Schenley Place

Pittsburgh, Pa

Technical Report 1

Structural Concepts and Existing Conditions

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Executive Summary

The following technical report describes the structural concepts and existing conditions of Schenley Place. The assumptions and simplifications necessary for the building's analysis are acknowledged within the included summaries and calculations.

Schenley Place contains a total of seven levels of office space above grade and approximately three and a half levels of parking garage mostly below grade. The gravity system above grade is designed as a typical wide-flanged steel frame system, while the gravity system below grade is designed as a combination of cast-in-place concrete columns and load bearing walls. Due to the depth of competent bedrock, the deep foundation is designed as a combination of drilled cast-in-place caissons coupled with grade beams, and a perimeter caisson wall to resist underlying soils. The lateral force resisting system located at the building's core is designed as both concentrically and eccentrically steel braced frames.

The snow, wind, and seismic analyses in this report were performed in accordance with the ASCE 7-05. The ASCE 7-05 is also the code used by the structural design professionals, Atlantic Engineering Services (AES), in the design of Schenley Place. Because AES failed to report the design base shear due to wind, a fair comparison of the 317 kips wind design base shear determined in the following report cannot be made. A seismic design base shear of 365 kips was reported by AES, while the following analysis reports a seismic design base shear of 645 kips. Because both this report and the AES used the ASCE 7-05, the apparent discrepancy of results must be attributable to some incongruence in applied variables: most likely the fundamental building period (T) or the effective seismic weight (w).

The following report includes spot checks for the composite metal deck floor slab on a typical level, as well as a wide-flange steel beam and column that are part of the typical steel frame. The spot checks performed accounted for gravity loads *only*. These spot checks conclude that the assumed design loads acting on the members were compiled in a comparable manner to AES. In accordance with these results, each component that AES designed is adequate. However, the large remaining strength capacities are attributable to lateral loads acting on the members subject to spot checking. This assumption will be further investigated in later reports, which will evaluate the effects of lateral loads on these structural members.

Architectural Concepts

Schenley Place is a new office located at 4420 Bayard Street, in the heart of Oakland, an inner-city neighborhood of Pittsburgh, Pennsylvania. The building contains a total of 7 levels above grade and 3.5 levels of parking garage mostly below grade. On the fourth and remaining levels of the building's north elevation, there is a substantial step-back in the building's original footprint, dictated by the specialty zoning constraints placed upon the building site. The site is fully landscaped and contains a small pocket park along the building's east elevation that is shared by the neighboring First Baptist Church of Pittsburgh.

Schenley Place has been designed to accommodate a variety of office type tenants. The first floor opens to a finished main building lobby, with remaining unfinished space available for tenant occupancy. The remaining floors have open, unfinished office space to accommodate tenants.

The exterior of Schenley Place is mainly brick and cast-stone veneer, architectural decisions driven by the design constraints based upon the location of the building site. The three-story façade, facing Bayard Street, is mainly Indiana limestone to mimic the neighboring First Baptist Church, whereas the seven-story façade, facing Ruskin Avenue, is primarily buff-colored brick to compliment Ruskin Hall, a dormitory belonging to the University of Pittsburgh. Found within the details of the building's façade are punched aluminum windows, cast-stone cornices, sills, and headers, brick details, and aluminum curtain walls. Masonry parapets occur at the step-back on the fourth level and the roof level. The roof top penthouse is clad in metal panels and the HVAC units are disguised by metal screenwall.

Structural System Summary

Foundation System Summary

The geotechnical engineering study for Schenley Place was completed by Construction Engineering Consultants, Inc. (CEC) on January 26, 2006. CEC reported competent bedrock at a depth of approximately forty-five feet. In addition, their report stated that the most economical deep foundation solution for Schenley Place included a system of cast-in-place drilled caissons, coupled with grade beams to support wall loads between caissons. Due to the below-grade excavation and the close proximity of neighboring structures, CEC also suggested a shoring system that would support overlying soils. As a result of CEC's geotechnical study, the foundation of Schenley Place incorporates a cast-in-place perimeter caisson wall designed to act as the shoring system, and drilled cast-in-place caissons and grade beams designed to support wall loads.

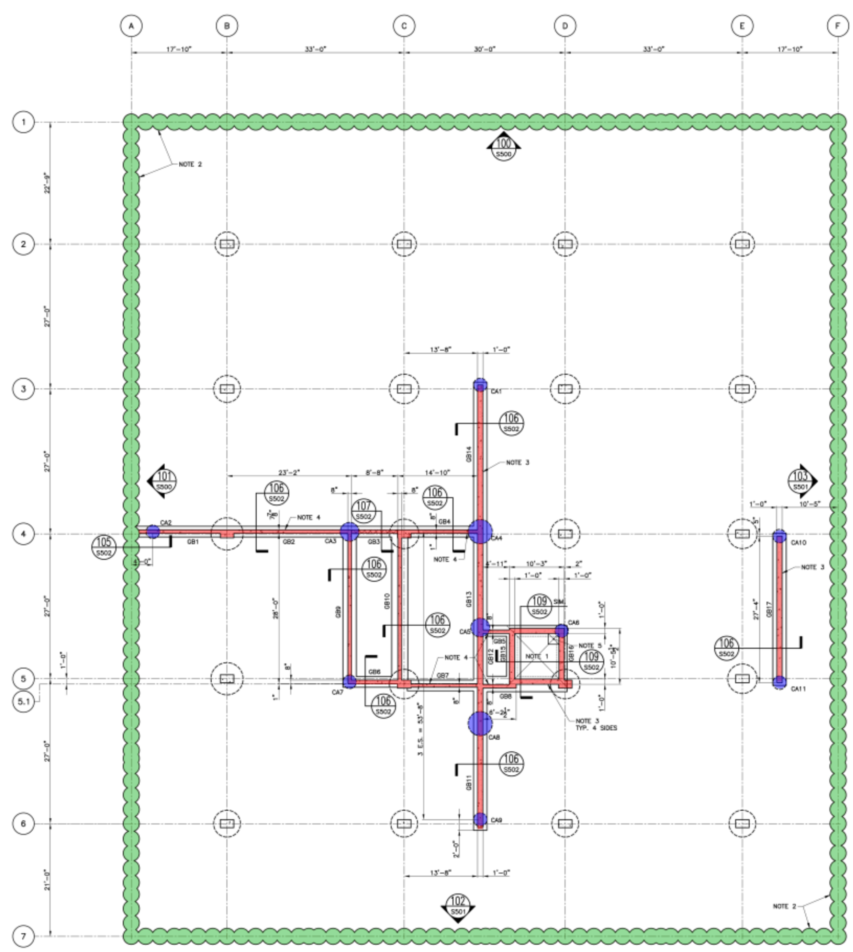


FIGURE 1: Cast-in-place perimeter caisson wall (green), drilled, cast-in-place caissons (blue), and grade beams (red).

The details of the cast-in-place perimeter caisson walls (e.g. reinforcing steel and the rock socket) will be designed by the perimeter caisson installer. The perimeter caisson walls terminate at the first floor and are tied to cap beams through the use of #6 dowels with standard 90º hooks. Each cap beam is 2'-0" in depth and varies in width according to the size of the perimeter caisson wall. The cast-in-place drilled caissons have a compressive strength of 4000 psi and are socketed at least three feet into sound bedrock as established by CEC's geotechnical report. In addition, the drilled caissons are designed with an end bearing capacity of 25 tsf. As Figure 1 illustrates, the drilled caissons vary in diameter from 30 to 54 inches. At the first floor, the drilled caissons terminate and are tied to 24"by 24" caisson caps with #6 dowels embedded at least two feet into the drilled caisson. The grade beams have a compressive strength of 4000 psi and range from 24 to 36 inches in width and 36 to 44 inches in depth. Each grade beam has top and bottom reinforcement with bar schedules that vary according to the size of the beam. The slab on grade is 4000 psi in strength and reinforced with 2x2-W1.4xW1.4 welded wire fabric, with a minimum thickness of 4".

Superstructure System Summary

The floor system of Schenley Place consists of both cast-in-place two-way concrete flat slab and cast-in-place concrete on composite steel deck. The floor system of the parking garage (the levels sub-grade) is designed as a two-way flat slab system. These slabs incorporate normal weight concrete with a 5000 psi compressive strength. Reinforcement that is primarily #5 and #7 top and bottom bars, is placed within a minimum slab thickness of 11". Additional reinforcement is placed at the cast-in-place concrete walls where necessary. The typical office spaces (the floors above grade) are designed as 3 ½" normal weight cast-in-place concrete slab on 3"-20 gauge composite steel decking, supported by composite steel beams. This floor system also acts as the diaphragm, which assists in transferring lateral loads (i.e. wind and seismic) to the lateral force resisting system.

The below grade gravity system of Schenley Place is a combination of cast-in-place concrete columns and load bearing wall. The typical above grade gravity system is designed as a steel frame consisting of primarily W-shapes that vary in size. The normal weight concrete columns have a compressive strength of 7000 psi. These concrete columns (either 24"x28" or 18"x30" in size) typically span the height of the parking garage and are reinforced with both #9 and #11 bars. The normal weight, cast-in-place concrete load bearing walls have a compressive strength of 5000 psi and range from 8 to 12 inches in thickness. They are reinforced with #5 or #6 bars at 12 or 16 inches on center. Figure 1 (pictured above) illustrates the location of these components. Each concrete column corresponds with the intersection of column lines, while the load bearing walls correspond with the grade beams. Both the concrete columns and load-bearing walls help transfer the slab loadings to the foundation. The typical steel frame gravity system above grade consists of wide-flange steel shapes with yield strengths of

50 ksi. Beams span the east-west direction at a maximum of 33'-0" while steel girders span the north-south direction at a maximum of 27'-0". Columns span up to three stories before a splice is required; typical story heights are 13'-4". Figure 2 (pictured below) illustrates the location of these components. Again, each steel column corresponds with the intersection of column lines, and the steel beams and girders span accordingly. The load path is the same of any typical steel frame gravity system: beam to girder to column to foundation.

The roof system at both the fourth level and main roof[1] requires 1½"-20 gage wide-rib, galvanized steel roof decking, supported by steel beams. Where the main roof houses the rooftop mechanical units and penthouse, additional steel beams are designed to support the increased loads.

[1] The "main roof" refers to the roof level above the seventh floor.

Lateral Force Resisting System Summary

The lateral force resisiting system, located at the building's core (Figure 2), is designed of both eccentrically and concentrically braced frames (Figure 3). Beginning on Garage Level 1[2], these frames continue to the main roof level. The eccentrically braced frames span the west-east direction at 30'-0", along column line 4 and 5.1 between C and D. The concentrically braced frames span the north-south direction at 27'0", along column line C and D between 4 and 5.1.

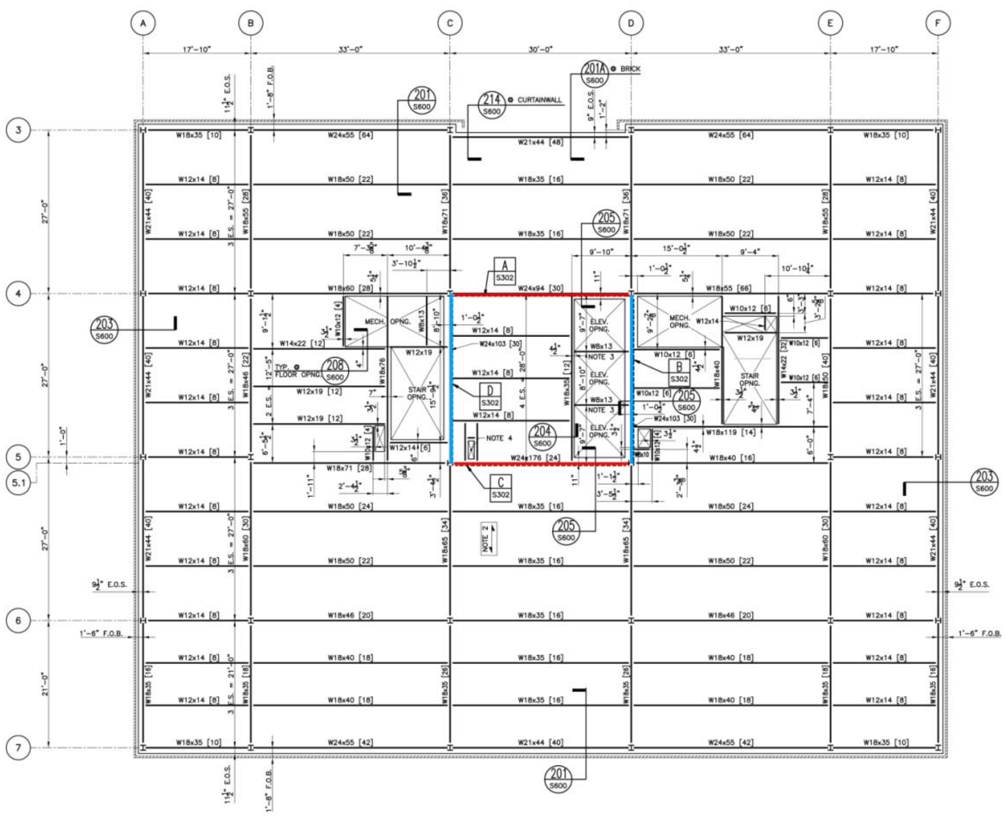


FIGURE 2: Location of lateral force resisting system. Concentrically braced frames (blue), and eccentrically braced frames (red).

[2] Garage Level 1 refers to the elevation 9'-0" below the first floor (on-grade).

The braces are designed as hollow steel shapes from HSS8x8x$^3/_8$ to HSS14x10x$^5/_8$ with yield strengths of 46 ksi. The composite steel deck system acts as the diaphragm, transferring the lateral loads acting on the exterior beams of the gravity system to columns that act as part of the gravity system as well as the lateral force resisting system. These columns, ranging from W14x311 at Garage Level 1 to W14x130 at roof level, then transfer the lateral load to the HSS-braces.

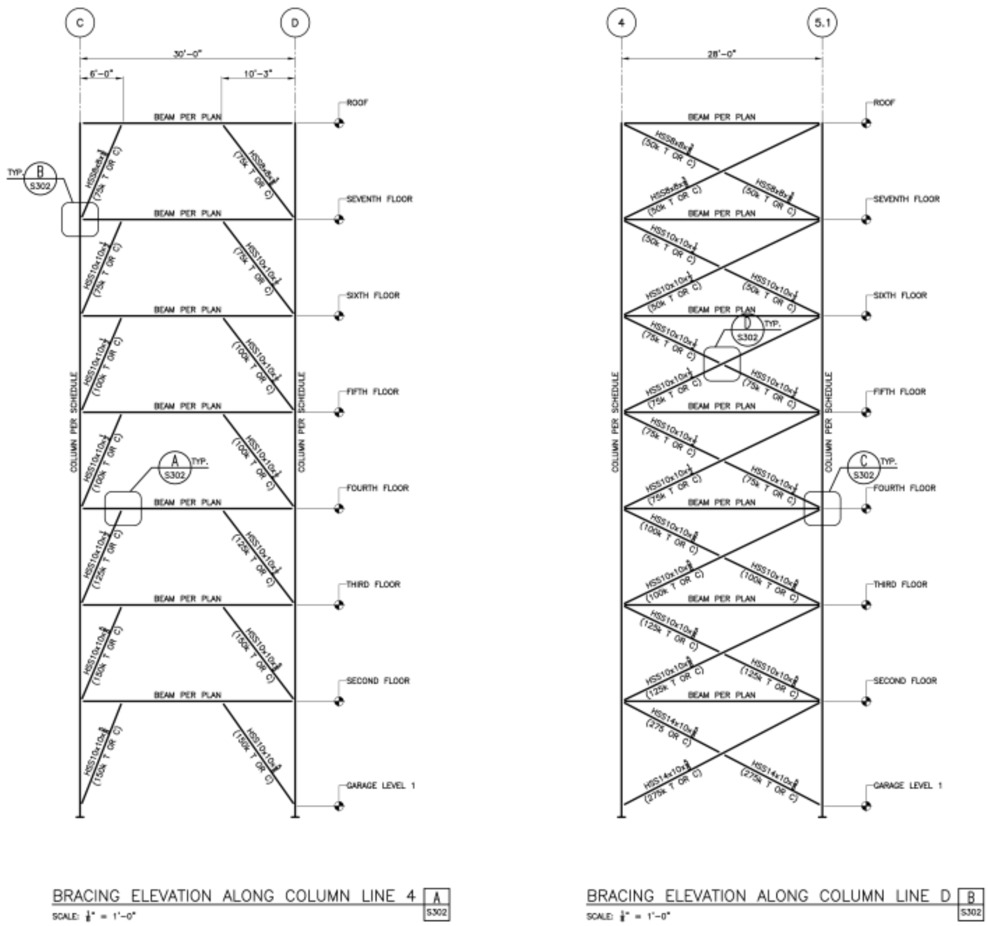


FIGURE 3: Eccentrically braced frame (left), concentrically braced frame (right).

Note that the steel systems have not been specifically detailed for seismic resistance.

Codes and Design Standards

Relevant Codes

- International Building Code (IBC), 2006
  (As amended by the city of Pittsburgh)
- Minimum Design Loads for Buildings and Other Structures (2005), American Society for Civil Engineers (ASCE 7-05)
- Building Code Requirements for Structural Concrete, American Concrete Institute (ACI 318-08)
- Specification for Structural Steel Buildings, American Institute of Steel Construction (AISC 360-05)

In the analysis of the original design, the same codes as provided above were used.

Material Strength Requirement Summary

Cast-in-place Concrete

Shallow foundations	f'c = 3000 psi
Caissons, grade beams, slabs on grade, and elevated floor slabs on deck	f'c = 4000 psi
Walls, beams, and formed elevated slabs	f'c = 5000 psi
Columns	f'c = 7000 psi

Structural Steel

Structural W-shapes and channels	$F_y = 50$ ksi
Steel tubes (HSS shapes)	$F_y = 46$ ksi
Angles and plates	$F_y = 36$ ksi
¾" bolts	ASTM A325
Composite steel deck (a minimum of 3"-20 gage)	$F_y \geq 33$ ksi
Steel roof deck (a minimum of 1½"-20 gage)	$F_y \geq 33$ ks

Summary of Design Loads

Live Loads (LL)			
Area	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
Public areas	100	100	100
Office lobbies	100	100	100
Office (first floor)	80	50	80
Office corridors above first floor	80	80	80
Offices above first floor	60	50	60
Partitions	20	≥ 15	20
Parking garage	40	40	40
Stairs	100	100	100
Roof	20	20	20
Dead Loads (DL)			
Material	**AES Design Load(PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
3 1/2" n.w.c. slab on 3"-20 gauge composite steel deck	Unknown	Section 3.1.1	*63
1 1/2" - 20 GA wide rib roof deck + MEP	Unknown		*10
3 5/8" masonry façade	Unknown		34
****Superimposed Dead Loads (SDL)**			
Area	**AES Design Load (PSF)**	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
Floor	Unknown	Not given	10
Roof	Unknown	Not given	10
Snow Load (SL)			
	AES Design Load (PSF)	**ASCE 7-05 Load (PSF)**	**Design Load (PSF)**
	24	22	24

*****The composite steel floor deck and roof deck manufacturers were not cited within the bid building specifications for Schenley Place. In order to perform spot checks of these various gravity members, it was necessary to assume a manufacturer and deck type. These assumptions are as follows:

3 ½” n.w.c. slab on 3”-20 gauge composite steel floor decking: Vulcraft, 3 VLI20

1 ½“ – 20 gauge wide rib roof decking: Vulcraft, 1.5B20

**The superimposed dead loads for both the floor and roof take into account the weight of finishes and MEP equipment.

Load Summary

Wind Load Summary

In the following wind analysis, wind loads were determined using ASCE 7-05, Chapter 6. This is the same code used by Atlantic Engineering Services (AES). Due to an overall building height of 104'-0", which exceeds the 60'-0" maximum building height outlined in Section 6.4, Method 1 – Simplified Procedure, wind loads were determined through the use of Section 6.5: Method 2 – Analytical Procedure. Figure 4 is a summary of the data used in calculating the design wind pressures. Design wind pressures are summarized in Figures A.4 and A.5 of Appendix A: Wind Loads. The unique wind pressures on the parapets of the fourth roof and main roof were accounted for as well in the wind analysis. These design wind pressures were used to determine both the individual story forces as well as the story shears. In a later analysis, it may be necessary to analyze the effects of the wind on cladding and components, as well as potential roof up-lift.

FIGURE 4: Data used to calculate wind loads

V(mph)	90		
K_d	0.85		
I	1.15		
Exposure	B		
K_{zt}	1.00		
Enclosure	Fully Enclosed		
n_1	1.44 (Rigid)		
G	0.85		
GC_{pn}	Windward	1.5	
	Leeward	-1.0	
Gc_{pi}	Enclosed Building	0.18	
		-0.18	
C_p	Windward	0.80	(All Values)
	Leeward	-0.47	(N/S Direction, L/B = 1.15)
		-0.50	(W/E Direction, L/B = 0.87)

At the time of design, the building was to be occupied by a single tenant who had intentions of using the building as a healthcare facility. Therefore, AES evaluated the wind loading using an Occupancy Category of III, which resulted in an Importance Factor, I of 1.15. To be consistent, an Importance Factor of 1.15 was maintained in the following wind analysis. This resulted in higher velocity pressures, and ultimately, higher design wind pressures. In the future, it may be valuable to reevaluate the wind loading with an Occupancy Category of II to optimize the design.

Both the north/south and west/east directions were evaluated. The resulting story forces and shears are summarized below (Figures 5 and 7). For detailed summaries of velocity pressure exposure coefficients, velocity pressures, design wind pressures, and an example story shear calculation, refer to Appendix A: Wind Load.

FIGURE 5: Story forces, shear, and moments due to wind (N/S direction)

Floor	Height above ground (ft)	Floor height (ft)	Force of total pressure, F (k)	Story shear, V (k)	Moment, M (ft-k)
Roof	104.00	5.00	54.66	54.66	5684.64
7	90.33	13.67	39.19	93.85	3540.0327
6	77.00	13.33	37.76	131.61	2907.52
5	63.67	13.33	36.41	168.02	2318.2247
4	50.33	13.33	47.57	215.59	2394.1981
3	37.00	13.33	33.69	249.28	1246.53
2	23.67	13.33	32.59	281.87	771.4053
1	9.00	14.67	25.56	307.43	230.04
Garage Level 1	0.00	9.00	9.67	317.1	0
			∑F= 317.1		**∑M= 19093**

The obvious increase in force at the fourth floor in the north/south direction is due to the unique wind pressure on the building's parapet. A windward force of 92.5 plf acts on the parapet located at and supported by the fourth floor. The main roof level sees the same unique parapet wind pressures. A windward force of 154 plf and a leeward force of 102.7 plf acts on the parapet located at the main roof level. Figure 6, a diagram of the windward and leeward pressures, better demonstrates the change in pressures as the height of the building increases, as well as the unique wind pressures acting on the building's parapets.

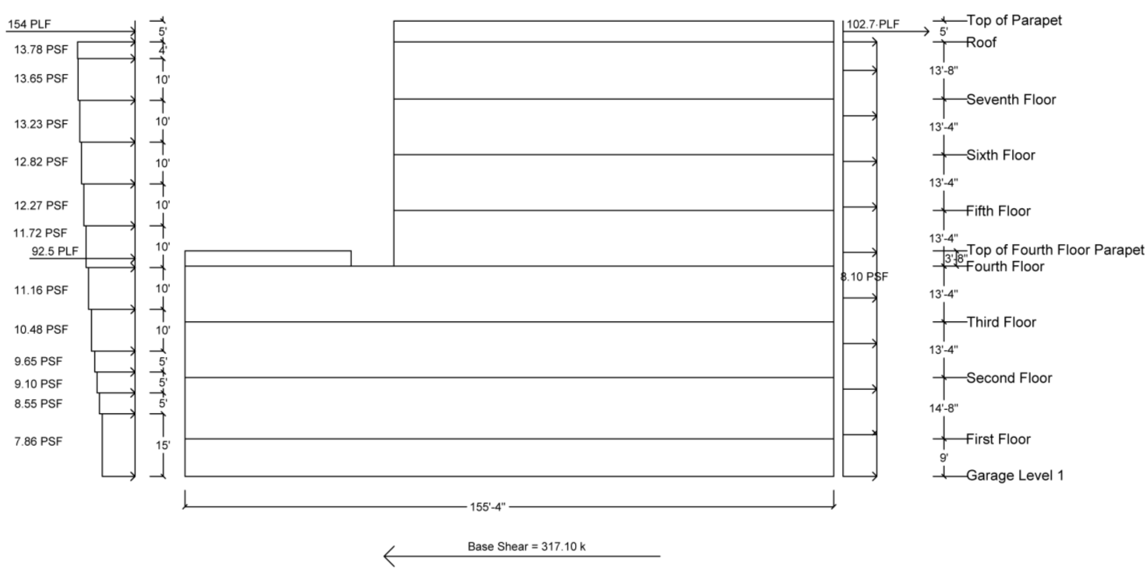


FIGURE 6: Overall windward and leeward pressures (N/S direction)

FIGURE 7: Story forces, shear, and moments due to wind (W/E direction)

Floor	Height above ground (ft)	Floor height (ft)	Force of total pressure, F (k)	Story shear, V (k)	Moment, M (ft-k)
Roof	104.00	5.00	43.12	43.12	4484.48
7	90.33	13.67	31.38	74.5	2834.5554
6	77.00	13.33	30.25	104.75	2329.25
5	63.67	13.33	29.17	133.92	1857.2539
4	50.33	13.33	42.49	176.41	2138.5217
3	37.00	13.33	39.92	216.33	1477.04
2	23.67	13.33	38.69	255.02	915.7923
1	9.00	14.67	30.42	285.44	273.78
Garage Level 1	0.00	9.00	11.51	296.95	0
			∑F= 296.95		**∑M= 16311**

The increase in forces at the fourth and main roof level in the west/east direction are due to the same unique wind pressures acting on the building's parapets as mentioned above. The decreases in story forces at the fifth level and above are a result of the step-back that occurs at the fourth level. The step-back reduces the length of wall that the wind is acting upon in the west/east direction. Consequently, the magnitude of these story forces are reduced. Figure 8, a diagram of the windward and leeward pressures in the west/east direction, better demonstrates the change in pressures as the height of the building increases, as well as the unique wind pressures acting on the building's parapets. To better understand the design wind pressures on the parapets, refer to Appendix A, Figure A.6.

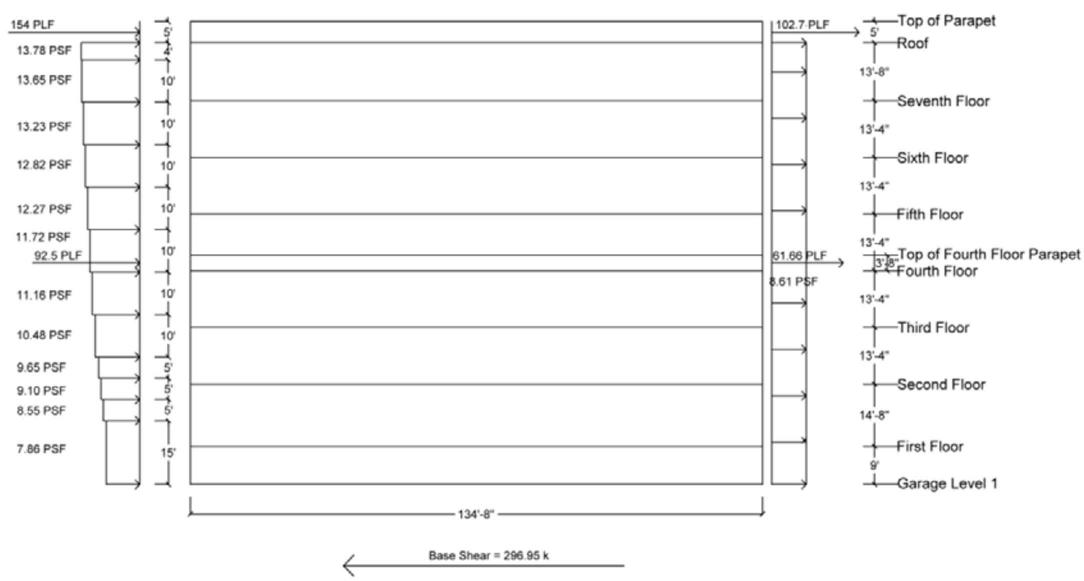


FIGURE 8: Overall windward and leeward pressure (W/E direction)

Seismic Load Summary.

The seismic analysis was performed in accordance with ASCE 7-05, Chapters 11, 12 and 22. As noted under the Lateral Force Resisting System Summary, the acting lateral system consists of two different types of bracing frames, both eccentrically and concentrically braced frames. As demonstrated by Figure9, a summary of the design factors used in the seismic analysis, several coefficients are dependent on the type of braced frame being analyzed. Therefore, it was necessary to calculate the story shears separately in both the north-south and west-east directions in order to determine the controlling base shear.

FIGURE 9: Seismic design factors

			ASCE 7-05 Reference
S_S	0.125		USGS
S_1	0.049		USGS
S_{MS}	0.200		Equation 11.4-1
S_{M1}	0.118		Equation 11.4-2
F_a	1.600		Table 11.4-1
F_v	2.400		Table 11.4-2
S_{DS}	0.133		Equation 11.4-3
S_{D1}	0.079		Equation 11.4-4
SDC	Table 11.6-1: A		The worst case between
	Table 11.6-2: **B**		Table 11.6-1 and 11.6-2
R	3		Table 12.2-1
Occupancy	III		
I	1.25		Table 11.5-1
h_n	104'		Table 12.8-2
T_L	12		Figure 22-15
	NORTH/SOUTH	**EAST/WEST**	
C_t	0.03	0.02	Table 12.8-2
x	0.75	0.75	
T_a=T	0.977	0.651	Equation 12.8-7
C_S	0.034	0.05	Equations 12.8-2 & 3
V	435	645	Equation 12.8-1
k	1.24	1.08	Section 12.8.3
W	12905		Refer to Appendix D

Figures 10 and 12 summarize the story weights, heights, story shears and moments of the north-south and west-east directions. Figures 11 and 13 better illustrate the lateral forces, story shears, and resulting base shears acting in the north-south and west-east directions.

FIGURE 10: Seismic forces, story shears, and moments in the N/S direction

Level	Story, weight w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, Fx	C_{vx}	Story shear, V_x (kips)	Moment, M_x (ft-k)
Roof	285	104	90358	27.4	0.063	27.4	2853.8
7	1330	81.32	310811	94.4	0.217	121.8	7675.6
6	1336	77	291780	88.6	0.204	210.4	6822.8
5	1336	63.66	230463	70.0	0.161	280.4	4455.4
4	1458	50.33	187942	57.1	0.131	337.5	2872.6
3	1960	37	172515	52.4	0.120	389.9	1938.4
2	1960	23.67	99143	30.1	0.069	420.0	712.7
1	3240	9	49409	15.0	0.034	435.0	135.0

$\sum F_x$ = 435 k **$\sum M_x$ = 27466 ft-k**

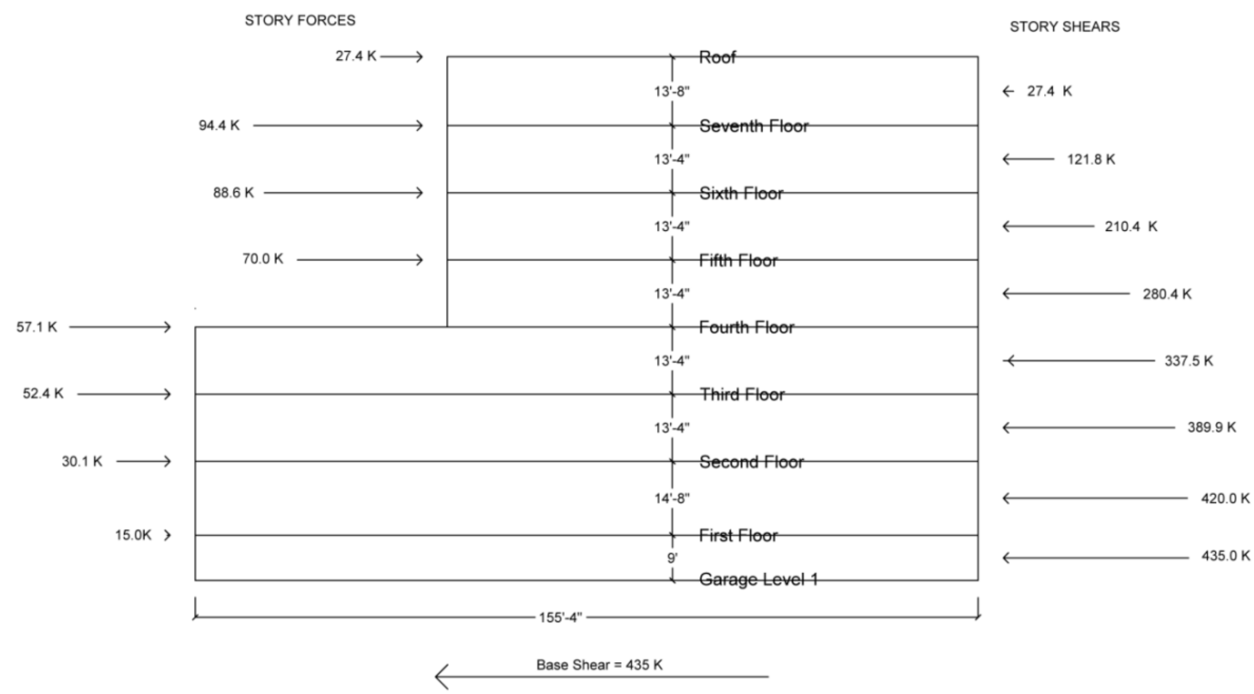


FIGURE 11: Story forces and shears due to seismic loading (N/S direction)

FIGURE 12: Seismic forces, story shears, and moments in the E/W direction

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x	C_{vx}	Story shear, V_x (kips)	Moment, M_x (ft-k)
Roof	285	104	42977	36.8	0.057	36.8	3830.3
7	1330	81.32	153768	131.8	0.204	168.6	10715.7
6	1336	77	145618	124.8	0.193	293.4	9608.7
5	1336	63.66	118572	101.6	0.158	395.0	6468.6
4	1458	50.33	100399	86.0	0.133	481.0	4330.3
3	1960	37	96809	83.0	0.129	564.0	3069.5
2	1960	23.67	59757	51.2	0.079	615.2	1212.1
1	3240	9	34764	29.8	0.046	645.0	268.1

$\sum F_x$ = 645 k **$\sum M_x$ = 39503 ft-k**

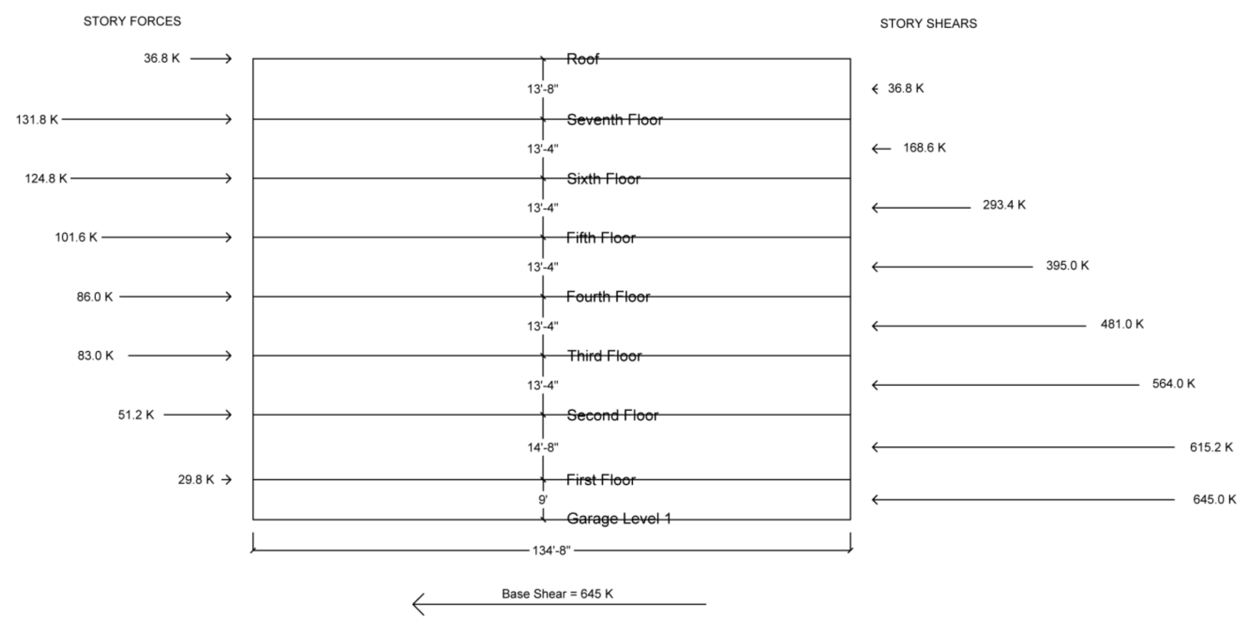


FIGURE 13: Story forces and shears due to seismic loading (W/E direction)

The analysis uncovered a design base shear of 435 kips in the north-south direction and a controlling design shear of 624 kips in the west-east direction. Atlantic Engineering Services (AES) reported a base shear of 365 kips.

The difference in the base shears reported by AES and the results of the contained analysis could be due to several factors. It is possible that AES may have had the ability to perform a more accurate analysis of the building, in which they may have been able to determine a more precise effective fundamental period, T, rather than use the approximate fundamental period, T_a, which was used this seismic analysis (Figure 9, $T=T_a$). In their favor, this could have resulted in a more flexible building system which would have ultimately resulted in a lower design shear value. Differences in effective seismic weight, w, could have also led to the difference between the design base shears reported. It is impossible to predict the method and values AES used in order to determine an effective seismic weight. The assumed dead loads that were used in this analysis can be found in Appendix B: Seismic Loads. Due to the topography of the site, the first floor is both on grade at the north elevation, and 9'-0" above grade at the south elevation. In order to simplify the calculations, the first floor weight was calculated as if it was entirely on grade. Only the columns whose base were at the first level, the brick façade, and the slab (which is normally left out of the weight if on grade) were included in the first floor story weight. Any elements below the first floor, which were primarily sub-grade, were ignored.

Upon comparison of the wind (317.1 kips, 19093 ft-kips) and seismic (645 kips 39503 ft-kips) design base shears and over-turning moments, it is evident that the seismic loading controls.

Snow Load Summary

Snow loads were determined in accordance with ASCE 7-05, Chapter 7. Figure 14 is a summary of the design criteria and resulting design flat roof snow load. The worst-case snow drift load on the roof level parapet was also determined. The resulting drift surcharge load, p_d is also summarized in Figure 14, and can conservatively be applied to the parapet on the fourth level as well.

FIGURE 14: Summary of snow load data

		ASCE 7-05 Reference
p_g	25	Figure 7-1
C_e	1.0	Table 7-2
	Terrain Category B	
	Partially Exposed	
C_t	1.0	Table 7-3
I	1.1	Table 7-4
p_f	22 psf	Equation 7-1
h_d	2.58 ft	Section 7.7.1
h_c	3.72 ft	
w	10.32 ft	
p_d	44.51 psf	

At the time of design, the building was to be occupied by a single tenant who had intentions of using the building as a healthcare facility. Therefore, AES evaluated the snow loading using an Occupancy Category of III, which resulted in an Importance Factor, I of 1.1. To be consistent, an Importance Factor of 1.1 was maintained in the following snow load analysis. This resulted in a higher flat roof snow load. In the future, it may be valuable to reevaluate the snow loading with an Occupancy Category of II to optimize the design.

FIGURE 15: Configuration of snow drifts on parapets

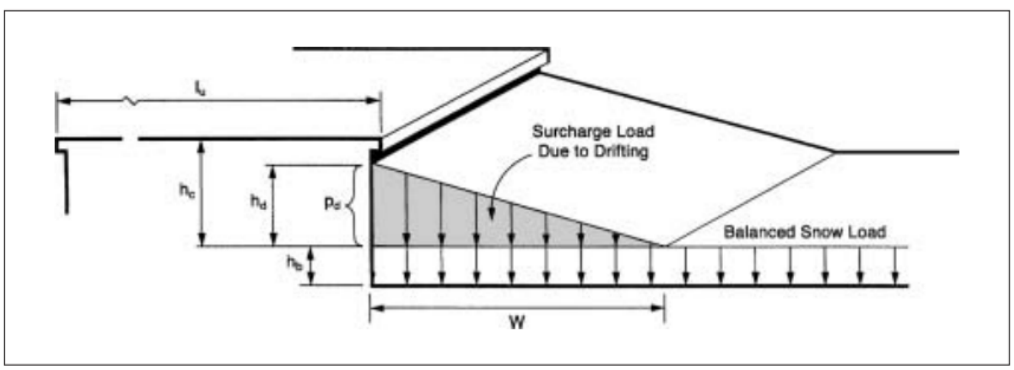


Figure 15, as provided in ASCE 7-05, better illustrates the configuration of snow drifts and defines the coefficients as provided in Figure 14.

In conclusion, the analysis performed indicates that a flat roof snow load of 22 psf should be designed for at both the fourth and main roof levels. Because AES used of a base ground snow load, p_g of 30 psf, they reported a flat roof snow load of 24 psf. Though AES's number is more conservative, the 22 psf flat roof snow load, as determined in this analysis, may be designed for. This number is based upon the lesser ground snow load of 25 psf as required by code (ASCE 7-05, Figure 7-1) An additional superimposed triangular surcharge snow drift load of 44.51 psf acts on the fourth level and roof level parapets. For more detailed information on the analysis of the flat roof and drift snow loads, refer to Appendix C: Snow Load.

Overview

In summary, the analysis conducted for wind, seismic, and snow was in accordance to ASCE 7-05. After comparing the controlling design base shears of both wind and seismic loadings, the results of the analysis performed suggest that the overall controlling base shear of 645 kips is due to seismic loads in the west-east direction. Though the design professionals of Atlantic Engineering Services computed a design base shear of 365 kips, various assumptions made while performing the analysis within this report could be the source of any discrepancies. In addition, a flat roof snow load of 22 psf should be accounted for in the gravity load design. AES computed a 24 psf flat roof snow load, based upon a 30 psf base ground snow load.

Appendices

Appendix A: Wind Load

FIGURE A.1: Data used to calculate wind loads

				ASCE 7-05 Reference
V(mph)	90			Figure 6-1
K_d	0.85			Table 6-4
I	1.15			Table 6-1
Exposure	B			Section 6.5.6
K_{zt}	1.00			Section 6.5.7.1
Enclosure	Fully Enclosed			Section 6.5.9
n_1	1.44 (Rigid)			*Equation C6-19
G	0.85			Section 6.5.8.1
GC_{pn}	Windward	1.5		Section 6.5.12.2.4
	Leeward	-1.0		
Gc_{pi}	Enclosed Building	0.18		Figure 6-5
		-0.18		
C_p	Windward	0.80	(All Values)	Figure 6-6
	Leeward	-0.47	(N/S Direction, L/B = 1.15)	
		-0.50	(W/E Direction, L/B = 0.87)	

***Equation C6-19:**

$f_{n1} = \frac{150}{H}$ where H = building height (ft)

$f_{n1} = \frac{150}{104} = 1.44 \geq 1$ Hz → The building is rigid

Note: *This expression over-estimates the frequency common in U.S. construction for smaller buildings less than 400 feet in height. This will result in conservative wind loadings.*

FIGURE A.2: Velocity pressure exposure coefficients K_z, K_h, and K_p

Height above ground level, z (ft)	K_z	K_h	K_p
15	0.57		
20	0.62		
25	0.66		
30	0.7		
40	0.76		
50	0.81		
54			0.83
60	0.85		
70	0.89		
80	0.93		
90	0.96		
100	0.99		
104		1.00	
109			1.01

ASCE 7-05 Reference: Table 6-3

FIGURE A.3: Velocity pressure, q_z, q_h, and q_p

Height above ground level, z (ft)	q_z	q_h	q_p
15	11.55		
20	12.57		
25	13.38		
30	14.19		
40	15.40		
50	16.42		
54			16.80
60	17.23		
70	18.04		
80	18.85		
90	19.46		
100	20.07		
104		20.27	
109			20.53

ASCE 7-05 Reference: Equation 6-15

FIGURE A.4: Design wind pressures in the N/S direction

Location	Height above ground level, z	q (psf)	External Pressure qGC_p (psf)	Internal Pressure $q_h(GC_{pi})$ (psf)	Net Pressure, p (psf)	
					(+) GC_{pi}	(-) GC_{pi}
Windward	104	20.27	13.78	3.65	10.13	17.43
	100	20.07	13.65	3.65	10.00	17.29
	90	19.46	13.23	3.65	9.58	16.88
	80	18.85	12.82	3.65	9.17	16.47
	70	18.04	12.27	3.65	8.62	15.92
	60	17.23	11.72	3.65	8.07	15.36
	50	16.42	11.16	3.65	7.52	14.81
	40	15.40	10.48	3.65	6.83	14.12
	30	14.19	9.65	3.65	6.00	13.30
	25	13.38	9.10	3.65	5.45	12.75
	20	12.57	8.55	3.65	4.90	12.19
	15	11.55	7.86	3.65	4.21	11.50
Leeward	All	20.27	-8.10	3.65	-11.75	-4.45

FIGURE A.5: Design wind pressures in W/E direction

Location	Height above ground level, z	q (psf)	External Pressure qGC_p (psf)	Internal Pressure $q_h(GC_{pi})$ (psf)	Net Pressure, p (psf)	
					(+) GC_{pi}	(-) GC_{pi}
Windward	104	20.27	13.78	3.65	10.13	17.43
	100	20.07	13.65	3.65	10.00	17.29
	90	19.46	13.23	3.65	9.58	16.88
	80	18.85	12.82	3.65	9.17	16.47
	70	18.04	12.27	3.65	8.62	15.92
	60	17.23	11.72	3.65	8.07	15.36
	50	16.42	11.16	3.65	7.52	14.81
	40	15.40	10.48	3.65	6.83	14.12
	30	14.19	9.65	3.65	6.00	13.30
	25	13.38	9.10	3.65	5.45	12.75
	20	12.57	8.55	3.65	4.90	12.19
	15	11.55	7.86	3.65	4.21	11.50
Leeward	All	20.27	-8.61	3.65	-12.26	-4.97

FIGURE A.6: Design wind pressure on the parapets

Location	Height above ground level, z	q_p (psf)	Net Pressure, p_p qGC_{pn} (psf)	Height of parapet (ft)	Force on parapet (plf)
Windward	109	20.53	30.80	5.00	154.0
	54	16.80	25.20	3.67	92.5
Leeward	109	20.53	-20.53	5.00	-102.7
	54	16.8	-16.80	3.67	-61.66

FIGURE A.7: Story forces, shear, and moments due to wind (N/S direction)

Floor	Height above ground (ft)	Floor height (ft)	Force of total pressure, F (k)	Story shear, V (k)	Moment, M (ft-k)
Roof	104.00	5.00	54.66	54.66	5684.64
7	90.33	13.67	39.19	93.85	3540.0327
6	77.00	13.33	37.76	131.61	2907.52
5	63.67	13.33	36.41	168.02	2318.2247
4	50.33	13.33	47.57	215.59	2394.1981
3	37.00	13.33	33.69	249.28	1246.53
2	23.67	13.33	32.59	281.87	771.4053
1	9.00	14.67	25.56	307.43	230.04
Garage Level 1	0.00	9.00	9.67	317.1	0
			∑F= 317.1		**∑M= 19093**

FIGURE A.8: Overall windward and leeward pressures (N/S direction)

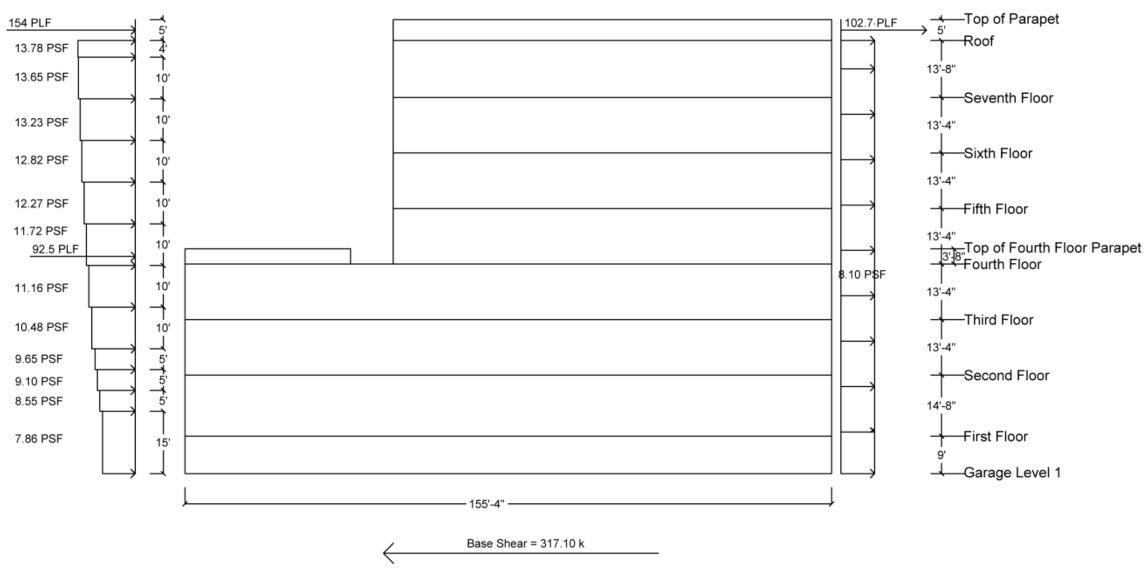
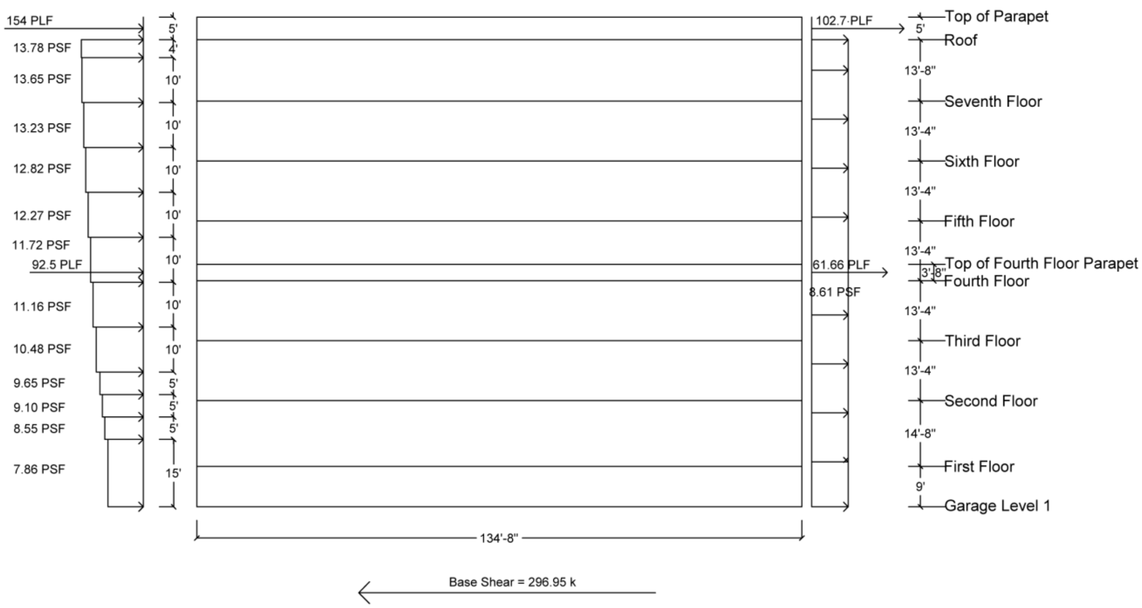


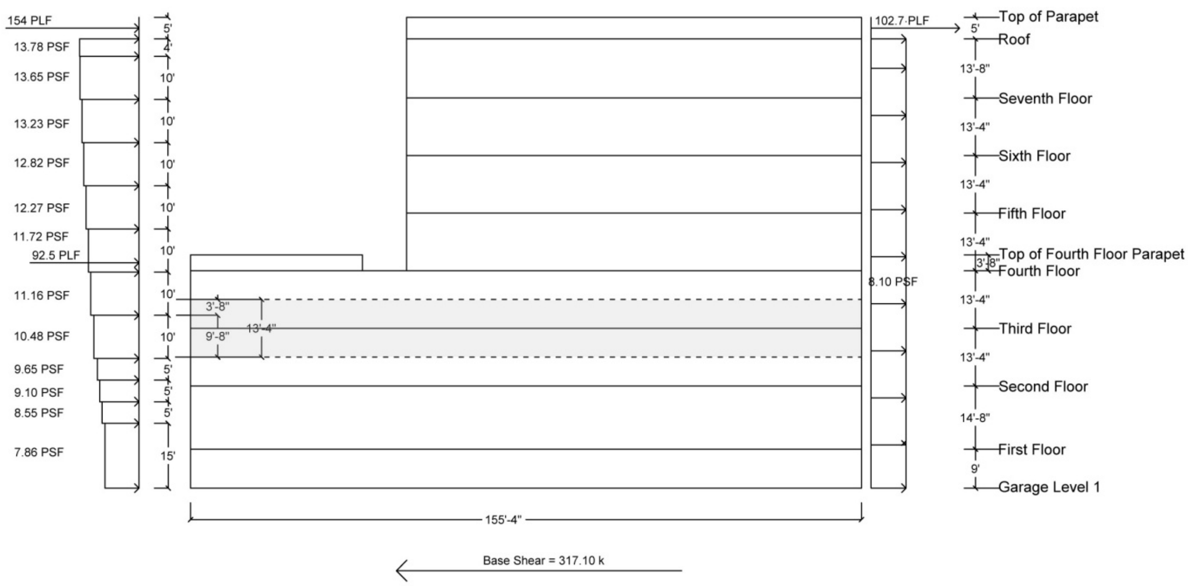
FIGURE A.9: Story forces, shear, and moments due to wind (W/E direction)

Floor	Height above ground (ft)	Floor height (ft)	Force of total pressure, F (k)	Story shear, V (k)	Moment, M (ft-k)
Roof	104.00	5.00	43.12	43.12	4484.48
7	90.33	13.67	31.38	74.5	2834.5554
6	77.00	13.33	30.25	104.75	2329.25
5	63.67	13.33	29.17	133.92	1857.2539
4	50.33	13.33	42.49	176.41	2138.5217
3	37.00	13.33	39.92	216.33	1477.04
2	23.67	13.33	38.69	255.02	915.7923
1	9.00	14.67	30.42	285.44	273.78
Garage Level 1	0.00	9.00	11.51	296.95	0
			∑F= 296.95		**∑M= 16311**

FIGURE A.10: Overall windward and leeward pressures (W/E direction)



Sample 3rd floor story force calculation (N/S Direction)



Windward 3rd floor story force:

$p_w = [(9.67')(10.48psf) + (3.67')(11.16psf)]134.67' = 19160\ lbs = 19.16\ k$

Leeward 3rd floor story force:

$p_l = [(13.33')(8.10psf)]134.67' = 14540\ lbs = 14.54\ k$

Total 3rd floor story force:

$P = p_w + p_l$

$P = 19.16\ k + 14.54\ k$

$\boldsymbol{P = 33.70\ k}$

Appendix B: Seismic Load

The following (Figure B.1) is a summary of the design data used when calculating the seismic loading, following the equivalent lateral force analysis procedure of the ASCE 7-05.

FIGURE B.1: Seismic design factors

			ASCE 7-05 Reference
S_S	0.125		USGS
S_1	0.049		USGS
S_{MS}	0.200		Equation 11.4-1
S_{M1}	0.118		Equation 11.4-2
F_a	1.600		Table 11.4-1
F_v	2.400		Table 11.4-2
S_{DS}	0.133		Equation 11.4-3
S_{D1}	0.079		Equation 11.4-4
SDC	Table 11.6-1: A		The worst case between
	Table 11.6-2: **B**		Table 11.6-1 and 11.6-2
R	3		Table 12.2-1
Occupancy	III		
I	1.25		Table 11.5-1
h_n	104'		Table 12.8-2
T_L	12		Figure 22-15
	NORTH/SOUTH	**EAST/WEST**	
C_t	0.03	0.02	Table 12.8-2
x	0.75	0.75	
T_a=T	0.977	0.651	Equation 12.8-7
C_S	0.034	0.05	Equations 12.8-2 & 3
V	435	645	Equation 12.8-1
k	1.24	1.08	Section 12.8.3
W	12905		Refer to Appendix D

Due to the size and detail of the charts used to calculate the effective seismic building weight, these have been omitted and a summary of assumed dead loads and total floor weights have been provided. Detailed effective seismic weight calculations can be presented at one's request.

FIGURE B.2: Summary of loads used to calculate effective seismic weight, W

Self-weights	
Beams	Sizes varied floor-to-floor
Columns	Sizes varied floor-to-floor
Composite decking	63 PSF 3 1/2" concrete & 3" 20-g steel deck Applied to all levels, except the 1st level
1st floor slab	138 PSF 11" N.W.C. slab Applied to 1st level
Masonry curtain wall	34 PSF 3 5/8" typical brick curtain wall Applied to all levels supporting façade
Superimposed dead loads	
Roof	10 PSF Applied to low and high roofs
Floor	10 PSF Takes into account partitions, finishes, and MEP Applied to all floors

FIGURE B.3: Summary of story weights

Level	Weight (kips)
Roof	285
7	1330
6	1336
5	1336
4	1458
3	1960
2	1960
1	3240
W=	12905

Example calculations of seismic forces, story shears, and moments in N/S direction on the 7th floor:

FIGURE B.4: Seismic forces, story shears, and moments in the N-S direction

Level	Story weight w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, Fx	C_{vx}	Story shear, V_x (kips)	Moment, M_x (ft-k)
Roof	285	104	90358	27.4	0.063	27.4	2853.8
7	1330	81.32	310811	94.4	0.217	121.8	7675.6
6	1336	77	291780	88.6	0.204	210.4	6822.8
5	1336	63.66	230463	70.0	0.161	280.4	4455.4
4	1458	50.33	187942	57.1	0.131	337.5	2872.6
3	1960	37	172515	52.4	0.120	389.9	1938.4
2	1960	23.67	99143	30.1	0.069	420.0	712.7
1	3240	9	49409	15.0	0.034	435.0	135.0
	12905	$w_i h_i^k$=	1432420	$\sum F_x$ = 435 k			$\sum M_x$ = 27466 ft-k

Story weight, w_x:

As summarized in Figure B.3

$$\boldsymbol{w_7 = 1330\ k}$$

Height, h_x:

As determined using construction documents)

$$\boldsymbol{h_7 = 81.32\ ft}$$

Exponent related to structure, k:

$$\boldsymbol{k = 0.75 + 0.5T}$$

$$k = 0.75 + 0.5(0.977) = \mathbf{1.24}$$

T can be found in Figure B.1

$w_x h_x^k$:

$$w_7 h_7^k = (1330)(81.32)^{1.24} = \mathbf{310811}$$

Lateral force, F_x:

$$\boldsymbol{F_7 = \frac{w_7 h_7^k}{\sum_{i=1}^{n} w_i h_i^k} V}$$

$$F_7 = \frac{310811}{1432420}(435) = \mathbf{94.4\ k}$$

$w_i h_i^k$ can be found in Figure B.4 and V in Figure B.1

Vertical distribution factor, C_{vx}:

$$\boldsymbol{C_{v7} = \frac{w_7 h_7^k}{\sum_{i=1}^{n} w_i h_i^k}}$$

$$C_{v7} = \frac{310811}{1432420} = \mathbf{0.217}$$

Story shear, V_x:

$$\boldsymbol{V_7 = \sum F_x = F_{roof} F_7}$$

$$V_7 = 27.4\ k + 94.4\ k = \mathbf{121.8}\ \boldsymbol{k}$$

Moment, Mx:

$$\boldsymbol{M_7 = h_7 F_7}$$

$$M_7 = (81.32')(94.4\ k) = \mathbf{7676}\ \boldsymbol{ft - k}$$

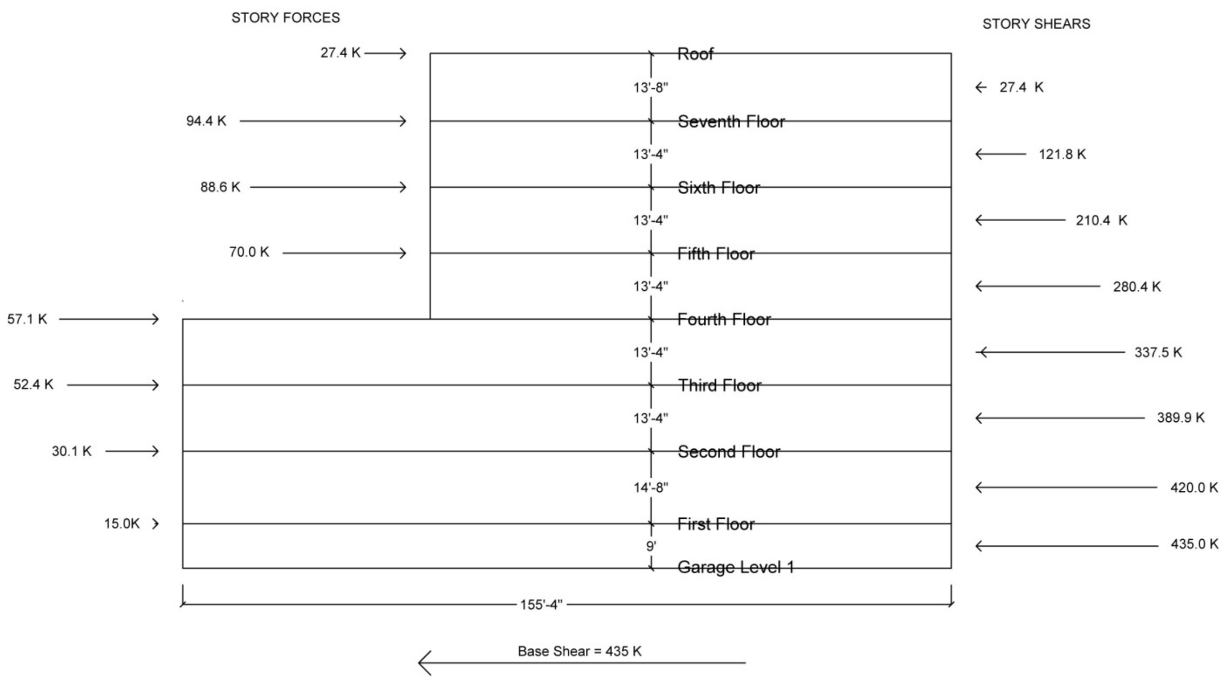


FIGURE B.5: Story forces and shears due to seismic loading (N/S direction)

FIGURE B.6: Seismic forces, story shears, and moments in the E/W direction

Level	Story weight, w_x (kips)	Height, h_x (ft)	$w_x h_x^k$	Lateral force, F_x	C_{vx}	Story shear, V_x (kips)	Moment, M_x (ft-k)
Roof	285	104	42977	36.8	0.057	36.8	3830.3
7	1330	81.32	153768	131.8	0.204	168.6	10715.7
6	1336	77	145618	124.8	0.193	293.4	9608.7
5	1336	63.66	118572	101.6	0.158	395.0	6468.6
4	1458	50.33	100399	86.0	0.133	481.0	4330.3
3	1960	37	96809	83.0	0.129	564.0	3069.5
2	1960	23.67	59757	51.2	0.079	615.2	1212.1
1	3240	9	34764	29.8	0.046	645.0	268.1

$\sum F_x$ = 645 k **$\sum M_x$ = 39503 ft-k**

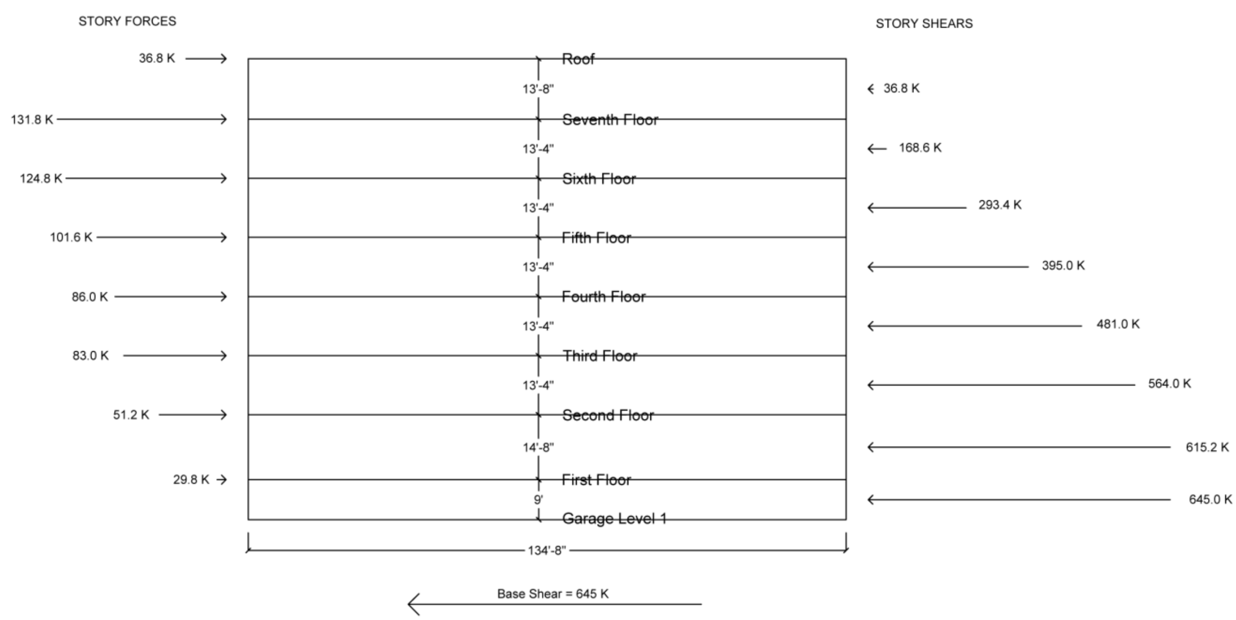


FIGURE B.7: Story forces and shears due to seismic loading (W/E direction)

Appendix C: Snow Load

Flat roof snow loads

FIGURE C.1: Data used to calculate flat roof snow loads, p_f

		ASCE 7-05 Reference
p_g	25	Figure 7-1
C_e	1.0	Table 7-2
	Terrain Category B	
	Partially Exposed	
C_t	1.0	Table 7-3
I	1.1	Table 7-4
p_f	22 psf	Equation 7-1

Flat roof snow loads, p_f (ASCE 7 -05 Reference: Section 7.3, Equation 7-1)

$$p_f = 0.7C_eC_tIp_g \geq 20psf(I)$$
$$p_f = (0.7)(1.0)(1.0)(1.1)(25\,psf) \geq 20psf(1.1)$$
$$p_f = 19.25\,psf \geq 22\,psf$$
$$\boldsymbol{p_f = 22\,psf}$$

Drift loads on roof parapet walls

FIGURE C.2: Data used to determine need for drift loads

		ASCE 7-05 Reference
p_g	25	Figure 7-1
C_e	1.0	Table 7-2
	Terrain Category B	
	Partially Exposed	
C_t	1.0	Table 7-3
I	1.1	Table 7-4
p_f	22 psf	Equation 7-1
C_s	1.0	Figure 7-2
p_s	22 psf	Equation 7-2
γ	17.25 pcf	Equation 7-3
h_b	1.28 ft	Section 7.1
h_c	3.72 ft	Section 7.1

Drift loads on parapet walls to be calculated as outline in Section 7.7.1.
Section 7.7.1 states: "If h_c/h_b is less than 0.2, drift loads are not required to be applied.

Sloped roof snow loads, p_s (ASCE 7 -05 Reference: Section 7.4, Equation 7-2)

$p_s = C_s p_f$

$p_s = (1.0)(22\ psf)$

$\boldsymbol{p_s = 22\ psf}$

Snow density, γ (ASCE 7 -05 Reference: Section 7.7.1, Equation 7-3)

$\gamma = 0.13p_g + 14\ < 30\ pcf$

$\gamma = 0.13(25\ psf) + 14\ < 30\ pcf$

$\boldsymbol{\gamma = 17.25\ pcf}\ < 30\ pcf$

Height of balanced snow load, h_b (ASCE 7 -05 Reference: Section 7.1)

$h_b = {}^{p_s}/_{\gamma}$

$h_b = {}^{22\ psf}/_{17.25\ pcf}$

$\boldsymbol{h_b = 1.28'}$

Clear height from top of h_b to top of parapet, h_c (ASCE 7 -05 Reference: Section 7.1)

$h_c = 5.00 - h_b$

$h_c = 5.00 - 1.28'$

$\boldsymbol{h_c = 3.72'}$

Drift load check:

${}^{h_c}/_{h_b} < 0.2$

${}^{3.72'}/_{1.28'} = 2.91 > 0.2$

Drift loads must be calculated

Figure C.3: Snow Drift Load on Roof Parapet

		ASCE 7-05 Reference
h_d	2.58 ft	Section 7.7.1
h_c	3.72 ft	
w	10.32 ft	
p_d	44.51 psf	

Height of drifts, h_d:

$$h_d = \left(0.43\sqrt[3]{l_u}\sqrt[4]{p_g + 10} - 1.5\right)0.75$$

Where, l_u = length of the roof upwind of parapet = 105'-4"

$$h_d = \left(0.43\sqrt[3]{105.33'}\sqrt[4]{25 + 10} - 1.5\right)0.75$$

$$h_d = 2.58'$$

$$\boldsymbol{h_d = 2.58' < h_c = 3.72'}:$$

$$w = 4h_d$$

$$w = 4(2.58')$$

$$\boldsymbol{w = 10.32'}$$

$$\boldsymbol{h_d = h_d = 2.58'}$$

$$p_d = h_d\gamma$$

$$p_d = (2.58')(17.25\ pcf)$$

$$\boldsymbol{p_d = 44.51\ psf}$$

Appendix D: Spot Checks

SPOT CHECK 1: Typical composite floor slab

Floor system: 3 ½" normal weight concrete slab on 3"-20 gauge composite steel deck
(Assumed to be Vulcraft 3VLI20, refer to "Summary of Design Loads", page 8)
Largest floor span: 9'-0"
Superimposed live load:

Floor live load = *80 psf + 20 psf = 100 psf

Superimposed live load $= 1.6LL$

Superimposed live load = $1.6(100\ psf) = \mathbf{160\ psf}$

*Because the office spaces are designed as unfinished spaces, you're unable to predict where the office corridors will be located when the spaces are occupied. Therefore, a live load of 80 psf was used for "office corridors above the first floor." *(Refer to Summary of Design Loads, page 8, for the chart of design loads)*

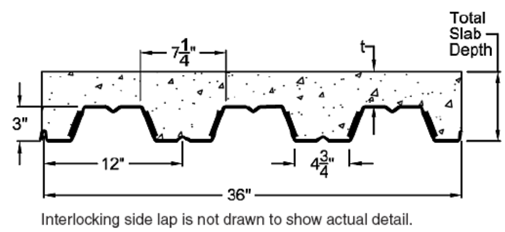


FIGURE D.1: Cross-section of composite floor deck

TOTAL SLAB DEPTH	DECK TYPE	SDI Max, Unshored Clear Span			Superimposed Live Load, PSF Clear Span (ft.-in.)														
		1 SPAN	2 SPAN	3 SPAN	7'-0	7'-6	8'-0	8'-6	9'-0	9'-6	10'-0	10'-6	11'-0	11'-6	12'-0	12'-6	13'-0	13'-6	14'-0
	3VLI22	8'-0	8'-3	9'-4	307	277	251	190	171	155	141	129	118	108	99	91	84	78	72
6.50	3VLI20	9'-3	11'-5	11'-9	343	307	278	253	232	174	158	144	132	121	111	103	95	87	81
(t=3.50)	3VLI19	10'-4	12'-8	13'-1	377	337	304	276	252	232	214	159	146	134	123	113	104	96	89
63 PSF	3VLI18	11'-4	13'-9	13'-10	400	371	338	309	285	264	246	229	215	162	151	140	131	122	115
	3VLI16	11'-7	13'-10	14'-3	400	400	378	345	317	293	272	253	237	222	169	157	146	136	128

FIGURE D.2: Chart of allowable superimposed live loads on Vulcraft 3VLI composite floor deck

Images courtesy of Vulcraft Steel Roof & Floor Deck 2008 Catalogue

As read from the chart, the uniform superimposed live load capacity for 3VLI20 at a 9'-0" span = 232 psf

$$232\text{ psf} > 160\text{ psf} \qquad \therefore \boldsymbol{OK}$$

The 22 gage composite steel decking would have worked according to the chart and the superimposed live load calculated, based on the design loads assumed (171 psf > 160 psf). However, these numbers are relatively close, and there may be other spans that carry a larger live load, which may exceed the 171 psf limit. Therefore, this is probably why the 20 gage composite steel decking was chosen.

SPOT CHECK 2: Gravity column (Column A.2) at the second floor

Loads carried by Column A.2 on the second floor

The loads used can be referenced on page 10 under "Summary of Design Loads"

Design Notes:

Column A.2 Supports the fourth level roof and the third floor
Column A.2: W10X45
Tributary Area = 182 (L = 20.415') x (W = 8.915')
Height = 26.66'

Level	DL (PSF)	*DL (K)	DL (PLF)	*DL (K)	LL (PSF)	LL(K)	L_rL (PSF)	L_rL (K)	SL (PSF)	SL (K)
Roof: Fourth Floor	20	3.640	48	0.428	---	---	20	3.640	24	4.368
Third Floor	73	13.286	42	0.374	80	14.560	---	---	---	---
∑ (K)	16.926		0.802		14.560		3.640		4.368	

*DL only includes beam self-weights, floor and roof systems, and superimposed loads

Item	DL (PSF)	DL (K)	DL (PLF)	DL (K)
Load due to brick façade	34	18.505	---	---
Column Self Weight	---	---	45	1.200
∑ (K)	18.505		1.200	

	DL	LL	L_rL	SL
TOTAL LOADS (K)	**37.433**	**14.560**	**3.640**	**4.368**

Load Combinations (LRFD):	Pu (K)
1.4D	52.41
$1.2D + 1.6L + 0.5L_r$	**70.04**
$1.2D + 1.6L_r + 0.5L$	58.02
$1.2D + 0.5L + 0.5L_r$	54.02
$1.2D + 0.5L + 0.2S$	53.07

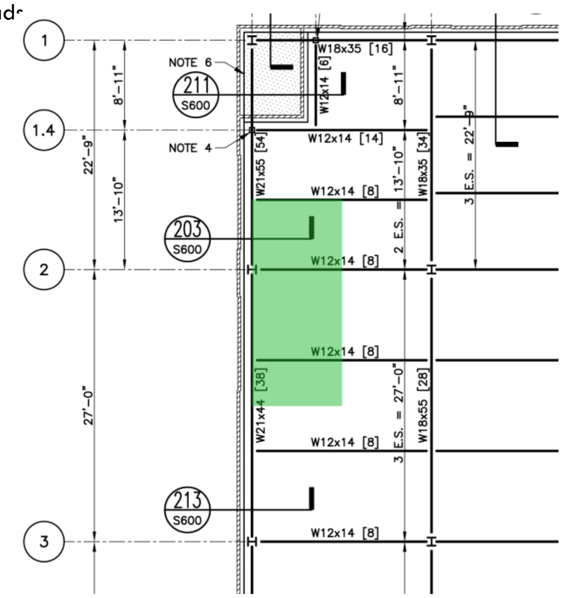


FIGURE D.3: Tributary area of Column A.2

Table 4-1: Available Strength in Axial Compression, kips

(KL)	ϕP_n
12	410
13.33	**401**
13	384
r_x/r_y	2.15

Fy =50 ksi,
K=1,
L=13.33'

$\phi P_n = 401 > 70.04 \text{ K} = P_u$

Because A.2 is an exterior column, the large remaining capacity is likely due to the lateral loads column A.2 must also carry, but not accounted for within this spot check

SPOT CHECK 3: Composite beam

Design notes:

Floor system: 3 ½" normal weight concrete slab on 3"-20 gauge composite steel deck *(Assumed to be Vulcraft 3VLI20, refer to "Summary of Design Loads", page 8)*
Deck is perpendicular to the beam

Concrete slab: f'c = 4000 psi
Slab thickness: 3½"

Steel beam: W18x35 [16]
- d = 17.7"
- t_w = 0.30"
- b_f = 6.0"
- t_f = 0.425"

A992: F_y = 50 ksi
F_u = 65 ksi
Tributary width: 9'-0"
Span = 30'-0"

Steel bolts: ASTM 325
¾" diameter
$Q_n = {}^{17.2\ k}/_{stud}$

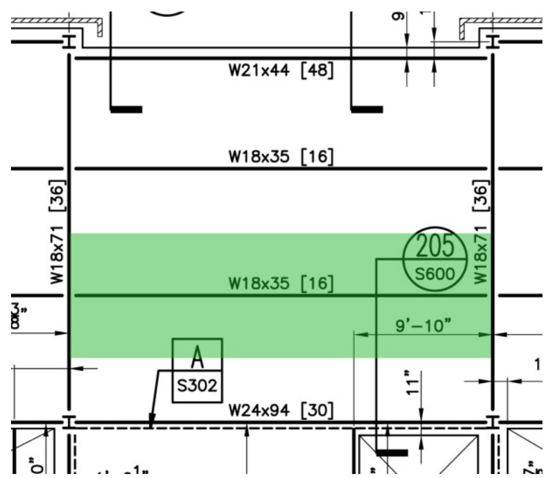


FIGURE D.4: Tributary area of W18x35

Design assumptions: (1) weak stud per rib

$$\boldsymbol{b_{eff} \le 2(b')}$$
$$b' \le \frac{span}{8} \ or \ \frac{1}{2} \ the\ distance\ to\ adjacent\ beam$$
$$b' \le \frac{(30')(12)}{8} \ or \ \frac{1}{2}(9')(12)$$
$$b' \le 45" \ or \ 54"$$
$$\boldsymbol{b_{eff} = (2)45" = 90"}$$

$$\boldsymbol{V'_c = 0.85 f'_c b_{eff} t}$$
$$V'_c = 0.85(4ksi)(90)(3) = \boldsymbol{918\ k}$$

$$\boldsymbol{V'_s = A_s F_y}$$
$$V'_s = (10.2in^2(50ksi) = \boldsymbol{515\ k}$$

$$\boldsymbol{V'_q = \sum Q_n}$$
$$V'_q = 17.2\ k\ (8\ studs) = \boldsymbol{137.6\ k}$$

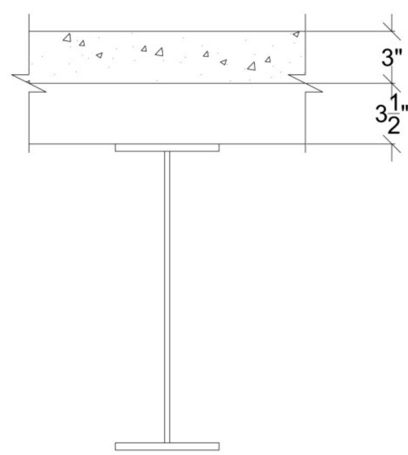


FIGURE D.5: Cross-section of composite beam and slab

∴ V'_q controls → Partially composite

Effective depth of concrete, a

$$\boldsymbol{a = \frac{\sum Q_n}{0.85f'_c b_{eff}}}$$

$$\text{a}=\frac{137.6\text{ k}}{0.85(4\text{ ksi})(90")} = \mathbf{0.455"} < 3.0"$$

Area of steel in compression, $A_{s\text{-}c}$:

$$\boldsymbol{A_{s-c} = \frac{T_s - C_c}{2F_y}}$$

$$A_{s-c} = \frac{515\ k - 137.6\ k}{2(50\ ksi)} = \mathbf{3.774\ in^2}$$

Location of PNA, x:

$$\boldsymbol{x = \frac{A_{s-c} - b_f t_f}{t_w} + t_f}$$

$$x = \frac{3.774in^2 - (6.0")(0.425")}{(0.30")} + 0.425" = \mathbf{4.505"}$$

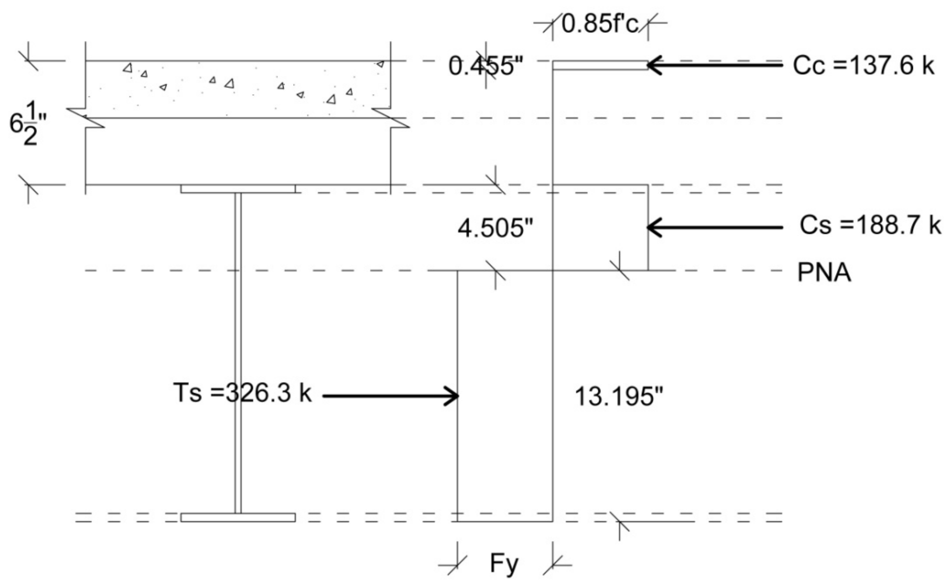


FIGURE 5.6: Diagram of compressive and tensile forces

Nomial moment, Mn:

$$M_n = (-188.7\ k)\left(\frac{4.505"}{2}\right) + (137.6\ k)(6.5" - \frac{0.455"}{2}) + (326.3\ k)(4.505" + \frac{13.195"}{2})$$

$$\boldsymbol{M_n = 4061\ ft - k}$$

$$\boldsymbol{\varphi M_n = (0.9)(4061\ ft - k) = 3655\ ft - k}$$

Design loads:

$$\boldsymbol{DL = beam\ self - weight + slab\ self - weight + superimposed\ DL}$$

$$DL = 0.035\ klf + 0.63\ ksf(9') + 0.01\ ksf(9') = \mathbf{0.692}\ \boldsymbol{{}^{k}/_{ft}}$$

$$\boldsymbol{LL = office\ corridors\ above\ first\ floor + partitions}$$

$$LL = 0.08\ ksf(9') + 0.02\ ksf(9') = \mathbf{0.90}\,\boldsymbol{{}^{k}/_{ft}}$$

Critical uniform load, w_u:

$$\boldsymbol{w_u = 1.2DL + 1.6LL}$$

$$w_u = 1.2(0.692\,{}^{k}/_{ft}) + 1.6(0.90\,{}^{k}/_{ft}) = \mathbf{2.27}\,\boldsymbol{{}^{k}/_{ft}}$$

Critical design moment, M_u:

$$\boldsymbol{M_u = \frac{wl^2}{8}}$$

$$M_u = \frac{(2.27\,{}^{k}/_{ft})(30^2)}{8} = \boldsymbol{255.4\ ft - k} < 3655\ ft - k \quad \therefore \boldsymbol{ok}$$

The large remaining available capacity is likely due to the lateral loads that were not accounted for in this spot check.

Verify that the deflections are acceptable:

Consider deflection during construction:

Assume that the bare beam must support its own self-weight and the self-weight of the slab, as well as a 20 psf construction live load.

First verify that the steel beam has the required strength to support these loads:

$$\boldsymbol{DL = beam\ self - weight + slab\ self - weight}$$

$$DL = 0.035\ klf + (0.063\ ksf)(9') = \mathbf{0.602}\ \boldsymbol{{}^{k}/_{ft}}$$

$$\boldsymbol{LL_{contruction} = 0.02\ ksf(9') = 0.18\,{}^{k}/_{ft}}$$

$$\boldsymbol{w_u = 1.2DL + 1.6LL}$$

$$w_u = 1.2(0.602\,{}^{k}/_{ft} + 1.6\left(0.18\,{}^{k}/_{ft}\right) = \mathbf{1.01}\,\boldsymbol{{}^{k}/_{ft}}$$

$$\boldsymbol{M_u = \frac{wl^2}{8}}$$

$$M_u = \frac{(1.01\,{}^{k}/_{ft})(30^2)}{8} = \boldsymbol{113.6\ ft - k} < 249\ ft - k = \varphi \boldsymbol{M_p} \quad \therefore \boldsymbol{ok}$$

Now check deflection during construction:

$$\Delta_{construction} = \frac{\mathbf{5 w_u l^4 1728}}{\mathbf{384 EI}} < \frac{\mathbf{l}}{\mathbf{360}}$$

$$\Delta_{construction} = \frac{5\left(0.782\ {}^{k}/_{ft}\right)(30^4)1728}{384(29000\ ksi)(510 in^4)} = \mathbf{0.964"} < \frac{(30' \text{x} 12)}{360} = \mathbf{1.0"}$$

$\therefore\ \Delta_{construction}\ \boldsymbol{is\ okay}$

Check live load deflection:

When checking the live load deflection, use the *service* live load.
I_{LB} was pulled from the ACI 318-08, Table 3-20

$$\Delta_{LL} = \frac{\mathbf{5 w_u l^4 1728}}{\mathbf{384 E I_{LB}}} < \frac{\mathbf{l}}{\mathbf{360}}$$

$$\Delta_{LL} = \frac{5(0.9)(30^4)1728}{384(29000\ ksi)(1000)} = \mathbf{0.566"} < \frac{(30' \text{x} 12)}{360} = \mathbf{1.0"}$$

$\therefore\ \Delta_{LL}\ \boldsymbol{is\ okay}$