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1000 CONNECTICUT AVENUE

Washington DC

Technical Report 3: Lateral System and Confirmation Design

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Executive Summary

1000 Connecticut Avenue is a 12 story, 565, 000 GSF commercial office building located at the corner of K Street and Connecticut Avenue in Washington D.C. The building is used primarily for office space, but also contains retail space on the first level, commercial office space on levels 3-12, a roof-top terrace with a green roof, and four levels of underground parking.

The purpose of this technical report is to further understand the existing structural system by determining which combination of lateral loads controlled the lateral system design; checking the story displacement and story drifts due to the controlling lateral loads and comparing the drift values to allowable code limits; analyzing the overturning moments due to the lateral loads and the resisting moments due to the total building weight; and, spot checking critical members for strength adequacy.

The wind loads were determined by using Analytical Procedure (method 2) outlined in ASCE 7-10 and the seismic loads were determined by using the Equivalent Lateral Force Procedure outlined in ASCE 7-10. The wind loads were calculated for both the North-South and East-West directions and it was found that the lateral forces due to the wind load were greatest in the N-S direction, resulting in a base shear of 1401 kips. One analysis was completed for determining the seismic story forces since the lateral force resisting system consists of a reinforced concrete moment frame in both the N-S and E-W directions. The seismic base shear was found to be 1001 kips, which was 55 % greater than the design base shear of 645 kips. This shows that the dead load assumptions and analysis simplifications were conservative.

Further, an ETABS computer model of the lateral system was created to determine which combination of lateral loads controlled the lateral system's design; to determine each frame's stiffness; and, to check the serviceability by determining the lateral displacements/story drifts due to the un-factored controlling lateral forces in both the N-S and E-W directions. It was found that the N-S wind load case 1 controlled the lateral load in the N-S direction and the seismic was the controlling lateral load in the E-W direction. Using the controlling lateral loads to determine the building drift, it was found that both the lateral displacements and story drifts were within the allowable code limits.

In addition, it was found that the columns do not transfer moment to the foundation since the spread footings will behave like pinned connections due to the their low rigidity; therefore the footings will not be able to carry the moment due to the lateral loads. It was determined that the slab-to-column moment frame systems below grade are adequate to carry the moments due to the lateral loads.

Lastly, a member spot check was performed on column 50, an interior column. The column was checked for both axial load and bending. ETABS was used to determine the in-plane bending moment acting on the column due to the factored wind load in the N-S direction. An interaction diagram was created to compare P_u and M_u to ϕP_n and ϕM_n and the column was found to be adequate to carry the combined axial and bending load.

The appendices in this report include hand calculations for wind, seismic, snow and gravity loads; frame spot checks; and, typical floor plans and a building section.

Introduction

1000 Connecticut Avenue, NW Office Building is a new 12 story office building located at the northwest intersection of K Street and Connecticut Avenue in Washington DC, as can be seen in Figure 1. The 1000 Connecticut Avenue Office building is designed to achieve LEED Gold certification upon completion. Despite being used primarily for office space, the building is comprised of mix occupancies, which include: office space, a gymnasium, retail, and parking garages. The structure has 4 levels of underground parking. The building's total square footage is 555,000 SF with 370,000 SF above grade and 185,000 SF below grade.

Figure 1 Building Site

To create a new Washington landmark, the building is designed to complement surrounding institutions by blending both traditional and modern materials. The facade consists of a glass, stainless steel and stone panel curtain wall system. Exterior and interior aluminum and glass storefront windows and doors are on the ground level. The lobby and retail space are located on the 1st level, which has a 12'-6 1/2" floor-to-floor story height. A canopy facing K Street brings attention to the main lobby entrance, as can be seen in Figure 2.

Figure 2 Main Lobby Entrance facing K Street (left) and perspective of curtain wall system (right)

Beyond the main entrance is a two story intricate lobby space with carrera marble and Chelmsford granite flooring, aluminum spline panels integrated with glass fiber reinforced gypsum (GFRG) ceiling tiles and European white oak wood screens, as can be seen in Figure 3.

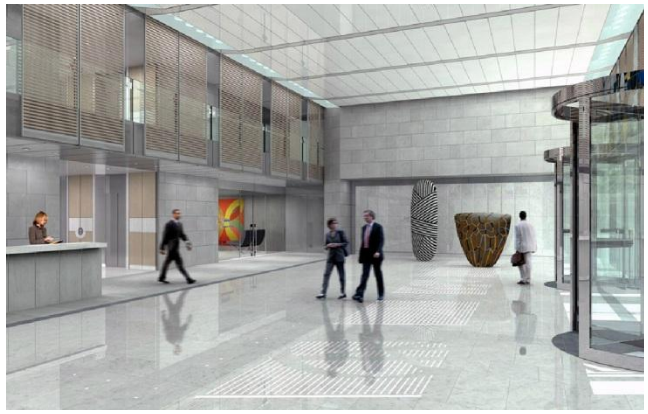


Figure 3 Perspective of lobby

The retail space is broken down into several retail stores facing K Street and Connecticut Avenue. These retail stores are housed behind storefront glass to enable display of merchandise to potential customers. The 2nd-12th levels have 10'-7 ½" floor-to-floor story heights. Housed on the typical levels (3rd-12th) is the office space. A combination of tall story heights and a continuous floor to ceiling glass façade enables natural daylight to enter the building space as well as provides scenery to the Washington monuments, Farragut Park , and the White House, as can be seen in Figure 4.

Figure 4 Perspective of typical office with floor-to-ceiling windows that supply views to the city

In addition, located on the penthouse level is a roof-top terrace with a green roof and a mechanical penthouse, as can be seen in Figure 5.

Figure 5 Perspective of green roof on roof-top terrace and mechanical penthouse

Housed on the basement levels (B1-B4) are underground parking and a fitness center. A total of 253 parking spaces are provided; level B1 has 19 parking spaces; level B2 has 74 parking spaces; level B3 has 78 parking spaces; level B4 has 82 parking spaces. In addition, the fitness center is located on level B1.

Structural Overview

1000 Connecticut Avenue Office Building's structural system is comprised of a reinforced concrete flat slab floor system with drop panels and a bay spacing of approximately 30 feet by 30 feet. The slab and columns combined perform as a reinforced concrete moment frame. The substructure and superstructure floor systems are both comprised of an 8" thick two-way system with #5 reinforcing bars spaced 12" on center in both the column and middle strips and 8" thick drop panels. The below grade parking garage ramp is comprised of a 14" thick slab with #5 reinforcing bars provided both top and bottom with a spacing of 12" on center.

Foundation

ECS Mid-Atlantic, LLC performed a geotechnical analysis of the building's site soil conditions as well as provided recommendations for the foundation. A total of five borings were observed in the geotechnical analysis. It was determined that a majority of the site's existing fill consists of a mixture of silt, sand, gravel, and wood. The natural soils consisted of sandy silt, sand with silt, clayey gravel, silty gravel, and silty sand. The soil varies from loose to extremely dense in relative density. Based on the samples recovered from the rock coring operations, the rock is classified as completely to moderately weathered, thinly bedded, and hard to very hard gneiss.

At the time of the study, the groundwater was recorded at a boring depth of 7.5 feet below the existing ground surface. The shallow water table is located at an elevation of 35 to 38 feet in the vicinity of the site.

1000 Connecticut Avenue, NW Office Building is supported by a shallow foundation consisting of column footings and strap beams, as can be seen in Figure 6. The typical column footing sizes are 4'-0" x 4'-0", 5'-0" x 5'-0", and 4'-0" x 8'-0".

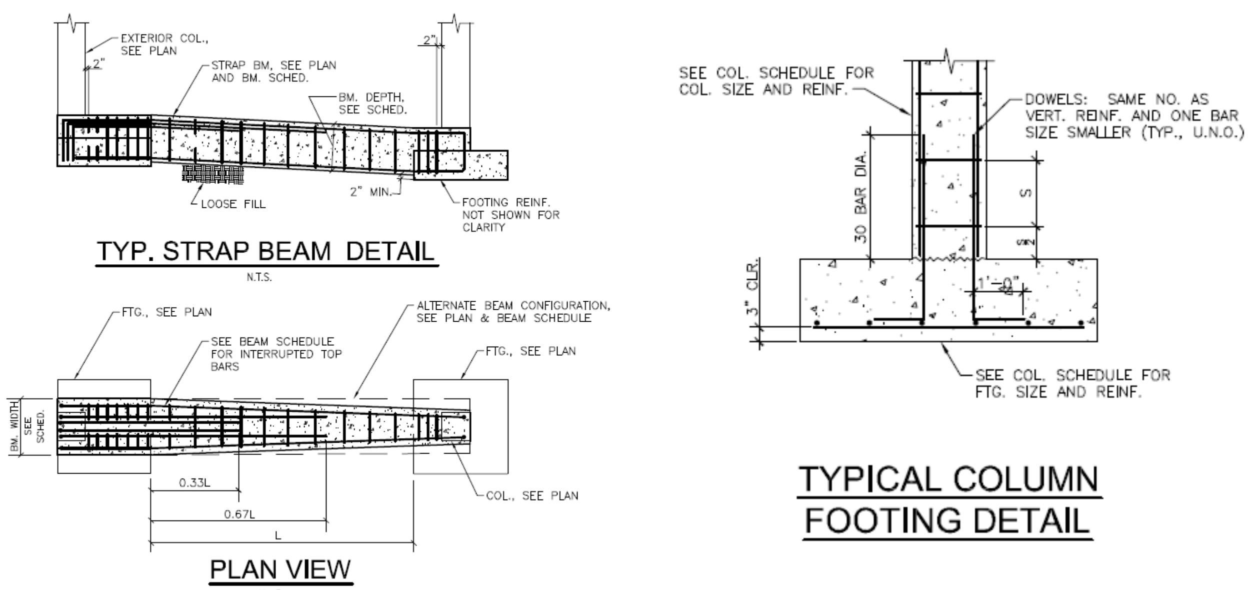


Figure 6 Details of typical strap beam and column footing

The footings bear on 50 KSF competent rock. The Strap beams (cantilever footings) are used to prevent the exterior footings from overturning by connecting the strap beam to both the exterior footing and to an adjacent interior footing. A simplified foundation plan can be seen in Figure 7.

The slab on grade is 5” thick, 5000 psi concrete with 6x6-W2.9xW2.9 wire welded fabric on a minimum 15 mil Polyethylene sheet over 6” washed crushed stone. The foundation walls consists of concrete masonry units vertically reinforced with #5 bars at 16” on center and horizontally reinforced with #4 bars at 12” on center and are subjected to a lateral load (earth pressure) of 45 PSF per foot of wall depth.

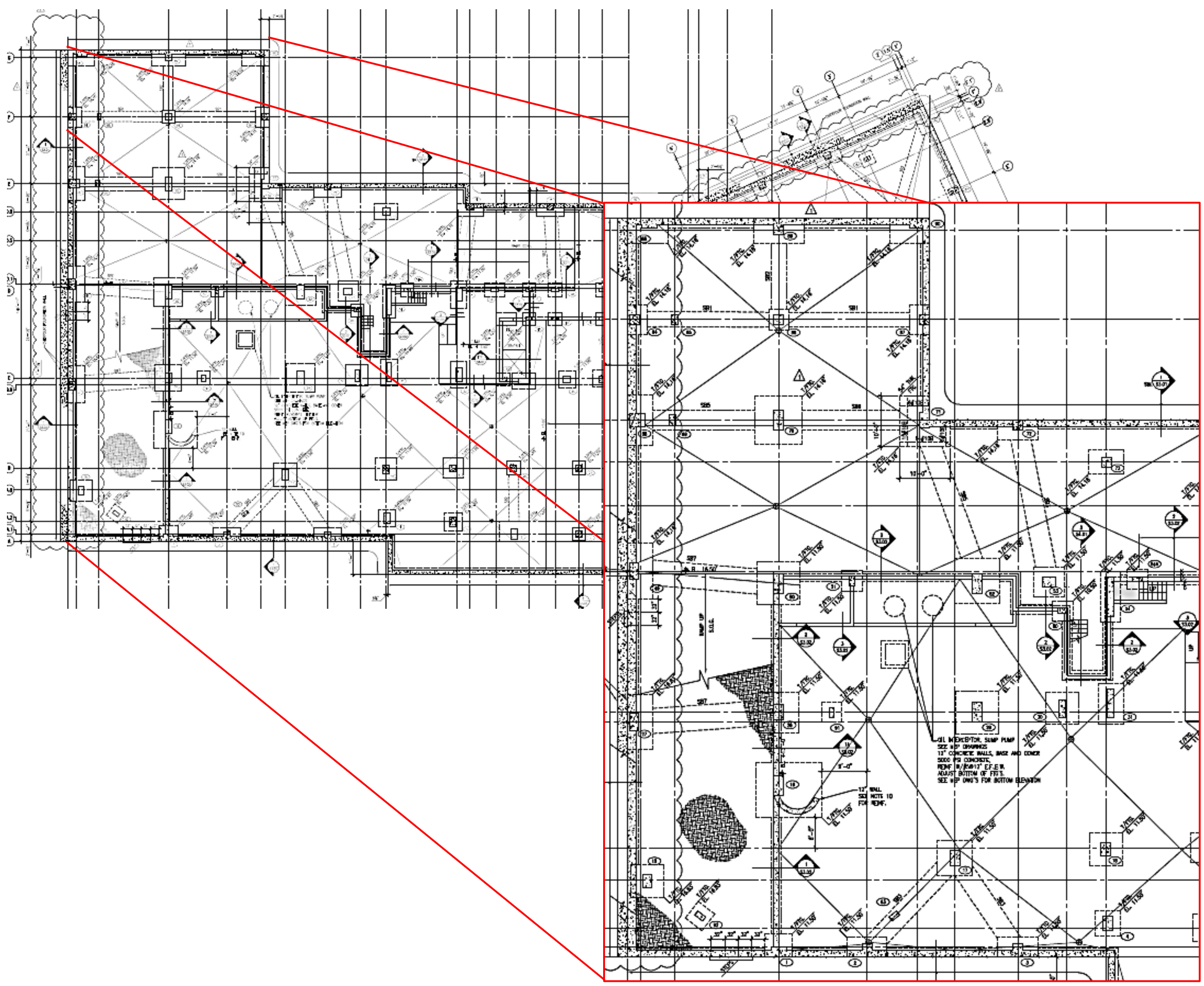


Figure 7 Foundation plan

Framing and Floor System

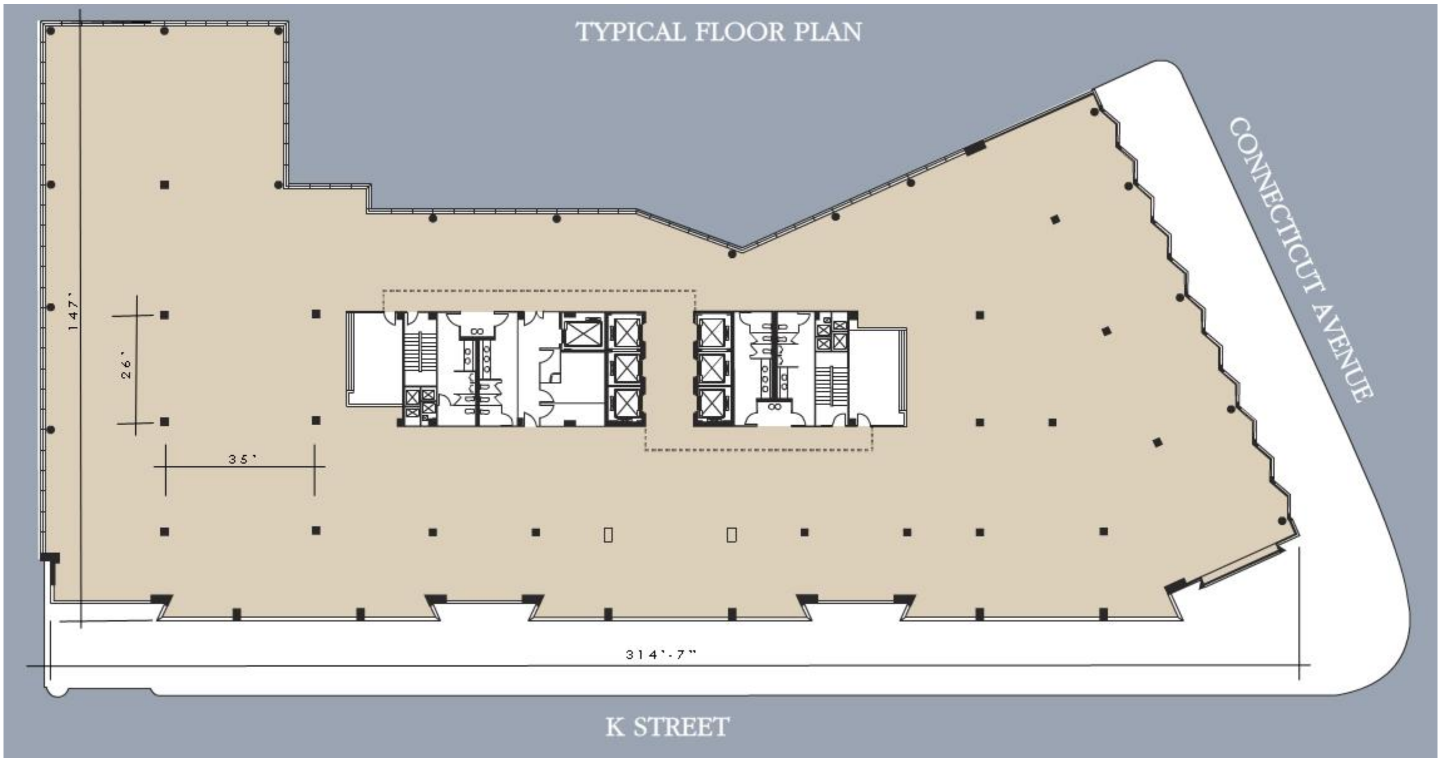
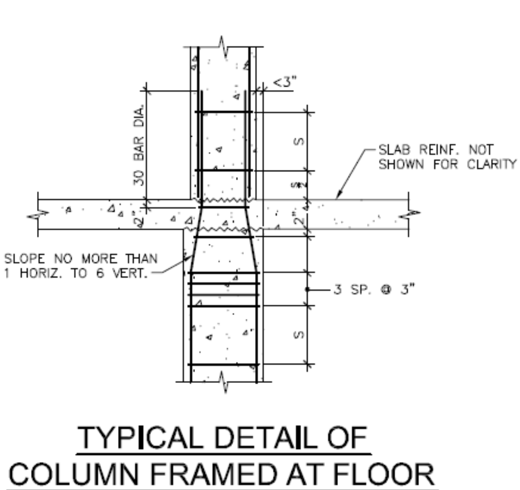


Figure 8 Floor plan displaying column locations and bays

The framing system is composed of reinforced concrete columns with an average column-to-column spacing of 30'x30', as can be seen in Figure 8. The columns have a specified concrete strength of f'c=8000 psi for columns on levels B4 to level 3, f'c=6000 psi for columns on levels 4-7, and f'c=5000 psi for columns on levels 8-mechanical penthouse. The columns are framed at the concrete floor, as can be seen in Figure 9, and the columns vary in size. The most common column sizes are 24"x24", 16"x48", and 24"x30". The column capitals are 6" thick, measured from the bottom of the drop panel, extending 6" all around the face of the column, as can be seen in Figure 10.



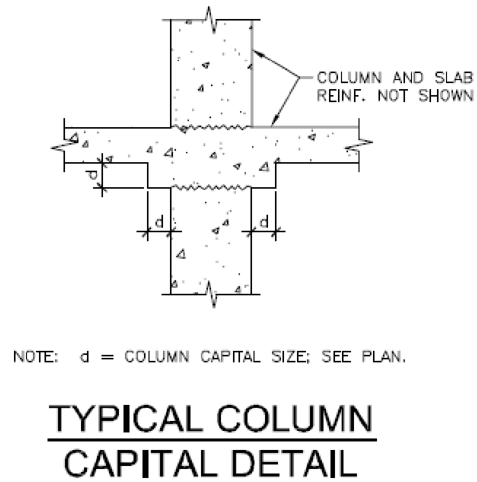


Figure 9 Typical Detail of column framed at the floor **Figure 10** Typical column capital detail

The typical floor system is comprised of an 8" thick two-way flat slab with drop panels reinforced with #5 bottom bars spaced 12" on center in both the column and middle strips, as can be seen in Figure 11.

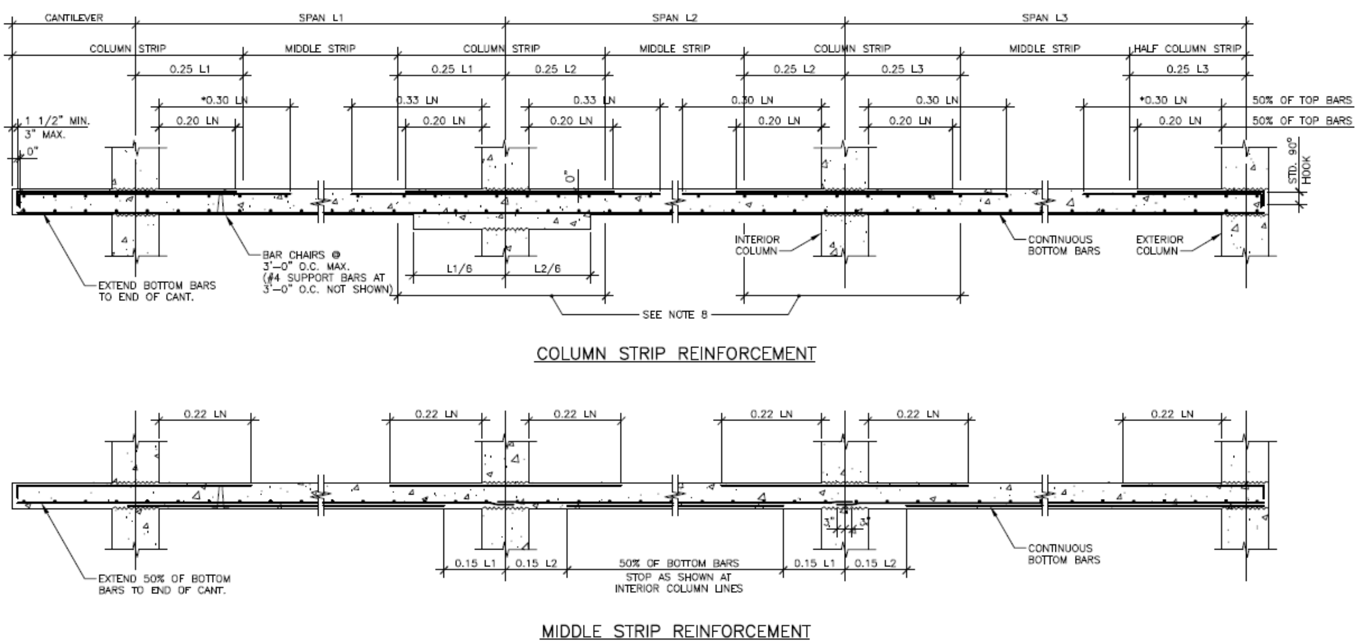


Figure 11 Typical two-way slab reinforcing detail

The individual drop panels are 8" thick, extending a distance d/6 from the centerline of the column, as can be seen in Figure 12.

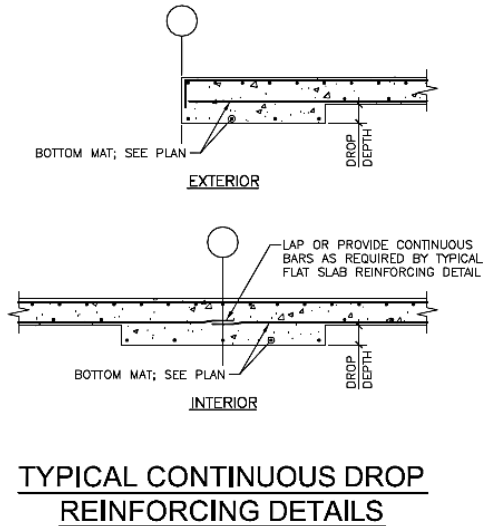


Figure 12 Typical Continuous drop panel

A 36" wide by 3 ½" deep continuous drop panel is located around the perimeter on all floor levels. Levels 3-12 are supported by four post-tension beams above the lobby area. Due to the two story lobby, there's a large column-to-column spacing. As a result, post tension beams are used to support the slab on levels 3-12 located above the lobby. In addition, four post-tension beams support the slab on levels 3-12 that are located above the two-story parking deck, which also has a large column-to-column spacing, as can be seen in Figure 13.

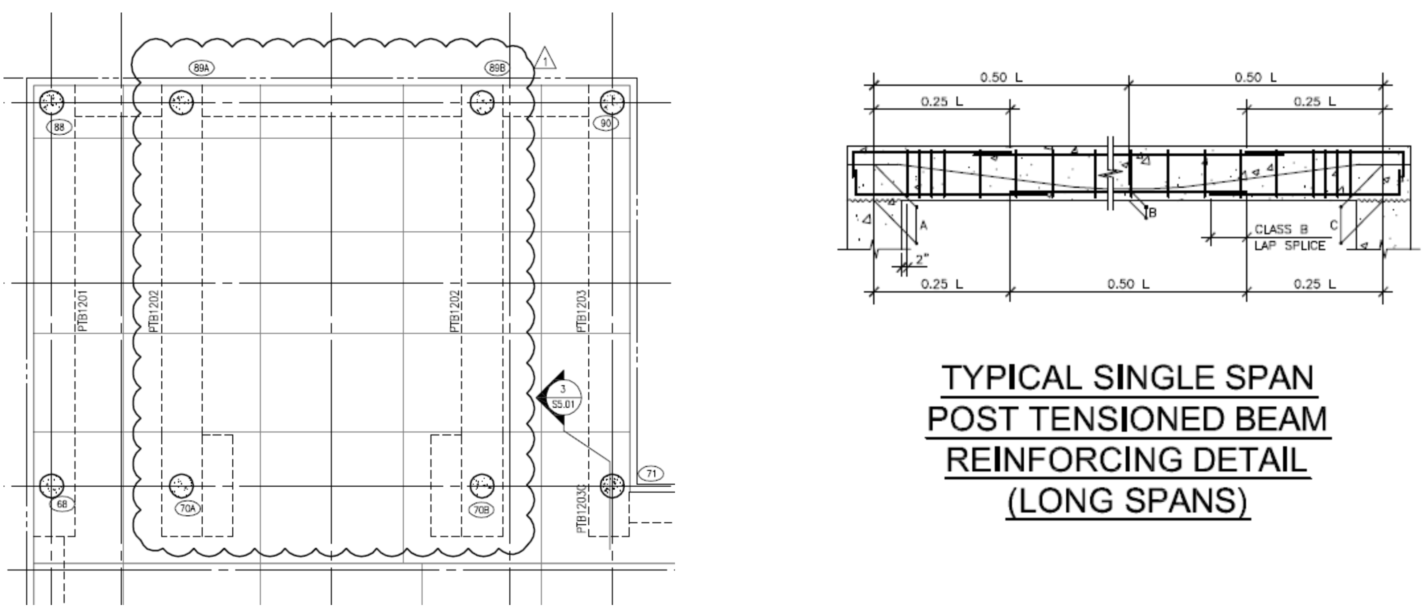


Figure 13 Plan view and typical detail of Post-tension beams supporting slab on levels above two-story loading dock

Lateral System

The lateral system is comprised of a reinforced concrete moment frame. The columns and slab are poured monolithically, thus creating a rigid connection between the elements. The curtain wall is attached to the concrete slab, which puts the slab in bending. The curtain wall transfers the lateral load to the slab. The slab then transfers the lateral load to the columns and in turn the columns transfer the load to the foundation. Transfer girders on the lower level are used to transfer the loads from the columns that do not align with the basement columns in order to transfer the load to the foundation. A depiction of how the lateral load is transferred through the system can be seen in Figure 14.

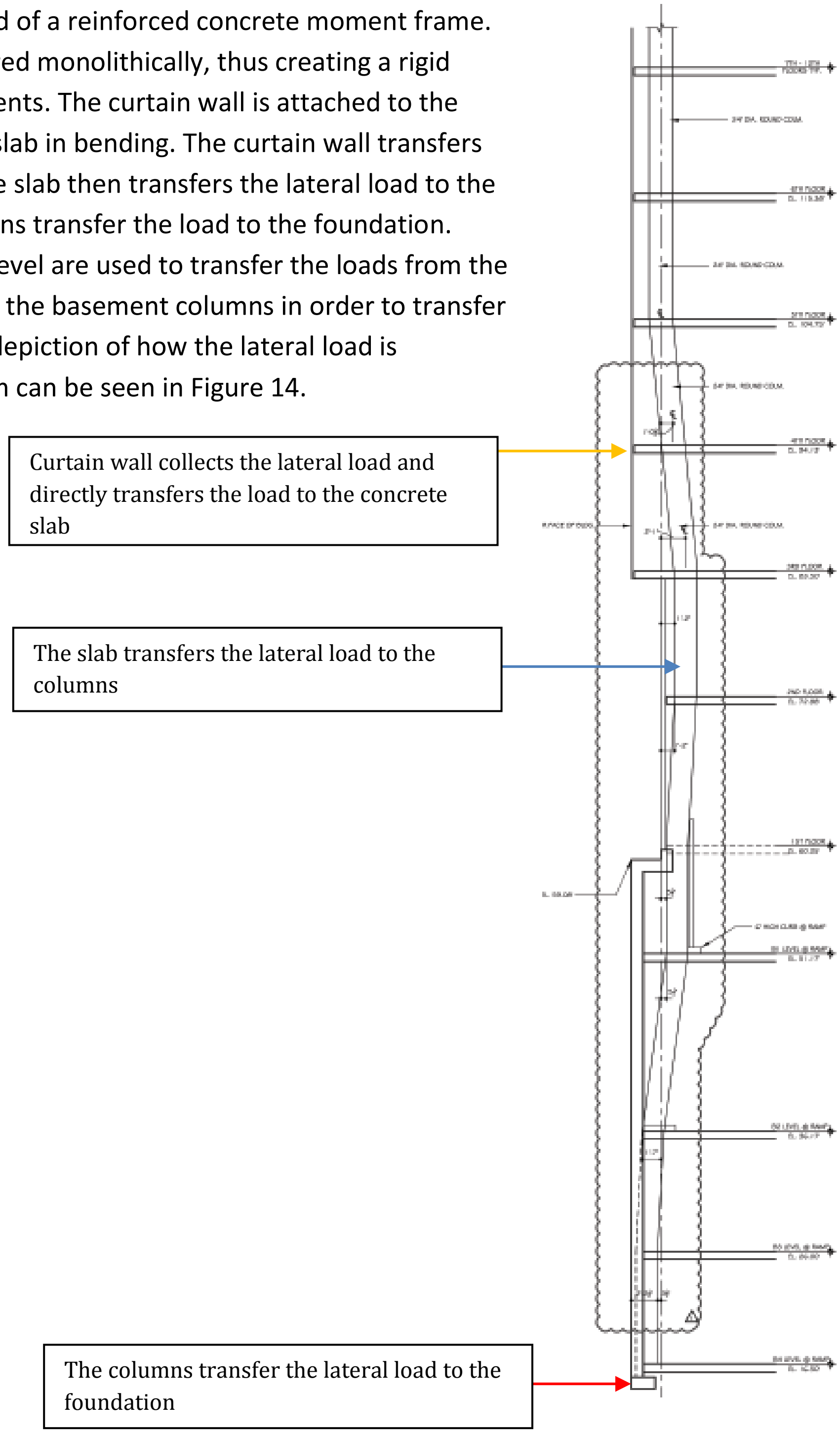


Figure 14 Lateral load path depiction

Roof System

The main roof framing system is supported by an 8”thick concrete slab with #5 bars spaced 12” on center at the bottom in the east-west direction. The slab also has 8” thick drop panels. The penthouse framing system is separated into two roofs: Elevator Machine Room roof and the high roof. The elevator machine room roof framing system is supported by 14” and 8” thick slab with #7 bars with 6” spacing on center top and bottom in the east-west direction.

Design Codes

According to sheet S601, the original building was designed to comply with the following:

- 2000 International Building Code (IBC 2000)
- Building Code Requirements for Structural Concrete (ACI 318)
- Specifications for Structural Concrete (ACI 301)
- Manual of Standard Practice for Detailing Reinforced Concrete Structures (ACI 315)
- Specification for the Design, Fabrication and Erection of Structural Steel for Buildings (AISC manual), Allowable Strength Design (ASD) method

The codes that were used to complete the analyses within this technical report are the following:

- ACI 318-08
- Minimum Design Loads for Building and Other Structures (ASCE 7-10)
- AISC Steel Construction Manual, 14th Edition, Load and Resistance Factor Design (LRFD) method
- Vulcraft Steel Roof and Floor Deck Manual, 2008
- Precast/Prestressed Concrete Institute (PCI) Handbook Manual, 7th Edition

Structural Materials

Table 1 below shows the several types of materials that were used for this project according to the general notes page of the structural drawings on sheet S601.

Concrete (Cast-in-Place)		
Usage	**Weight**	**Strength (psi)**
Spread Footings	Normal	4000
Strap Beams	Normal	4000
Foundation Walls	Normal	4000
Formed Slabs and Beams	Normal	5000
Columns	Normal	Varies (based on column schedule)
Concrete Toppings	Normal	5000
Slabs on Grade	Normal	5000
Pea-gravel concrete (or grout)	Normal	2500 (for filling CMU units)
All other concrete	Normal	3000
Reinforcing Steel		
Type	**Standard**	**Grade**
Deformed Reinforcing Bars	ASTM A615	60
	ASTM A775	N/A
Welded Wire Fabric	ASTM A185	N/A
Reinforcing Bar Mats	ASTM A184	N/A
Post-Tensioning (Unbonded)		
Type	**Standard**	**Strength (ksi)**
Prestressed Steel (seven wire low-relaxation or stressed relieved strand)	ASTM A416	270
Miscellaneous Steel		
Type	**Standard**	**Grade**
Structural Steel	ASTM A36	N/A
Bolts	ASTM A325	N/A
Welds	AWS	N/A

Table 1 Design materials

Gravity Loads

For this technical report, live loads and snow loads were compared to the loads listed on the structural drawings. In addition, dead loads were calculated and assumed in order to spot check gravity members and typical columns. The system evaluations were then compared to the original design. The hand calculations for the gravity member checks can be found in Appendix A.

Dead and Live Loads

Table 2 below is a list of the live loads in which the project was designed for compared to the minimum design live loads outlined in ASCE 7-10.

Floor Live Loads		
Occupancy	Design Load (psf)	ASCE 7-10
Parking Levels	50	40
Retail	100	100
Vestibules & Lobbies	100	100
Office Floors	100=(80 psf+ 20 psf partitions)	70= (50 psf + 20 psf partitions)
Corridors	100	100 on ground level 80 above 1st level
Stairs	100	100
Balconies & Terraces	100	100
Mechanical Room	150	-
Pump Room, Generator Room	150	-
Light Storage	125	125
Loading Dock, Truck Bays	350	250
Slab On Grade	100	-
Green Roof Areas	30	-
Terrace	100	100

Table 2 Summary of design live loads compared to minimum design live loads on ASCE 7-10

Note: - Means the load for the specified occupancy was not provided

Based on the above design live loads, certain spaces were designed for higher loads to create a more conservative design and to allow for design flexibility. For this technical report, the design live loads were used for the gravity member analyses.

Snow Load

The snow load was determined in conformance to chapter 7 in ASCE 7-10. A summary of the snow drift parameters are shown in table 3.

Flat Roof Snow load Calculations	
Variable	Value
Ground Snow, p_g (psf)	25
Temperature, Factor C_t	1.0
Exposure Factor, C_e	0.9
Importance Factor, I_s	1.0
Flat Roof Snow Load, p_f	15.75

Table 3 Summary of roof snow calculations

According to structural drawing sheet S601, the flat roof snow load was 22.5 psf whereas 15.75 psf was calculated in this technical report. According to ASCE 7-10, $p_f=0.7C_eC_tI_sP_g$, whereas according to IBC 2000, $p_f=C_eC_tI_sP_g$. The difference in the calculated flat roof snow load and the design flat roof snow load is due to a 0.7 reduction factor. The 15.75 psf value was used to determine the snow load and snow drifts. These subsequent calculations can be found in Appendix A.

Table 4 below is a list of the dead loads that were used for the gravity spot checks. The superimposed dead loads for the floor levels and roofs were assumed.

Dead Loads	
Normal Weight Concrete	150 pcf
Curtain Wall	250 plf
Precast Panels	450 plf
Floor Superimposed Dead Load (ceiling, lights, MEP, miscellaneous)	10 psf
Main Roof Superimposed Dead Load (ceiling, lights, MEP, miscellaneous)	10 psf
Penthouse Roof Superimposed Dead Loads	5 psf

Table 4 Summary of dead loads

Flat Slab Interior Panel Gravity Check

The interior flat slab panel outlined in figure 15 was checked for slab thickness and column strip reinforcement. I chose to check this panel because it is a typical interior panel with a long span of 35 feet in the east-west direction. Due to the panel's long span, it would require a thick slab in order to control deflection and thus the slab thickness chosen for this panel will also be applicable throughout the remainder of the flat slab system.

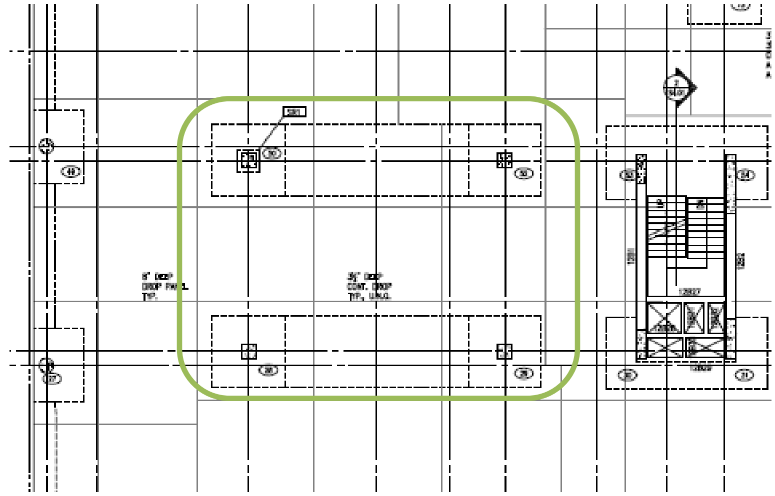


Figure 15 Interior flat slab panel

I simplified my analysis by using ACI 318 Direct Design Method (DDM) to determine the column strip moments as well as analyzed the slab as a flat plate system, neglecting the drop panels.

To begin my analysis, I determined the slab thickness according to table 9.5(c) in ACI 318. The determined slab thickness was 11". Next, I calculated the factored load w_u=337 psf and the uniform panel moment M=1193 k-ft. Using the direct design method, the uniform moment was longitudinally distributed to determine the panel's negative moment and midspan moment. The longitudinal moments were then distributed transversely to the column strip. After determining the column strip moments, I then proceeded to determine the column strip's reinforcement.

The simplified analysis resulted in a slab thickness of 11" and (24) #8 bars were determined to resist the column strip positive moment and (13) #8 bars were determined to resist the column strip negative moment. The original design uses an 8" slab thickness reinforced with #5 bars. The gravity spot check resulted in a different slab thickness and reinforcement bar size because the analysis was oversimplified. The system was analyzed as a flat plat instead of a flat slab as well as the direct design method was used to determine longitudinal and transverse moments, which is a conservative method for analyzing this slab panel. I will complete a more thorough analysis of this system in technical report 3 by treating the slab as a flat slab as well as using the Equivalent Frame Method to determine the exact moments.

Column # 50 Gravity Spot Check

Column 50 is an interior column that starts at the basement level and expands up to the roof level. I sized the column at the 1st and 5th levels. I chose these two locations because the slab cross section changes at the 5th level. As a design aid, I used the interaction diagrams from Reinforced Concrete: Mechanics and Design, 5th edition. After the analysis, it was determined that a 30"x30" column would be required to resist the axial load on the 1st level and a 24"x30" column would be required to resist the axial load on the 5th level. The original design used a 24"x36" column on the 1st level. Based on the gross area, my cross section has a percent error of 4%, which is very close to the cross sectional area of the original design. This error may be the result of the fact that the 1st level column has a slope, and I neglected this slope to simplify the analysis. The original column size for the 5th level is a 24"x24" column. Based on the gross cross-sectional area, my cross section has a percent error of 25%, which is relatively close to original design section. The result of this error could be a combination of dead load assumptions and simplified column analysis. In technical report 3, a more thorough analysis will be performed to determine the column size.

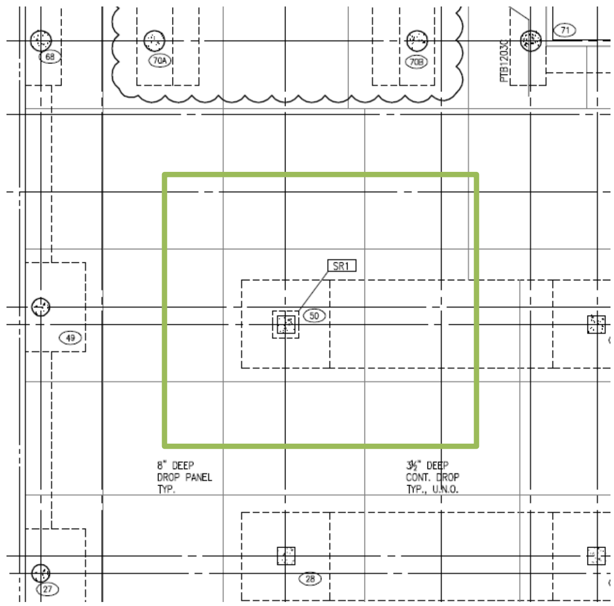


Figure 16 Column 50 with approximate tributary area

Lateral Loads

In this report, wind and seismic lateral loads were calculated to determine the loads acting on the structure's lateral system. To perform manual calculations for determining the lateral loads, simplifying assumptions were made. In addition, it was determined how much of the story force was distributed to each moment frame, which will be discussed later in this report. The hand calculations associated with the wind and seismic loads determination can be found in Appendices B and C.

Wind Loads

Wind loads were determined using the Main Wind Force Resisting System (MWFRS) procedure (method 2) in conformance to Chapters 26 and 27 outlined in ASCE 7-10. Due to the building's complex geometry, a rectangular building shape was assumed to simplify the wind load analysis, as can be seen in Figure 17.

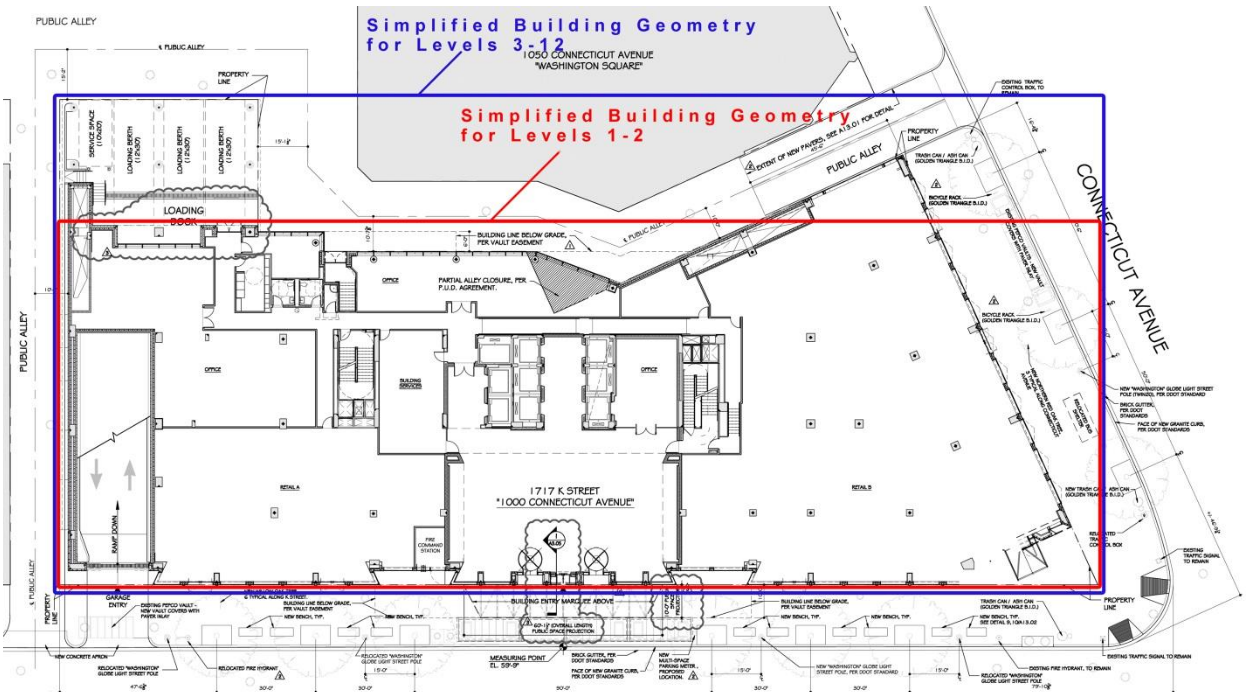


Figure 17 Simplified building shape for wind load analysis

Most of the calculations for determining the wind pressures and story forces were performed in Microsoft Excel. In the analysis, windward, leeward, sidewall, and roof suction pressures were determined. Internal pressures were neglected in calculating the design wind pressure because internal pressures do not contribute towards the external wind pressures acting on the building.

The general wind load design criteria and guest effect factors can be found in Tables 5 and 6. The calculated approximate lower- bound natural frequency for the building was 0.544 Hz, which is less than 1 Hz, therefore the gust factors were calculated in the event the building is flexible.

Further, wind pressures in the N-S and E-W directions can be seen in Tables 7 and 8 with the corresponding vertical profile sketch of the wind pressures shown in Figures 18 and 19. The story forces were then determined based on the wind pressures. The resulting base shears were 1401 k for the N-S direction and 553 k in the E-W direction. The story forces and overturning moments for both the N-S and E-W directions can be found in Tables 9 and 10 along with the vertical profile of the story forces in Figures 20 and 21.

General Wind Load Design Criteria		
Design Wind Speed, V	115 mph	ASCE 7-10, Fig. 26.5-1A
Directionality Factor, K_d - MWFRS	0.85	ASCE 7-10, Tbl. 26.6-1
Directionality Factor, K_d - Mechanical PH	0.9	ASCE 7-10, Tbl. 26.6-1
Exposure Category	B	ASCE 7-10, Sect. 26.7.3
Topographic Factor, K_{zt}	1.0	ASCE 7-10, Sect. 26.8.2
Internal Pressure Coeficient, GC_{pi}	0.18	ASCE 7-10, Tbl. 26.11-1

Table 5 General wind design criteria

Gust Factor-MWFRS			
N-S Wind		E-W Wind	
Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
0.861	0.861	0.945	0.926
Gust Factor-Mechnical Penthouse			
N-S Wind		E-W Wind	
0.85		0.85	

Table 6 Guest Factors

Wind Pressures - N-S Direction			
Type	Floor	Distances (ft)	Wind Pressure (psf)
	1	0	11.30
	2	12.54	11.30
	3	23.17	13.08
	4	33.79	15.06
	5	44.42	16.06
	6	55.04	16.85
	7	65.67	17.64
	8	76.29	18.43
	9	86.92	19.03
	10	97.54	19.62
	11	108.17	20.61
	12	118.79	20.61
Windward Walls	Main Roof	130	21.61
Leedward Walls	Levels 1-2	0 to 23.17	-13.50
	Level 3 -12	23.17 to 130	-13.50
Side Walls	All	All	-18.91
	N/A	0 to 65	-32.52
Roof	N/A	65 to 130	-20.20
	N/A	130-260	-17.61
	N/A	>260	N/A

Table 7 N-S Wind Pressures

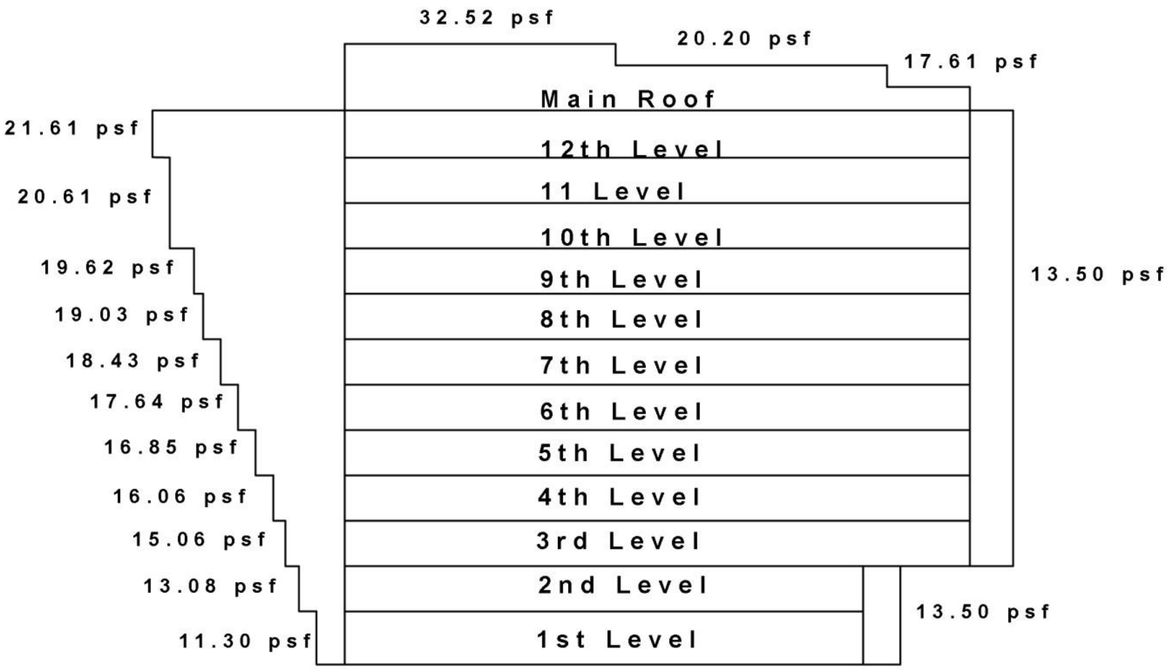


Figure 18 N-S wind pressure vertical pressure sketch

Wind Pressures - E-W Direction			
Type	Floor	Distances (ft)	Wind Pressure (psf)
	1	0	12.40
	2	12.54	12.40
	3	23.17	14.07
	4	33.79	16.20
	5	44.42	17.27
	6	55.04	18.12
	7	65.67	18.97
	8	76.29	19.83
	9	86.92	20.47
	10	97.54	21.11
	11	108.17	22.17
	12	118.79	22.17
Windward Walls	Main Roof	130	23.24
Leedward Walls	Levels 1-2	0 to 23.17	-8.03
	Level 3 -12	23.17 to 130	-8.51
Side Walls	Levels 1-2	0 to 23.17	-20.75
	Levels 3-12	23.17 to 130	-20.33
	N/A	0 to 65	-26.14
Roof	N/A	65 to 130	-26.14
	N/A	130-260	-14.52
	N/A	>260	-8.71

Table 8 E-W wind pressures

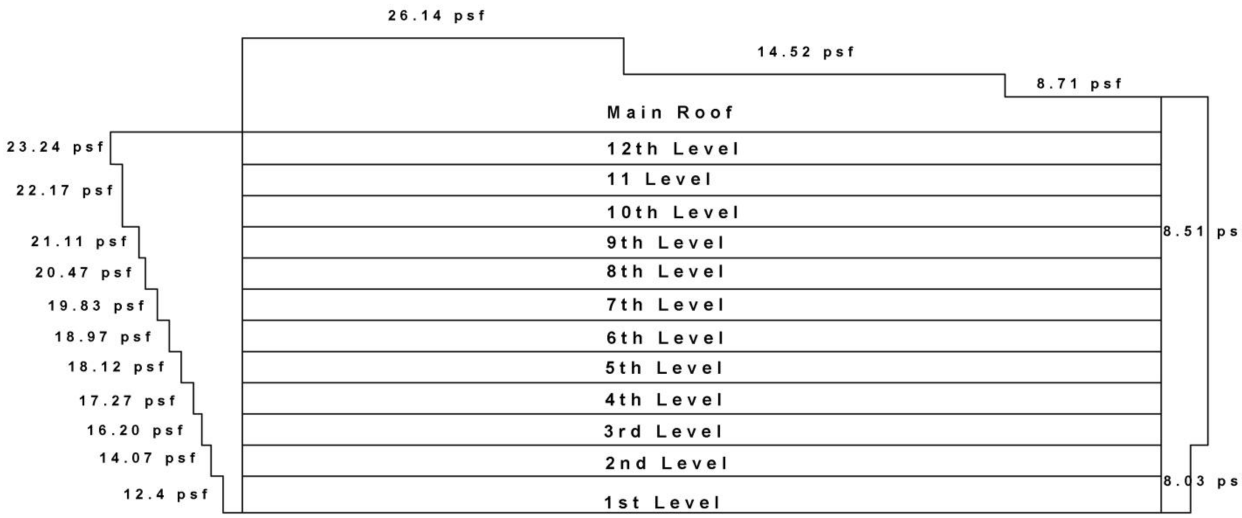


Figure 19 E-W vertical wind pressure profile

Wind Forces - N-S Direction										
Floor	Elevation (ft)	Tributary Below			Tributary Above			Story Force (Kips)	Story Shear (Kips)	Overturning Moment (K-ft)
		Height (ft)	Length (ft)	Area (ft^2)	Height (ft)	Length (ft)	Area (ft^2)			
PH Roof	148.5	18.5	199.83	3696.86	0	199.83	0	142.82	142.82	21208.42
Main Roof	130	5.31	314.58	1671.21	0	314.58	0	58.68	201.49	7627.83
12	118.79	5.31	314.58	1671.21	5.31	314.58	1671.21	115.69	317.19	13743.40
11	108.17	5.31	314.58	1671.21	5.31	314.58	1671.21	114.04	431.23	12335.55
10	97.54	5.31	314.58	1671.21	5.31	314.58	1671.21	112.38	543.61	10961.76
9	86.92	5.31	314.58	1671.21	5.31	314.58	1671.21	109.73	653.34	9537.91
8	76.29	5.31	314.58	1671.21	5.31	314.58	1671.21	107.74	761.09	8219.83
7	65.67	5.31	314.58	1671.21	5.31	314.58	1671.21	105.43	866.51	6923.30
6	55.04	5.31	314.58	1671.21	5.31	314.58	1671.21	102.78	969.29	5656.76
5	44.42	5.31	314.58	1671.21	5.31	314.58	1671.21	100.13	1069.41	4447.57
4	33.79	5.31	314.58	1671.21	5.31	314.58	1671.21	97.14	1166.56	3282.49
3	23.17	5.31	314.58	1671.21	5.31	314.58	1671.21	92.17	1258.73	2135.69
2	12.54	6.27	314.58	1972.42	5.31	314.58	1671.21	93.35	1352.08	1170.63
1	0	0	314.58	0.00	6.27	314.58	1972.42	48.92	1401.00	0.00
									Total Base Shear =	1401 K
									Total Overturning Moment =	107,251 K-ft

Table 9 N-S Story forces, base shear, and overturning moment

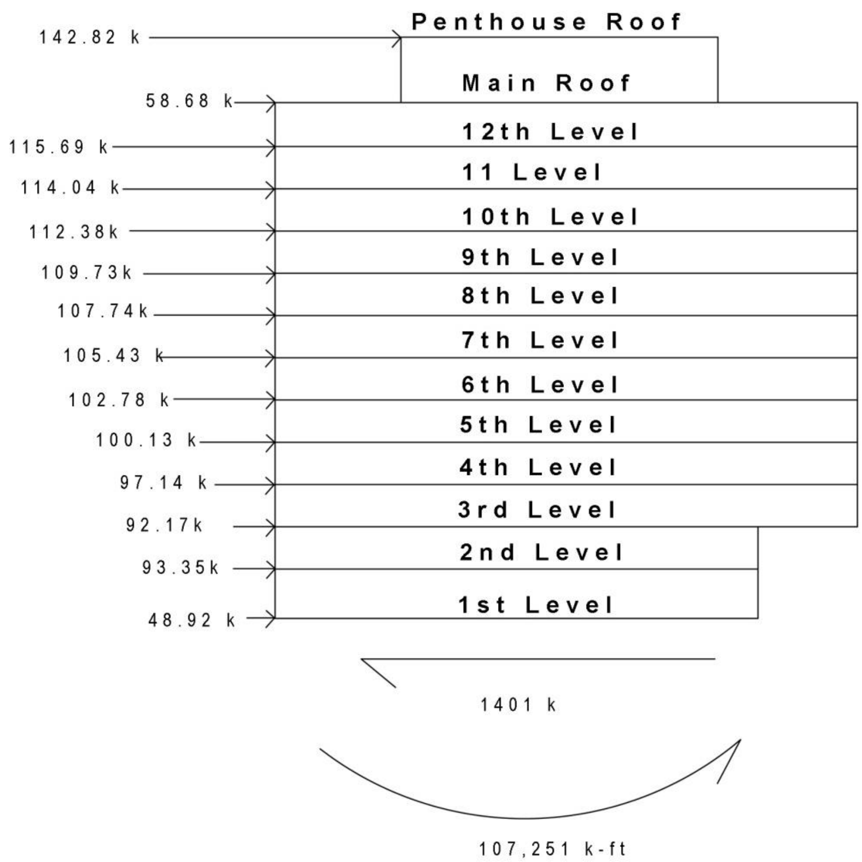


Figure 20 Vertical profile of story forces in N-S direction

Wind Forces - E-W Direction										
		Tributary Below			Tributary Above			Story Force	Story Shear	Overturning
Floor	Elevation (ft)	Height (ft)	Length (ft)	Area (ft^2)	Height (ft)	Length (ft)	Area (ft^2)	(Kips)	(Kips)	Moment (K-ft)
PH Roof	148.5	18.5	59.83	1106.86	0	59.83	0	42.76	42.76	6349.90
Main Roof	130	5.31	147	780.94	0	147	0	27.57	70.33	3583.67
12	118.79	5.31	147	780.94	5.31	147	780.94	48.75	119.08	5791.43
11	108.17	5.31	147	780.94	5.31	147	780.94	47.92	167.00	5183.62
10	97.54	5.31	147	780.94	5.31	147	780.94	47.09	214.09	4593.03
9	86.92	5.31	147	780.94	5.31	147	780.94	45.76	259.85	3977.18
8	76.29	5.31	147	780.94	5.31	147	780.94	44.76	304.60	3414.58
7	65.67	5.31	147	780.94	5.31	147	780.94	43.59	348.20	2862.72
6	55.04	5.31	147	780.94	5.31	147	780.94	42.26	390.46	2326.03
5	44.42	5.31	147	780.94	5.31	147	780.94	40.93	431.39	1818.06
4	33.79	5.31	147	780.94	5.31	147	780.94	39.43	470.82	1332.35
3	23.17	5.31	147	780.94	5.31	147	780.94	36.56	507.38	847.10
2	12.54	6.27	121.75	763.37	5.31	121.75	646.80	29.90	537.27	374.88
1	0	0	121.75	0.00	6.27	121.75	763.37	15.60	552.87	0.00
									Total Base Shear =	553 K
									Total Overturning Moment =	42,455 K-ft

Table 10 E-W Story forces, base shear, and overturning moment

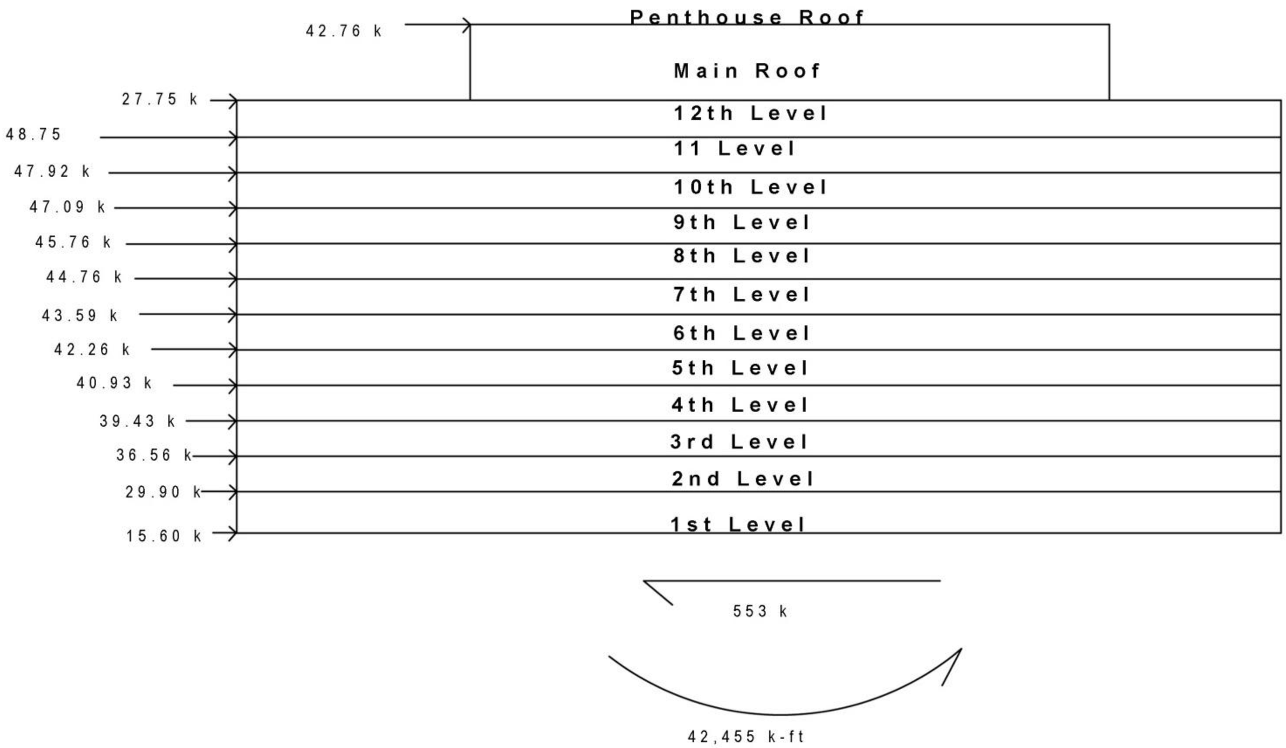


Figure 21 Vertical profile of story forces in E-W direction

Seismic Loads

Seismic loads were determined using the Equivalent Lateral Force Procedure outlined in Chapters 11 and 12 in ASCE 7-10. To simplify the analysis, slab openings due to the stairwells and elevator shafts were neglected, therefore resulting in more conservative calculations. In addition, the 1st level weight was neglected and thus the 2nd-12th levels, main roof, and penthouse were considered for building weight calculations. The typical floor level slab thickness is 8” with small areas consisting of 12” slabs. For calculation simplification, a uniform slab thickness of 8” was used.

Since the lateral resisting system consists of a reinforced concrete moment frame in both the N-S and E-W directions, one analysis was performed to determine the seismic story forces and base shear for both directions.

Since this building has several stories above grade, building weight was determined by calculating the dead weight for the typical floor level and applying that story weight to the other floor levels (levels 2-12). The weight on the main roof and penthouse roof were calculated separately. The weight included for summing the total building weight were the weight of the slabs, columns, drop panels, and superimposed dead loads.

After the analysis, the determined base shear was 1001 kips, while the original design base shear was 645 kips. The calculated base shear results in a percent error of 55%. Based on this significant difference, it is possible that the dead load assumptions were conservative. In addition, all existing slab openings were neglected, also resulting in a conservative seismic base shear determination. Refer to Table 11 for seismic force analysis results.

Seismic Forces								
level i	Height to level i h_i (ft)	Story Height h_x (ft)	Story Weight w_x (kips)	$w_x * h_x^k$	C_{VX}	Story Force f_i (kips)	Story Shear V_i (kips)	Overturning Moment M_z (k-ft)
PH Roof	0	148.0	754	779331	0.034	34	34	5036
Main Roof	0	129.5	4000	3434311	0.150	150	184	19417
12	10.63	118.8	4737	3610992	0.157	158	342	18741
11	10.63	108.2	4737	3170303	0.138	138	480	14982
10	10.63	97.6	4737	2746158	0.120	120	600	11703
9	10.63	87.0	4737	2339639	0.102	102	702	8884
8	10.63	76.3	4737	1952037	0.085	85	788	6506
7	10.63	65.7	4737	1584929	0.069	69	857	4547
6	10.63	55.1	4737	1240295	0.054	54	911	2982
5	10.63	44.4	4737	920716	0.040	40	951	1786
4	10.63	33.8	4737	629751	0.027	28	979	930
3	10.63	23.2	4737	372723	0.016	16	995	377
2	12.54	12.5	4453	149344	0.007	7	1001	82
		Σ=	56577	22930529		1001		95973

Table 11 Story forces, base shear, and overturning moment due to seismic loads

Computer Model

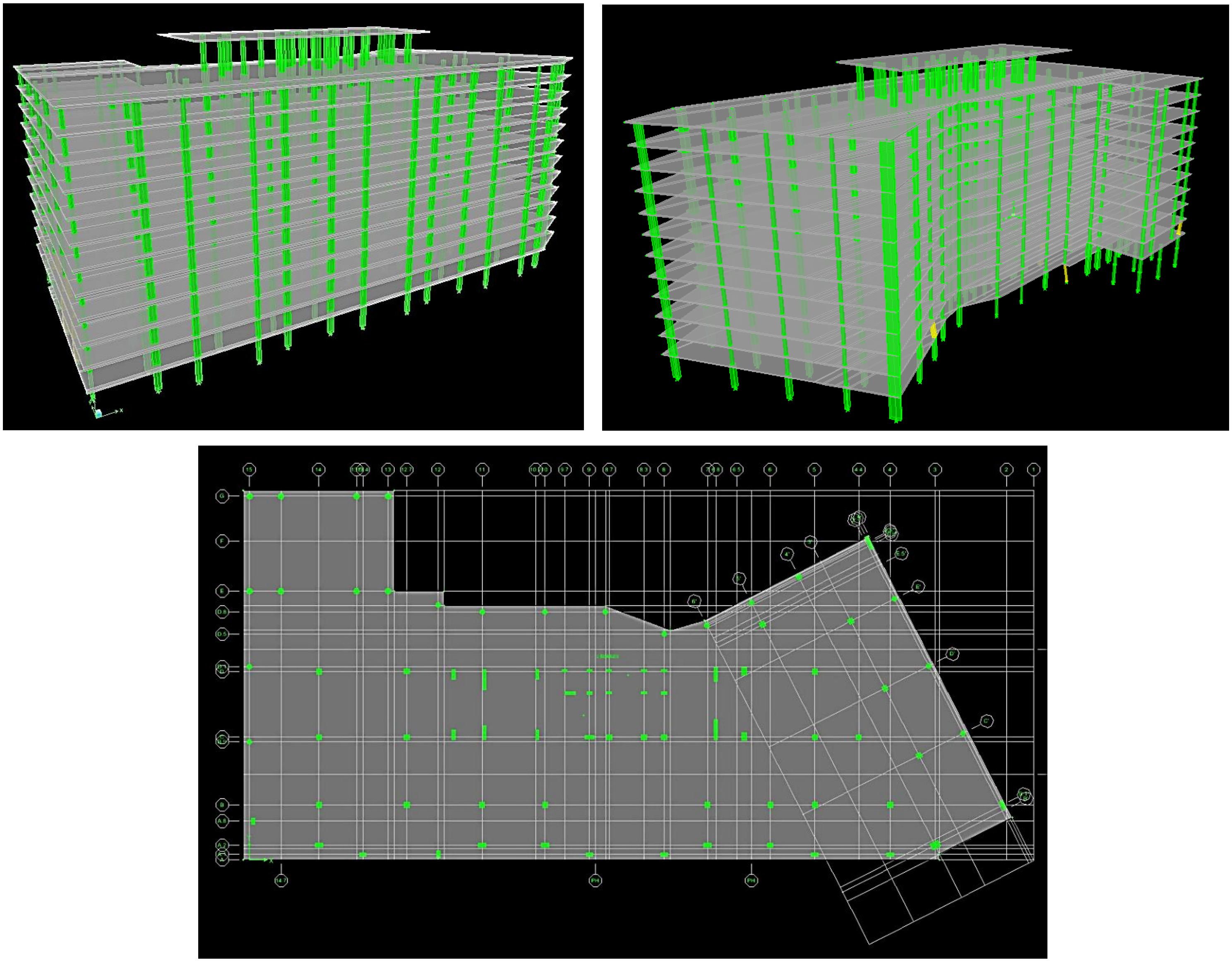


Figure 22 3D perspectives (top) and plan view (bottom) of the existing lateral system modeled in ETABS

To analyze the existing lateral system, two computer models were created using ETABS, which is a computer and structures modeling and analysis program. The models were used to determine:

- the structure's story drifts;
- each moment frame's stiffness;
- which combination of lateral loads controlled the lateral system's design

Several assumptions were made when creating the lateral models. The columns were modeled as line elements and were then assigned section properties according to the column schedule. The base supports were modeled as pin supports since the foundation consists of spread footings, which are not very rigid and thus do not carry much moment. Each floor level was modeled as an area element and assigned a rigid diaphragm since the floor system consists of a two-way flat slab system. In addition, material properties were modified by eliminating the self-mass from the material definitions and applying the actual floor mass to the diaphragm by using the Additional Area Mass function.

For the first model, as can be seen in Figure 22, a shell element with a membrane and bending thickness of 8" was used to define the slab, but this model was unstable because there was a connectivity issue between the slab (area element) and the columns (line elements); essentially the model was analyzed as a series of pin based columns without lateral stability (the slab) supporting the columns. This model failed to represent the slab as a part of the lateral system.

A second model was created to model the slab more accurately by creating concrete moment frames and representing the slabs with equivalent beams modeled as line elements. An equivalent frame was used to determine the beam width. Since the average column-to-column spacing is 30 feet, the column strip width was determined to be 15 feet; therefore a beam width of 15 feet was chosen to represent the slab. In addition, columns that did not align with the major column lines were shifted to align with them to better create representative moment frames. The equivalent concrete moment frames, shown in yellow, can be seen in Figures 23 and 24.

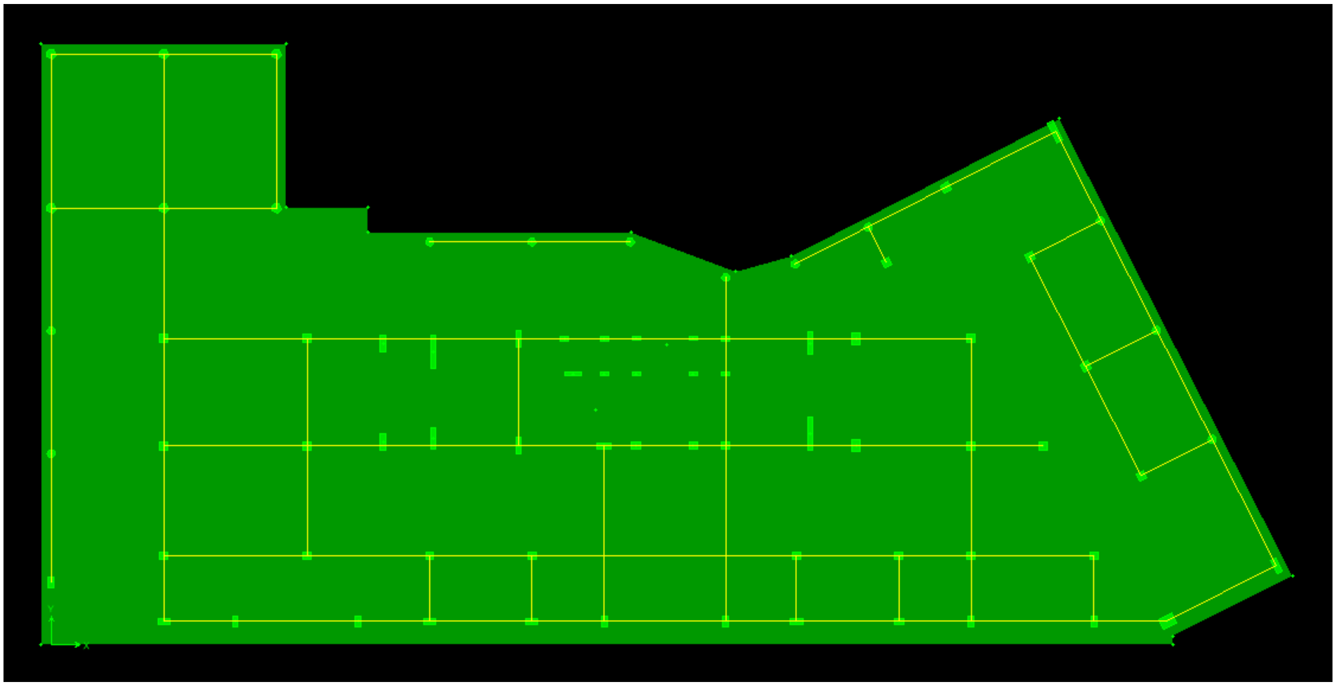


Figure 23 Plan view of the rigid floor diaphragm (green) and moment frame locations (yellow lines)

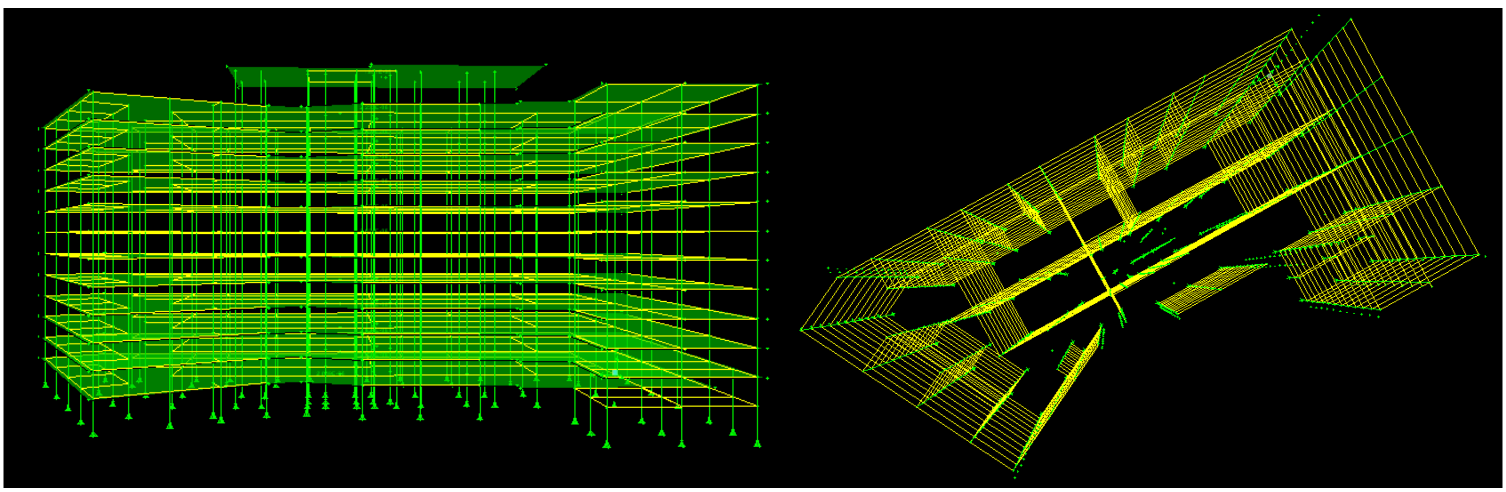


Figure 24 3D perspectives of equivalent frames with rigid diaphragm (left) and bird's eye view of the moment frames (right) where the vertical green lines are the columns and horizontal yellow lines are the beams

Relative Stiffness and Rigidity

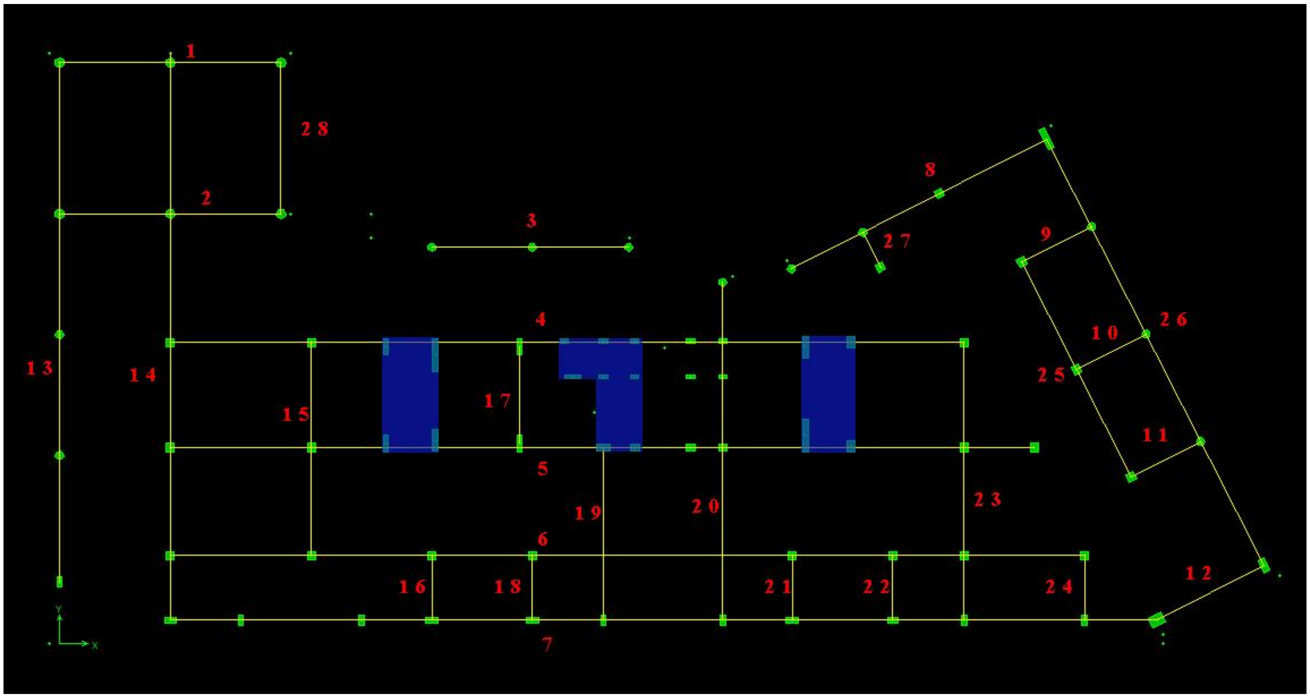


Figure 25 Moment frame layout with frame assignments

The distribution of lateral story forces at a given story level to the lateral force resisting systems at that story is done according to the relative stiffness of each lateral system. The stiffness of each system is determined by applying a unit load at the top story of each lateral force resisting system element. The stiffer the system, the more lateral load it will resist. The location and orientation of each moment frame can be seen in Figure 25. The stiffness of each frame was found in order to complete an analysis of both the direct and torsional shears, which will be discussed later in this technical report.

Each frame's stiffness was determined by applying a 1000 kip story load in the X –direction at the main roof level, which is the top level of the lateral force resisting system, and using ETABS to find the shear and displacement of each frame at the main roof level due to the 1000 kip story load. This same procedure was also applied to the Y-direction. The shear force and displacement in each frame at the main roof level were used to determine the frame's stiffness, K, where:

$K_i = P/\delta$, where P is the shear force in the frame at the main roof level and δ is the frame's displacement due to the 1000 k story load.

After determining each frame's stiffness, the relative stiffness was calculated by comparing the stiffness of each frame to the frame with the greatest stiffness. Firstly, the frame with the largest stiffness was set to have a relative stiffness of 1. The remaining frames' relative rigidity was determined by dividing each frame's stiffness by the highest stiffness. This procedure was also applied to the Y-direction. Each frame's relative stiffness can be seen in Table 12.

Frame 7 has the highest stiffness for the X-direction and frame 14 has the highest stiffness for the Y-direction. As a result, these two frames will resist the largest portion of the story lateral load in the X- and Y-directions.

Relative Stiffness of Concrete Moment Frames								
Frame	Displacement (12th story)		shear force (12th story)		Stiffness, K		Relative Stiffness (%)	
	X dir (in)	Y dir (in)	X dir (Kips)	Y dir (Kips)	X dir (kip/in)	Y dir (kip/in)	X dir	Y dir
1	0.981	1.33	48.01	0.45	48.93	0.34	18.25	0.28
2	0.963	1.33	45.84	3.57	47.62	2.68	17.76	2.21
3	0.959	1.33	32.96	0.17	34.39	0.13	12.83	0.11
4	0.947	1.33	143.76	1.05	151.82	0.79	56.63	0.65
5	0.934	1.33	191.68	2.20	205.25	1.65	76.56	1.36
6	0.921	1.33	144.19	2.93	156.62	2.20	58.42	1.81
7	0.913	1.33	244.70	1.51	268.10	1.14	100.00	0.93
8	0.972	1.33	46.73	24.14	48.08	18.18	17.93	14.97
9	0.961	1.33	14.74	4.02	15.34	3.03	5.72	2.49
10	0.948	1.33	18.65	13.87	19.68	10.45	7.34	8.60
11	0.935	1.33	18.88	13.04	20.20	9.83	7.54	8.09
12	0.919	1.33	37.18	2.90	40.44	2.19	15.08	1.80
13	0.981	1.33	10.15	116.66	10.35	87.60	3.86	72.11
14	0.981	1.33	4.50	161.71	4.59	121.47	1.71	100.00
15	0.981	1.33	0.46	72.05	0.47	54.14	0.17	44.57
16	0.981	1.33	0.78	22.33	0.80	16.79	0.30	13.82
17	0.981	1.33	1.13	44.34	1.15	33.34	0.43	27.45
18	0.981	1.33	0.22	27.93	0.22	21.00	0.08	17.29
19	0.981	1.33	2.44	22.95	2.49	17.26	0.93	14.21
20	0.981	1.33	0.05	62.94	0.05	47.36	0.02	38.99
21	0.981	1.33	0.08	28.29	0.08	21.29	0.03	17.53
22	0.981	1.33	1.01	20.66	1.03	15.56	0.38	12.81
23	0.981	1.33	5.72	89.70	5.83	67.54	2.17	55.60
24	0.981	1.33	8.21	18.00	8.37	13.56	3.12	11.16
25	0.930	1.33	15.64	71.47	16.81	53.82	6.27	44.31
26	0.919	1.33	15.00	109.37	16.31	82.43	6.09	67.86
27	0.956	1.33	7.83	10.35	8.19	7.79	3.05	6.41
28	0.981	1.33	0.97	36.74	0.99	27.61	0.37	22.73

∑ Shears in X-direction=	1062 ≈1000
∑ Shears in Y-direction=	985 ≈1000

Table 12 Relative stiffness of the concrete moment frames

Load Combinations

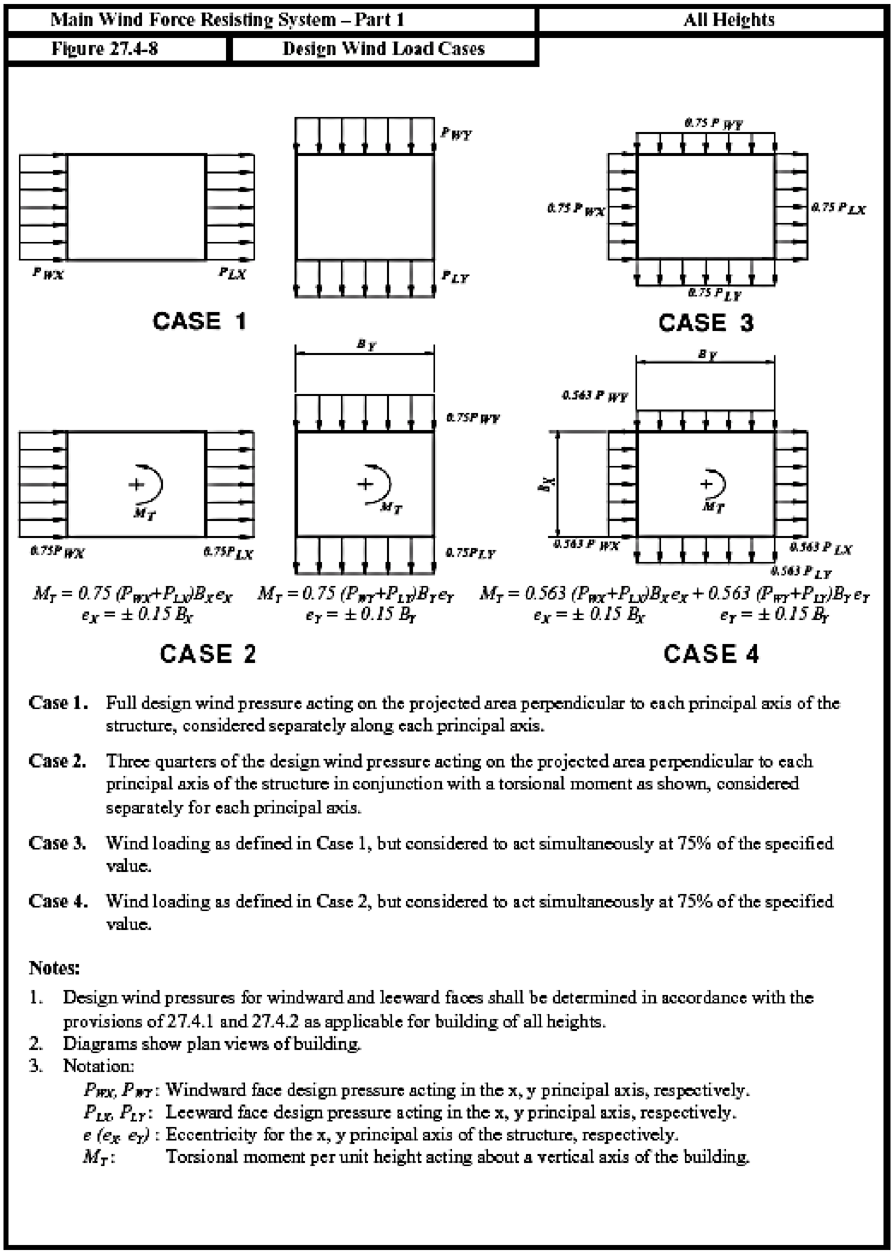
To determine which lateral loads or combinations of lateral loads controlled the existing lateral system's design, several load combinations were considered using ASCE 7-10, as can be seen in Figure 26.

2.3.2 Basic Combinations. Structures, components, and foundations shall be designed so that their design strength equals or exceeds the effects of the factored loads in the following combinations:

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

Figure 26 Load Resisting Factor Design (LRFD) load combinations from Chapter 2 of ASCE 7-10

First, the four possible wind load cases were analyzed to determine which controlled the lateral system. This was done by using the ETABS model to find the shear forces in each frame due to each wind case. The 12th (main roof) story was used as a trial level to find the shear forces in the frames. The wind case that resulted, on average, in the highest shear forces was selected as the controlling wind case. The four possible wind cases can be seen in Figure 27. All four wind load cases can be found in Appendix B and the forces in each frame for each wind case can be found in Appendix D.



Main Wind Force Resisting System – Part 1		All Heights
Figure 27.4-8	Design Wind Load Cases	

$M_T = 0.75\ (P_{WX}+P_{LX})B_X e_X$
$e_X = \pm\ 0.15\ B_X$

$M_T = 0.75\ (P_{WY}+P_{LY})B_Y e_Y$
$e_Y = \pm\ 0.15\ B_Y$

$M_T = 0.563\ (P_{WX}+P_{LX})B_X e_X + 0.563\ (P_{WY}+P_{LY})B_Y e_Y$
$e_X = \pm\ 0.15\ B_X$ $e_Y = \pm\ 0.15\ B_Y$

CASE 2

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 27.4.1 and 27.4.2 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:

P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
$e\ (e_X, e_Y)$: Eccentricity for the x, y principal axis of the structure, respectively.
M_T: Torsional moment per unit height acting about a vertical axis of the building.

Figure 27 Design wind load cases from ASCE 7-10

To simplify the analysis, the only load combinations that were considered in this technical report were those that include wind and/ or seismic. This includes combinations 4-7 in Figure 26. In addition, only the lateral loads were compared, therefore the only combinations compared were 1.0E and 1.6W.

Based on the four wind cases, it was found that case 1 in the N-S direction controlled. This controlling wind case was then compared to the N-S and E-W seismic loads. The controlling wind load case was multiplied by a factor of 1.6 and the seismic loads were multiplied by a factor of 1.0.

Using ETABS, it was found that the North-South case 1 wind lateral load controlled the design in the N-S direction and seismic controlled the design in the East-West direction. This is consistent with the base shears discussed earlier in this report, where the base shear due to the North-South wind was 1401 kips and the base shear due to the seismic loads in the East-West direction was 1001 kips. Appendix D shows the forces in each frame at the 8^{th} story due to the checked load combinations.

Building Torsion

When the Center of Mass (COM) and Center of Rigidity (COR) do not coincide, the building will be subjected to torsional effects caused by the lateral loads. These torsional effects must be accounted for in design. To determine the total building torsion, one must consider the torsion due to the location difference between the COR and COM and accidental torsion.

The accidental torsion is calculated by multiplying the lateral load by 5% of the building width, where the building width is perpendicular to the acting lateral load. The total torsion the building is subjected to is determined by adding the torsional moment to the accidental torsional moment. The total torsional moments were found in both the North-South and East-West directions, which can be seen in Tables 13 and 14. The North-South lateral load is controlled by the North-South wind load case 1 and the East-West direction is controlled by the seismic loads. The North-South direction is subjected to a 28,496 k-ft torsional moment and the E-W direction is subjected to a 9,431 k-ft torsional moment.

Building Torsion, N-S Direction (Wind Controlling)							
Level	Story Force	Center of Mass (ft)	Center of Rigidity (ft)	eccentricity, e (ft)	Torsional Moment, M_t (k-ft)	Accidental Moment, M_a (k-ft)	Total Moment, M_T (k-ft)
Main Roof	58.68	124.2	136	-11.8	-692.4	891	198.9
12	115.69	145.94	136	9.94	1150.0	1757	2907.3
11	114.04	145.94	136	9.94	1133.6	1732	2865.8
10	112.38	145.94	136	9.94	1117.1	1707	2824.1
9	109.73	145.94	136	9.94	1090.7	1667	2757.5
8	107.74	145.94	136	9.94	1070.9	1637	2707.5
7	105.43	145.94	136	9.94	1048.0	1601	2649.5
5	102.78	145.94	136	9.94	1021.6	1561	2582.9
6	100.13	145.94	136	9.94	995.3	1521	2516.3
4	97.14	145.94	136	9.94	965.6	1476	2441.1
3	92.17	145.94	136	9.94	916.2	1400	2316.2
2	93.35	137.33	134	3.33	310.9	1418	1728.8
						∑=	**28496.0**

Accidental Torsion, M_a (k-ft)- Y direction (N-S)			
Level	Bx (ft)	0.05Bx (ft)	M_a
Main Roof	303.8	15.19	891
12	303.8	15.19	1757
11	303.8	15.19	1732
10	303.8	15.19	1707
9	303.8	15.19	1667
8	303.8	15.19	1637
7	303.8	15.19	1601
5	303.8	15.19	1561
6	303.8	15.19	1521
4	303.8	15.19	1476
3	303.8	15.19	1400
2	303.8	15.19	1418

Table 13 Total building torsion in the N-S direction (top) and the accidental torsion (bottom)

Building Torsion, E-W Direction (Earthquake Controlling)							
Level	Story Force	Center of Mass (ft)	Center of Rigidity (ft)	eccentricity,e (ft)	Torsional Moment, M_t (k-ft)	Accidental Moment, M_a (k-ft)	Total Moment, M_T (k-ft)
Main Roof	150	57.48	55	2.48	372	1102	1474
12	158	57.48	55	2.48	391.84	1159	1550.84
11	138	57.48	55	2.48	342.24	1018	1360.24
10	120	57.48	55	2.48	297.6	881	1178.6
9	102	57.48	55	2.48	252.96	751	1003.96
8	85	57.48	55	2.48	210.8	627	837.8
7	69	57.48	55	2.48	171.12	509	680.12
5	54	57.48	55	2.48	133.92	398	531.92
6	40	57.48	55	2.48	99.2	296	395.2
4	28	57.48	55	2.48	69.44	202	271.44
3	16	57.48	55	2.48	39.68	120	159.68
2	7	51.87	59.4	-7.53	-52.71	40	-12.71
						∑=	**9431.09**

Accidental Torsional,M_a (k-ft)- X direction (E-W)				
level i	By ft	5%By ft	Ax	M_T k-ft
Main Roof	147	7.35	1.0	1102
12	147	7.35	1.0	1159
11	147	7.35	1.0	1018
10	147	7.35	1.0	881
9	147	7.35	1.0	751
8	147	7.35	1.0	627
7	147	7.35	1.0	509
6	147	7.35	1.0	398
5	147	7.35	1.0	296
4	147	7.35	1.0	202
3	147	7.35	1.0	120
2	121.75	6.09	1.0	40

Table 14 Total building torsion in the E-W direction (top) and the accidental torsion (bottom)

Lateral Load Distribution

Lateral force resisting systems resist lateral loads through direct shear and torsional shear. For 1000 Connecticut Avenue, to determine the portion of the story lateral force resisted by each frame, sample calculations were completed by solving for both the direct and torsional shears in each frame. The total shear in each frame was determined by adding the direct shear to the torsional shear.

Direct Shear

The frames that are parallel to the direct shear will participate in resistance. For example, the lateral loads acting in the North-South direction will be resisted directly by frame 8-28 and the lateral loads acting in the East-West direction will be resisted directly by frame 1-12 and 25-27.

The direct shear of each frame was calculated by multiplying the relative stiffness of each frame by the lateral load. The relative stiffness represents the portion of the story lateral load resisted by the frame.

Relative stiffness= $= \frac{\mathrm{Ki}}{\sum \mathrm{ki}}$

Where,

K_i is the stiffness of the frame parallel to the lateral load

In the North-South direction, wind load case 1 was the controlling direct shear and seismic was the controlling direct shear in the East-West direction. A sample distribution of the lateral force acting on the 10^{th} level can be found in table 15.

Torsional Shear

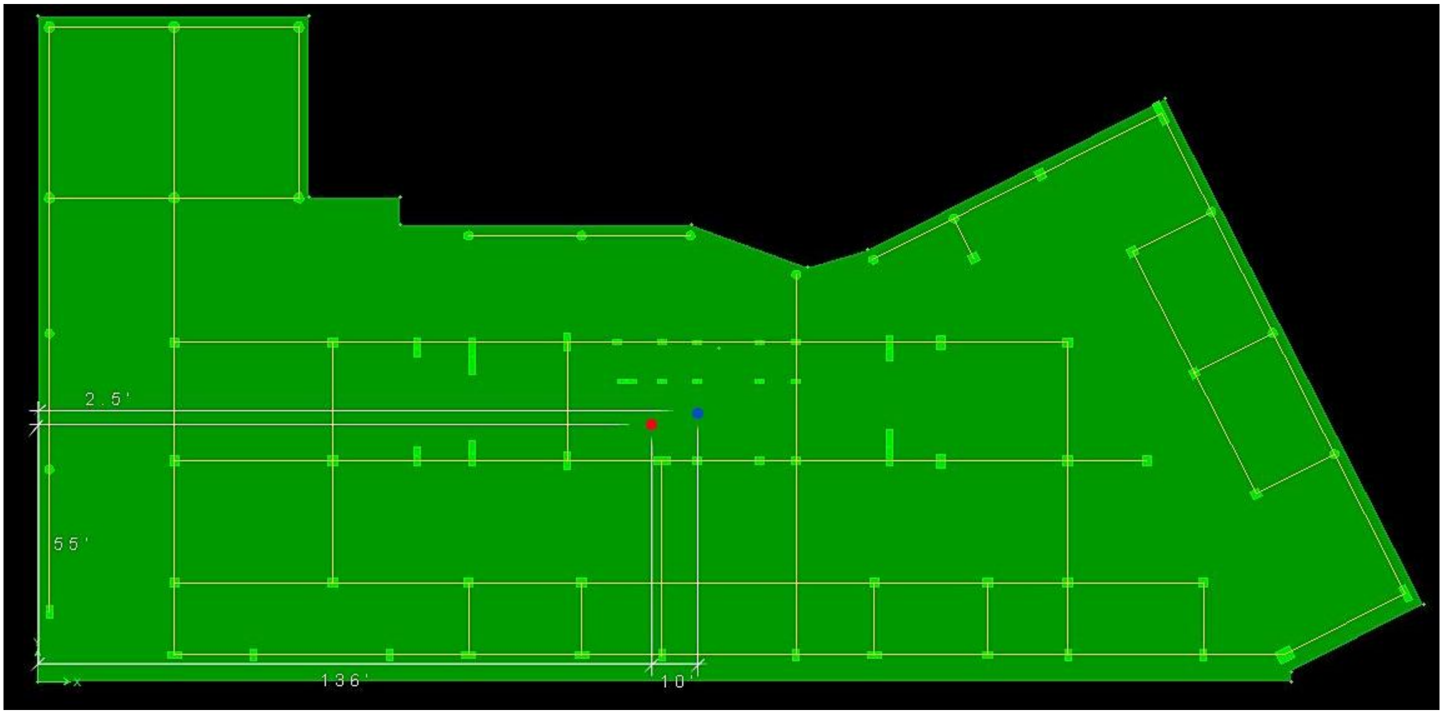


Figure 28 Plan view showing the location of the Center of Mass (blue dot) and the Center of Rigidity (red dot)

If the Center of Mass and Center of Rigidity do not coincide, then the lateral loads will cause torsional effects; lateral loads act through the COM, but are resisted through the COR. Contrast to direct shear, all of the frames will participate in resisting these torsional effects. The torsional shear in each frame was first determined by finding the eccentricity between the COM and COR. Next, the distance between the frame and COR was determined where the distance is the moment arm between the COR and the frame. The torsional Shear equation with corresponding variable definitions can be seen below.

Torsional Shear, $V_i = \frac{V e d_i K_i}{\sum K_i d_i^2}$

Where,

V- story lateral load
e- eccentricity (distance between the Center of Mass and Center of Rigidity)
Ki- stiffness of the lateral force resisting system element
di- moment arm between COR to the lateral force resisting system element

To determine d for frames 8-12 and 25-27, the frames had to be broken down into X and Y components since they are located at an angel. Frames 8-12 are at a 27 degree angel from the positive X-axis and frames 25-27 are at a 117 degree angel from the positive X-axis. The frames separated into their corresponding X and Y components can be seen in Figure 29. A sample calculation of the torsional shears in each frame on the 10th level can be seen in Table 15. Graphs showing the direction of the direct and torsional shears acting on each frame due to a lateral load applied in the North-South direction can be found in Appendix E.

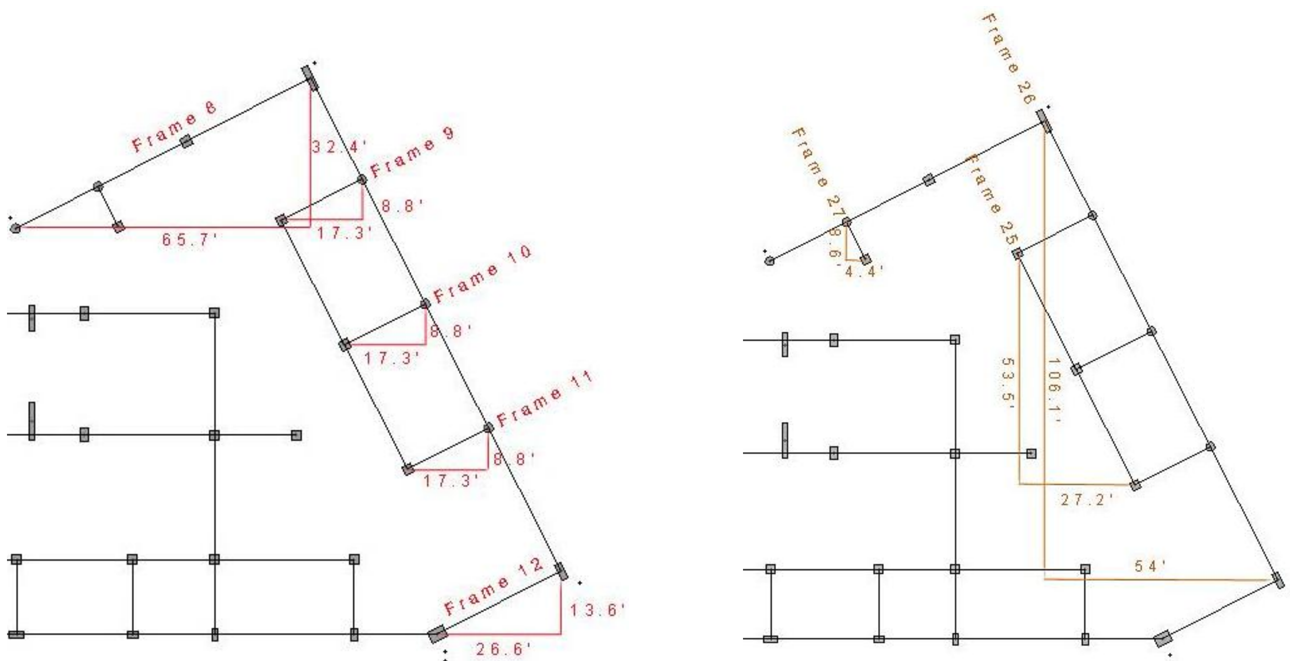


Figure 29 Frames 8-12 (left) and frame 25-27 (right) separated into their corresponding X-and Y-components

North- South Wind									
Frame	Lateral Force (kips)	Stiffness , K (kips/in)	e_x (ft)	e_y (ft)	d (ft)	$K*d^2$	Direct Shear (kips)	Torsional Shear (kips)	Total Shear (kips)
1	112.40	0.34	10	2.5	89.5	2710.2	0	0.0055	0.005
2	112.40	2.68	10	2.5	51.8	7202.4	0	0.0251	0.025
3	112.40	0.13	10	2.5	43.5	241.9	0	0.0010	0.001
4	112.40	0.79	10	2.5	19.9	312.6	0	0.0028	0.003
5	112.40	1.65	10	2.5	-6.3	65.7	0	-0.0019	-0.002
6	112.40	2.20	10	2.5	-33.2	2428.2	0	-0.0132	-0.013
7	112.40	1.14	10	2.5	-49.2	2748.2	0	-0.0101	-0.010
8	112.40	18.18	10	2.5	43.5	34402.0	2.8	0.1428	2.934
9	112.40	3.03	10	2.5	36.7	4078.4	0.5	0.0201	0.485
10	112.40	10.45	10	2.5	13	1765.9	1.6	0.0245	1.629
11	112.40	9.83	10	2.5	-15.1	2240.4	1.5	-0.0268	1.482
12	112.40	2.19	10	2.5	-49.2	5290.4	0.3	-0.0194	0.316
13	112.40	87.60	10	2.5	-136	1620184.6	13.4	-2.1506	11.296
14	112.40	121.47	10	2.5	-105.9	1362262.9	18.6	-2.3222	16.325
15	112.40	54.14	10	2.5	-70.9	272134.5	8.3	-0.6929	7.618
16	112.40	16.79	10	2.5	-40.9	28081.8	2.6	-0.1239	2.453
17	112.40	33.34	10	2.5	-19.6	12807.1	5.1	-0.1180	5.000
18	112.40	21.00	10	2.5	-15.9	5310.2	3.2	-0.0603	3.164
19	112.40	17.26	10	2.5	1.8	55.9	2.6	0.0056	2.655
20	112.40	47.36	10	2.5	31.5	46989.1	7.3	0.2693	7.539
21	112.40	21.29	10	2.5	48.8	50695.2	3.3	0.1875	3.455
22	112.40	15.56	10	2.5	73.8	84722.4	2.4	0.2072	2.595
23	112.40	67.54	10	2.5	91.5	565475.2	10.4	1.1156	11.484
24	112.40	13.56	10	2.5	121.5	200151.0	2.1	0.2974	2.379
25	112.40	53.82	10	2.5	109	639457.1	8.3	1.0590	9.321
26	112.40	82.43	10	2.5	113.8	1067442.6	12.7	1.6933	14.346
27	112.40	7.79	10	2.5	69.4	37523.0	1.2	0.0976	1.294
28	112.40	27.61	10	2.5	-78.4	169678.1	4.2	-0.3907	3.847
					$\sum K*d^2$=	6226457.0			111.6

Table 15 Sample calculation of direct and torsional shears in each frame for a story lateral force acting on the 10th level

Story Drift and Lateral Displacement

The lateral displacements and story drifts were obtained from ETABS. This was done by using only un-factored wind and seismic loads. The inter-story drifts due to the un-factored wind load case 1 were compared to the H/400 allowable displacement, from ASCE 7-10, where H is the story-to-story- height. For the un-factored seismic loads, the inter-story drifts were compared to 0.020H from table 12.12-1 of ASCE 7-10, as can be seen in Figure 30. 1000 Connecticut Avenue has a risk category of II and has a reinforced concrete moment frame structural system, therefore the allowable drift will be 0.02H, where H is the story-to-story height.

Table 12.12-1 Allowable Story Drift, $\Delta_a^{a,b}$

Structure	Risk Category I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less above the base as defined in Section 11.2, with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures[d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$\boxed{0.020h_{sx}}$	$0.015h_{sx}$	$0.010h_{sx}$

[a] h_{sx} is the story height below Level x.
[b] For seismic force-resisting systems comprised solely of moment frames in Seismic Design Categories D, E, and F, the allowable story drift shall comply with the requirements of Section 12.12.1.1.
[c] There shall be no drift limit for single-story structures with interior walls, partitions, ceilings, and exterior wall systems that have been designed to accommodate the story drifts. The structure separation requirement of Section 12.12.3 is not waived.
[d] Structures in which the basic structural system consists of masonry shear walls designed as vertical elements cantilevered from their base or foundation support which are so constructed that moment transfer between shear walls (coupling) is negligible.

Figure 30 Table of allowable story drift for seismic loads

The serviceability for both the wind and seismic loads were found to be within the allowable limits. The story displacements and story drifts in the N-S and E-W directions can be found in Table 16.

Story Displacement/ Drift Due to Unfactored Wind Loads						
Story	Height Above Grade	Actual Displacement		H/400	Story Drift	
	(ft)	X (in)	Y (in)	(in)	X (in)	Y (in)
Main Roof	129.42	0.3966	1.2078	0.31875	0.012	0.0532
11	118.79	0.3846	1.1546	0.31875	0.0143	0.0553
10	108.17	0.3703	1.0993	0.31875	0.018	0.0625
9	97.64	0.3523	1.0368	0.31875	0.0217	0.0716
8	86.92	0.3306	0.9652	0.31875	0.0253	0.081
7	76.29	0.3053	0.8842	0.31875	0.0281	0.089
6	65.67	0.2772	0.7952	0.31875	0.0312	0.0973
5	55.04	0.246	0.6979	0.31875	0.0341	0.1053
4	44.42	0.2119	0.5926	0.31875	0.0361	0.1125
3	33.79	0.1758	0.4801	0.31875	0.0383	0.1202
2	23.17	0.1375	0.3599	0.31875	0.0467	0.1338
1	12.54	0.0908	0.2261	0.3762	0.0908	0.2261

Story Displacement/ Drift Due to Unfactored Seismic Loads						
Story	Height Above Grade	Actual Displacement		0.02H	Story Drift	
	(ft)	X (in)	Y (in)	(in)	X (in)	Y (in)
Main Roof	129.42	0.7061	0.9809	0.2125	0.0198	0.04
11	118.79	0.6863	0.9409	0.2125	0.029	0.0486
10	108.17	0.6573	0.8923	0.2125	0.0379	0.0589
9	97.64	0.6194	0.8334	0.2125	0.0456	0.0686
8	86.92	0.5738	0.7648	0.2125	0.0521	0.0766
7	76.29	0.5217	0.6882	0.2125	0.0556	0.0821
6	65.67	0.4661	0.6061	0.2125	0.0593	0.0866
5	55.04	0.4068	0.5195	0.2125	0.1619	0.0895
4	44.42	0.2449	0.43	0.2125	-0.0378	0.0903
3	33.79	0.2827	0.3397	0.2125	0.0617	0.0904
2	23.17	0.221	0.2493	0.2125	0.0717	0.0952
1	12.54	0.1493	0.1541	0.2125	0.1493	0.1541

Table 16 Story displacements/drifts due to un-factored wind and seismic loads

Overturning and Stability Analysis

A building's foundation must be designed to support both axial loads and bending moments caused by the lateral loads. The support base of lateral force resisting columns is subjected to uplift forces caused by the lateral loads. As a result, these uplift forces subject the building to overturning moments.

1000 Connecticut Avenue's foundation is comprised of spread footings, which behave as pinned connections due to their low rigidity. As a result, the foundation does not participate in resisting moments caused by the lateral loads. The concrete slab combined with the columns behaves as a reinforced concrete moment frame where the slab-to-column connection is rigid. The rigid connection between the slab and columns are designed to resist the moments due to the lateral loads.

Through the analysis of the lateral system, the foundation was checked to determine if it is adequate to carry the moment due to the lateral forces on the slab, which transfers the load to the columns. The overturning moments were found by using the controlling lateral loads in each direction. It was determined in preceding sections of this technical report that wind load case 1 was the controlling lateral load for the North-South direction and the seismic load was the controlled the East-West direction. The wind and seismic loads were used to calculate the overturning moments by multiplying the lateral loads by the story height. The resisting moments were calculated by multiplying the total building weight by half of the building length, where the building length is in the direction in which the resisting moment is acting.

The overturning moment has to be less than 2/3 the resisting moment due to the dead load. It was found that the resisting moments in both directions were much greater than the overturning moments. Therefore, it was found that the slab-to-column moment frame systems below grade are adequate to carry the moments due to the lateral loads. Since the spread footings will behave as pinned connections, the columns will not transfer any moment to the foundation. Therefore the rigid connection between the slab and columns will carry the overturning moment. The overturning and resisting moments can be seen in Table 17.

Overturning Moment					
		N-S Wind		E-W Seismic	
Floor	Height (ft)	Lateral Force (kips)	Moment (k-ft)	Lateral Force (kips)	Moment (k-ft)
PH Roof	148	143	21164.0	34	5032.0
Main Roof	129.42	59	7635.8	150	19413.0
12	118.79	116	13779.6	158	18768.8
11	108.17	114	12331.4	138	14927.5
10	97.64	112	10935.7	120	11716.8
9	86.92	110	9561.2	102	8865.8
8	76.29	108	8239.3	85	6484.7
7	65.67	105	6895.4	69	4531.2
6	55.04	103	5669.1	54	2972.2
5	44.42	100	4442.0	40	1776.8
4	33.79	97	3277.6	28	946.1
3	23.17	92	2131.6	16	370.7
2	12.54	93	1166.2	7	87.8
Overturning Moment=		∑=	**107229**		**95893**

Resisting Moment				
Bulding Weight kips)	N-S Wind		E-W Seismic	
	Length- Y direction (ft)	Moment (k-ft)	Length- X direction (ft)	Moment (k-ft)
56577	147	**4158410**	314.6	**8899562**

Table 17 Overturning and resisting moments in the N-S and E-W directions

Frame Checks

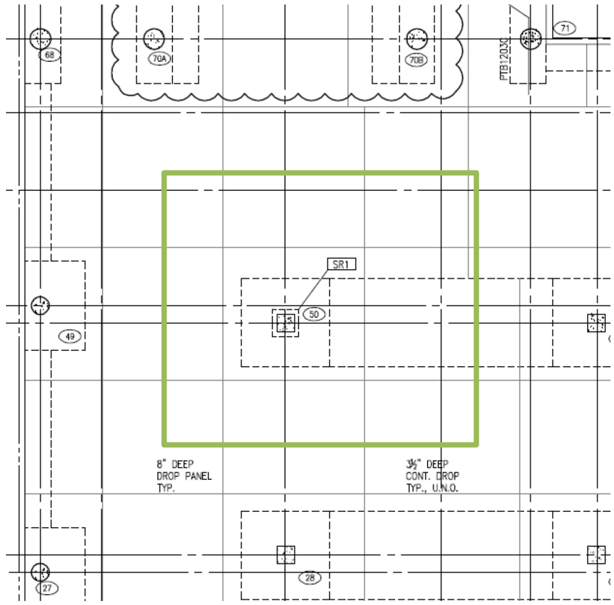


Figure 31 Critical column member with approximate tributary area

Spot checks were performed on column 50 on the 1st level, as can be seen in Figure 31. This column was considered a critical member because it supports a large tributary area of 970.3 ft^2 and as a result is subjected to a large axial load of P_u =2362 kips, which was calculated in Technical Report 1. The column was checked for both axial and bending capacity.

To analyze the column, an interaction diagram was created to determine whether the column was able to support the required axial load, P_u, and bending moment, M_u. The interaction diagram design values, ϕP_n and ϕM_n, were compared to the moment obtained from ETABS and the 2362 kip axial load. It was found that the column is subjected to an in-plane bending moment of M_u=176 k-ft. This moment is due to the factored wind load case of 1.6W in the North-South direction.

After the analysis, it was shown that column 50 was adequate to support both the axial and bending loads. The spot check calculations for this column can be found in Appendix F.

Conclusion

Technical Report 3 analyzed 1000 Connecticut Avenue, NW Office Building's existing lateral system and confirmed its design by determining which combination of lateral loads controlled the lateral system design; checking the story displacement and story drifts for serviceability; analyzing the overturning moments due to the lateral loads and the resisting moments due to the total building weight; and spot checking critical members for strength adequacy.

The wind loads were determined by using Analytical Procedure (method 2) outlined in ASCE 7-10 and the seismic loads were determined by using the Equivalent Lateral Force Procedure outlined in ASCE 7-10. The wind loads were calculated for both the North-South and East-West directions and it was found that the lateral forces due to the wind load were greatest in the N-S direction, resulting in a 1401 kip base shear. One analysis was completed for determining the seismic story forces since the lateral force resisting system consists of a reinforced concrete moment frame in both the N-S and E-W directions. The calculated seismic base shear of 1001 kips compared to the design base shear of 645 kips resulted in a 55 % error. This shows that the dead load assumptions and analysis simplifications were conservative.

Further, a computer model of the lateral force resisting system was created in ETABS. The model was used to determine which combination of lateral loads controlled the lateral system's design; frame stiffness; and, to check the serviceability by determining the lateral displacements/story drifts due to the un-factored controlling lateral forces in both the N-S and E-W directions. It was found that the N-S wind load case 1 controlled the lateral load in the N-S direction and the seismic was the controlling lateral load in the E-W direction. Using the controlling lateral loads to determine drifts, it was found that the lateral displacements and story drifts were within the allowable code limits.

In addition, it was found that the columns do not transfer moments to the foundation since the spread footings will behave like pinned connections due to the footings' low rigidity. It was determined that the slab-to-column moment frame systems below grade are adequate to carry the moments due to the lateral loads.

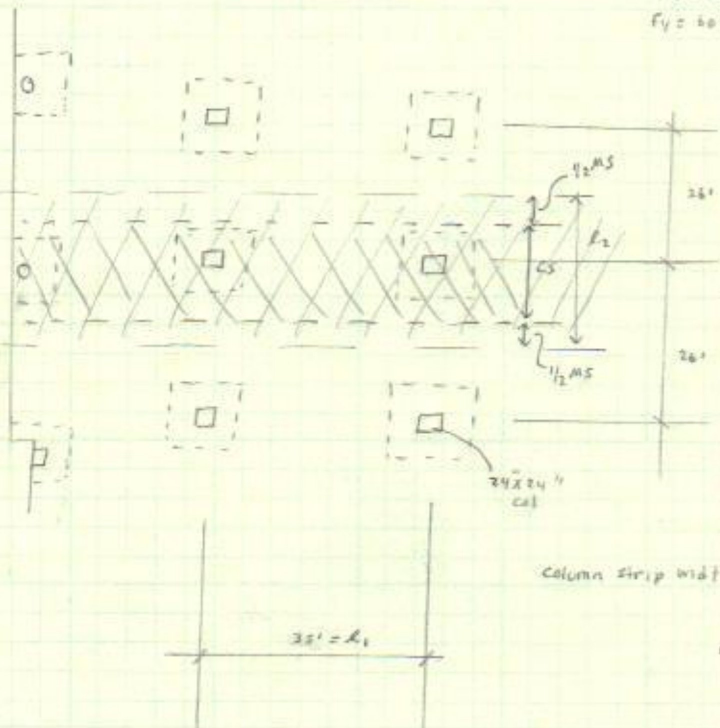
Lastly, a member spot check was performed on column 50, an interior column. The column was checked for both axial load and moment. ETABS was used to determine the in-plane bending moment acting on the column due to the factored wind load in the N-S direction. An interaction diagram was created to compare P_u and M_u to ϕP_n and ϕM_n and the column was found to be adequate to carry the combined axial and bending load.

Appendix A: Gravity Load Calculations

Gravity spot check – flat slab interior panel | Tech 1 | page 1 of 4

check typical interior flat slab panel in E-W direction for slab thickness and column strip reinforcement

$f'_c = 5000$ psi
$f_y = 60$ ksi



column strip width = min $\left| \frac{\ell_1}{2} = 17.5' \right.$, $\frac{\ell_2}{2} = 13'$

step 1: slab thickness

from Table 9.5(c) in ACI 318-08:

interior panel with drop panels and $f_y = 60$ ksi

$$t_{min} = \frac{\ell_n}{36} = \frac{(35-2)ft\,(12\,in/ft)}{36} = 11''$$

step 2: determine moments in column strip

simplifying assumption – determine column strip moments by analyzing the slab as a flat plate system (neglecting the drop panels)

Gravity spot check – flat slab interior panel	Tech 1	page 2 of 4

– total load

dead:

slab – 150 pcf ($^{11}/_{12}$ ft) = 137.5 psf

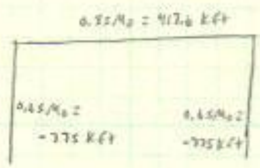
SDL = 10 psf

live load – 100 psf

$w_u = 1.2\,(137.5 + 10)\,\text{psf} + 1.6\,(100\,\text{psf}) = 337\,\text{psf}$

$– M_0 = \dfrac{w_u \ell_2 \ell_n^2}{8} = \dfrac{1}{8}(337\,\text{psf})(26\,\text{ft})(33\,\text{ft})^2 = 1193\,\text{k}\cdot\text{ft}$

– distribute M_0 longitudinally using ACI direct design moment coefficients, sect 13.6.3.2



Interior span

– transverse distribution of moments in column strip → from ACI sect 13.6.4

percentage of longitudinal moment going to col. strip

negative moment @ interior support

$\dfrac{\ell_2}{\ell_1} = \dfrac{26}{35} = 0.74$ $\qquad \dfrac{\alpha \ell_2}{\ell_1} = 0$ (no longitudinal beam support)

	ℓ_2/ℓ_1		
	0.5	0.74	1.0
$\dfrac{\alpha \ell_2}{\ell_1}$	75	75	75

– 775 kft —— 75 % to col. strip = –581.3 kft

positive moment

$\dfrac{\ell_2}{\ell_1} = 0.74 \qquad \alpha \ell_2/\ell_1 = 0$

	0.50	0.74	1.0
$\alpha \ell_2/\ell_1$	60	60	60

+ 417.6 kft —— 60 % to col. strip = 250.6 kft

Gravity Spot check – flat slab interior panel | Tech 1 | page 3 of 4

step 3: reinforcement design of column strip in interior panel

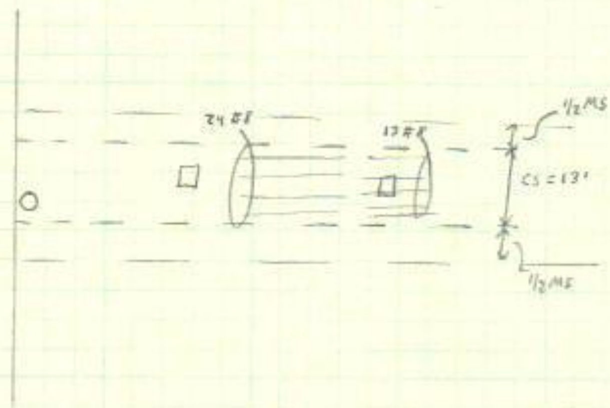
item no.	description	interior span M_u^-	interior span M_u^+
1.	M_u (k-ft)	-775	+417.6
2.	width of strip, b (in)	156"	156"
3	effective depth, d (in) $11 - 0.75 - 0.5(0.625)$ t; clear cover; reinf. #5 bar dia.	9.94"	9.94"
4.	$M_n = \frac{M_u}{0.9}$ (k-ft)	-861	464
5	$R = \frac{M_n}{bd^2}$ (psi) $= \frac{M_n \times 12000}{156''(9.94'')^2}$	670	361
6.	ρ from table A-3 in Reinforced concrete, 8th edit from interpolation, $\rho = \left(\frac{R-R_1}{R_2-R_1}\right)(\rho_2-\rho_1)+\rho_1$	0.0122	0.0063
7.	$A_s = \rho bd$ (in²) $= \rho(156)(9.94)$	18.92	9.77
8.	$A_{s,min} = 0.0018bt$ $= 0.0018(156)(11)$	3.1	3.1
9.	$N = \frac{\text{larger of 7 or 8}}{0.31}$ #5 bar	61	31.5 ≅ 32
10.	$N_{min} = \frac{\text{width of strip}}{2t}$ $= \frac{156}{2(11)}$	7.09 ≅ 8	8

To minimize the number of reinforcing bars required, increase bar size to #8.

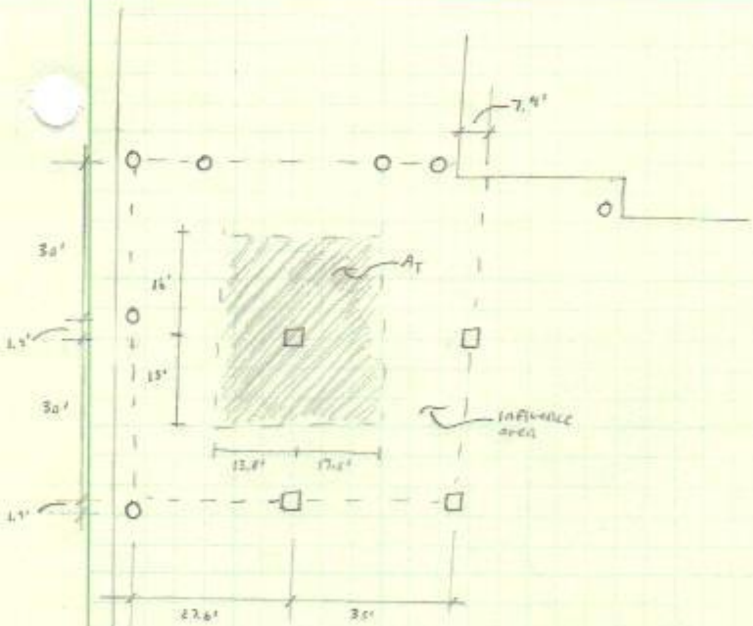
Therefore to resist the positive moment in the middle of the interior slab span, use $N = \frac{18.92}{0.79} = 24$ bars

To resist the negative moment at the supports use $N = \frac{9.77}{0.79} = 13$ bars

Gravity spot check - Flat slab interior panel | Tech 1 | page 4 of 4



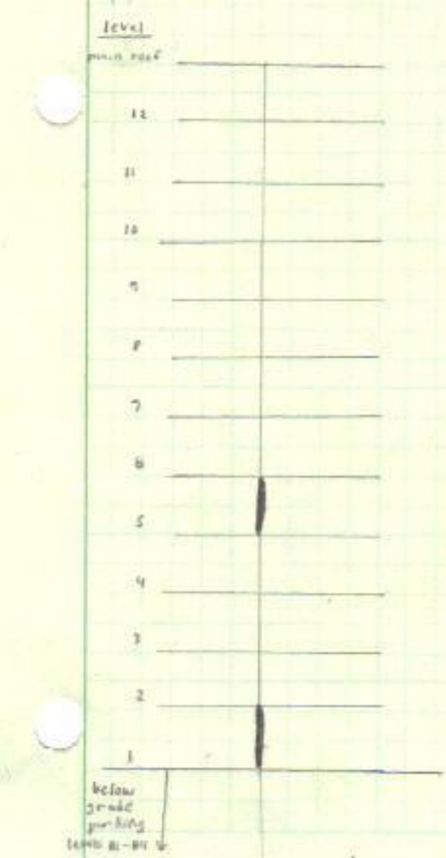
Gravity spot check – column #50 | Tech 1 | Page 1 of 6



tributary area / floor, A_T

$A_T = (31.3')(31') = 970.3 \text{ ft}^2$

influence area/floor $= (70')(61.9') = 4333 \text{ ft}^2$



Loads

main roof: $L_r = 150$ psf ← pump / generator room

assumption: $L_r = 150$ psf > $S = 20$ psf, ∴ use L_r in all applicable load combinations

DL = 100 psf + 10 psf = 110 psf

levels 2–12: LL = 100 psf

DL = 100 psf + 10 psf = 110 psf

Analysis procedure:

1. determine axial load and unbalanced moment on column at levels 1 and 5
2. determine preliminary size of column at levels 1 and 5 based on gravity loads

Gravity spot check - column #50 | Tech 1 | page 2 of 6

Axial load on column at level 1

- load above level 1: roof + 11 floors

$LL_{reduction}$ factor = max $\begin{cases} 0.4 \\ 0.25 + \dfrac{15}{\sqrt{4 \times 4 \times 777}} = 0.32 \end{cases}$ ∴ use $LL_{red} = 0.40$

$P_L = LL_{red} \times LL \times A_T = 0.40\,(100\text{ psf})(11\text{ flrs})(970.3\text{ ft}^2/\text{flr}) = 427\text{ K}$

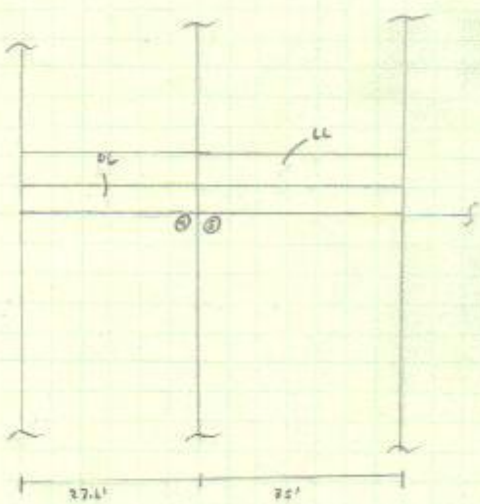
$P_D = 110\text{ psf}\,(970.3\text{ ft}^2/\text{flr})(11\text{ flrs}) + 110\text{ psf}\,(970.3\text{ ft}^2/\text{roof}) = 1281\text{ K}$

$P_{Lr} = 150\text{ psf}\,(970.3\text{ ft}^2/\text{roof}) = 145.5\text{ K}$

$P_u = 1.2P_D + 1.6P_L + 0.5L_r = 1.2(1281\text{ K}) + 1.6(470\text{K}) + 0.5(145.5\text{ K}) = 2362\text{ K}$

Unbalanced moment for column at level 1

use the ACI moment coefficient method to determine the maximum moments and shears at the critical sections



5 negative moment at exterior face of 1st interior support

$FEM = \dfrac{w_u \ell_n^2}{10}$ - more than 2 spans

8 negative moment at other faces of interior supports

$FEM = \dfrac{w_u \ell_n^2}{11}$

note: ℓ_n is the clear distance between the supports, but for preliminary sizing purposes, I will use the ctr-to-ctr distance with the assumption that the column sizes are unknown at this stage of preliminary column sizing

$w_{LL} = 100\text{ psf}\,(31\text{ ft}) = 3100\text{ plf} = 3.1\text{ klf}$
$w_{DL} = 110\text{ psf}\,(31\text{ ft}) = 3400\text{ plf} = 3.4\text{ klf}$

$w_u = 1.2\,(3.4\text{ klf}) + 1.6\,(3.1\text{ klf}) = 9.04\text{ klf}$

Gravity Spot check - column #50	Tech 1	page 3 of 6

$$Mu_{\text{left of support}} = \frac{9.04\ \text{klf}\left(\frac{27.6+35}{2}\right)^2}{10} = 886\ \text{kft}$$

$$Mu_{\text{right of support}} = \frac{9.04\ (31.3)^2}{11} = 805\ \text{kft}$$

use $M_U = 886$ kft to be conservative

Preliminary column size for level 1

assume bars on all 4 faces, $f'_c = 8000$ psi and $f_y = 60$ ksi

$$e = \frac{M_u}{P_u} = \frac{886 \times 12}{2362} = 4.5''$$

assume $d' = 2.5''$ $\qquad \gamma = \dfrac{h - 2d'}{h}$

- set target reinforcement ratio to about $\rho_g = 3\% = 0.03$

h	γ	e/h
22	0.773	0.205
24	0.792	0.188
30	0.833	0.15
36	0.861	0.125

assumptions: to enable use of design aid interaction diagrams for determining the preliminary column size for level 1, use a specific concrete strength of $f'c = 6000$ psi in place of the existing column's specific concrete strength of $f'c = 8000$ psi

- using fig. A-11b from Reinforced Concrete: Mechanics and Design, 5th edition, for $\gamma = 0.75$, $\rho_g = 0.03$ and $e/h \approx 0.17$

then $\dfrac{\phi P_n}{bh} = 2.9$ for which $b \cdot h = \dfrac{\phi P_n}{2.9} = \dfrac{2362}{2.9} = 814.5\ \text{in}^2$

$\Rightarrow b = h = 28.5''$ $\rightarrow$ try $30'' \times 30''$ column

from fig. A-11b

$$\frac{\phi P_n}{b \cdot h} = \frac{2362}{30 \times 30} = 2.62$$

$$\frac{\phi M_n}{bh^2} = \frac{886 \times 12}{30 \times 30^2} = 0.40$$

requires $\rho_g = 1.3\%$

from fig. A-11c ($\gamma = 0.90$)

$$\frac{\phi P_n}{bh} = 2.62$$

$$\frac{\phi M_n}{bh^2} = 0.40$$

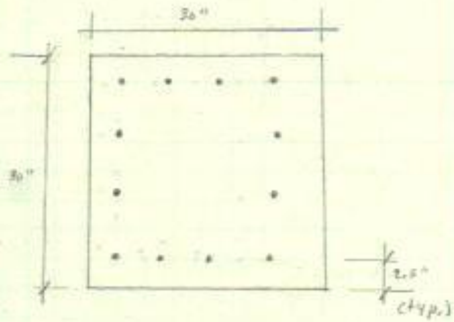
requires $\rho_g \simeq 1.1\%$

interpolation gives $\rho = \frac{0.833 - 0.75}{0.90 - 0.75}(1.1 - 1.3) + 1.3 = 1.19\%$ for $\gamma' = 0.833$

$- \; A_{s_{min}} = \rho_g (A_g) = 0.0119(30 \times 30) = 10.71 \text{ in}^2$

$\frac{10.71 \text{ in}^2}{8 \text{ bars}} = 1.34 \text{ in}^2$ use (12) #9 = 12 in^2

Use a 30" x 30" column reinforced with (12) #10



assumption: column is subjected to only gravity load ∴ check the column for pure axial compressive strength

pure compression: ($c = \infty$, $\varepsilon_c = \varepsilon_s = 0.003$ (section is compression controlled)

for tied columns, $\phi P_n = 0.8\phi\,[f'_c(bh - \Sigma A_{si}) + \Sigma A_{si} f_y]$
$= 0.80(0.65)[6(30 \times 30 - 12) + 12(60)]$
$= 3145\text{ K} > P_u = 2362\text{ K}$ OK ✓

existing design uses a 24"x36" column on level 1 with f'c = 8000 psi and (16) #11 reinforcing bars. In addition, the column at level 1 has a slope, but for simplification purposes the slope was neglected

cross-section percent error $= \frac{|24 \times 36 - 30 \times 30|}{24 \times 36} \times 100 = 4\%$

Gravity spot check – column # 50 | Tech 1 | page 5 of 6

Axial load on column at level 5

- load above level 5: roof + 7 floors

$$LL_{reduct.}\ factor = \max \begin{cases} 0.40 \\ 0.25 + \frac{15}{\sqrt{7 \times 4333}} = 0.336 \end{cases} \quad \therefore \text{ use } 0.40$$

$P_L = 0.40 (100\ psf)(7\ flrs)(970.3\ ft^2/flr) = 272$

$P_D = 110\ psf\ (970.3\ ft^2/flr)(7\ flrs) + 110\ psf\ (970.3\ ft^2/roof) = 854\ K$

$P_{Lr} = 150\ psf\ (970.3\ ft^2/roof) = 145.5\ K$

$P_u = 1.2(854) + 1.6(272) + 0.5(145.5) = 1533\ K$

Unbalanced moment for column at level 5

– Same as UBM for column at level 1 (refer to pg. 2)

Preliminary column size for level 5

assume bars on all 4 faces, $f'c = 6000\ psi$ and $f_y = 60\ ksi$

$$e = \frac{M_u}{P_u} = \frac{856 \times 12}{1533} = 6.94''$$

assume $d' = 2.5''$

set target reinforcement ratio to $\rho_g = 0.03$

h	γ	e/h
22"	0.773	0.315
24"	0.792	0.289
30"	0.833	0.231
36"	0.861	0.193

– using fig. A-8b from Reinforced Concrete: Mechanics and Design, 5th edition, for $\gamma = 0.75$, $\rho_g = 0.03$ and $e/h \approx 1.86 \rightarrow$ (average of e/h values above)

then $\frac{\phi P_n}{bh} \approx 2.8$ for which $bh = \frac{1533}{2.8} = 547.5\ in^2$

$\Rightarrow b = h = 23.4 \rightarrow$ try 24" x 30" column

Gravity Spot check – column #50 | Tech 1 | Page 6 of 6

from fig. A-11b

$$\frac{\phi P_n}{bh} = \frac{1533}{24 \times 30} = 2.13$$

$$\frac{\phi M_n}{bh^2} = \frac{856 \times 12}{24 \times 30^2} = 0.50$$

requires $\rho_g = 1.5\%$

from fig. A-11c ($\gamma = 0.90$)

$$\frac{\phi P_n}{bh} = 2.13$$

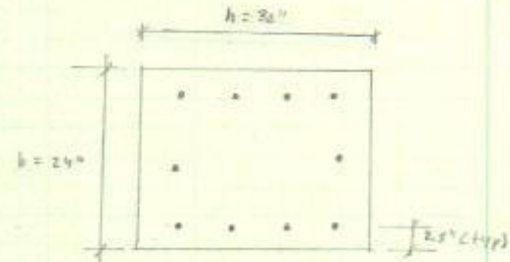
$$\frac{\phi M_n}{bh^2} = 0.50$$

requires $\rho_g = 1.2\%$

$$\rho_g = \frac{0.833 - 0.75}{0.90 - 0.75}(1.2 - 1.5) + 1.5 = 1.33$$

– $A_{s,min} = \rho_g (A_g) = 0.0133 (24 \times 30) = 9.58 \text{ in}^2$

$$\frac{9.58 \text{ in}^2}{8 \text{ bars}} = 1.20 \text{ in}^2$$ use (10) #9 = 10.0 in²



assumption: column is subjected to only gravity load ∴ check the column for pure axial compressive strength

Pure compression: $c = \infty$, $\varepsilon_c = \varepsilon_s = 0.003$

for tied columns, $\phi P_n = 0.8(0.65)[6(24 \times 30 - 10) + 10(60)]$
$= 2527 \text{ k} > P_u = 1533 \text{ k}$ ok ✓

The existing column at level 5 is 24"x24" with (12)#11 reinforcing bars and f'c = 6000 psi.

cross-section percent error $= \frac{|24 \times 24 - 24 \times 30|}{24 \times 24} \times 100 = 25\%$

Snow Drift	tech 1	page 1 of 2

step 1: ground snow load, p_g → from fig. 7-1, $p_g = 25$ psf

step 2: Exposure factor, C_e, → from table 7-2, Terrain category B
roof fully exposed
⇒ $C_e = 0.90$

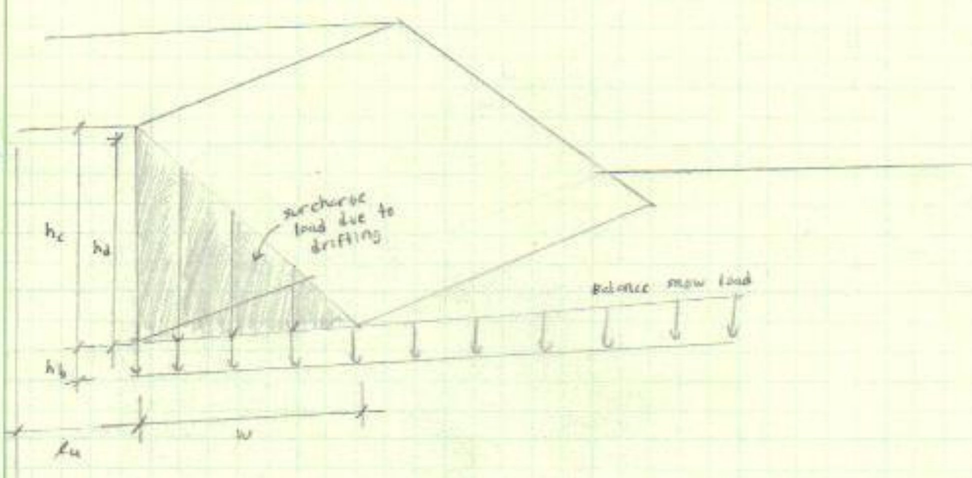
step 3: Thermal factor, C_t → from table 7-3, $C_t = 1.0$

step 4: importance factor, I → from table 1.5-1, $I = 1.0$ (occ II)

step 5: flat roof snow load, p_f → sect 7.3,

$$p_f = 0.7\, C_e C_t I_s p_g = 0.7(0.90)(1.0)(1.0)(25\text{ psf}) = 15.75\text{ psf}$$

Snow drift : Penthouse level



step 6: maximum intensity of the drift surcharge load, p_d → sect 7.7.1

$$p_d = h_d \gamma \Rightarrow \text{snow density } \gamma = 0.13 p_g + 14 = 0.13(25\text{ psf}) + 14 = 17.25\text{ pcf} < \gamma_{max} = 30\text{ pcf}$$

leeward drift
E-W wind : $l_u = 135'$ ⇒ snow drift height, h_d → from fig. 7-9,

$$h_d = 0.43 \sqrt[3]{135} \sqrt[4]{25+10} - 1.5 = 3.7\text{ ft}$$

leeward drift
N-S wind: $l_u = 55.9'$ ⇒ $h_d = 0.43 \sqrt[3]{55.9} \sqrt[4]{25+10} - 1.5 = 2.5$ ft

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leeward drift
E-W wind: $p_d = 3.9\ ft(17.25\ pcf) = 67.3\ psf$; width of snow drift, $w = 4h_d = 4(3.9\ ft) = 15.6\ ft$
leeward drift
N-S wind: $p_d = 2.5\ ft(17.25\ pcf) = 43.1\ psf$; $w = 4(2.5) = 10\ ft$

step 7: balanced snow load height, h_b → from sect 2.1

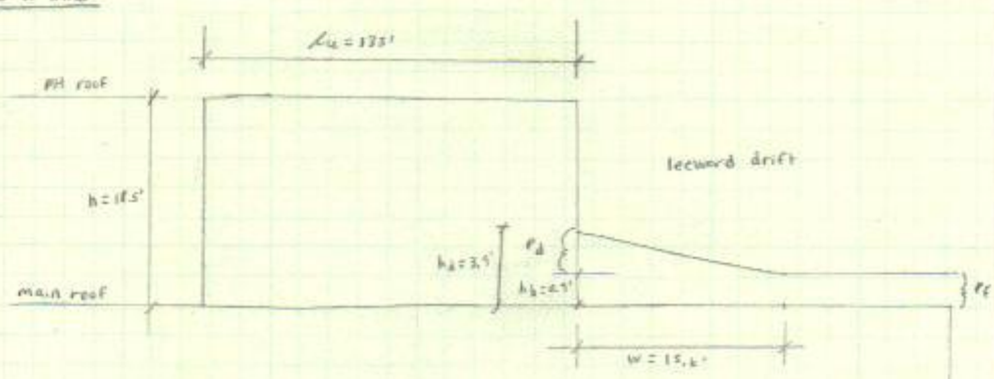
$$h_b = \frac{p_f}{\gamma} = \frac{15.75\ psf}{17.25\ pcf} = 0.9\ ft$$

step 8: $h = \underbrace{148.5'}_{\text{PH roof}} - \underbrace{130'}_{\text{main roof}} = 18.5\ ft$

step 9: $h_c = h - h_b = 18.5' - 0.9' = 17.6\ ft$

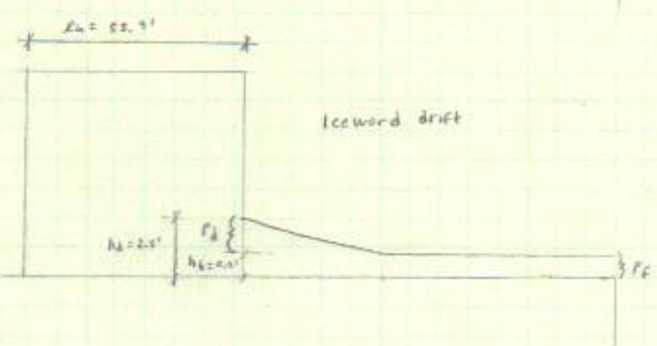
step 10: total snow load, $p = p_d + p_f$

E-W wind



$p_{total} = 67.3\ psf + 15.75\ psf = 83\ psf$

N-S wind



$p_{total} = 43.1 + 15.75 = 59\ psf$

since $p_{tot,E-W} > p_{tot,N-S}$, use $p_{tot} = 83\ psf$ for a conservative design

Appendix B: Wind Load Calculations

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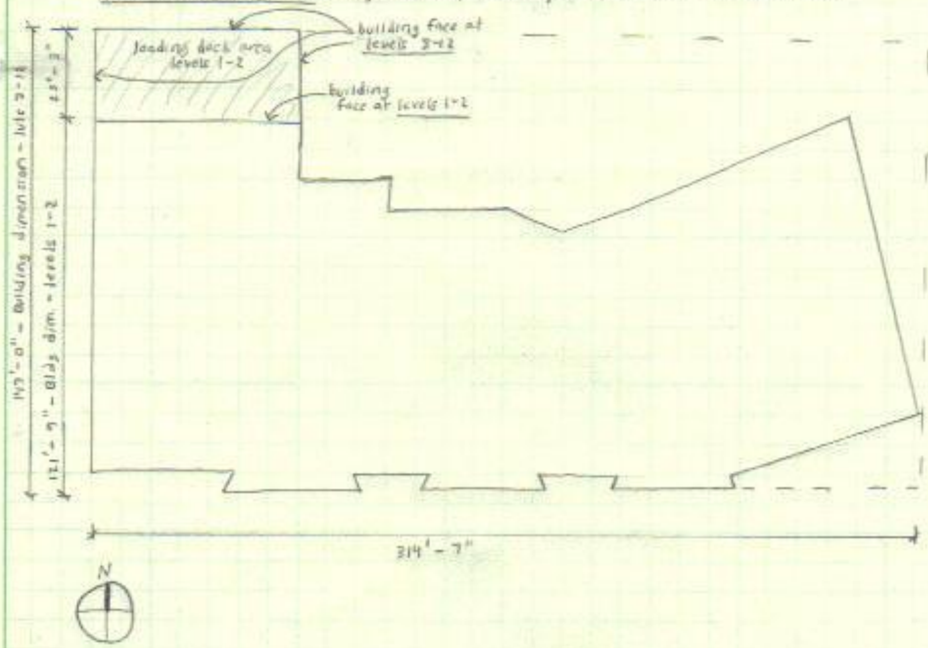
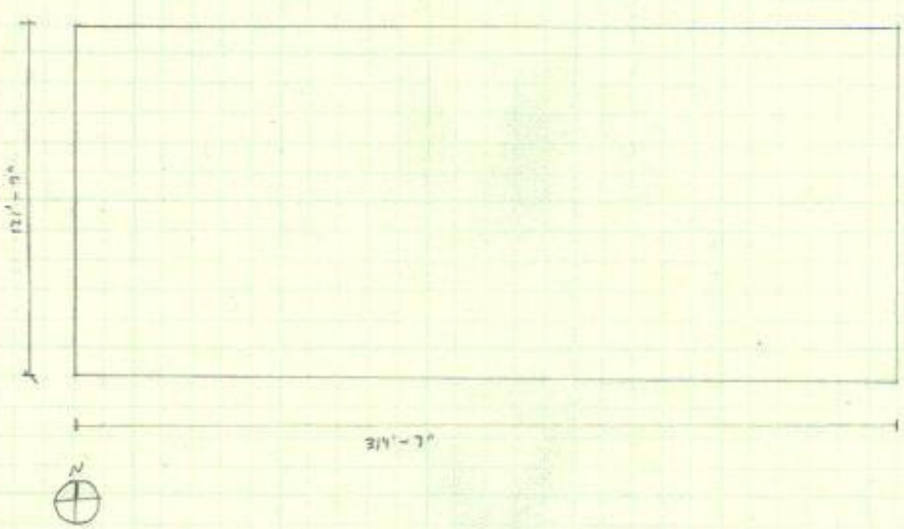
simplifying assumptions: change building geometry to rectangular for calculations



Levels 1-2 simplified building footprint



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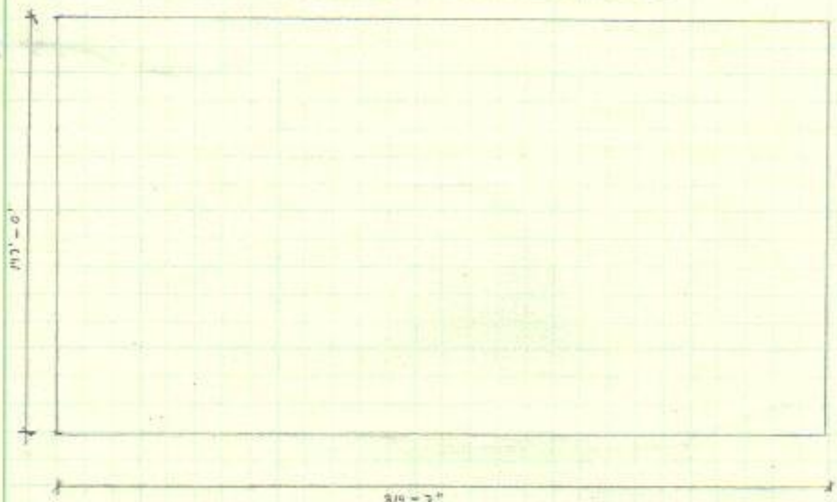
Levels 3-12 simplified building footprint



For analysis, use method 2

step 1: wind speed (V) → using Fig. 26.5-1A , V = 115 mph

step 2: directionality factor, K_d → using Table 26.6-1 , $K_d = 0.85$

step 3: exposure category - B (urban, suburban)

step 4: topographic factor , K_{zt} → from sect 26.8.2 , $K_{zt} = 1.0$ for homogeneous topography

step 5: gust effect factor, G_f → from sect 26.9

- since bldg ht < 300' and bldg ht < 4Leff in both N-S and E-W directions, use n_a to determine the natural frequency. the approximate lower-bound natural frequency, n_a, for a concrete moment-resisting frame (sect 26.9.3)

$$n_1 = n_a = \frac{43.5}{h^{0.9}}$$

$$= \frac{43.5}{130^{0.9}} = 0.544 \text{ hz} < 1 \text{ Hz}$$

∴ calculate G_f in the event the building is flexible

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

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$g_Q = g_v = 3.4$

$$g_r = \sqrt{2\ln(3{,}600\,(0.544))} + \frac{0.577}{\sqrt{2\ln(3{,}600\,(0.544))}} = 3.8936 + \frac{0.577}{3.8936} = 4.042$$

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

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

$$N_1 = \frac{n_1 L_{\bar{z}}}{V_{\bar{z}}}$$

$\bar{z} = 0.6h = 0.6(130) = 78\text{ ft} > z_{min} = 30\text{ ft}$ ok ✓

from table 26.9-1, $\bar{\alpha} = 1/4.0$, $\bar{b} = 0.45$, $c = 0.30$, $\ell = 320\text{ ft}$, $\bar{\epsilon} = 1/3.0$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{78}\right)^{1/6} = 0.26$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{78}{33}\right)^{1/3} = 426.26$$

$$V_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{78}{33}\right)^{1/4}\left(\frac{88}{60}\right)(115) = 94.11$$

$$N_1 = \frac{0.594\,(426.26)}{94.11} = 2.46$$

$$R_n = \frac{7.47\,(2.46)}{(1 + 10.3(2.46))^{5/3}} = 0.0788$$

damping ratio, $\beta = 0.010$ (from section C26.9 in ASCE 7-10, 1% is recommended for concrete buildings)

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North-South Direction (levels 1-2)

h = 130 ft

L = 121.75 ft

B = 314.58 ft

$$\eta_h = 4.6 n_1 \frac{h}{V_{\bar{z}}} = 4.6(0.544)\frac{(130)}{94.11} = 3.46$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{3.46} - \frac{1}{2(3.46)^2}(1-e^{-2(3.46)})$$

$$= 0.289 - 0.0418(0.999) = 0.247$$

$$\eta_B = 4.6 n_1 \frac{B}{V_{\bar{z}}} = 4.6(0.544)\frac{(314.58)}{94.11} = 8.36$$

$$R_B = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{8.36} - \frac{1}{2(8.36)^2}(1-e^{-2(8.36)})$$

$$= 0.1196 - 0.00715(1.00) = 0.112$$

$$\eta_L = 15.4 n_1 \frac{L}{V_{\bar{z}}} = 15.4(0.544)\frac{(121.75)}{94.11} = 10.84$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{10.84} - \frac{1}{2(10.84)^2}(1-e^{-2(10.84)})$$

$$= 0.0923 - 0.00426(1.00) = 0.088$$

$$R = \sqrt{\frac{1}{0.01}(0.0788)(0.247)(0.112)[0.53+0.47(0.088)]}$$

$$= 0.353$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{314.58+130}{426.26}\right)^{0.63}}} = 0.78$$

$$G_f = 0.925\left(\frac{1+1.7(0.26)\sqrt{3.4^2(0.78)^2+4.042^2(0.353)^2}}{1+1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.861}$$

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East-West Direction (levels 1-2)

$h = 130$ ft

$L = 314.58$ ft

$B = 121.75$ ft

$\eta_h = 3.96$ (see N-S direction)

$R_h = 0.247$ (see N-S direction)

$$\eta_B = 4.6(0.544)\frac{(121.75)}{94.11} = 3.24$$

$$R_B = \frac{1}{3.24} - \frac{1}{2(3.24)^2}\left(1 - e^{-2(3.24)}\right)$$

$$= 0.309 - 0.0476(0.998) = 0.261$$

$$\eta_L = 15.4(0.544)\frac{(314.58)}{94.11} = 28.00$$

$$R_L = \frac{1}{28.00} - \frac{1}{2(28)^2}\left(1 - e^{-2(28)}\right)$$

$$= 0.0357 - 0.000638 = 0.035$$

$$R = \sqrt{\frac{1}{0.01}(0.0788)(0.247)(0.261)[0.53 + 0.47(0.035)]}$$

$$= 0.527$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{121.75 + 130}{426.26}\right)^{0.63}}} = 0.83$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.83)^2 + 4.042^2(0.527)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.945}$$

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North - South Direction (levels 3-12)

$h = 130\text{ ft}$

$L = 147\text{ ft}$

$B = 314.58\text{ ft}$

$$\eta_h = 4.6(0.544)\frac{(130)}{94.11} = 3.46$$

$$R_h = \frac{1}{3.46} - \frac{1}{2(3.46)^2}\left(1 - e^{-2(3.46)}\right) = 0.247$$

$$\eta_B = 4.6(0.544)\frac{(314.58)}{94.11} = 8.36$$

$$R_B = \frac{1}{8.36} - \frac{1}{2(8.36)^2}\left(1 - e^{-2(8.36)}\right) = 0.112$$

$$\eta_L = 15.4(0.544)\frac{(147)}{94.11} = 13.09$$

$$R_L = \frac{1}{13.09} - \frac{1}{2(13.09)^2}\left(1 - e^{-2(13.09)}\right) = 0.073$$

$$R = \sqrt{\frac{1}{0.01}(0.0788)(0.247)(0.112)[0.53 + 0.47(0.073)]}$$

$$= 0.351$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{314.58 + 130}{424.26}\right)^{0.63}}} = 0.78$$

$$G_F = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.78)^2 + 4.092^2(0.351)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.861}$$

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East-west Direction (levels 3-12)

$h = 130\ ft$

$L = 314.58\ ft$

$B = 147\ ft$

$n_h = 3.96$ (see N-S direction)

$R_h = 0.247$ (see N-S direction)

$$n_B = 4.6\,(0.549)\frac{(147)}{94.11} = 3.91$$

$$R_B = \frac{1}{3.91} - \frac{1}{2(3.91)^2}\left(1 - e^{-2(3.91)}\right) = 0.22$$

$$n_L = 15.4\,(0.549)\frac{(314.58)}{94.11} = 28.00$$

$$R_L = \frac{1}{28} - \frac{1}{2(28)^2}\left(1 - e^{-2(28)}\right) = 0.035$$

$$R = \sqrt{\frac{\sigma}{0.01}(0.0788)(0.247)(0.22)(0.53 + 0.47(0.035))}$$

$$= 0.484$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{147 + 130}{426.26}\right)^{0.63}}} = 0.82$$

$$G_f = 0.925\left(\frac{1 + 1.7(0.26)\sqrt{3.4^2(0.82)^2 + 4.042^2(0.484)^2}}{1 + 1.7(3.4)(0.26)}\right)$$

$$= \boxed{0.926}$$

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step 6: Enclosure classification – Fully enclosed

step 7: Internal pressure coefficient → using table 26.11-1, $GC_{pi} = \pm 0.18$

step 8: Velocity pressure exposure coefficients, K_z or K_h → using table 27.3-1, see excel spread-sheet

step 9: Velocity pressure q_z or q_h → from sect. 27.3.2, $q_z = 0.00256 K_z K_{zt} K_d V^2$ (lb/ft²)

see excel spread sheet for values

step 10: external pressure coefficient, C_p → using Fig. 27.4-1

wall pressure coefficients, C_p

Windward wall: → all L/B values → $C_p = 0.8$

Side wall: → all L/B values → $C_p = -0.7$

Leeward wall: levels 1-2

N-S wind

$L/B = \frac{121.75}{314.58} = 0.39$, $0 < 0.39 < 1 \Rightarrow C_p = -0.5$

E-W wind

$L/B = \frac{314.58}{121.75} = 2.58$, $2 < 2.58 < 4 \Rightarrow C_p \rightarrow$ interpolate based on L/B values

$$\frac{C_p - C_{p_i}}{C_{p_f} - C_{p_i}} = \frac{(L/B) - (L/B)_i}{(L/B)_f - (L/B)_i}$$

$$C_p = \frac{(L/B) - (L/B)_i}{(L/B)_f - (L/B)_i}(C_{p_f} - C_{p_i}) + C_{p_i}$$

$$= \frac{2.58 - 2}{4 - 2}(-0.2 + 0.3) + (-0.3)$$

$$= -0.271$$

levels 3-12

N-S wind

$L/B = \frac{147}{314.58} = 0.47$, $0 < 0.47 < 1 \Rightarrow C_p = -0.5$

E-W wind

$L/B = \frac{314.58}{147} = 2.14$, $2 < 2.14 < 4 \Rightarrow C_p \rightarrow$ interpolate based on L/B values

$$C_p = \frac{2.14 - 2}{4 - 2}(-0.2 + 0.3) + (-0.3)$$

$$= -0.297$$

Roof pressures coefficients, C_p

$\theta = 0°$

N-S wind

$h/L = \frac{130}{147} = 0.88 \qquad 0.5 < 0.88 < 1.0$

horizontal distance from windward edge: 0 to $h/2 \rightarrow 0'$ to $65' \Rightarrow C_p \rightarrow$ interpolate based on h/L values

$C_p = \frac{0.88 - 0.5}{1 - 0.5}(-1.3 + 0.9) + (-0.9)$
$= -1.204$

$h/2$ to $h \rightarrow 65'$ to $130' \Rightarrow C_p = \frac{0.88 - 0.5}{1 - 0.5}(-0.7 + 0.9) + (-0.9)$
$= -0.748$

h to $3h \rightarrow 130'$ to $230' \Rightarrow C_p = \frac{0.88 - 0.5}{1 - 0.5}(-0.7 + 0.5) + (-0.5)$
$= -0.652$

$> 2h \rightarrow N/A$

E-W wind

$h/L = \frac{130}{314.58} = 0.41 < 0.5$

horizontal distance from windward edge: 0 to $\frac{h}{2} \rightarrow 0'$ to $65' \Rightarrow C_p = -0.9$

$h/2$ to $h \rightarrow 65' - 130' \Rightarrow C_p = -0.9$

h to $2h \rightarrow 130' - 260' \Rightarrow C_p = -0.5$

$> 2h \rightarrow > 260' \Rightarrow C_p = -0.3$

step II: Design wind pressures $\rightarrow$ from sect. 27.4.1, $p = q G_f C_p - q_i (G C_{p_i})$ (lb/ft²)

windward walls $\rightarrow p = \underbrace{q_z G_f C_p}_{\text{external pressure}} - \underbrace{q_h (G C_{p_i})}_{\text{internal pressure}}$

- see excel spread sheet for pressures

leeward walls, side walls, roof $\rightarrow p = \underbrace{q_h G_f C_p}_{\text{external pressure}} - \underbrace{q_h (G C_{p_i})}_{\text{internal pressure}}$

note: internal wind pressures were neglected in calculating the design wind pressures since the internal pressures do not contribute towards the external wind pressures acting on the building

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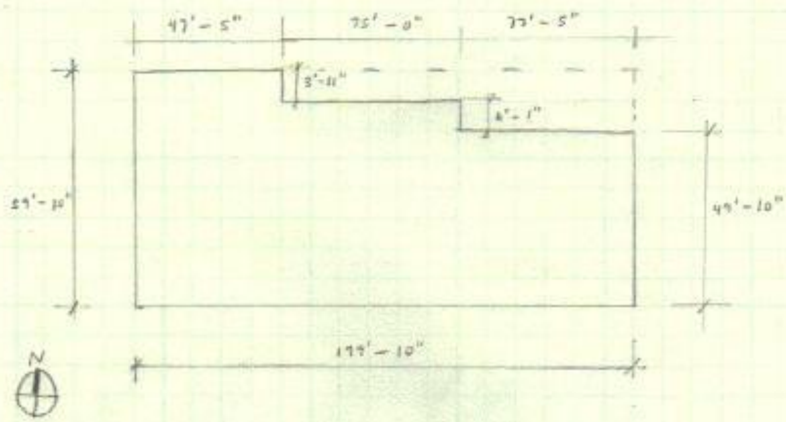
Mechanical Penthouse

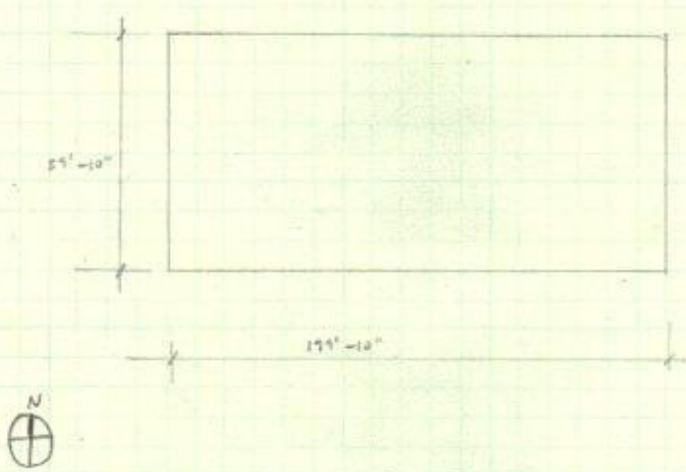
Assumption: mech. Penthouse is not a part of the MWFRS

– the penthouse will be analyzed as a separate structure and therefore the guidelines listed in chapter 29 (wind loads on other structures and building appurtenances – MWFRS) will be followed for analysis

h = 18'–6"



simplifying assumptions: change penthouse geometry to rectangular for calculations



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step1: risk category → using table 1.5-1, risk category II
step 2: V = 115 mph
step 3: $K_d = 0.90$
step4: exposure category, B
step 5: $K_{zt} = 1.0$
step6: Gust factor, G_f → from sect. 26.9.1, $G_f = 0.85$ for other structures
step 7: K_z or K_h → using table 29.3-1, $K_z = 1.13$

step 8: q_z → using sect. 29.3.2, $q_z = 0.00256\, K_z K_{zt} K_d V^2$

$$= 0.00256\,(1.13)(1.0)(0.90)(115)^2$$
$$= 34.43 \text{ psf}$$

step 9: force coefficient, C_f → using Fig. 29.5-1

$$h/D = \frac{\text{height of structure}}{\text{least dim. of sq. cross-section}} = \frac{130}{59.83} = 2.17 \Rightarrow C_f = \frac{2.17-1}{7-1}(1.4-1.3)+1.3 = 1.32$$

step 10: wind force, F → using sect. 29.5, $F = q_z G_f C_f A_f$

A_f – projected area normal to the wind

N-S Direction

$A_f = Bh = 199.83(18.5) = 3696.86 \text{ ft}^2$

$F = 34.43 \text{ psf}\,(0.85)(1.32)(3696.86 \text{ ft}^2) = 142.8 \text{ K}$

E-W Direction

$A_f = 59.83(18.5) = 1106.86 \text{ ft}^2$

$F = 34.43\,(0.85)(1.32)(1106.86) = 42.8 \text{ K}$

– see excel spread sheet for calculated story forces, base shear, and overturning moment

N-S Direction				E-W Direction			
Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i	Level	h_i (ft) (Height above grade of level i)	L_i (ft) (Building Length at level i)	h_i*L_i
1	0	121.75	0	1	0	314.58	0
2	12.54	121.75	1526.75	2	12.54	314.58	3944.833
3	23.17	147	3405.99	3	23.17	314.58	7288.819
4	33.79	147	4967.13	4	33.79	314.58	10629.66
5	44.42	147	6529.74	5	44.42	314.58	13973.64
6	55.04	147	8090.88	6	55.04	314.58	17314.48
7	65.67	147	9653.49	7	65.67	314.58	20658.47
8	76.29	147	11214.63	8	76.29	314.58	23999.31
9	86.92	147	12777.24	9	86.92	314.58	27343.29
10	97.54	147	14338.38	10	97.54	314.58	30684.13
11	108.17	147	15900.99	11	108.17	314.58	34028.12
12	118.79	147	17462.13	12	118.79	314.58	37368.96
Main roof	130	147	19110	Main roof	130	314.58	40895.4
∑=	852.34		124977.35	∑=	852.34		268129.1
L_{eff}=	146.63			L_{eff}=	314.58		

Velocity Pressure Coefficients, K_z, and Velocity Pressures, q_z			
Level	Elevation (ft)	K_z	q_z
1	0	0.57	16.40
2	12.54	0.57	16.40
3	23.17	0.66	18.99
4	33.79	0.76	21.87
5	44.42	0.81	23.31
6	55.04	0.85	24.46
7	65.67	0.89	25.61
8	76.29	0.93	26.76
9	86.92	0.96	27.63
10	97.54	0.99	28.49
11	108.17	1.04	29.93
12	118.79	1.04	29.93
Main Roof	130	1.09	31.37
PH Roof	148.5	1.13	34.43

Gust Factor-MWFRS				
Variable	N-S Wind		E-W Wind	
$n_1=n_a$	0.544			
$g_Q=g_v$	3.4			
g_R	4.042			
Z_{mean}	78			
$I_{z, mean}$	0.26			
$L_{z, mean}$	426.26			
$V_{z, mean}$	94.11			
N_1	2.46			
R_n	0.0788			
β	0.01			
η_h	3.46			
R_h	0.247			
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
η_B	8.36	8.36	3.24	3.91
R_B	0.112	0.112	0.261	0.22
η_L	10.84	13.09	28	28
R_L	0.088	0.073	0.035	0.035
R	0.353	0.351	0.527	0.484
Q	0.78	0.78	0.83	0.82
G_f	0.861	0.861	0.945	0.926
Gust Factor-Mechnical Penthouse				
Variable	N-S Wind		E-W Wind	
G_f	0.85		0.85	

Wall Pressure Coeffcients, C_p				
Description	N-S Wind		E-W Wind	
	Levels 1-2	Levels 3-12	Levels 1-2	Levels 3-12
L/B	0.39	0.47	2.58	2.14
Windward Walls	0.8			
Side Walls	-0.7			
Leeward Walls	-0.5	-0.5	-0.271	-0.293

Force Coefficient, C_f		
Description	N-S Wind	E-W Wind
Mechnical Penthouse		
h/D	1.32	1.32

Roof Pressure Coefficients, Cp		
Description	N-S Wind	E-W wind
h/L	0.88	0.41
0 to h/2	-1.204	-0.9
h/2 to h	-0.748	-0.9
h to 2h	-0.652	-0.5
>2h	N/A	-0.3

CASE 3 WIND LOAD			
Wind Forces - N-S Direction		Wind Forces - E-W Direction	
Floor	Story Force (Kips)	Floor	Story Force (Kips)
PH Roof	107.11	PH Roof	32.07
Main Roof	44.01	Main Roof	20.68
12	86.77	12	36.57
11	85.53	11	35.94
10	84.29	10	35.32
9	82.30	9	34.32
8	80.81	8	33.57
7	79.07	7	32.69
6	77.08	6	31.70
5	75.09	5	30.70
4	72.86	4	29.57
3	69.13	3	27.42
2	70.01	2	22.42
1	36.69	1	11.70

CASE 2 WIND LOAD				
Wind Forces - N-S Direction			Wind Forces - E-W Direction	
Floor	Story Force (Kips)	M_T (k-ft)	Story Force (Kips)	M_T (k-ft)
PH Roof	107.11	3210.7	32.07	287.8
Main Roof	44.01	2076.5	20.68	455.9
12	86.77	4094.5	36.57	806.3
11	85.53	4035.9	35.94	792.5
10	84.29	3977.2	35.32	778.7
9	82.30	3883.4	34.32	756.7
8	80.81	3813.1	33.57	740.2
7	79.07	3731.0	32.69	720.9
6	77.08	3637.3	31.70	698.9
5	75.09	3543.5	30.70	676.9
4	72.86	3437.9	29.57	652.1
3	69.13	3262.1	27.42	604.6
2	70.01	3303.7	22.42	409.5
1	36.69	1731.3	11.70	213.7

CASE 4 WIND LOAD					
Wind Forces - N-S Direction			Wind Forces - E-W Direction		
Floor	Story Force (Kips)	M_T (k-ft)	Story Force (Kips)	M_T (k-ft)	$M_{T\,N\text{-}S} + M_{T\,E\text{-}W}$ (k-ft)
PH Roof	80.41	2410.1	24.07	216.1	2626.2
Main Roof	33.03	1558.8	15.52	342.2	1901.0
12	65.14	3073.6	27.45	605.2	3678.8
11	64.20	3029.6	26.98	594.9	3624.5
10	63.27	2985.6	26.51	584.6	3570.1
9	61.78	2915.2	25.76	568.0	3483.2
8	60.66	2862.4	25.20	555.6	3418.0
7	59.35	2800.8	24.54	541.2	3341.9
6	57.86	2730.4	23.79	524.6	3255.0
5	56.37	2660.0	23.04	508.1	3168.1
4	54.69	2580.8	22.20	489.5	3070.2
3	51.89	2448.7	20.58	453.9	2902.6
2	52.56	2480.0	16.83	307.4	2787.4
1	27.54	1299.6	8.78	160.4	1460.0

Appendix C: Seismic Load Calculations

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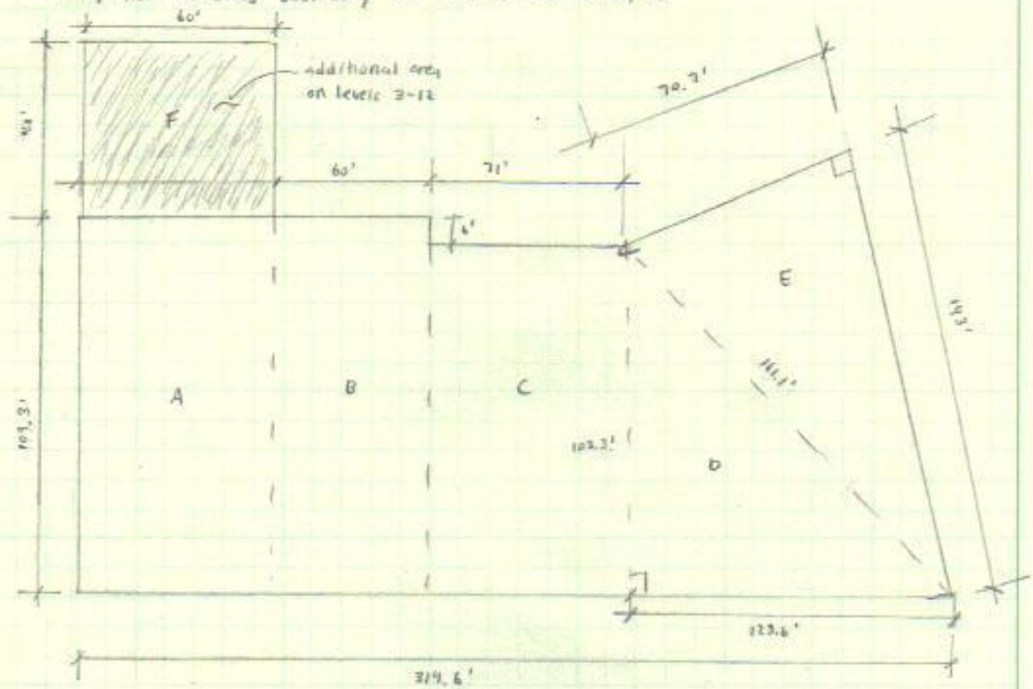
simplified building geometry for seismic load analysis



Area A = 60' (109.3') = 6558 ft²

Area B = 60' (109.3') = 6558 ft²

Area C = 71' (103.3') = 7334.3 ft²

Area D = $\frac{1}{2}$ (123.6')(103.3) = 6383.9 ft²

Area E = $\frac{1}{2}$ (143')(70.7') = 5055.1 ft²

Area F = 60(40) = 2400 ft²

building perimeter (levels 1-2) = 834.6 ft

building perimeter (levels 3-12, main roof) = 919.6 ft

floor areas for levels 1-2 = $A_A + A_B + A_C + A_D + A_E$ = 6558 (2) + 7334.3 + 6383.9 + 5055.1 = 31,889 ft²

floor area for levels 3-12 = 31,889 + A_F = 31,889 + 2400 = 34,289 ft²

main roof area = 34,289 ft²

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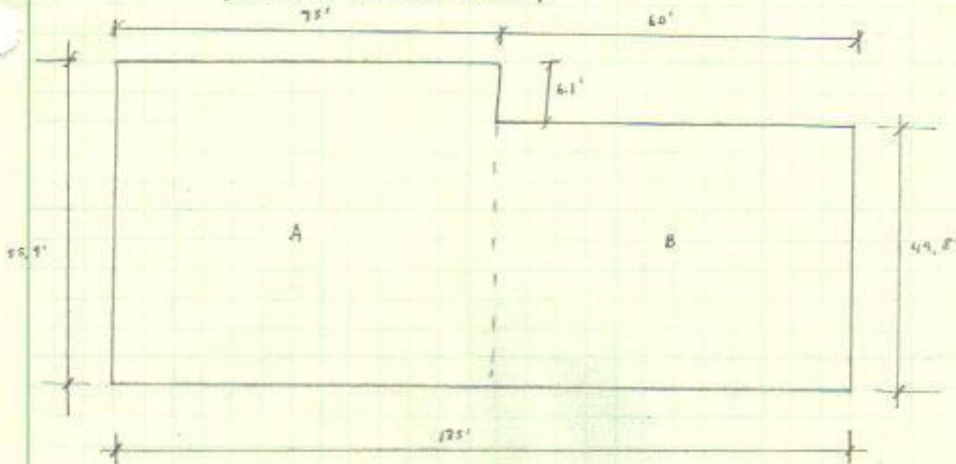
Mechanical Penthouse geometry



Area A $= 75'(55.9') = 4192.5\ ft^2$

Area B $= 49.8'(60') = 2988\ ft^2$

total Mech. PH area $= A_A + A_B = 4192.5 + 2988 = 7180.5$

Mech. PH perimeter $= 381.8\ ft$

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Building weight:

- neglect 1st level weight because level 1 will not contribute towards resisting the seismic loads
- for calculation simplicity, slab openings due to stairways and elevator shafts were neglected, therefore resulting in a more conservative calculation

2nd level

Dead loads:

8" thick normal wt concrete = 150 pcf $\times \frac{8}{12}$ ft = 100 psf

SDL = 10 psf

curtain wall = 250 plf

column wt = 488 K

Drop panel wt = 248 K

$W_{2nd\ level}$ = (100 + 10) psf (31,289 ft²) + 250 plf (834.6 ft) + 488 K + 248 K = 4452 K

levels 3 – 12

- dead loads are the same as on level 2

W = (110 psf)(34,289 ft²) + 250 plf (714.6 ft) + 488 K + 248 K = 4736 K/flr (10 flrs) = 47364 K

main roof

Dead loads;

8" thick normal wt concrete slab = 100 psf

SDL = 10 psf

curtain wall = 250 plf

$W_{main\ roof}$ = (110 psf)(34,289 ft²) + 250 plf (714.6 ft) = 4000 K

Mechanical Penthouse roof

Dead loads:

8" slab = 100 psf

SDL = 5 psf

$W_{PH\ roof}$ = (105) psf (7181 ft²) = 754 K

Total building dead load = 4452 + 47364 + 4000 + 754 = 56,570 K step1: site class → given in geotechnical report, "C" (very dense soil and hard rock, from table 20.3-1)

step2: spectral response acceleration

at short periods, S_S → from Fig. 22-1, $S_S = 0.20$

at 1-second period, S_1 → from Fig. 22-2, $S_1 = 0.06$

step3: site coefficients and adjusted maximum considered E.Q. spectral response acceleration parameters

from table 11.4-1

$S_{MS} = F_a S_S$

with $S_S < 0.25$ and site class C

$F_a = 1.2 \Rightarrow S_{MS} = 1.2(0.20) = 0.24$

from table 11.4-2

$S_{M1} = F_v S_1$

with $S_1 < 0.1$ and site class C

$F_v = 1.7 \Rightarrow S_{M1} = 1.7(0.06) = 0.102$

step4: Design spectral response acceleration parameters at short periods, S_{DS}, and at 1-sec. period, S_{D1}, for 5% damp ⇒ from sect. 11.4.4

$S_{DS} = 2/3\ S_{MS} = 2/3\ (0.24) = 0.16$

$S_{D1} = 2/3\ S_{M1} = 2/3\ (0.102) = 0.068$

step5: occupancy category and importance factors

occupancy category II → from table 1.5-1

importance factor, I → from table 1.5.2, $I = 1.0$

step6: seismic design category, SDC

SDC based on short period response acceleration parameter → from table 11.6-1

for $S_{DS} = 0.16$ and occ. II ~ SDC = "A"

SDC based on 1-sec. response acceleration parameter

for $S_{D1} = 0.068$ and occ. II ~ SDC = "B"

since risk category B is more severe than risk category A, use SDC = "B"

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step 7: response modification factor, R → from table 12.2-1

for ordinary reinforced concrete moment frames, R = 3

Equivalent Lateral Force Procedure used for analysis

step 8: approximate fundamental period, T → from sect. 12.8.2.1

$T_a = C_t h_n^x$ from table 12.8-2, "concrete moment resisting frames", $C_t = 0.016$, $x = 0.9$

$T_a = 0.016\,(130)^{0.9}$
$= 1.278\ s$

long transitional period → from fig. 22-12, $T_L = 6\ s$

step 9: seismic response coefficient, C_s → from sect. 12.8.1.1

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.16}{(3/1.0)} = 0.0533$$

$$T = 1.278\ s < T_L = 6\ s \Rightarrow C_s < \frac{S_{D1}}{(R/I)T} = \frac{0.068}{(3/1)(1.278)} = 0.0177$$ not ok

> 0.01 ✓ ok

since $C_s = 0.0533 > 0.0177$, C_s of 0.0177 controls and thus it is the value that is used for calculating the base shear V.

step 10: base shear, V

$V = C_s W = 0.0177\,(56570) = 1001\ k$

Step 11: distribute seismic base shear, V, to story levels → from sect. 12.8.3

$F_x = C_{vx} V$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$k = \frac{1.278 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.389$$

* story forces and overturning moment calculated in excel spreadsheet

Floor Weight Calculations						
Floor	Area (ft^2)	Perimeter (ft)	8" slab weight (psf)	Superimposed DL (psf)	Curtain Wall Weight (plf)	Total Weight (Kips)
2	31889.00	834.6	100	10	250.00	3716
3	34289.00	914.6	100	10	250.00	4000
4	34289.00	914.6	100	10	250.00	4000
5	34289.00	914.6	100	10	250.00	4000
6	34289.00	914.6	100	10	250.00	4000
7	34289.00	914.6	100	10	250.00	4000
8	34289.00	914.6	100	10	250.00	4000
9	34289.00	914.6	100	10	250.00	4000
10	34289.00	914.6	100	10	250.00	4000
11	34289.00	914.6	100	10	250.00	4000
12	34289.00	914.6	100	10	250.00	4000
Main Roof	34289.00	914.6	100	10	250.00	4000
PH Roof	7181.00	381.8	100	5	N/A	754
					Total Floor Weight=	48475

Typical Floor Column Weight (4th Level)					
Size (in x in)	Quantity	Length Clear Span(ft)	Unit Weight (lbs/ft^3)	Volume (ft^3)	Weight (Kips)
18X36	4	9.96	150	179.25	26.89
16X32	7	9.96	150	247.85	37.18
18X28	1	9.96	150	34.85	5.23
18X42	3	9.96	150	156.84	23.53
24X30	6	9.96	150	298.75	44.81
24X24	10	9.96	150	398.33	59.75
24 dia	12	9.96	150	375.2	56.28
16X48	2	9.96	150	106.22	15.93
16X66	1	9.96	150	73.03	10.95
14X48	2	9.96	150	92.94	13.94
12X24	5	9.96	150	99.58	14.94
22X26	1	9.96	150	39.56	5.93
20.5X24	1	9.96	150	34.02	5.10
22X24	1	9.96	150	36.51	5.48
14X96	1	9.96	150	92.94	13.94
24X36	2	9.96	150	119.50	17.93
28X28	3	9.96	150	162.65	24.40
12X48	1	9.96	150	39.83	5.98
11X24	4	9.96	150	73.03	10.95
16X96	1	9.96	150	106.22	15.93
14X66	1	9.96	150	63.90	9.58
28 dia	8	9.96	150	340.5	51.07
18X64	1	9.96	150	79.67	11.95
		Column Weight per floor (11 total flrs)=			488
				Total Column Weight=	5365

Typical Floor Drop Panel Weight (4th Level)					
Size (in x in)	Quantity	Thickness (in)	Unit Weight (lbs/ft^3)	Volume (ft^3)	Weight (Kips)
36 wide	Continuous Drop (around perimeter of all floors)	3.50	150	800.28	120
min. 68X68	40	8.00	150	856.3	128
		Drop Panel Weight per floor (11 total flrs)=			248
				Total Drop Panel Weight=	2733

Appendix D: Controlling Wind Load Case and Controlling Load Combination

Controlling Wind Case

Wind Load Case 1- story 12		
Frame	X-Direction	Y-Direction
	Shear Force (kips)	Shear Force (kips)
1	5.44	1.74
2	5.06	1.45
3	3.13	0.41
4	13.2	1.86
5	8.57	2.53
6	16	0.25
7	22.71	5.71
8	3.86	11.57
9	0.065	2.56
10	1.01	9.4
11	1.37	9.14
12	2.98	0.63
13	3.02	49.77
14	1.57	68.44
15	0.16	30.73
16	0.08	6.12
17	0.048	3.79
18	0.014	8.81
19	0.56	8.57
20	0.17	26.31
21	0.02	8.65
22	0.48	4.72
23	1.34	37.11
24	1.08	2.71
25	3.81	10.4
26	3.56	10.87
27	2.43	7.08
28	0.16	16.26
Average Shear=	3.6	$\boxed{12.4}$ kips

Wind Load Case 2- level 12		
Frame	X-Direction	Y-Direction
	Shear Force (kips)	Shear Force (kips)
1	3.22	7.92
2	3.26	2.91
3	2.1	2.29
4	9.08	6.94
5	6.45	1.35
6	12.65	5.04
7	19.77	25.1
8	2.81	6.46
9	0.11	3.42
10	0.31	10.21
11	0.54	9.53
12	2.48	2.96
13	0.33	22.33
14	0.76	35.68
15	0.36	18.78
16	0.03	4.57
17	0.41	1.46
18	0.12	7.01
19	0.5	6.83
20	0.13	21.3
21	0.045	7.1
22	0.33	4
23	0.15	34.63
24	1.1	1.52
25	1.8	10.82
26	2.01	13.65
27	1.48	5.81
28	0.06	10.34
Average Shear=	2.6	10.4 kips

Controlling wind case

Wind Load Case 3- level 12	
Frame	Shear Force (kips)
1	2.78
2	4.9
3	2.04
4	8.51
5	8.33
6	11.81
7	21.31
8	11.69
9	2.25
10	5.83
11	6.08
12	2.63
13	39.6
14	52.51
15	23.18
16	4.53
17	2.88
18	6.6
19	6.85
20	19.61
21	6.5
22	3.9
23	26.82
24	1.22
25	6.87
26	5.52
27	5.6
28	12.31

Average Shear= 11.2 kips

Wind Load Case 4- level 12	
Frame	Shear Force (kips)
1	3.5
2	0.3
3	0.1
4	1.6
5	5.9
6	13.3
7	33.7
8	6.9
9	2.5
10	7.0
11	6.7
12	4.1
13	17.0
14	26.2
15	13.8
16	3.5
17	1.4
18	5.4
19	5.5
20	16.1
21	5.4
22	3.3
23	25.9
24	0.3
25	7.7
26	8.5
27	4.7
28	7.7

Average Shear= 8.5 kips

Controlling Load Combination

Earthquake- East-West - Story 8	
Load Combination- 1.0E	
Frame	Shear Force (kips)
1	33.5
2	32.2
3	24.9
4	91.0
5	138.3
6	100.6
7	161.6
8	36.7
9	13.8
10	15.9
11	15.4
12	27.4
13	5.5
14	2.9
15	0.6
16	0.6
17	0.7
18	0.0
19	1.0
20	0.4
21	0.0
22	0.3
23	3.0
24	5.6
25	7.3
26	11.5
27	2.6
28	0.5

Average Shear= 26.2 kips

Earthquake- North-South - story 8	
Load Combination- 1.0E	
Frame	Shear Force (kips)
1	1.0
2	2.2
3	0.4
4	1.1
5	0.3
6	3.0
7	2.5
8	14.3
9	1.3
10	6.9
11	5.5
12	1.8
13	76.4
14	107.7
15	47.3
16	19
17	34
18	22.2
19	19.2
20	48.3
21	21.7
22	17
23	60.4
24	19.2
25	43.4
26	69.6
27	11.7
28	22.3

Average Shear= 24.3 kips

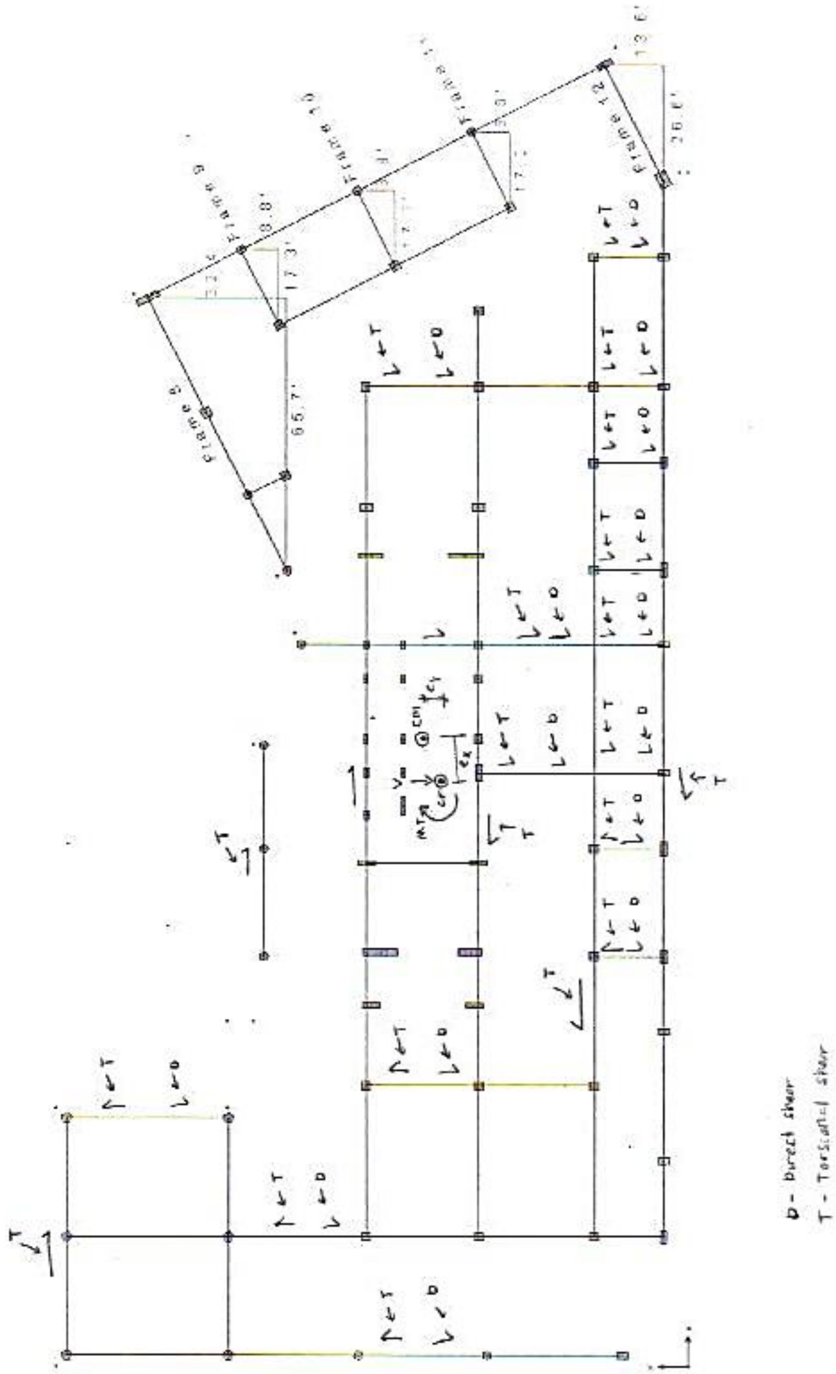
Wind Load Case 1- North-South - story 8	
Load Combination- 1.6W	
Frame	Shear Force (kips)
1	6.9
2	1.1
3	2.5
4	5.2
5	2.3
6	2.6
7	19.7
8	18.9
9	4.3
10	13.3
11	11.2
12	6.7
13	99
14	144
15	66.2
16	26
17	48.5
18	31.9
19	29.2
20	76.1
21	33.7
22	26.8
23	102
24	31.2
25	76
26	125
27	16.7
28	31.3

Average Shear= 37.8 kips

Controlling load combination in E-W direction

Controlling load combination in N-S direction

Appendix E: Direction of Direct and Torsional Shears Acting on Lateral Resisting Moment Frames



Plan view showing the direction of the direct shears and torsional shears acting on the 0° and 90° moment frames due to a N-S story lateral load

Appendix F: Frame Spot Checks

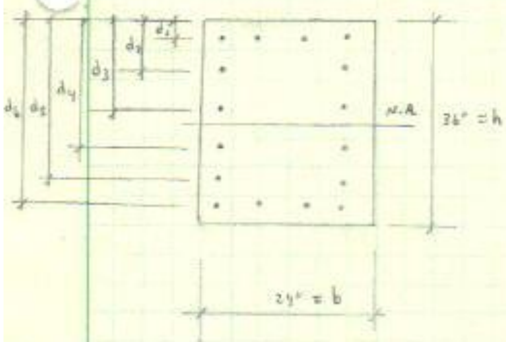
Frame Spot checks | Tech report 3 | page 1 of 4

column 50

reinforcement: 16 #11's
$66_1 45$

$f'_c = 8000$ psi
$f_y = 60$ ksi

$\epsilon_y = \frac{60}{29000} = 0.00207$



assumption: $d_1 = 3"$
$d_2 = 9"$
$d_3 = 15"$
$d_4 = 21"$
$d_5 = 27"$
$d_6 = 33"$

$P_u = 2342$ k , $M_u = 176$ kft

- Axial strength, P_o

$P_o = 0.85 f'_c \, (bh - \Sigma A_{s_i}) + \Sigma A_{s_i} f_{s_i}$

$= 0.85(8)[24(36) - 1.56(16)] + 1.56(16)(60)$
$= 7203$ k

- Balanced - strain strength, M_b, P_b

$c = \frac{0.003}{0.003 + \epsilon_y} d_{max} = \frac{0.003}{0.003 + 0.00207}(33) = 19.53"$

$\epsilon_{s_1} = \frac{0.003}{c}(c - d_1) = \frac{0.003}{19.53}(19.53 - 3) = 0.0025 > \epsilon_y$

$f_{s_1} = \epsilon_{s_1} \cdot E_s = 0.0025(29000 \text{ ksi}) = 72.5$ ksi [C]

$\epsilon_{s_2} = \frac{0.003}{c}(c - d_2) = 0.0016 < \epsilon_y$

$f_{s_2} = \epsilon_{s_2} \cdot E_s = 46.4$ ksi [C]

$\epsilon_{s_3} = 0.000696 < \epsilon_y$

$f_{s_3} = 20.2$ ksi [C]

$\epsilon_{s_4} = -0.000226 < \epsilon_y$

$f_{s_4} = -6.6$ ksi [T]

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$\varepsilon_{s5} = \frac{0.003}{c}(c - d_5) = -0.00115 < \varepsilon_y$

$f_{s5} = \varepsilon_{s5} \cdot E = -33.4$ ksi [T]

$\varepsilon_{s6} = -0.00207 = \varepsilon_y$

$f_{s6} = -60$ ksi [T]

$P_b = 0.85 f'_c \cdot b \cdot \beta_1 \cdot c + \Sigma A_{si} f_{si}$

$\beta_1 = 0.85 - \frac{0.05}{1000}(8000 - 4000) = 0.65$

$= 0.85(8)(24)(0.65)(19.53) + 4(1.56)(72.5) + 2(1.56)(46.4) + 2(1.56)(20.2) + 2(1.56)(-6.6)$
$+ 2(1.56)(-33.4) + 4(1.56)(-60)$

$= 2233$ k

$M_b = 0.85 f'_c \cdot b \cdot \beta_1 \cdot c \left(\frac{h}{2} - \frac{\beta_1 c}{2}\right) + \Sigma\left(A_{si} f_{si} \left(\frac{h}{2} - d_i\right)\right)$

$= 0.85(8)(24)(0.65)(19.53)(11.65) + 6786 + 1303 + 189 + 61.8 + 938 + 5616$

$= 37{,}030$ k-in $= 3{,}252.5$ k-ft

- pure tension

$c = \infty$; $\varepsilon_s = -\varepsilon_y$

$T_0 = \Sigma A_{si} f_{si}$

$= 1.56(16)(-60\text{ ksi}) = -1497.6$ K

- $\varepsilon_t = \varepsilon_6 = 0.005$

$c = \frac{0.003}{0.003 + 0.005}(\overset{d_{max}}{33}) = 12.38''$

$\varepsilon_{s1} = \frac{0.003}{12.38}(12.38 - d_1) = 0.00227 > \varepsilon_y$; $f_{s1} = \varepsilon_{s1} \cdot E_s = 65.8$ ksi [C]

$\varepsilon_{s2} = 0.000819 < \varepsilon_y$; $f_{s2} = 23.6$ ksi [C]

$\varepsilon_{s3} = -0.000635 < \varepsilon_y$; $f_{s3} = -18.4$ ksi [T]

$\varepsilon_{s4} = -0.00209 > \varepsilon_y$; $f_{s4} = -60.6$ ksi [T]

$\varepsilon_{s5} = -0.00354 > \varepsilon_y$; $f_{s5} = -102.7$ ksi [T]

$\varepsilon_{s6} = -0.005 > \varepsilon_y$; $f_{s6} = -145$ ksi [T]

$P_n = 0.85(8)(24)(0.65)(12.38) + 6.24(65.8)$
$+ 3.12(23.6) + 3.12(-18.4) + 3.12(-60.6)$
$+ 3.12(-102.7) + 6.24(-145)$
$= 325.8$ K

$M_n = 0.85(8)(24)(0.65)(12.38)(14) + 6159 + 662.7$
$- 172.2 + 567.2 + 2883.8 + 13572$
$= 42{,}058$ k-in $= 3505$ k-ft

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- Pure bending, M_o

- assume ε_{s_1} and ε_{s_2} do not yield
- assume $f_{s_3} - f_{s_6}$ yield

$$f_{s_1} = \frac{0.003}{c}(c-3)(29000)$$

$$f_{s_2} = \frac{0.003}{c}(c-9)(29000)$$

$$f_{s_3} = f_{s_4} = f_{s_5} = f_{s_6} = -60$$

$$\Sigma F = 0.85(8)(24)(0.65)c + 4(1.56)\left[87 - \frac{261}{c}\right] + 2(1.56)\left[87 - \frac{783}{c}\right] + 2(1.56)(-75)$$

$$+ 2(1.56)(-75) + 2(1.56)(-75) + 4(1.56)(-75)$$

$$= 166.1c + 542.9 - \frac{1628.6}{c} + 271.4 - \frac{2443}{c} - 1170$$

$$= 166.1c^2 + 542.9c - 1628.6 + 271.4c - 2443 - 1170c = 0$$

$$166.1c^2 - 355.7c - 4071.6 = 0$$

$$c = 8.1''$$

verify assumptions: $f_{s_1} = 54.8$ ksi $< f_y = 60$ ksi ok

$f_{s_2} = -9.7$ ksi $> f_y = -60$ (does not yield) ok

$f_{s_3} = -74$ ksi $< f_y = -60$ (yields) ok

$f_{s_4} = -139$ ksi $< f_y = -60$ (yields) ok

$f_{s_5} = -203$ ksi $< f_y = -60$ (yields) ok

$f_{s_6} = -267$ ksi $< f_y = -60$ (yields) ok

$$M_o = 0.85(8)(24)(0.65)(8.1)(15.32) + 4(1.56)(54.8)\overset{h/2 - d_1}{(15)} + 2(1.56)(-9.7)(9) + 2(1.56)(-74)(3)$$

$$+ 2(1.56)(-139)(-3) + 2(1.56)(-203)(-9) + 4(1.56)(-267)(-15)$$

$$= 49{,}363 \text{ k-in} = 4{,}114 \text{ kft}$$

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for tied columns:

$\phi = 0.65 + 0.25\left(\frac{\varepsilon_t - \varepsilon_y}{0.005 - \varepsilon_y}\right)$

- axial strength, P_o

$c = \infty \therefore \varepsilon_c = \varepsilon_s = 0.003$

since $0.002 < \varepsilon_s < 0.005$ $\quad \phi = 0.65 + 0.25\left(\frac{0.003 - 0.00207}{0.005 - 0.00207}\right) = 0.73$

$\phi P_n = 0.73(7203) = 5258\ k$

- balanced strain strength, M_b, P_b

$c = \frac{0.003}{0.003 + \varepsilon_y} d_{i_{max}} = 17.53"$

$\varepsilon_{s_6} = \varepsilon_t = -0.00207 = \varepsilon_y$ $\quad \phi = 0.65 + 0.25\left(\frac{0.00207 - 0.00207}{0.005 - 0.00207}\right) = 0.65$

$\phi P_n = 0.65(2233) = 1451\ k$

$\phi M_n = 0.65(3252.5) = 2114\ k\,ft$

- Pure tension,

$c = \infty$, $\varepsilon_s = -\varepsilon_y = -0.00207$, $\phi = 0.65$

$\phi P_n = 0.65(-1497.6) = -973\ k$

- M_n and P_n @ $\varepsilon_t = 0.005$

since $\varepsilon_t \geq 0.005$, $\phi = 0.90$

$\phi P_n = 0.9(325.8) = 293\ k$

$\phi M_n = 0.9(3505) = 3155\ k\,ft$

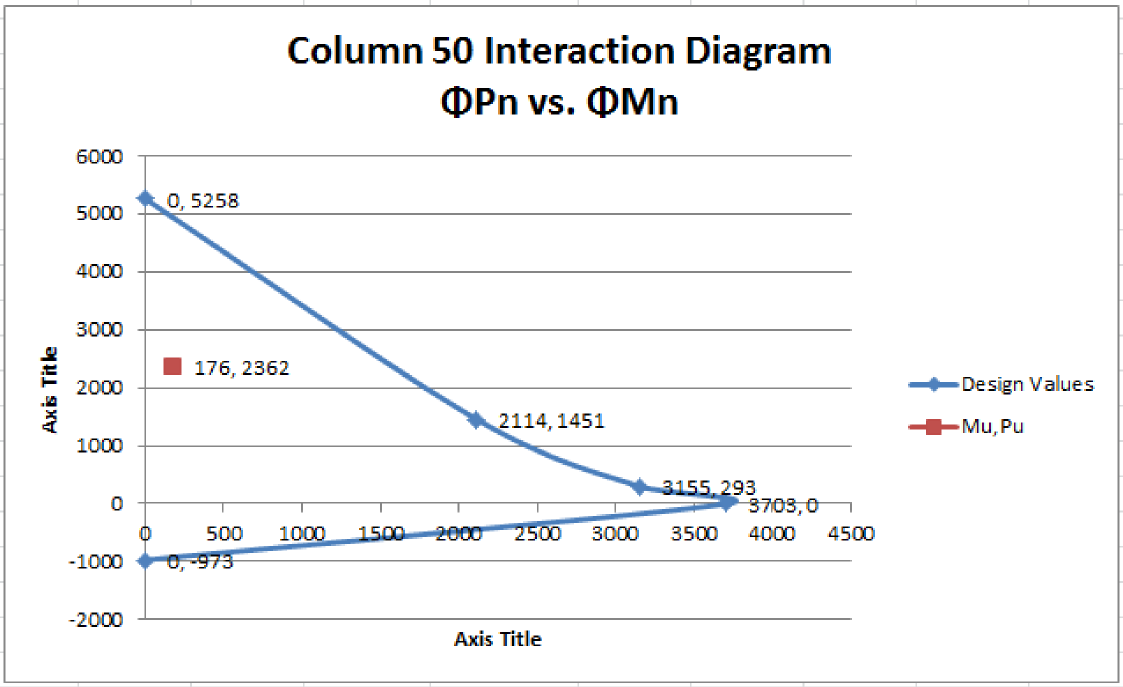
- pure bending, M_o

$\varepsilon_t = \varepsilon_6 = \frac{0.003}{8.1}(8.1 - 33) = -0.0092 > 0.005$, $\phi = 0.90$

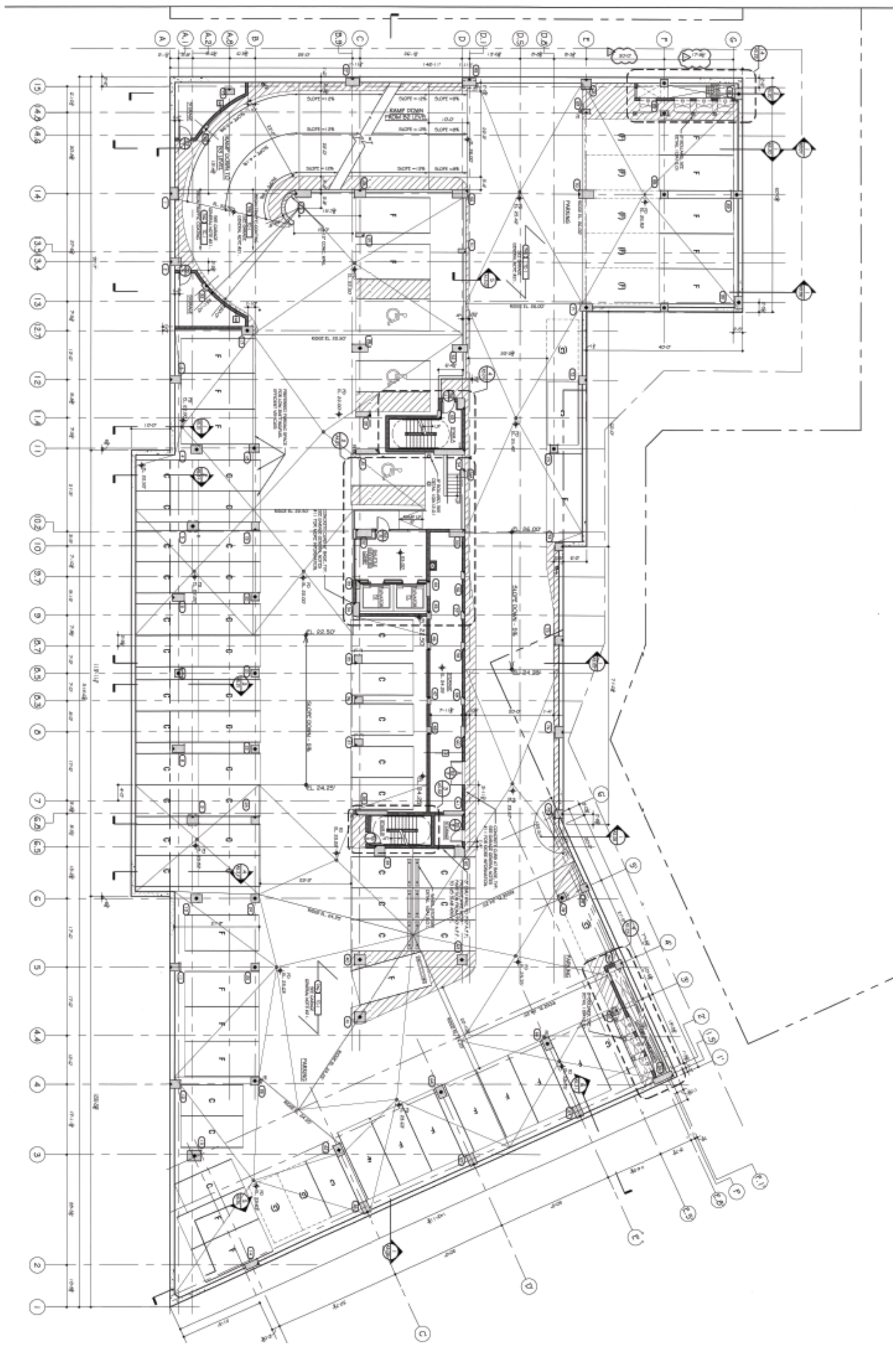
$\phi M_n = 0.90(4114) = 3703\ k\,ft$

ΦPn (kips)	ΦMn (k-ft)
-973	0
0	3703
293	3155
1451	2114
5258	0

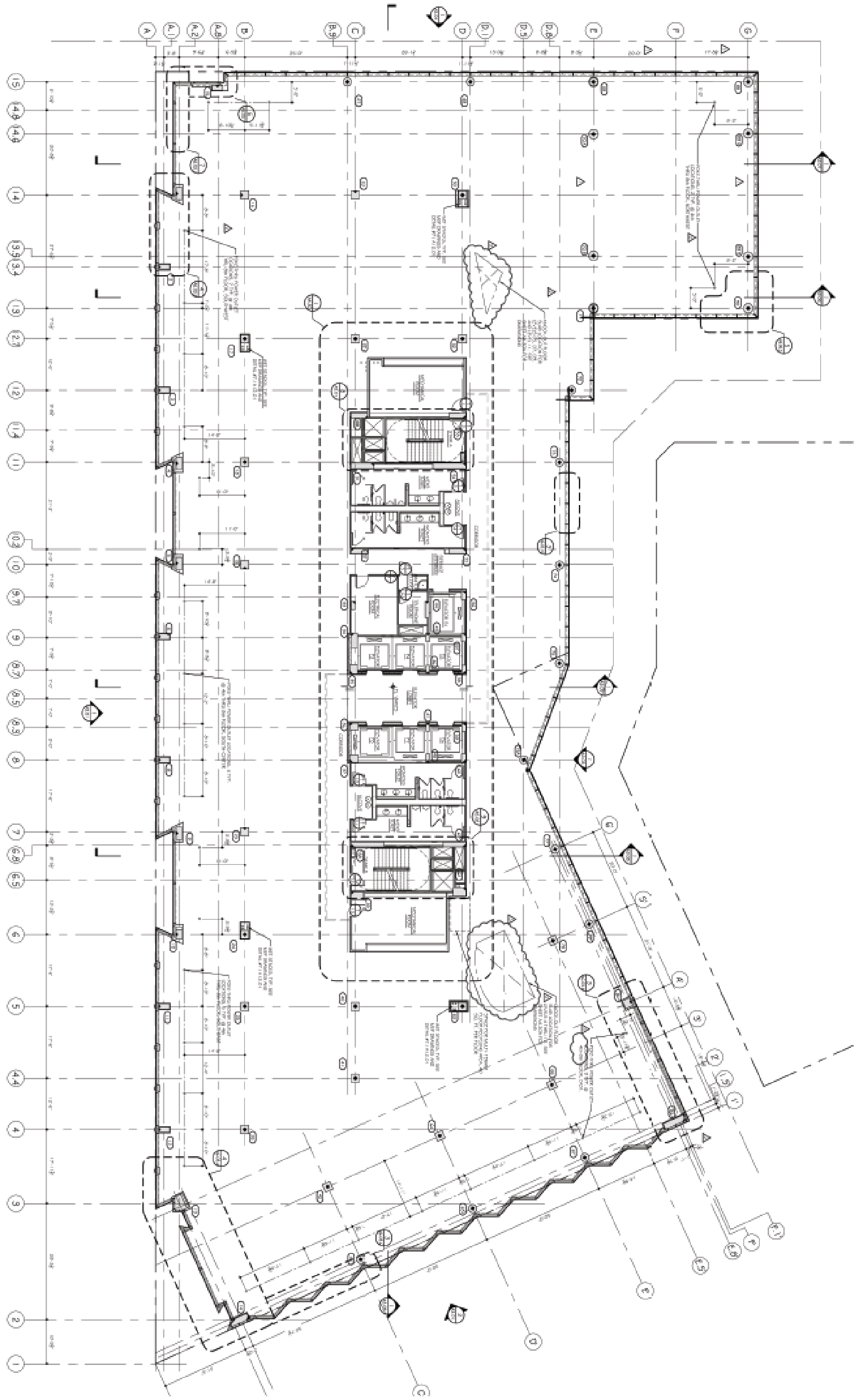
Pu (kips)	Mu (k-ft)
2362	176



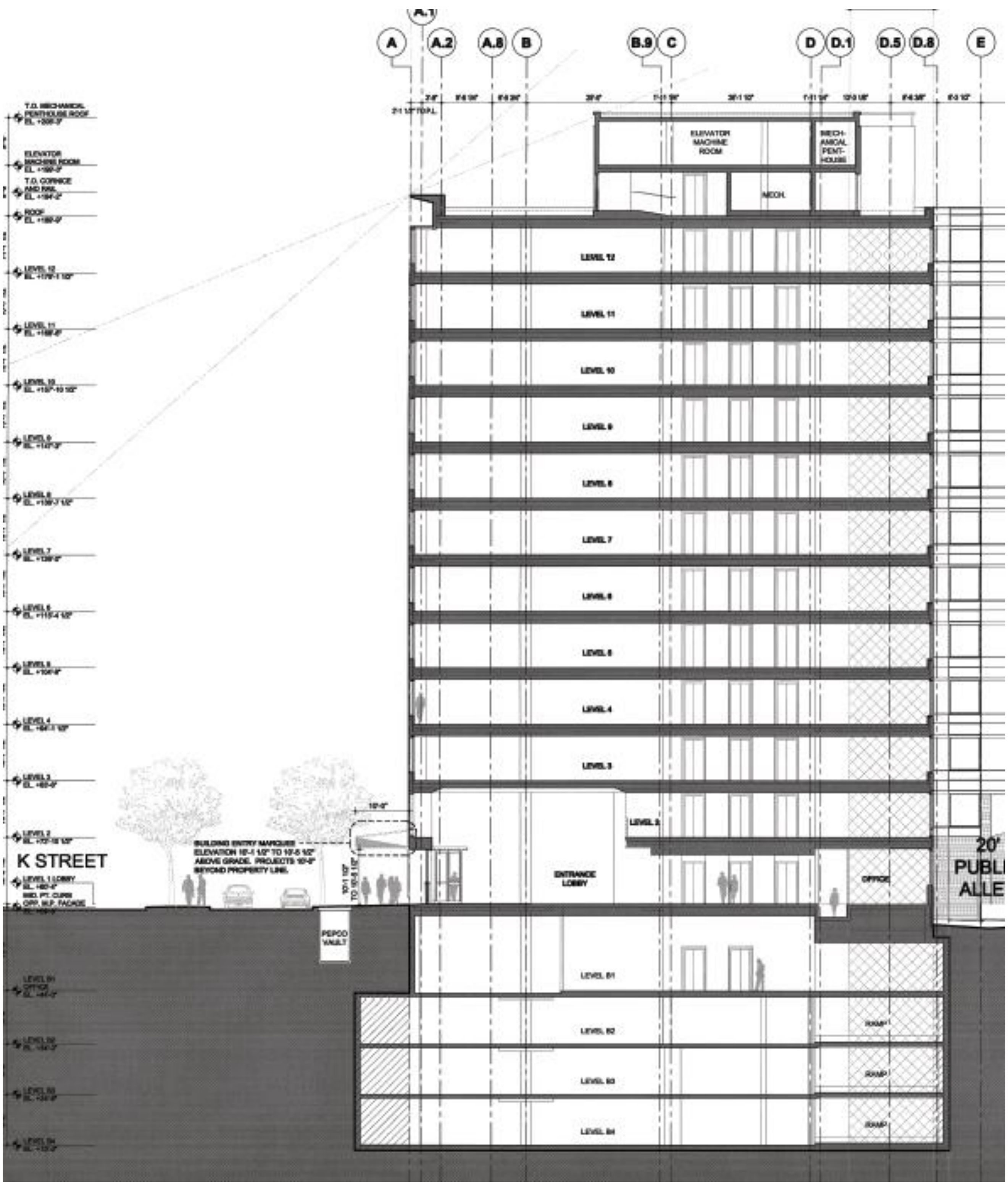
Appendix G: Typical Floor Plans



Typical underground parking plan rotated 90 degrees CW



Typical Floor plan oriented 90 degrees CW



Building Section