materials and building codes are compiled for reference.

Gravity loads are calculated according to ASCE 7-05. When possible, the loads are compared to the design loads provided by the engineers of record in the structural general notes. Initially, it appears that the engineers were slightly more conservative than required by ASCE 7-05. However, upon performing spot checks of members for gravity loading, it is concluded that the live loads used in the checks may be more conservative than those used in the design of the building.

Wind and seismic loads are also calculated according to ASCE 7-05. It is not possible to compare the base shears from the wind and seismic analyses to those used in the design of the 8th Street Office Building. However, it is concluded that the results should be similar since the engineers used ASCE 7-02 in their design, and a few of the variables provided by the engineers are identical to those found in this report. Finally, it is determined that the wind loads control over the seismic loads as expected.

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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 four 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.

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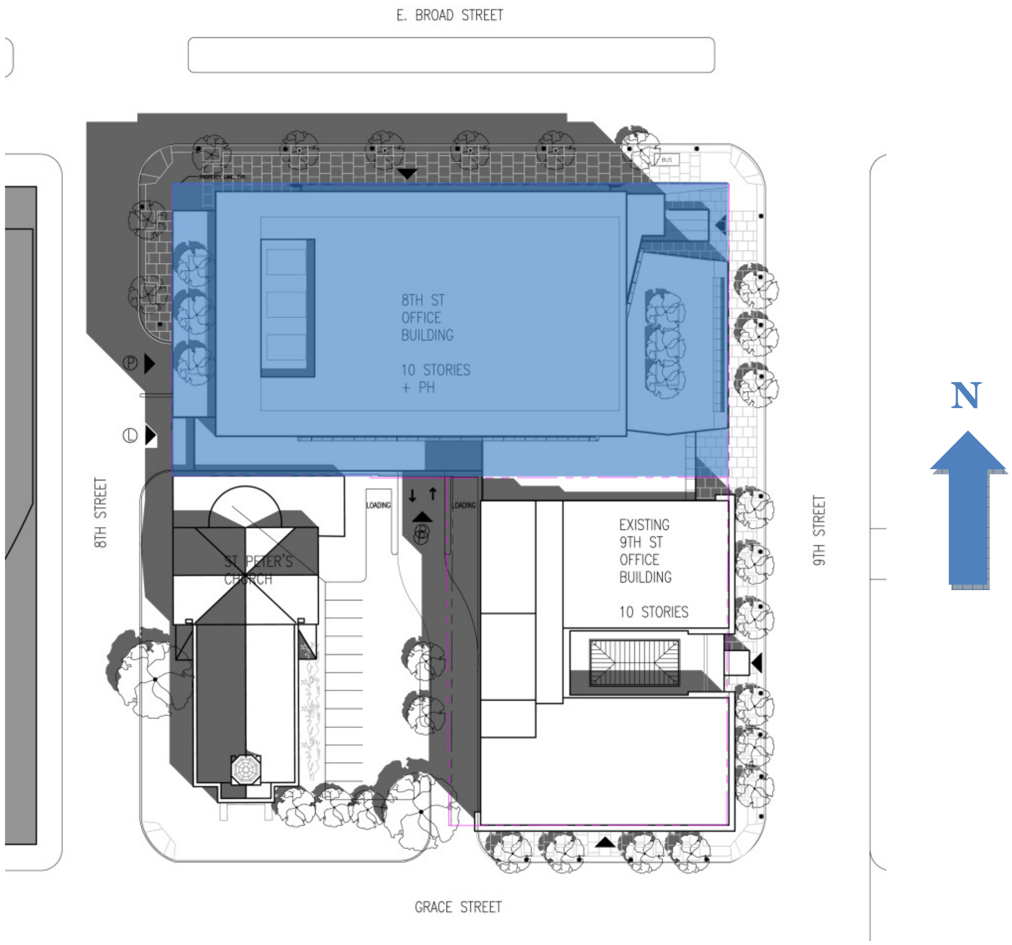

Figure 1 – Site plan

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

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 level of the underground parking garage is only located on the western half of the site. See Figures 4 and 5 for visual representations of the mat foundations locations. Based on the elevations, it was recommended that the 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.

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.

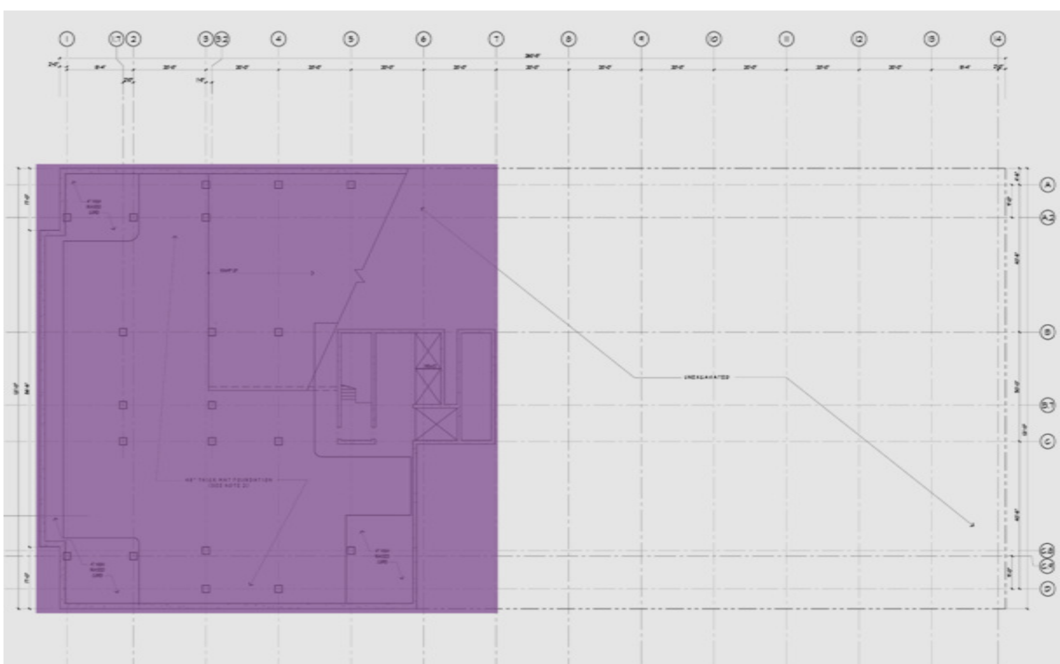

Figure 4 – 4th Level of Parking Garage with General Mat Foundation Location

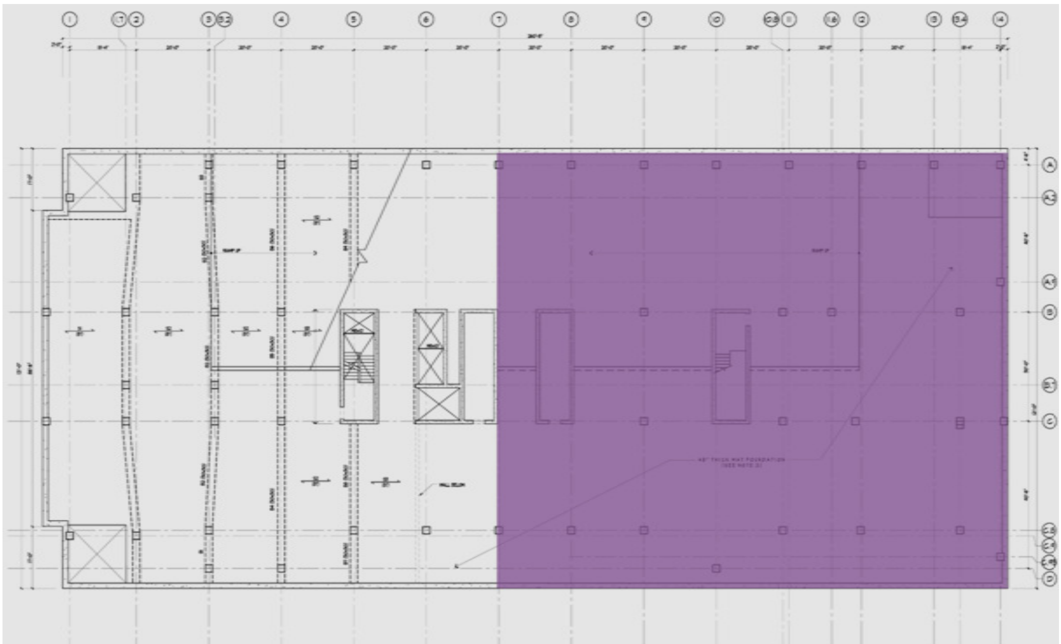

Figure 5 – 3rd Level of Parking Garage with General Mat Foundation Location

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. Reinforcement for the beams ranges anywhere from #7 to #11 bars. The majority of the one way concrete slabs are 8" thick and reinforced with #5 bars spaced at 12".

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. Composite action is achieved between the floor system and the beams through ¾" diameter, 4" long headed shear studs. See Figure 6 for a detail of the floor system.

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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. A typical bay is also shown in Figure 17 in the Typical Spot Checks section.

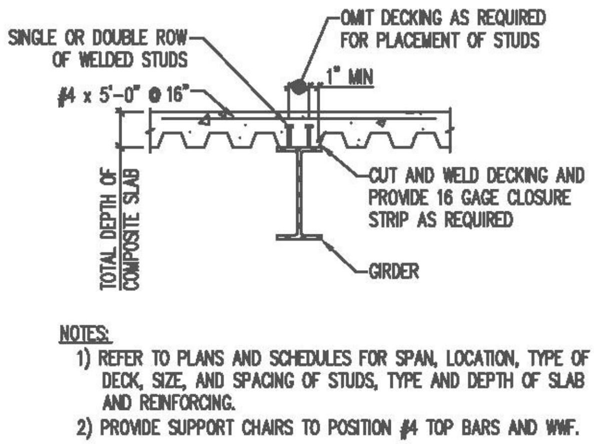

Figure 6 – “Concrete Steel Deck Parallel to Beam” Detail

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. See Figure 7 for the exact locations of the shear walls. 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. There are a total of 16 shear walls. All of the shear walls are located on the 3rd level of the parking garage through the 10th floor. However, only 8 shear walls extend downwards to the 4th level of the parking garage, only 12 shear walls extend upwards to the Penthouse level, and only 4 shear walls extend upwards to the Penthouse Mezzanine level. 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 completely to the shear walls. The shear walls then carry those loads down to the mat foundation.

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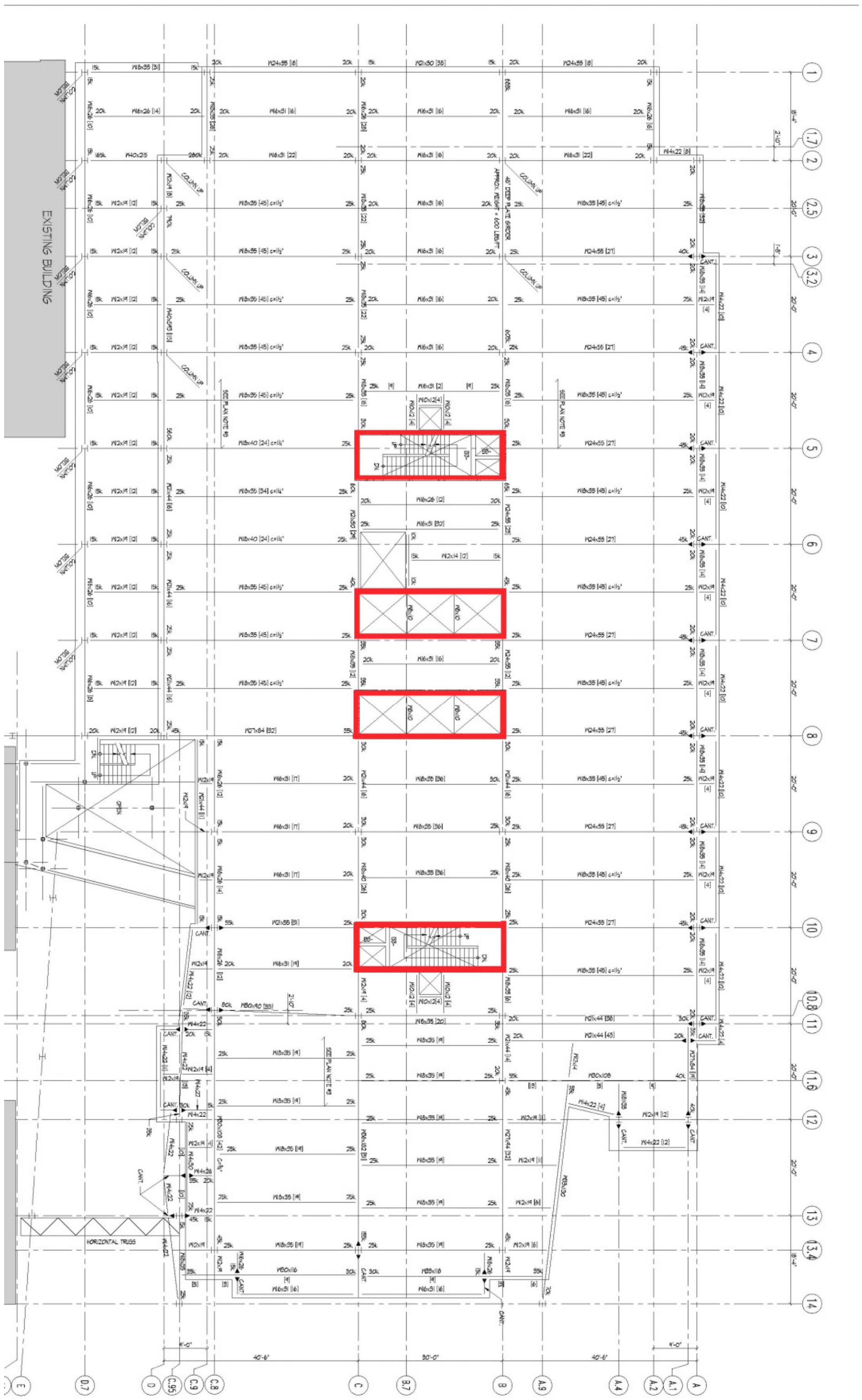

Figure 7 – Locations of Reinforced Concrete Shear Walls

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Materials

Structural Steel:

Rolled Shapes……………………………………………………………………………………ASTM A992, Grade 5
Channels, Angles and Plates…………………………………………………………………………………ASTM A36
Pipes…………………………………………………………………………………………ASTM A53, Grade B, F_y=35 ksi
Tubes (Square and Rectangular HSS)…………………………………………ASTM A500, Grade B, F_y=46 ksi

Metal Decking:

3 $^1/_4$" Lightweight Concrete over 2" Composite Deck (5 $^1/_4$" total depth)……ASTM A653, 18 Gage
1 $^1/_2$" Roof Deck………………………………………………………………………………ASTM A653, 20 Gage

Headed Shear Studs:

$^3/_4$" diameter………………………………………………………………………………………………ASTM A108

High Strength Bolts:

$^3/_4$" Bolts……………………………………………………………………………………………………...ASTM A-325N

Welding Electrodes:

E70XX…………………………………………………………………………………………..Tensile Strength = 70 ksi

Cast-in-Place Concrete:

Slabs on Grade (Interior)………………………………………………………………………………f'_c=3000 psi
Slabs on Grade (Exterior)………………………………………………………………………………f'_c=3500 psi
Reinforced Slabs……………………………………………………………………………………………f'_c=5000 psi
Reinforced Beams…………………………………………………………………………………………f'_c=5000 psi
Fill on Metal Deck…………………………………………………………………………………………f'_c=3500 psi
Columns……………………………………………………………………………………………f'_c=5000/7000 psi
Walls……..f'_c=4000 psi
Mat Foundation……………………………………………………………………………………………f'_c=4000 psi

Reinforcement:

Deformed Reinforcing Bars……………………………………………………………………………ASTM A615, Grade 60
Welded Wire Fabric……………………………………………………………………………………………ASTM A185

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Codes and References

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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Loads

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

Gravity Loads

Dead Loads:

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	7 psf
Curtain Walls and Precast Concrete Panels	25 psf
Total for Floor System Design	68 psf
Total for Seismic Analysis	75 psf

Note: Self weight of concrete shear walls is based on 150 lb/ft^3 and varies by floor based on height and length. See Appendix B for inclusion of the shear walls in the calculation of dead loads.

Superimposed Dead Loads:

Typical Floor:

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

Atrium:
To account for finishes and catwalks, 20 psf is assumed for each level that the atrium extends upwards. Structural slabs, partitions and ceiling loads are not included.

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:

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 for mechanical rooms. Since ASCE 7-05 does not provide a live load value for mechanical rooms, 150 psf will be used in future analyses.

Snow Loads:

Ground Snow Load	20 psf
Flat Roof Snow Load	22 psf
Penthouse Level Roof Snow Drift	46 psf
Typical Terrace Snow Drift	50 psf

See Appendix C for snow load and drift calculations.

Wind Loads

Wind loads for the 8th Street Office Building were determined using Method 2, also known as the Analytical Procedure, in ASCE 7-05 Section 6.5. Because the building has a significant setback that occurs at the 7th floor, two analyses were conducted. The first analysis utilized the first floor dimensions, and the second analysis utilized average dimensions from the 7th through the 10th floors. The controlling pressure was selected for each floor in order to calculate the forces. Generally, the second analysis produced the controlling pressures, although the results were not significantly different. Detailed calculations for each of the analyses can be found in Appendix D.

It was determined that the total controlling pressures in the North-South direction are slightly larger than those in the East-West direction. Furthermore, the base shear controls in the North-South direction since the length of the building in that direction produces a larger façade area.

The wind variables common to both of the analyses conducted can be found below in Figure 8. The values of the controlling pressures and the corresponding lateral loads, shears and moments are then tabulated by level in Figure 9.

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)

Figure 8 – Wind Variables

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

Figure 9 – Wind Pressures and Forces

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Wind Pressure Diagrams:

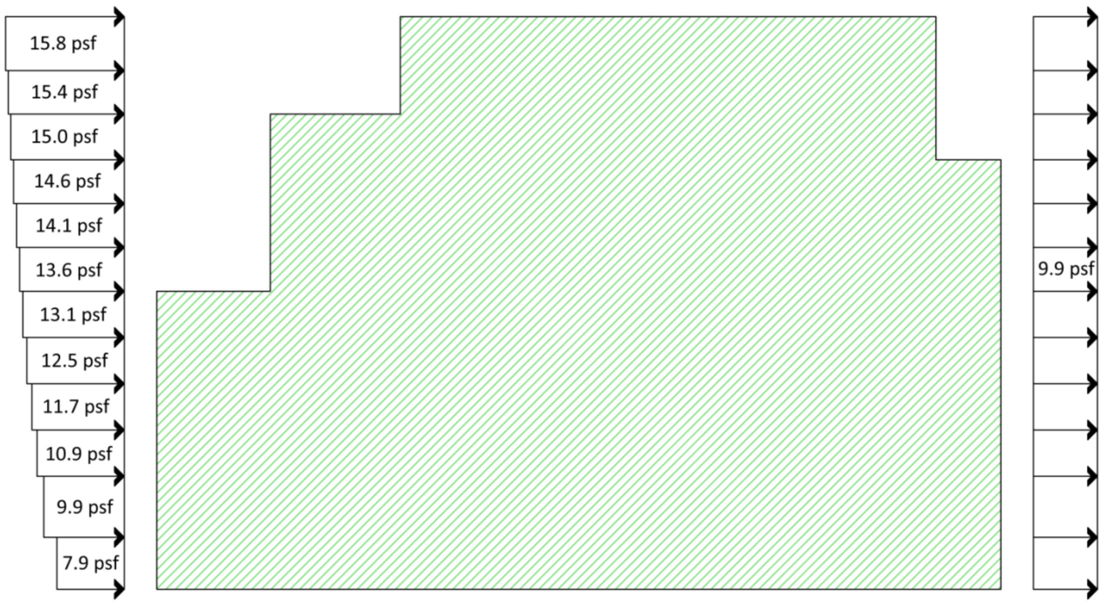

Figure 10 – North-South Wind Pressure Diagram

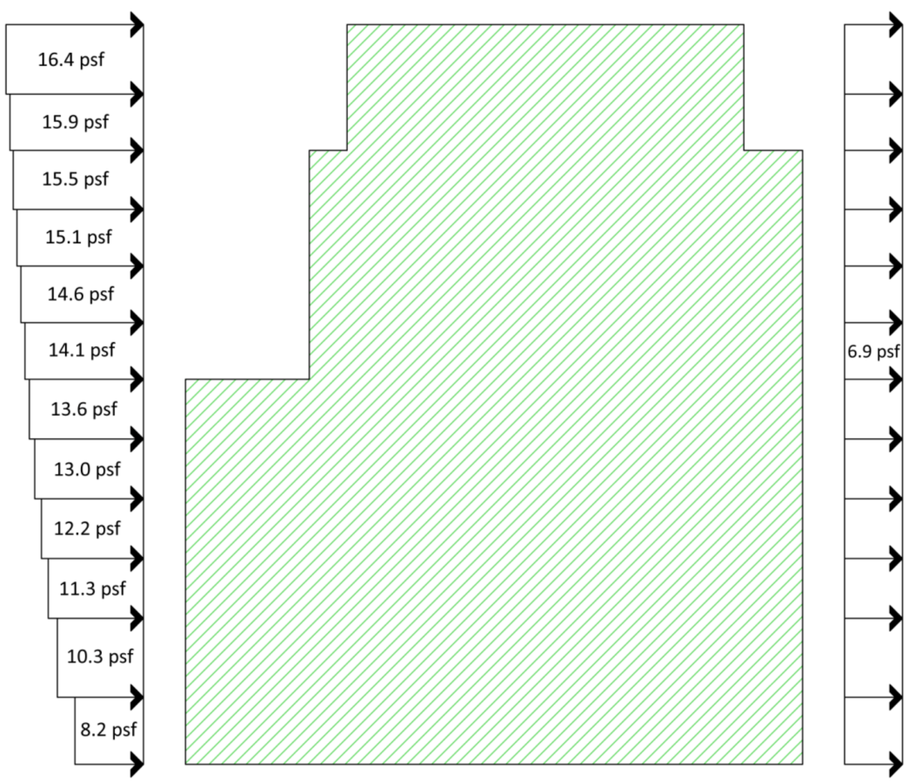

Figure 11 – East-West Wind Pressure Diagram

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Wind Load Diagrams:

Wind pressures were converted to concentrated loads by utilizing the tributary area of the building's façade at each level. It has been assumed that the floor diaphragms will transfer the lateral loads to the shear walls surrounding four cores in the building. See Figures 12 and 13 for the distribution of the wind loads and the base shear in each direction.

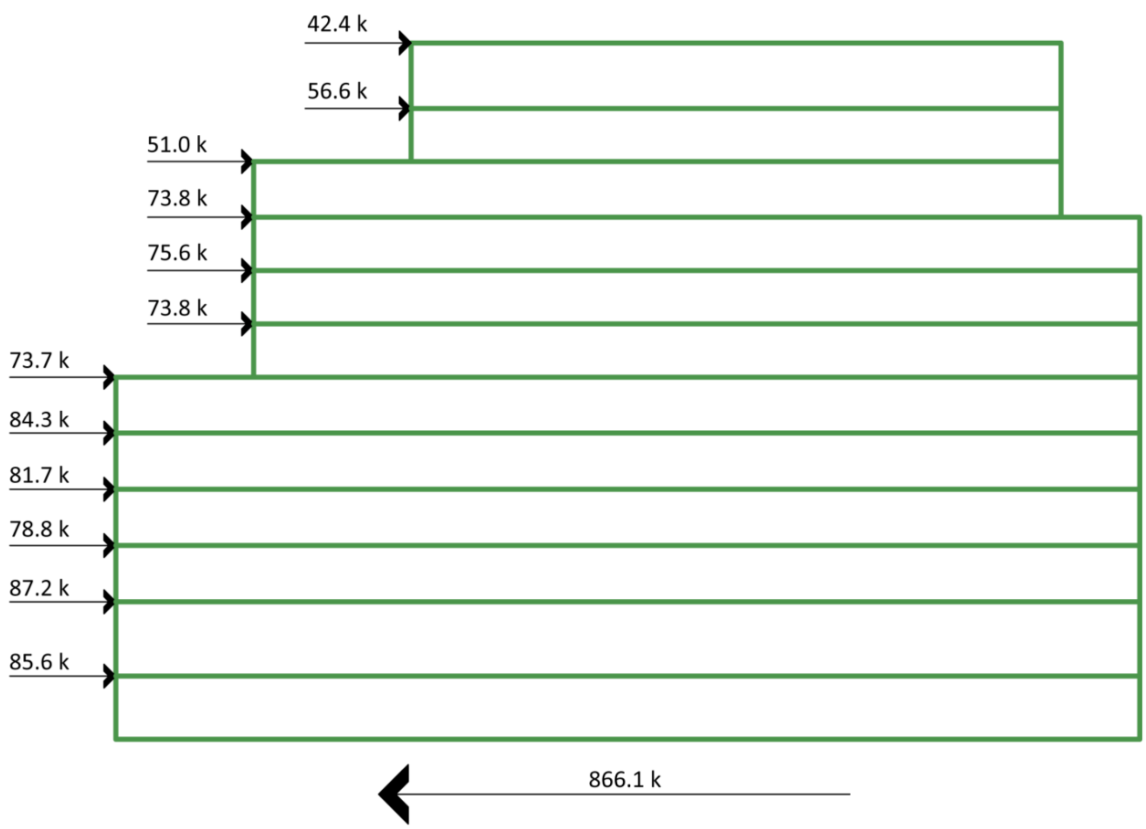

Figure 12 – North-South Wind Load Diagram

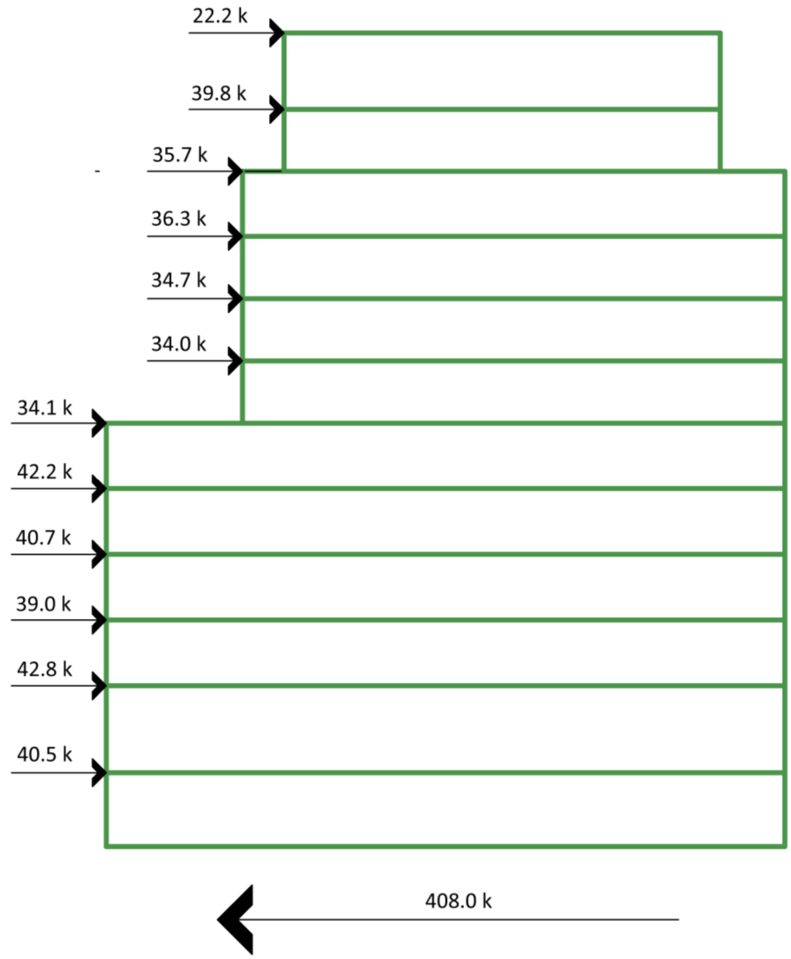

Figure 13 – East-West Wind Load Diagram

As indicated earlier, it can be seen that the base shear of 866 k in the North-South direction controls over the base shear of 408 k in the East-West direction. The controlling base shear calculated by the engineers of record is not available for a comparison. However, ASCE 7-02 was used in the design of the building, so it is reasonable to assume that the wind analysis performed by the engineers produced similar results. In addition, the basic wind speed, importance factor, exposure category and internal pressure coefficient used in this analysis are identical to those listed in the structural general notes for the project.

Finally, it should be noted that the 9th Street Office Building and St. Peter's Church abut the 8th Street Office Building and block the wind on lower levels. However, wind was still examined in these areas in the event that the adjacent buildings no longer exist at some point in the future.

Seismic Loads

Seismic loads for the 8th Street Office Building were determined using Chapters 11 and 12 of ASCE 7-05. It was determined that the Equivalent Lateral Force Procedure could be used in the calculation of seismic forces. The analysis includes dead loads from floor slabs, steel framing, concrete shear walls, glass curtain walls and superimposed dead loads. An additional allowance was also provided for the penthouse mechanical areas and the roof terraces. See Appendix B for assumptions and calculations related to the building's total dead load. Detailed calculations related to the seismic analysis are available in Appendix E. A summary of the seismic variables can be found below in Figure 14.

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)

Figure 14 – Seismic Variables

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A load distribution table is provided below in Figure 15. Once again, it has been assumed that the floor diaphragms will transfer the lateral loads to the shear walls surrounding four cores in the building. It is evident that the seismic forces and base shear are less than those produced by the wind pressures. See Figure 16 on the next page for a seismic load diagram.

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

Figure 15 – Seismic Forces, Shears and Moments by Level

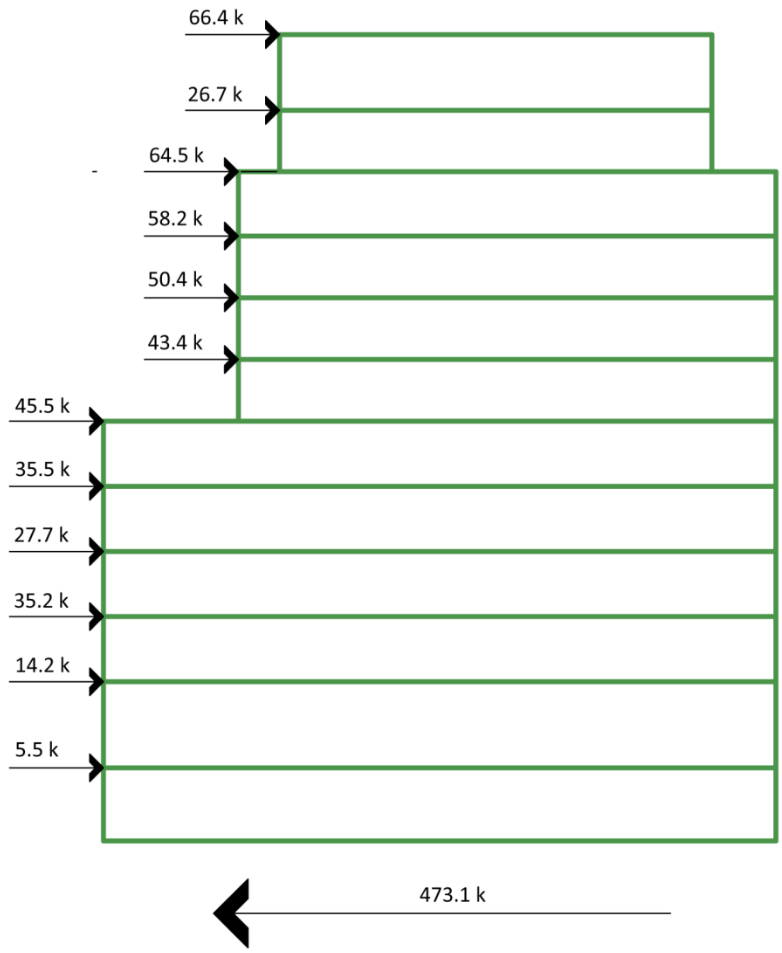


Figure 16 – Seismic Load Diagram

The seismic base shear of 473 k is significantly less than the controlling wind base shear of 866 k. Therefore, it can be concluded that the wind loads will be the controlling load case over the seismic loads for the 8th Street Office Building.

The seismic base shear calculated by the engineers of record is unavailable for a comparison. However, ASCE 7-02 was used in the design of the building, and it has been indicated on the general notes that the Equivalent Lateral Force Procedure was used. Therefore, it is reasonable to assume that the seismic analysis performed by the engineers produced similar results to those presented above. Any discrepancies should only be found in the calculation of the building's dead load, as an extremely detailed takedown was not performed in this report.

Typical Spot Checks

The typical bay that was analyzed for gravity loads can be seen below in Figure 17. The beam, girder and column that were checked are outlined in red. Because the 8th Street Office Building has been designed with the utmost flexibility for its occupants in mind, it is not uncommon for long spans to dictate member sizes. Typical beams for the longer 40'-6" spans are W18x35, while shorter spans of 20'-0" only require W16x31. These beams frame into W18x35 girders. W14 columns are used and spliced every three floors.

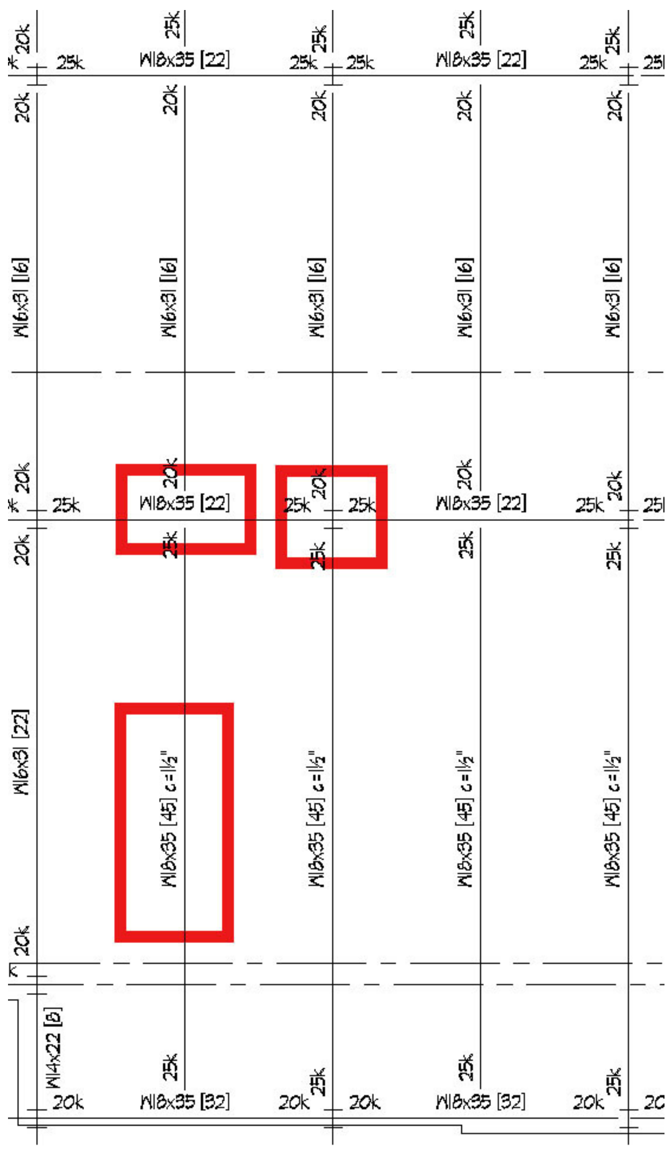


Figure 17 – Typical Bay Indicating Spot Checks

Slab/Metal Deck

It was determined from the structural general notes and the framing plan notes that the metal decking is 2" deep with a minimum thickness of 18 gage. The slab is of lightweight concrete and has a total depth of 5 ¼". Furthermore, it was stipulated that the deck be provided by United Steel Deck with the following properties:

DECK PROPERTIES									
Gage	t	w	As	I	S_p	S_n	R_b	ϕV_n	studs
22	0.0295	1.5	0.440	0.338	0.284	0.302	714	1990	0.43
20	0.0358	1.8	0.540	0.420	0.367	0.387	1010	2410	0.52
18	0.0474	2.4	0.710	0.560	0.523	0.529	1680	3180	0.69

Figure 18 – United Steel Deck Properties

The maximum unshored span of 10.97 feet was obtained from Figure 19 below. In the 8th Street Office Building, beams are typically spaced 10 feet on center, so the clear span must be less than 10.97 feet. Therefore, the decking is adequate to span the beams.

	COMPOSITE PROPERTIES									Max. unshored spans, ft.			
	Slab Depth	ϕM_{nf} in.k	A_c in²	Vol. ft³/ft²	W psf	S_c in³	I_{av} in⁴	ϕM_{no} in.k	ϕV_{nt} lbs.	1span	2span	3span	A_{wwf}
18 gage	4.50	62.08	32.6	0.292	34	1.53	5.4	42.99	4560	9.20	11.33	11.71	0.023
	5.25	77.02	40.0	0.354	41	1.95	8.3	54.72	5590	8.54	10.62	10.97	0.029
	6.00	91.95	48.0	0.417	48	2.39	12.1	67.07	6530	8.01	10.02	10.36	0.036
	6.25	96.93	50.8	0.438	50	2.54	13.6	71.29	6730	7.86	9.84	10.17	0.038
	6.50	101.91	53.6	0.458	53	2.69	15.2	75.55	6920	7.71	9.68	10.00	0.041
	7.00	111.87	59.5	0.500	58	3.00	18.8	84.17	7340	7.44	9.36	9.67	0.045
	7.25	116.85	61.9	0.521	60	3.16	20.7	88.52	7500	7.32	9.21	9.52	0.047
	7.50	121.83	64.3	0.542	62	3.31	22.8	92.91	7670	7.24	9.07	9.38	0.050

Figure 19 – United Steel Deck Composite Properties

Finally, the maximum uniform live service load was obtained from Figure 20 below. The metal deck and slab can support 235 pounds per square foot for an 11'-0" span and a total depth of 5 ¼". This is greater than the total service load of 190 pounds per square foot, so the metal deck and slab are sufficient. In fact, the load provided by United Steel Deck already takes into account the self weight of the deck and slab, so it was conservative to use 190 pounds per square foot.

Technical Report #1

	Slab Depth	ϕMn in.k	L, Uniform Live Service Loads, psf * 6.00	6.50	7.00	7.50	8.00	8.50	9.00	9.50	10.00	10.50	11.00	11.50	12.00
18 gage	4.50	62.08	400	400	400	400	375	330	290	260	230	205	180	155	135
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	5.25	77.02	400	400	400	400	400	400	365	325	290	260	235	210	190
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	6.00	91.95	400	400	400	400	400	400	400	385	345	310	280	250	230
	6.25	96.93	400	400	400	400	400	400	400	400	365	325	295	265	240
	6.50	101.91	400	400	400	400	400	400	400	400	385	345	310	280	255
	7.00	111.87	400	400	400	400	400	400	400	400	400	380	340	310	280

Figure 20 – United Steel Deck Uniform Live Service Loads

Typical Composite Beam

As stated earlier, typical composite beam sizes for the 8th Street Office Building tend to depend on the span length. The beam that was checked was designed by the engineers to be a W18x35 [45] with a camber of 1 ½”. The beam spans 40’-6” and carries load from a tributary width of 10’-0”. Detailed calculations that check bending, shear and deflection can be found in Appendix F.

It was found that a W18x40 [50] is actually needed to meet bending requirements. The reason a slightly larger beam is needed is most likely due to the amount of live load that was assumed in the spot check. New tenants will move in after approximately five years, so it was decided to use a live load of 80 psf as designated by ASCE 7-05 for corridors above the first floor instead of 50 psf for offices. The new tenants may wish to rearrange their open office spaces with the partitions, and areas that used to be offices may become corridors and vice versa. It is also anticipated that the new tenants may wish to create more meeting/assembly areas on the higher floors that require a larger live load. Therefore, no live load reductions were utilized in order to remain conservative. Another indication that the loads used in the check are larger than the design loads is that the engineers used Allowable Stress Design rather than Load and Resistance Factor Design, and ASD is more conservative than LRFD.

Finally, the W18x40 [50] alone does not meet deflection criteria. Therefore, either a larger beam or a cambered W18x40 [50] is necessary. It was concluded that the camber of 1 ½” designated by the engineers is accurate.

Technical Report #1

Typical Composite Girder

The girder that was checked was designed by the engineers to be a W18x35 [22] with a span of 20'-0". In the spot check, the girder was designed to carry one concentrated load equal to 94.6 k at the middle of the span. The composite beam that was checked earlier and a W16x31 [16] composite beam contribute to the concentrated load. Detailed calculations that check bending, shear and deflection of the girder can be found in Appendix F.

It was discovered during the girder check that a W18x35 [54] is needed. Although the same size was obtained, 54 shear studs is a significantly larger number than the 22 studs required by the engineers. It is assumed again that the reason for the difference is the amount of live load used in the spot check. It is also worthwhile to note that it may be impractical to place 54 studs on a 20'-0" span girder, and the choice of a larger size member may be recommended. Finally, there were no deflection issues with the girder as expected.

Typical Column

Due to the splicing of columns every three floors, the column that was checked is located on the 8th floor. The column is at the bottom of a group of W14x68 columns, so it must be designed to carry the greatest load out of the group. Specifically, column B-3 was chosen because it is located in the typical bay where the composite beam and girder were checked.

Table 4-1 of the 13th Edition Steel Construction Manual was used to size the column. The unbraced length of the column was assumed to be the floor-to-floor height, and it was also assumed that the column is pinned at both the top and the bottom. In order to remain consistent with the beam and girder spot checks, an 80 psf live load was used and it was not reduced. Refer to Appendix F for a rough column load takedown and other calculations.

Ultimately, the beam was designed to be a W14x74 carrying an axial load of 710 k in the check. This is only slightly larger than the W14x68 column designated by the engineers. Once again, the reason for the slight increase in size is most likely due to the fact that a larger, unreduced live load was used to conservatively account for the demands of new tenants. Furthermore, it is possible that a larger mechanical room dead load was used in the column load takedown for the spot check, and that lead to the increase in size of the column.

Conclusion

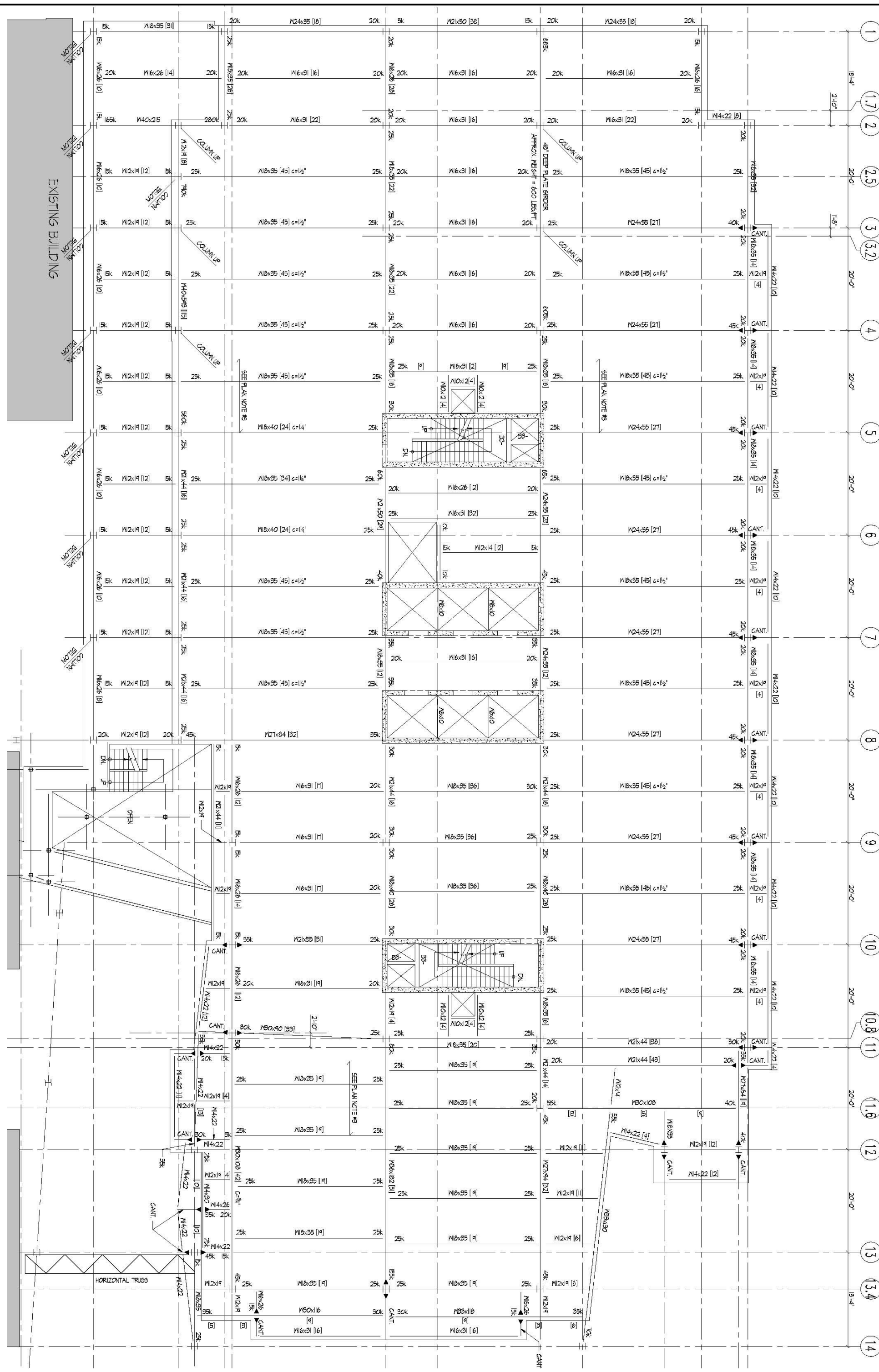
The existing structural conditions of the 8th Street Office Building have been thoroughly investigated. The building has been introduced through detailed descriptions of its various spaces and functions, foundation system, superstructure and lateral system. In addition, a variety of plans, elevations and details have been provided to enhance the descriptions. The types of materials and building codes and references have also been listed. Gravity and lateral loads were all analyzed using ASCE 7-05. They were compared, when possible, to the forces used by the original designers of the building. Finally, spot checks were performed on a typical bay in order to ascertain the accuracy of the gravity loads that were determined earlier.

It was concluded after the lateral analyses were performed that the wind loads control over the seismic loads for the 8th Street Office building located in Richmond, VA. Wind and seismic base shears used by the engineers of record were not available for comparison. However, it was deemed reasonable to assume that the results should be similar since the engineers used ASCE 7-02. Any discrepancies are most likely a result of differing design loads or tributary areas.

It was concluded after the spot checks were performed that a higher live load may have resulted in slightly larger typical members than those specified by the engineers. Furthermore, the assumed mechanical room loads located at the penthouse level may have been larger than those used by the engineers since the column was sized larger in the spot check.

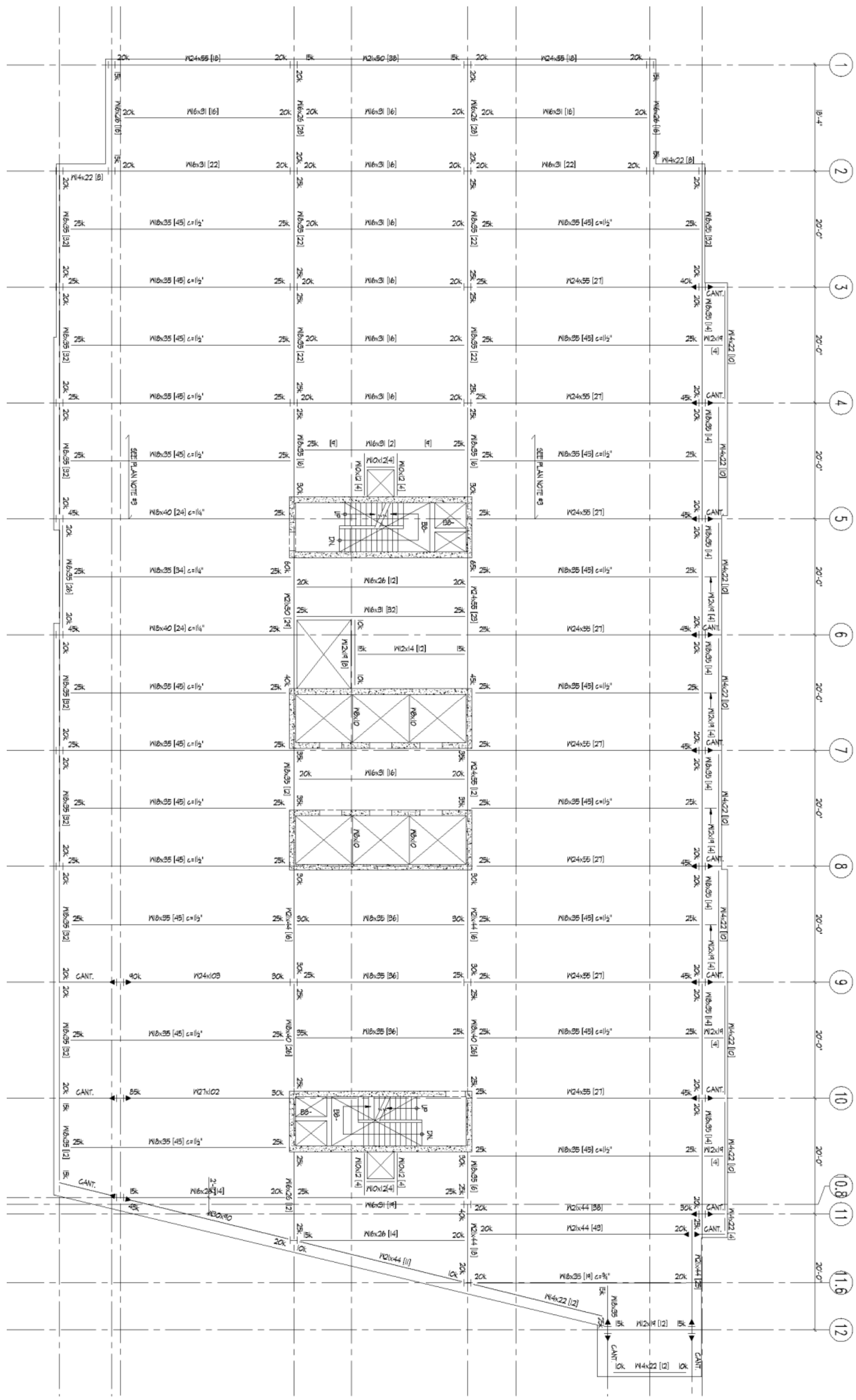
Technical Report #1

Appendix A – Typical Framing Plans



3rd Floor Framing Plan

Technical Report #1



8th Floor Framing Plan

Appendix B – Dead Load Calculations

Building Weight

Dead Loads

→ 2" 18 gage composite metal deck with 3 1/4"
LW concrete ⇒ 5 1/4" total depth

Use 2 psf for deck

Use $\frac{(3\,1/4" + 1")}{12\ in/ft} \times 115\ \frac{lb}{ft^3} = 41\ psf$

∴ Use $\boxed{43\ psf}$ total for floor

→ Steel Framing

Perform a couple spot calculations:

3rd floor between gridlines C-D and 6-8

beams
- (3) W18×35 40.5' long
- (1) W18×40 40.5'
- (2) W21×44 20'

columns (3) W14×99 14.25' tall

$$weight = 3(35)(40.5) + 1(40)(40.5) + 2(44)(20) + 3(99)(14.25) = \frac{11505\ lb}{40' \times 40.5"} = 7.1\ psf$$

Area

8th floor between gridlines C-D and 6-8

beams same as above

columns (3) W14×68 13.5' tall

$$weight = 3(35)(40.5) + 1(40)(40.5) + 2(44)(20) + 3(68)(13.5) = \frac{10027\ lb}{40' \times 40.5'} = 6.2\ psf$$

∴ Use $\boxed{7\ psf}$ total for steel framing

→ Shear Walls, 12" thick, 16 walls total

Length of walls per floor:

Levels 1-10 have 323'

PH has 242'

PH mezzanine has 81'

Example calc: Level 1

$150 \frac{lb}{ft^3} (1')(323')(16') = 775.2\ k$

Remainder calculated in spreadsheet

→ Curtain walls & precast concrete panels

Assume $\boxed{25\text{ psf}}$ (distributed to floor area)

Superimposed Dead Loads

→ Fireproofing $\boxed{2\text{ psf}}$

→ Finishes $\boxed{10\text{ psf}}$ slightly high to account for granite, tile, etc.

→ Partitions $\boxed{20\text{ psf}}$ for levels 1-10

→ Ceiling $\boxed{5\text{ psf}}$

→ MEP $\boxed{5\text{ psf}}$

Penthouse & Penthouse Mezzanine levels:

Heavy mechanical equipment loads not available

→ Assume an additional $\boxed{100\text{ psf}}$ (inplace of 5 psf)

accounts for built up concrete
there is still 150 psf LL

Atrium areas

→ Assume $\boxed{20\text{ psf}}$ to account for finishes & catwalks

* do not include partitions, ceiling, flooring

Technical Report #1

3rd, 7th, 10th floor terraces / roof
→ Use $\boxed{125\text{ psf}}$

SUMMARY:

Levels 1 – 10 ⇒ Use 43 + 7 + 25 + 2 + 10 + 20 + 5 + 5
= $\boxed{117\text{psf + shear walls}}$

PH & PH Mezzanine ⇒ Use 43 + 7 + 25 + 2 + 10 + 5 + 100
= $\boxed{192\text{ psf + shear walls}}$

Atrium ⇒ Use 7 + 25 + 2 + 5 + 20 = $\boxed{59\text{ psf}}$

Terraces / Roof ⇒ Use $\boxed{125\text{ psf}}$

Level	Floor-to-Floor Height (ft)	Floor Area (sq ft)	Atrium Area (sq ft)	Terrace/Roof Area (sq ft)	Shear Wall Length (ft)	Floor Loading (psf)	Atrium Loading (psf)	Terrace/Roof Loading (psf)	Shear Wall Weight (pcf)	Weight (kips)
2	18.83	29130	4296	0	323	117	59	0	150	4574
3	14.25	28697	2968	2469	323	117	59	125	150	4532
4	14.25	28534	3159	0	323	117	59	0	150	4215
5	14.25	28724	2968	0	323	117	59	0	150	4226
6	14.25	28517	3233	0	323	117	59	0	150	4218
7	13.50	24615	0	6886	323	117	0	125	150	4395
8	13.50	24635	0	0	323	117	0	0	150	3536
9	13.50	24649	0	0	323	117	0	0	150	3538
10	14.08	22883	0	1781	323	117	0	125	150	3582
PH	13.42	11664	0	6212	242	192	0	125	150	3503
PH Mezz.	16.58	5715	0	0	81	192	0	0	150	1299
Roof	-	0	0	22904	0	0	0	125	0	2863
									Total W:	44481

Appendix C – Snow Load Calculations

Snow Analysis

ground snow load (Fig. 7-1) $p_g = \boxed{20 \text{ psf}}$

flat roof snow load (Eqn 7-1) $p_f = 0.7\, C_e C_t I p_g$

exposure factor (Table 7-2) $C_e = \boxed{1.0}$ Category B, partially exposed

thermal factor (Table 7-3) $C_t = \boxed{1.0}$

Importance factor (Table 7-4) $I = \boxed{1.1}$ Category III

$\Rightarrow p_f = 0.7(1.0)(1.0)(1.1)(20) = \boxed{15.4 \text{ psf}}$

$p_g \leq 20$ psf so $p_f \geq I p_g = 1.1(20) = \boxed{22 \text{ psf}}$

CONTROLS

Technical Report #1

Snow Drift

Check east side of the Penthouse roof level where the metal roof cantilevers over:
(between gridlines 10 - 13 and A - B)

Assume Penthouse is a "Roof Projection"

(Sections 7.7.1 and 7.8) Use average dimensions to be conservative regarding cantilever.

Do not need to check every side of Penthouse because east side is worst case scenario:

total $h = 30'$; upper roof $l_u = 153'-2''$; upper roof $l_u = 49'$

(Eqn 7-3) $\gamma = 0.13\, p_g + 14 = 0.13(20\text{ psf}) + 14 = \boxed{16.6\text{ pcf}}$

< 30 pcf ✓

$h_b = \dfrac{p_s}{\gamma} = \dfrac{22\text{ psf}}{16.6\text{ pcf}} = \boxed{1.33\text{ ft}}$ = height of balanced snow load

$h_c = h_{total} - h_b = 30' - 1.33' = \boxed{28.67'}$

$\dfrac{h_c}{h_b} = \dfrac{28.67}{1.33} = 21.6 > 0.2 \Rightarrow$ drift loads required

LEEWARD DRIFT: $l_u = 153'-2''$

(Fig. 7-9) $h_d \approx 3.7 \Rightarrow$ use $h_d = 0.75(3.7)$

$\boxed{h_d = 2.78}$

WINDWARD DRIFT: $l_u = 49'$

(Fig. 7-9) $h_d \approx 2.0 \Rightarrow$ use $h_d = 0.75(2)$

$\boxed{h_d = 1.5}$

$2.78 > 1.5 \Rightarrow \boxed{h_{d\,design} = 2.78}$

$h_d = 2.78' < h_c = 28.67' \Rightarrow w = 4h_d = 4(2.78') = \boxed{11.12'}$

and h_d still $= 2.78'$

$w = 11.12' < 8h_c$ so okay

$p_d = h_d \gamma = 2.78'(16.6\text{ pcf}) = \boxed{46.1\text{ psf}}$

maximum drift surcharge load all around PH (to be conservative)

Technical Report #1

Check Typical Terrace snow drift using 3rd floor terrace:
(between gridlines 1 - 2 and C.9 - D.7)
(Section 7.7.1)

Short direction: total $h = 97.5'$
3rd fl to 10th fl terrace

upper roof $l_u = 96'-8''$

lower roof $l_u = 25'-8''$

Already know: $\gamma = 16.6$ pcf ; $h_b = 1.33'$

$h_c = 97.5' - 1.33' = \boxed{96.17'} \Rightarrow \frac{h_c}{h_b} > 0.2 \therefore$ drift req'd

LEEWARD DRIFT: $l_u = 96.67$ ft

(Fig. 7-9) $\boxed{h_d \approx 3.0}$

WINDWARD DRIFT: $l_u = 25.67$ ft

(Fig. 7-9) $h_d \approx 1.4 \Rightarrow h_d = 0.75(1.4)$

$\boxed{h_d = 1.05}$

$3.0 > 1.05 \Rightarrow$ use $\boxed{h_{d\,\text{design}} = 3.0'}$

$h_d = 3.0' < h_c = 96.17' \Rightarrow w = 4h_d = 4(3.0) = \boxed{12'}$
and h_d still $= 3.0$

$w = 12' < 8h_c$ so okay

$p_d = h_d \gamma = 3(16.6) = \boxed{49.8 \text{ psf}}$

maximum drift surcharge load in short direction

Long direction: (extend area to gridlines 1 - 8 and D - D.7)

total $h = 57'$
3rd fl to atrium roof

upper roof $l_u =$ approx. $16'$ due to protruding skylight
lower roof $l_u = 138'$

Technical Report #1

$h_c = 57' - 1.33' = 55.67' \Rightarrow \frac{h_c}{h_b} > 0.2$ ∴ drift required

LEEWARD DRIFT: $l_u = 16'$

(Fig. 7-9) $\boxed{h_d \approx 1.0}$

WINDWARD DRIFT: $l_u = 138'$

(Fig. 7-9) $h_d \approx 3.5 \Rightarrow h_d = 0.75(3.5)$

$\boxed{h_d = 2.63}$

$2.63 > 1.0 \Rightarrow$ use $\boxed{h_{d\,design} = 2.63}$

$h_d = 2.63 < h_c = 55.67' \Rightarrow w = 4h_d = 4(2.63) = \boxed{10.5'}$

and h_d still $= 2.63$

$w = 10.5' < 8h_c$ so okay

$p_d = h_d \gamma = 2.63(16.6) = \boxed{43.66 \text{ psf}}$

maximum drift surcharge in long direction

Appendix D – Wind Analysis

Analysis 1

Wind Analysis - Method 2 in Chapter 6 of ASCE 7-05

Location: Richmond, VA

Basic wind speed (Figure 6-1) $\boxed{V = 90 \text{ mph}}$

Wind directionality factor (Table 6-4) $\boxed{K_d = 0.85}$

Importance Factor (Table 6-1) $\boxed{I = 1.15}$ ...based on • Occupancy Category III (Table 1-1)
• V = 85-100 mph

$\boxed{\text{Exposure Category} = B}$ ......based on • Surface Roughness B (urban & suburban areas)

Velocity pressure exposure coeff. (Table 6-3): ...based on Exposure B, Case 2

Height above ground level, z (ft)	K_z
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04
140	1.09
160	1.13
176'-5"	1.16

interpolate in spreadsheet to get values by level

Topographic factor $\boxed{K_{zt} = 1.0}$ per 6.5.7.1 & 6.5.7.2

Gust effect factor: frequency $n_1 = \frac{100}{h\ (ft)}$ (Eqn C6-17)

$n_1 = \frac{100}{176'\text{-}5''} = \boxed{0.567 \text{ Hz}}$

$\Rightarrow \boxed{\text{FLEXIBLE}}$ since $n_1 < 1$

Technical Report #1

Gust effect factor for Flexible Structures:

→ $g_Q = g_v = 3.4$

$$g_R = \sqrt{2 \ln(3600 n_1)} + \frac{0.577}{\sqrt{2 \ln(3600 n_1)}} \quad \text{(Eqn. 6-9)} \quad ; n_1 = 0.567$$

→ $\boxed{g_R = 4.052}$

Section 6.3 : B = horiz. dim. of building measured normal to wind direction

L = horiz. dim of bldg measured parallel to wind direction

h = mean roof height

	N-S	E-W
B	260' - 8" = 260.67	145.25'
L	145.25'	260' - 8" = 260.67
h	176'-5" = 176.42	176'-5" = 176.42

$\bar{z}$ = equivalent height of structure

$\bar{z}$ = max of 0.6h or z_{min}

$0.6h = 0.6(176.42) = \boxed{105.85 \text{ ft}}$ ← controls

$z_{min} = 30$ ft (Table 6-2)

$\bar{\alpha} = \frac{1}{4.0}$; $\bar{b} = 0.45$ (Table 6-2)

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V \left(\frac{88}{60}\right) \quad \text{(Eqn 6-14)}$$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{105.85}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right) = \boxed{79.49 \text{ ft/s}}$$

mean hourly wind speed

Intensity of Turbulence $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$ (Eqn 6-5)

$c = 0.30$ (Table 6-2) $I_{\bar{z}} = 0.3\left(\frac{33}{105.85}\right)^{1/6} = \boxed{0.247}$

Technical Report #1

the integral length scale of turbulence $L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\varepsilon}}$ (Eqn 6-7)

$\ell = 320$ ft ; $\bar{\varepsilon} = \frac{1}{3.0}$ (Table 6-2)

$$L_{\bar{z}} = 320\left(\frac{105.85}{33}\right)^{1/3} = \boxed{471.93}$$

Background response $Q = \sqrt{\dfrac{1}{1+0.63\left(\dfrac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$ (Eqn 6-6)

N-S $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{260.67+176.42}{471.93}\right)^{0.63}}} = \boxed{0.790}$$

E-W $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{145.25+176.42}{471.93}\right)^{0.63}}} = \boxed{0.818}$$

R_h : use $\eta = \dfrac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \dfrac{4.6(0.567)(176.42)}{79.49} = \boxed{5.789}$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{(Eqn 6-13a)}$$

$$R_h = \frac{1}{5.789} - \frac{1}{2(5.789)^2}\left(1-e^{-2(5.789)}\right) = \boxed{0.158}$$

R_B : use $\eta = \dfrac{4.6 n_1 B}{\bar{V}_{\bar{z}}}$; $R_B =$ (Eqn 6-13a)

N-S $$\eta = \frac{4.6(0.567)(260.67)}{79.49} = \boxed{8.553}$$

$$R_B = \frac{1}{8.553} - \frac{1}{2(8.553)^2}\left(1-e^{-2(8.553)}\right) = \boxed{0.110}$$

E-W $$\eta = \frac{4.6(0.567)(145.25)}{79.49} = \boxed{4.766}$$

Technical Report #1

R_B: E-W $\quad R_B = \frac{1}{4.766} - \frac{1}{2(4.766)^2}\left(1 - e^{-2(4.766)}\right)$

$= \boxed{0.188}$

R_L: use $\eta = \frac{15.4\, n_1\, L}{\bar{V}_{\bar{z}}}$

N-S $\quad \eta = \frac{15.4(0.567)(145.25)}{79.49} = \boxed{15.955}$

$R_L = \frac{1}{15.955} - \frac{1}{2(15.955)^2}\left(1 - e^{-2(15.955)}\right) = \boxed{0.061}$

E-W $\quad \eta = \frac{15.4(0.567)(260.67)}{79.49} = \boxed{28.634}$

$R_L = \frac{1}{28.634} - \frac{1}{2(28.634)^2}\left(1 - e^{-2(28.634)}\right) = \boxed{0.034}$

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$ (Eqn 6-12) $\quad N_1 = \frac{0.567(471.93)}{79.49} = \boxed{3.366}$

$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}}$ (Eqn 6-11)

$R_n = \frac{7.47(3.366)}{[1 + 10.3(3.366)]^{5/3}} = \boxed{0.065}$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$ (Eqn 6-10)

β = damping ratio, percent of critical

According to pg. 294 in ASCE 7-05 under Structural Damping: 1 to 2% for damping ratio

$\Rightarrow$ assume $\beta = 1.5\% = 0.015$

Technical Report #1

N-S $R = \sqrt{\frac{1}{0.015}(0.065)(0.158)(0.110)(0.53 + 0.47 \times 0.061)}$

$\boxed{R = 0.205}$

E-W $R = \sqrt{\frac{1}{0.015}(0.065)(0.158)(0.188)(0.53 + 0.47 \times 0.034)}$

$\boxed{R = 0.265}$

Finally, gust effect factor $G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$

N-S $G_f = 0.925\left[\frac{1 + 1.7(0.247)\sqrt{(3.4)^2(0.79)^2 + (4.052)^2(0.205)^2}}{1 + 1.7(3.4)(0.247)}\right]$

$\boxed{G_f = 0.831}$

E-W $G_f = 0.925\left(\frac{1 + 1.7(0.247)\sqrt{(3.4)^2(0.818)^2 + (4.052)^2(0.265)^2}}{1 + 1.7(3.4)(0.247)}\right]$

$\boxed{G_f = 0.858}$

Enclosure classification (6.5.9) - Enclosed (definition in 6.2)

Internal pressure coeff. GC_{pi} (Fig. 6-5)

$\boxed{= \pm 0.18}$

External pressure coeff. C_p (Fig. 6-6)

Windward wall $\boxed{C_p = 0.8}$ use with q_z

Leeward wall N-S $\frac{L}{B} = \frac{145.25}{260.67} = 0.557$

$\Rightarrow \boxed{C_p = -0.5}$ use w/ q_h

Leeward wall E-W $\frac{L}{B} = \frac{260.67}{145.25} = 1.795 \Rightarrow \boxed{C_p = -0.341}$

from interpolation
use w/ q_h

Technical Report #1

Velocity pressure (section 6.5.10)

(Eqn 6-15) q_z evaluated @ z $= 0.00256\, K_z K_{zt} K_d V^2 I \left(\frac{lb}{ft^2}\right)$

q_h = velocity pressure using Eqn 6-15 @ mean roof height h

already found

K_z = veloc. press. exposure coeff, VARIES

$K_{zt} = 1.0$

$K_d = 0.85$

$V = 90$ mph

$I = 1.15$

Design wind pressures: (for Flexible Bldg)

(Eqn 6-19) $p = q G_f C_p - q_i (GC_{pi})$ answer in psf

windward: $q = q_z$ @ each level

$q_i = q_h$

Leeward: $q = q_h$; $q_i = q_h$

See spreadsheet for K_z, q_z, pressures for each level

Technical Report #1

Summary of 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

Technical Report #1

Analysis 2

Do second wind analysis with different B & L because levels 7 thru roof are smaller.

	N-S	E-W
B	≈ 228'	≈ 118'
L	≈ 118'	≈ 228'
h	176.42'	176.42'

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} \quad \text{(Eqn 6-6)}$$

N-S $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{228+176.42}{471.93}\right)^{0.63}}} = \boxed{0.798}$$

E-W $$Q = \sqrt{\frac{1}{1+0.63\left(\frac{118+176.42}{471.93}\right)^{0.63}}} = \boxed{0.825}$$

R_B : $$\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} \; ; \; R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1-e^{-2\eta}\right) \quad \text{(Eqn. 6-13a)}$$

N-S $$\eta = \frac{4.6(0.567)(228)}{79.49} = \boxed{7.481}$$

$$R_B = \frac{1}{7.481} - \frac{1}{2(7.481)^2}\left(1-e^{-2(7.481)}\right) = \boxed{0.125}$$

E-W $$\eta = \frac{4.6(0.567)(118)}{79.49} = \boxed{3.872}$$

$$R_B = \frac{1}{3.872} - \frac{1}{2(3.872)^2}\left(1-e^{-2(3.872)}\right) = \boxed{0.225}$$

Technical Report #1

R_L: $\eta = \dfrac{15.4\, n_1 L}{\bar{V}_{\bar{z}}}$; R_L = (Eqn 6-13a)

N-S $\quad \eta = \dfrac{15.4(0.567)(118)}{79.49} = \boxed{12.962}$

$$R_L = \frac{1}{12.962} - \frac{1}{2(12.962)^2}\left(1 - e^{-2(12.962)}\right) = \boxed{0.074}$$

E-W $\quad \eta = \dfrac{15.4(0.567)(228)}{79.49} = \boxed{25.045}$

$$R_L = \frac{1}{25.045} - \frac{1}{2(25.045)^2}\left(1 - e^{-2(25.045)}\right) = \boxed{0.039}$$

External Pressure Coeff. C_p (Fig. 6-6)

Leeward wall N-S $\quad \dfrac{L}{B} = \dfrac{118}{228} = 0.518 \Rightarrow \boxed{C_p = -0.5}$

Leeward wall E-W $\quad \dfrac{L}{B} = \dfrac{228}{118} = 1.932 \Rightarrow \boxed{C_p = -0.314}$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} \quad \text{(Eqn 6-10)}$$

N-S $\quad R = \sqrt{\dfrac{1}{0.015}(0.065)(0.158)(0.125)(0.53 + 0.47 \times 0.074)} = \boxed{0.220}$

E-W $\quad R = \sqrt{\dfrac{1}{0.015}(0.065)(0.158)(0.225)(0.53 + 0.47 \times 0.039)} = \boxed{0.291}$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) \quad \text{(Eqn 6-8)}$$

N-S $\quad G_f = 0.925\left(\dfrac{1 + 1.7(0.247)\sqrt{(3.4)^2(0.798)^2 + (4.052)^2(0.220)^2}}{1 + 1.7(3.4)(0.247)}\right)$

$\boxed{G_f = 0.838}$

E-W $\quad G_f = 0.925\left(\dfrac{1 + 1.7(0.247)\sqrt{(3.4)^2(0.825)^2 + (4.052)^2(0.291)^2}}{1 + 1.7(3.4)(0.247)}\right)$

$\boxed{G_f = 0.868}$

All other variables remain unchanged from the first analysis.
See spreadsheet for new pressures.

Technical Report #1

Summary of 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)

	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

Appendix E – Seismic Analysis

Seismic Analysis – Equivalent Lateral Force Procedure

Occupancy Category : III (Table 1-1)

S_s = 23% g for Richmond, VA (Fig. 22-1)

S_1 = 6% g (Fig. 22-2)

Site Class : (Table 20.3-1) C from geotech. report

(Table 11.4-1) $F_a = 1.2$

(Table 11.4-2) $F_v = 1.7$

$S_{MS} = F_a S_s = 1.2(0.23) = 0.276$ (Eqn 11.4-1)

$S_{M1} = F_v S_1 = 1.7(0.06) = 0.102$ (Eqn 11.4-2)

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.276) = 0.184$ (Eqn 11.4-3)

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.102) = 0.068$ (Eqn 11.4-4)

Importance Factor : (Table 11.5-1) $I = 1.25$

Seismic Design Category : (Table 11.6-1) ⇒ B
(Table 11.6-2) ⇒ B

∴ Equivalent Lateral Force Procedure permitted by Table 12.6-1

(Fig. 22-15) $T_L = 8$

(Table 12.8-2) $C_t = 0.02$, $x = 0.75$ } for "All other structural systems"

$T_a = C_t h_n^x$ (Eqn 12.8-7) h_n = height of bldg = 176.42′

$T_a = 0.02(176.42)^{0.75} = 0.968$ sec.
approximate fundamental period

$T = T_a = 0.968$ per Section 12.8.2

Check $T \le C_u T_a$ where $C_u = 1.7$ (Table 12.8-1)

so use $T = (1.7)(0.968) = \boxed{1.645}$

$R = 5$ (Table 12.2-1) "Ordinary reinforced concrete shear walls" under Building Frame Systems for $T \leq T_L$ (Eqn 12.8-3) $C_s \leq \dfrac{S_{D1}}{T\left(\frac{R}{I}\right)}$

$1.645 < 8$

(Eqn 12.8-2) $C_s = \dfrac{S_{DS}}{\left(\frac{R}{I}\right)}$

(Eqn 12.8-5 $C \geq 0.01$

$$C_s = \frac{0.184}{\left(\frac{5}{1.25}\right)} = 0.046 > \frac{0.068}{1.645\left(\frac{5}{1.25}\right)} = \boxed{0.0103}$$

controls

also 0.0103 > 0.01 so okay

(Eqn. 12.8-1) $V = C_s W$ Seismic Base Shear

W = effective seismic weight per Sec. 12.7.2

W = DL + 25% LL storage + min 10psf partitions in offices + 20% snow load

where $P_f > 30$ psf

* Partitions were included in DL
$P_f = 22$ psf < 30 psf
Storage areas are negligible

∴ W = DL   see spreadsheet for weights by level
= 44481k

$V = 0.0103\,(44481) = \boxed{458\text{ k}}$

Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$ (Eqn 12.8-11)

$C_{vx} = \dfrac{w_x h_x^{\,k}}{\sum_{i=1}^{n} w_i h_i^{\,k}}$ (Eqn 12.8-12)

k = 1 for T = 0.5 and 2 for T = 2.5

for T = 0.968 interpolate ⇒ $\boxed{k = 1.234}$

See spreadsheet for forces, shears & moments

Appendix F – Typical Spot Checks

Typical Spot Checks

Typical Slab/ Metal Deck:

2" lightweight concrete fill on 3 1/4" 18 gage composite metal deck

Deck properties specified by engineer:

$I_p = 0.560\ in^4$; $S_p = 0.523\ in^3$; $S_n = 0.529\ in^3$

match 2×12" deck w/ 115 pcf concrete in the United Steel Deck Design Manual & Catalog
2" LOK-FLOOR

Max. unshored span:

5 1/4", 3 span ⇒ $\boxed{10.97\ ft}$ ← considered clear span

beams typically spaced 10 ft on center

∴ typical clear span < 10.97 ft okay

Uniform Live Service Load:

Studs spaced 1 per ft., 5 1/4" slab, 11 ft span to be conservative

⇒ $\boxed{235\ psf}$

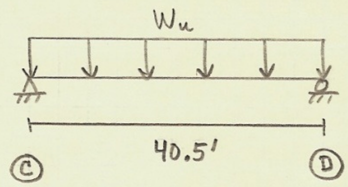
use unfactored loads to check

DL = 68 psf
SDL = 42 psf
LL = 80 psf ← corridors above 1st floor

Total = 190 psf* < 235 psf okay

* includes selfweight of slab/metal deck even though United Steel Deck accounts for it so was conservative

Typical Beam:



Trib. width = 10'

Loads: DL = 68 psf × 10' = 0.68 k/ft
SDL = 42 psf × 10' = 0.42 k/ft
LL = 80 psf × 10' = 0.8 k/ft

$W_u = 1.2D + 1.6L = 1.2(0.68 + 0.42) + 1.6(0.8) = 2.6$ k/ft

$M_u = \frac{Wl^2}{8} = \frac{2.6(40.5)^2}{8} = 533.1$ ft-k

$V_u = \frac{Wl}{2} = \frac{2.6(40.5)}{2} = 52.7$ k

Assume $a = 1.5$ in then $Y2 = 5\,1/4'' - \frac{1.5''}{2} = 4.5''$

Table 3-19 in Steel Manual:

Try a $\boxed{W18 \times 40}$ → $\phi M_p = 294$ ft-k
$\phi M = 538$ ft-k
$Y1 = 3$
$\Sigma Q_n = 430$ k
studs $= \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{430}{17.2} \times 2$

Table 3-21
$Q_n = 17.2$ k

$\boxed{\text{\# studs} = 50}$

Equivalent weight = (40.5')(40 lb/ft) + (50 studs)(10 lb/stud)
= 2.12 k
↑ assumed

check assumptions: $b_{eff} = 10$ ft or $\frac{40.5'}{4} = 10.125'$
∴ $b_{eff} = 10' \times 12 = 120''$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{430}{0.85(3.5)(120)} = 1.20'' < 1.5''$ so conservative ✓

check deflection:

construction Δ DL = 0.68 k/ft + 0.04 k/ft + ≈ 0.01 k/ft
= 0.73 k/ft

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$$\Delta_{\substack{construction\\DL}} = \frac{5}{384}\,\frac{(0.73)(40.5)^4(1728)}{(29000)(612)} = 2.49''$$

I

$\frac{L}{360}$ or 1" is criteria $\Rightarrow$ 2.49" > 1" so NOT okay

however, can $\boxed{\text{camber}}$ the beam $\boxed{1\tfrac{1}{2}''}$ which is the design by the engineer (w/ a diff. beam size & # studs)

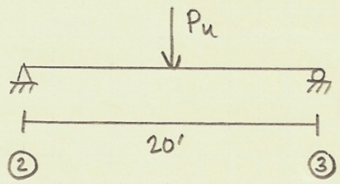
$$\text{live } \Delta = \frac{5wL^4}{384EI} = \frac{5(0.8)(40.5)^4(1728)}{384(29000)(1510)} = 1.12''$$

I from Table 3-20

$$\frac{L}{360} = \frac{40.5 \times 12}{360} = 1.35'' > 1.12'' \text{ okay}$$

Finally, $\phi V_n = 169\text{ k} > 52.7\text{ k}$ okay (Table 3-2)

Typical Girder:



Beam 1: $W_{u_1} = 1.2(\underset{DL}{0.75} + \underset{SDL}{0.42}) + 1.6(\underset{LL}{0.8}) = 2.684 \text{ k/ft}$

10' trib width

$$V_{u_1} = \frac{2.684(40.5')}{2} = 54.35 \text{ k}$$

Beam 2: $W_{u_2} = 2.684 \text{ k/ft}$

10' trib width $V_{u_2} = \frac{2.684(30')}{2} = 40.26 \text{ k}$

$\therefore P_u = 54.35 + 40.26 = 94.6 \text{ k}$

$$M_u = \frac{Pl}{4} = \frac{94.6(20')}{4} = 473 \text{ ft-k}$$

$$V_u = \frac{P}{2} = \frac{94.6}{2} = 47.3 \text{ k}$$

Assume $a = 1.5$ in. so $Y2 = 5\,1/4 - \frac{1.5}{2} = 4.5$

(Table 3-19) Try a W18x35 → $\phi M_p = 249$ ft-k
$\phi M = 494$ ft-k
$Y1 = 2$
$\Sigma Q_n = 451$ k

(Table 3-21) $Q_n = 17.1$ k ← based on $f'_c = 3$ ksi so slightly conservative

$$\#\text{studs} = \frac{\Sigma Q_n}{Q_n} \times 2 = \frac{451}{17.1} \times 2 = 54 \text{ studs}$$

Equivalent weight = $(20')(35 \text{ lb/ft}) + (54 \text{ studs})(10 \text{ lb/stud})$
$= 1.24$ k

Check assumptions: $b_{eff} = 10$ ft. or $\frac{20'}{4} = 5'$

$\therefore b_{eff} = 5(12) = 60''$

Technical Report #1

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{451}{0.85(3.5)(60)} = 2.53'' > 1.5'' \quad \text{so NOT okay}$$

Use $a = 2.5''$ so $Y2 = 5.25 - \frac{2.5}{2} = 4''$

$\Rightarrow$ can still use $\boxed{W18\times35}$ with $\phi M = 477$ ft-k at $Y1 = 2$
with $\boxed{\text{54 shear studs}}$

because $2.5 \approx 2.53$ okay

Check deflection:

$$DL = \underset{}{0.68\ k/ft} + \underset{beam}{0.035\ k/ft} + \underset{studs}{0.027\ k/ft} = 0.742\ k/ft$$

$$\Delta_{construction} = \frac{5}{384}\,\frac{(0.742)(20)^4(1728)}{(29000)(510)} = 0.18''$$

$\leftarrow$ I

$$\frac{L}{360} = \frac{20\times12}{360} = 0.67'' > 0.18'' \ ; \quad 1'' > 0.18'' \quad \text{so okay}$$

$$\Delta_{live} = \frac{5(0.8)(20)^4(1728)}{384(29000)(1300)} = 0.076'' < \frac{L}{360} \quad \text{okay}$$

$\leftarrow$ Table 3-20 I

Finally, $\phi V_n = 159k > 47.3k$ okay (Table 3-2)

Technical Report #1

Typical Column:

Check column 3-B on the 8th floor due to splicing at middle of 7th floor. Furthermore, the beam and then girder that were checked transfer load to that column.

Rough Load Takedown:

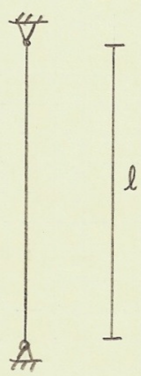
Roof → none
PH Mezzanine → open to below } due to cooling towers
PH → 192 psf DL, 150 psf LL over 20' × 35.25'

$$P_u = [1.2(0.192) + 1.6(0.15)](20)(35.25) = 331.6\ k$$

9th-10th floors → 117 psf DL, 80 psf LL over 20' × 35.25'

$$P_u = [1.2(0.117) + 1.6(0.08)](20)(35.25) = 189.2\ k$$

$$\text{total } P_u = 331.6 + 2(189.2) = 710\ k$$



K = 1.0

l = 13.5 ft.

unbraced length assumed to be floor-to-floor height

(Table 4-1) Use a $\boxed{W14 \times 74}$

$\phi P_n = 734\ k > 710\ k$ @ 13 ft.