Technical Report 1

Falls Church Tower

Falls Church, VA

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Structural Option

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Executive Summary

The structural concepts and existing conditions report describes the structural system of the Falls Church Tower luxury apartment building. The structure is located in Falls Church, Virginia and utilizes spread footings under both the concrete columns and retaining walls. The floor systems of the building are comprised of 5 inch slab on grade, 7 inch post-tensioned slabs, and 9 inch slabs supported by concrete beams. In addition to the floor systems the building contains concrete columns in a variety of sizes arranged in alternating directions so as to support both gravity loads and lateral loads.

The gravity loads and lateral loads of Falls Church Tower were calculated using ASCE 7-05 and compared to the loads provided by the structural engineers. The controlling lateral load proved to be the seismic base shear (V=1933 K) which was significantly greater the the wind base shear of 951.93 K.

Spot checks were done on column 120, which runs from the level 2 floor slab to the level 11 floor slab with a consistent tributary area throughout the height of the structure. The results from these spot checks verified the design loads used by the designing engineer and the design loads used throughout the technical report. However, it must be kept in mind that neither live load reductions nor lateral forces where included in these spot checks in order to simplify the calculations.

Introduction

The Falls Church Tower is a luxury apartment building located in Falls Church, Virginia. The high rise apartment building stand eleven stories tall with penthouse on the main roof. Three and a half levels of parking are offered beneath the building and private pool sits adjacent to the plaza. The building encloses 364,000 square feet of gross floor area which excludes mechanical rooms, underground rooms, and garage space. The first floor contains the lobby, a residential gym, and a lounge as well as some living space with the remaining floors serving as strictly residential space. Overall the building contains 213 residential units with a wide view of the surrounding area courtesy of the building's curved facade. The structural system of the building is primarily concrete consisting of retaining walls, columns, post-tensioned slabs, and beams. The lateral system is composed of the aforementioned columns and slabs which form and ordinary concrete moment frame.

Foundation

The foundation system of Falls Church Tower was designed in accordance with the geotechnical report provided by Whitlock, Dairymple, Poston and Associates. The report indicated a soil bearing pressure of 4 ksf along the southern face of the tower and a bearing pressure of 10 ksf for the remainder of the structure.

The foundation system from levels B3 Ext. through B1 consist of retaining walls, spread footings, and a precast slab on grade. The retaining wall runs the full perimeter of the building with a thickness of 1'-4"on the B3 Ext. level and 1'-0" for B3 through B1. The footings under the retaining walls have a width ranging from 2' to 3'. The 2' width is used for sections of the buildings where the B1 retaining wall is offset towards the interior of the building by 3'-6". A section of a typical retaining wall can be seen in Figure 1-2 and Figure 1-3.

The column footings have a range of 6'x6' to 12'x12' throughout the structure. The larger footings (10'x10' to 12'x12') being located in the basement parking section beneath the plaza. A typical footing detail can be seen in Figure 1-1. The slab on grade is 5 ksi, normal weight concrete that is 5" thick with 6x6-W2.0xW2.0 welded wire fabric placed on a vapor barrier on top of 6" of #57 washed crushed stone

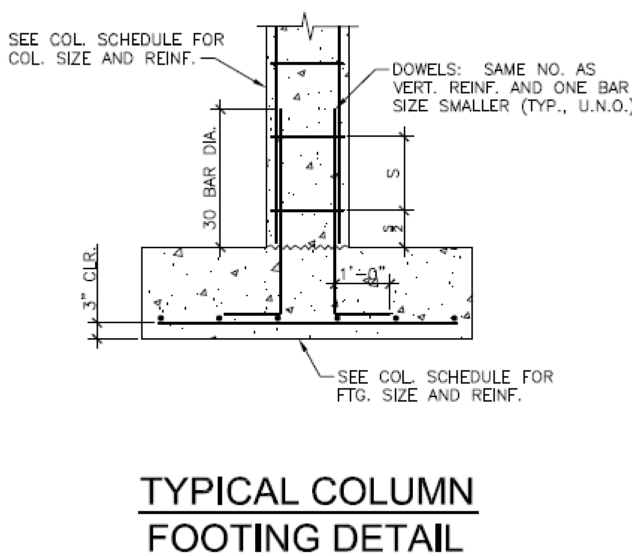


Figure 1-1

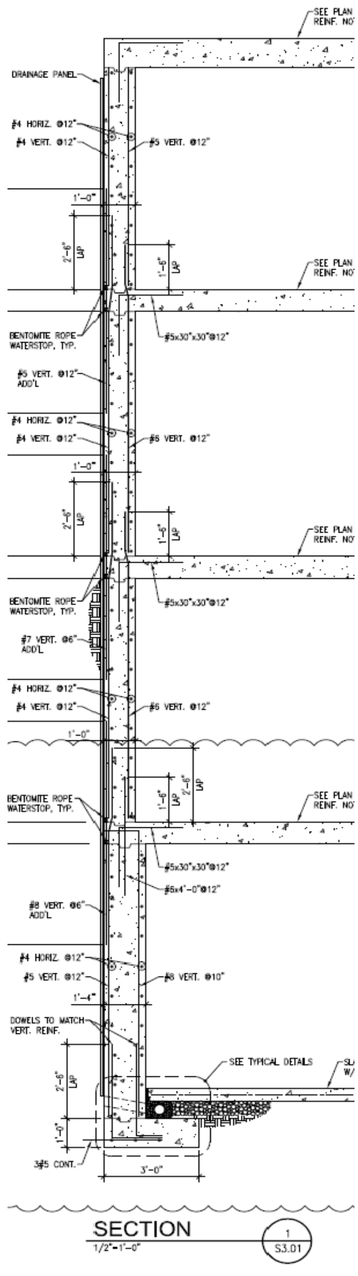


Figure 1-2

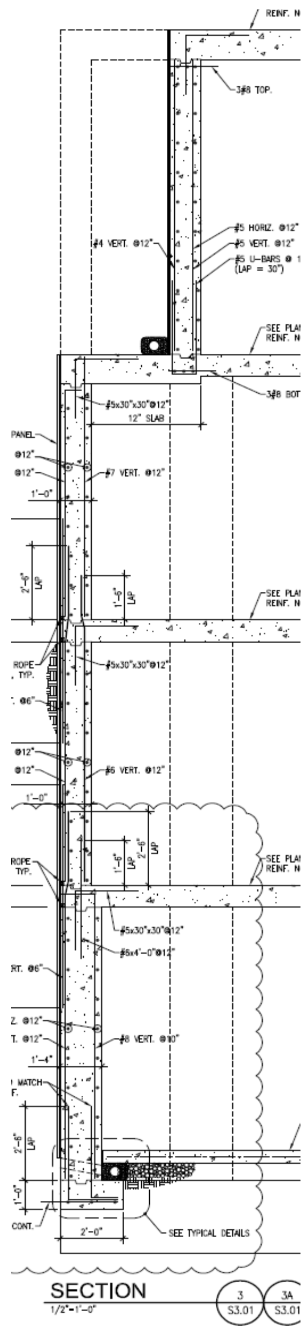


Figure 1-3

Gravity Load System

The main gravity load resisting system is composed of a flat plate supported by an intricate array of columns. Levels B3 Ext. through B1 plate systems are typically a 5 ksi, 9" thick, normal weight slab with a two way mat of #4 bottom bars at 12" on center except for slabs on grade which are 5 kis, 5" thick normal weight concrete. The penthouse roof and the elevator machine room roof use a 6" thick, one-way slab with the same properties and is support by a system of concrete beams. The plate systems from level 1 through the main roof utilize a 7" thick post tensioned slab. The typical tendons are two to three strands thick and spaced 5' on center. For a typical post tension layout plan refer to Figure 1-4.

The tower columns don't necessarily have a standard bay size due to the building's curved shape and the stair cases in both the east and west wings which interrupt any attempt at a rectilinear layout. The most typical bay size established throughout the building would be the 28'x24' bays located in the western half of the building's curved section. A standard column layout can be seen in Figure 1-5

In addition to the flat plate system the structural engineers also incorporated concrete beams into the design where necessary. As previously mentioned a system of beams is used to support the penthouse and mechanical room roofs. There are also strap (grade) beams used in the west section of B3 Ext. foundation and the east edge of B3 foundation which can be seen in Figure 1-6. Lastly, beams are used to frame all stairs and elevator shafts.

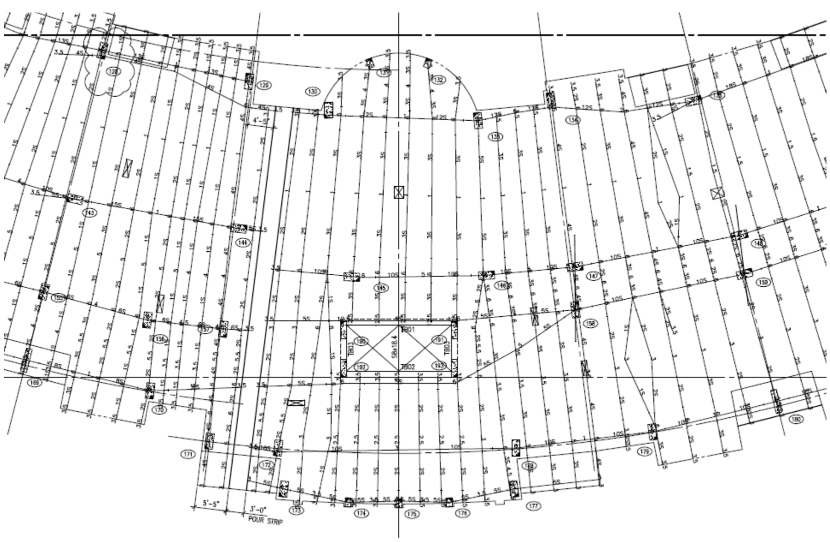


Figure 1-4
(for a larger view refer to Appendix A)

FOURTH FLOOR POST TENSION LAYOUT PLAN
SCALE: 1/8"=1'-0"

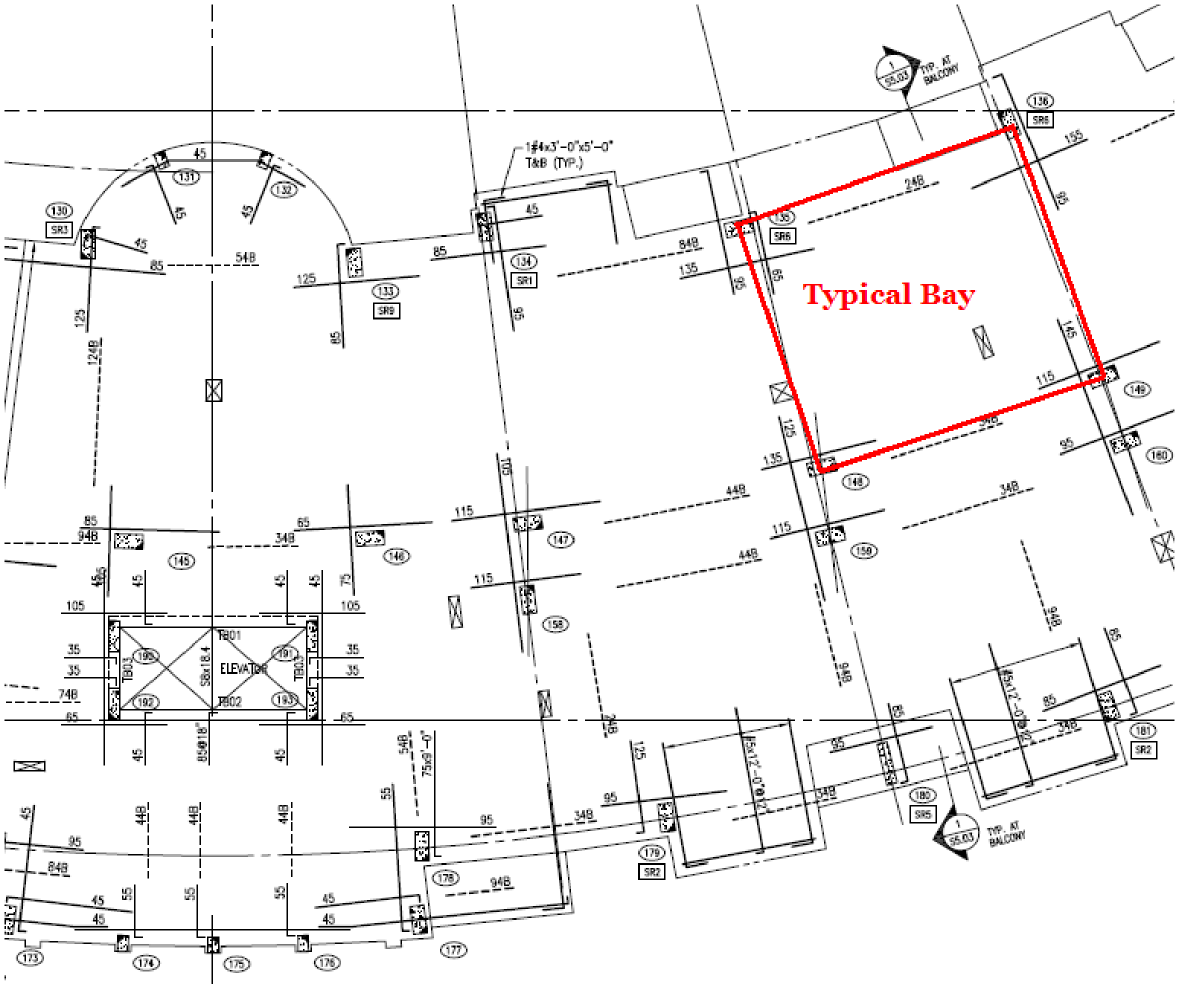


Figure 1-5

Lateral Load System

The lateral system of the building is an ordinary concrete moment frame. The tower columns' dimensions range from 12" to 24" on the short face and 12" to 48" on the long face. The two most typical columns that occur throughout the building are 16"x32" and 12"x36". The 16"x32" dimension is common for most of the interior columns whereas the 12"x36" columns are used to frame the stairs and elevator shafts.

Applicable Codes

Codes Used for Original Design

- International Building Code 2000
- Arlington County Building Code
- American Concrete Institute (ACI 318 and ACI 301)
- American Society for Testing and Materials
- American Institute of Steel Construction Manual

Codes Substituted for Thesis Analysis

- American Society of Civil Engineers (ASCE 7-05)
- International Building Code 2006

Materials and Properties

Concrete

• Footings	3000 psi
• Retaining Wall Footings	5000 psi
• Foundation Walls	
○ B3 and B3 Ext. Level	5000 psi
○ B2 and B1 Level	4000 psi
○ Site Retaining Wall	5000 psi
• Formed Slabs and Beams	5000 psi
• Columns	5000, 6000, and 8000 psi
• Slabs on Grade	5000 psi
• Pea-Gravel Concrete	2500 psi
• All Other Concrete	4000 psi

Reinforcing Steel

• Reinforcing Bars	ASTM A615
• Welded Wire Fabric	ASTM A185
• Reinforcing Bar Mats	ASTM A185
• Reinforcing Bars in Garage Slabs	ASTM A775

Steel

• Wide Flange Members	ASTM A992
• Stiffener Plates	ASTM A572
• Other	ASTM A36

Design Loads

All of the design loads for Falls Church Tower were calculated using the values and methods provided in sections three and four. These values can be found in tables 1-1 and 1-2 below and include live load and dead load values. Snow loads have been excluded from this section but can be found in Appendix C. Live load reductions were not taken into consideration for this design.

Table 1-1: Gravity Live Loads

Live Load Areas	ASCE 7-05 Required Loading		Loads Used By Engineer
Private Rooms	40 psf	ASCE 7-05 Table 4-1	40 psf + 20 psf (Partition Allowance)
Public Rooms/Corridors	100 psf	ASCE 7-05 Table 4-1	100 psf
Tenant Storage	125 psf	ASCE 7-05 Table 4-1	125 psf
Roof	20 psf	ASCE 7-05 Table 4-1	30 psf
Stairways	100 psf	ASCE 7-05 Table 4-1	100 psf
Balconies	100psf	ASCE 7-05 Table 4-1	-
Theater	60 psf	ASCE 7-05 Table 4-1	-
Garage	40 psf	ASCE 7-05 Table 4-1	50 psf
Plaza	100 psf	ASCE 7-05 Table 4-1	350 psf
Mechanical	-		150 psf
Elevator Machine Room	-		125 psf

Table 1-2: Gravity Dead Loads

Dead Loads	Load Values
Floor Finish	16 psf
Slab: B3 - 1	109 psf
Slab: 2 - Main Roof	85 psf
MEP	15 psf
Steel	15 psf
Misc	10 psf
Roof Waterproofing	5.5 psf

Wind Loads

Wind loads for Falls Church Tower were calculated using the Analytical Procedure from ASCE 7-05. Variables used in the wind load calculations can be found below in Table 2-1. Calculations used to determine these values can be found in Appendix D.

Table 2-1: Wind Design Variables

Wind Variables		
Basic Wind Speed	V	90 mph
Exposure	B	-
Building Classification	II	-
Importance Factor	I	1.00
Directionality Factor	K_d	0.85
Topographic Factor	K_{zt}	1.00
Pressure Exposure Coefficient	K_z	Varies
Pressure at Height z	q_z	Varies
Pressure at Mean Roof Height	q_h	18.24 psf
Gust Effect Factor	G_f	0.886
External Pressure Coefficient (Windward)	C_{pw}	0.80
External Pressure Coefficient (Leeward)	C_{pl}	-0.50
Internal Pressure Coefficient	GC_{pi}	0.18

While performing calculations to determine the gust effect factor and the leeward external pressure coefficient coefficients, it was found that each factor had the same value for the North-South and east-west directions. Given this information only one directional analysis was performed, that being in the North-South direction as these faces provide a larger surface area and therefore larger story forces which control design.

The final design wind pressures for the tower are provided in Table 2-2 and Figure 2-1 illustrates the distribution of these pressures across the face of the building. The shear forces produced by these pressures are provided in Table 2-3 and the distribution illustrated in Figure 2-2.

Table 2-2: Design Wind Pressures

Design Wind Pressure P (psf)							
Floor	Height Above Ground (ft)	K_z	q_z (psf)	q_h (psf)	Windward (psf)	Leeward (psf)	Total Pressure (psf)
B1	0.000	0.570	10.05	18.24	10.41	-11.36	21.77
1.000	10.000	0.570	10.05	18.24	10.41	-11.36	21.77
2.000	21.000	0.628	11.07	18.24	11.13	-11.36	22.49
3.000	30.580	0.704	12.41	18.24	12.08	-11.36	23.44
4.000	40.170	0.761	13.41	18.24	12.79	-11.36	24.15
5.000	49.750	0.809	14.26	18.24	13.39	-11.36	24.75
6.000	59.330	0.847	14.93	18.24	13.87	-11.36	25.23
7.000	68.920	0.886	15.62	18.24	14.35	-11.36	25.72
8.000	78.500	0.924	16.29	18.24	14.83	-11.36	26.19
9.000	88.080	0.954	16.81	18.24	15.20	-11.36	26.56
10.000	97.670	0.983	17.33	18.24	15.57	-11.36	26.93
11.000	107.250	1.008	17.77	18.24	15.88	-11.36	27.24
Penthouse	118.830	1.035	18.24	18.24	16.21	-11.36	27.58

Table 2-3: Story Shear Forces

Floor	Floor Height (ft)	Total Pressure (psf)	Story Force (K)	Story Shear (K)	Height Above Grade (ft)	Moment (ft-K)
B1	11.000	21.77	39.73	951.94	0.00	0
1.000	11.000	21.77	79.46	912.21	10.00	794.64
2.000	9.583	22.49	75.49	832.74	21.00	1585.3
3.000	9.583	23.44	73.03	757.25	30.58	2233.19
4.000	9.583	24.15	75.67	684.23	40.17	3039.55
5.000	9.583	24.75	77.75	608.56	49.75	3868.06
6.000	9.583	25.23	79.47	530.81	59.33	4714.78
7.000	9.583	25.72	81.01	451.34	68.92	5583.17
8.000	9.583	26.19	78.23	370.33	78.50	6140.84
9.000	9.583	26.56	79.5	292.1	88.08	7002.54
10.000	9.583	26.93	73.35	212.6	97.67	7163.62
11.000	10.583	27.24	75.12	139.26	107.25	8056.97
Main Roof	18.500	27.58	51.88	64.13	118.83	6165.39
Penthouse Roof	-	27.58	12.25	12.25	137.33	1681.68
		Base Shear =	**951.93**		**Overturning Moment =**	**58029.73**

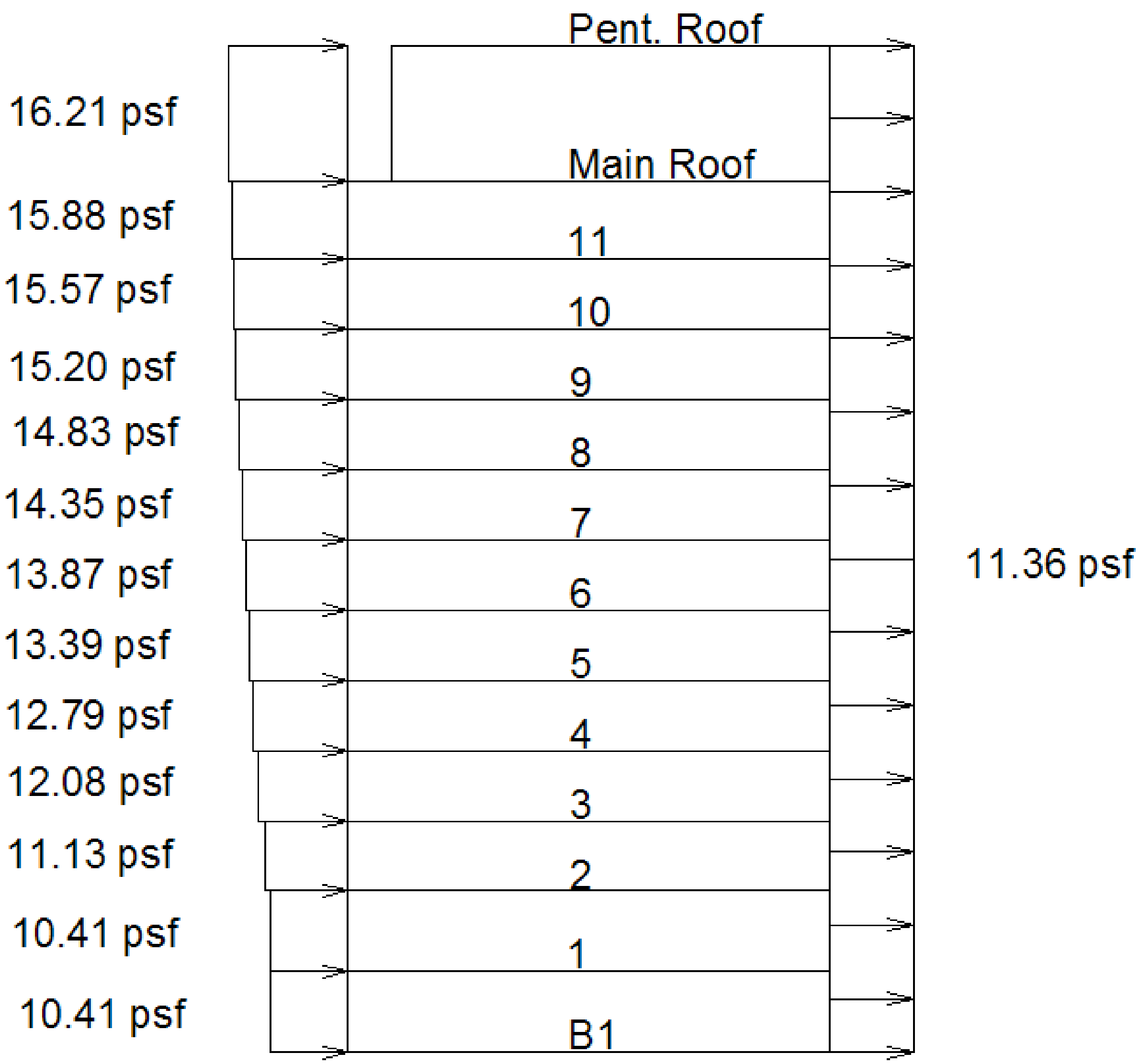


Figure 2-1: Design Wind Pressures in the N-S Direction

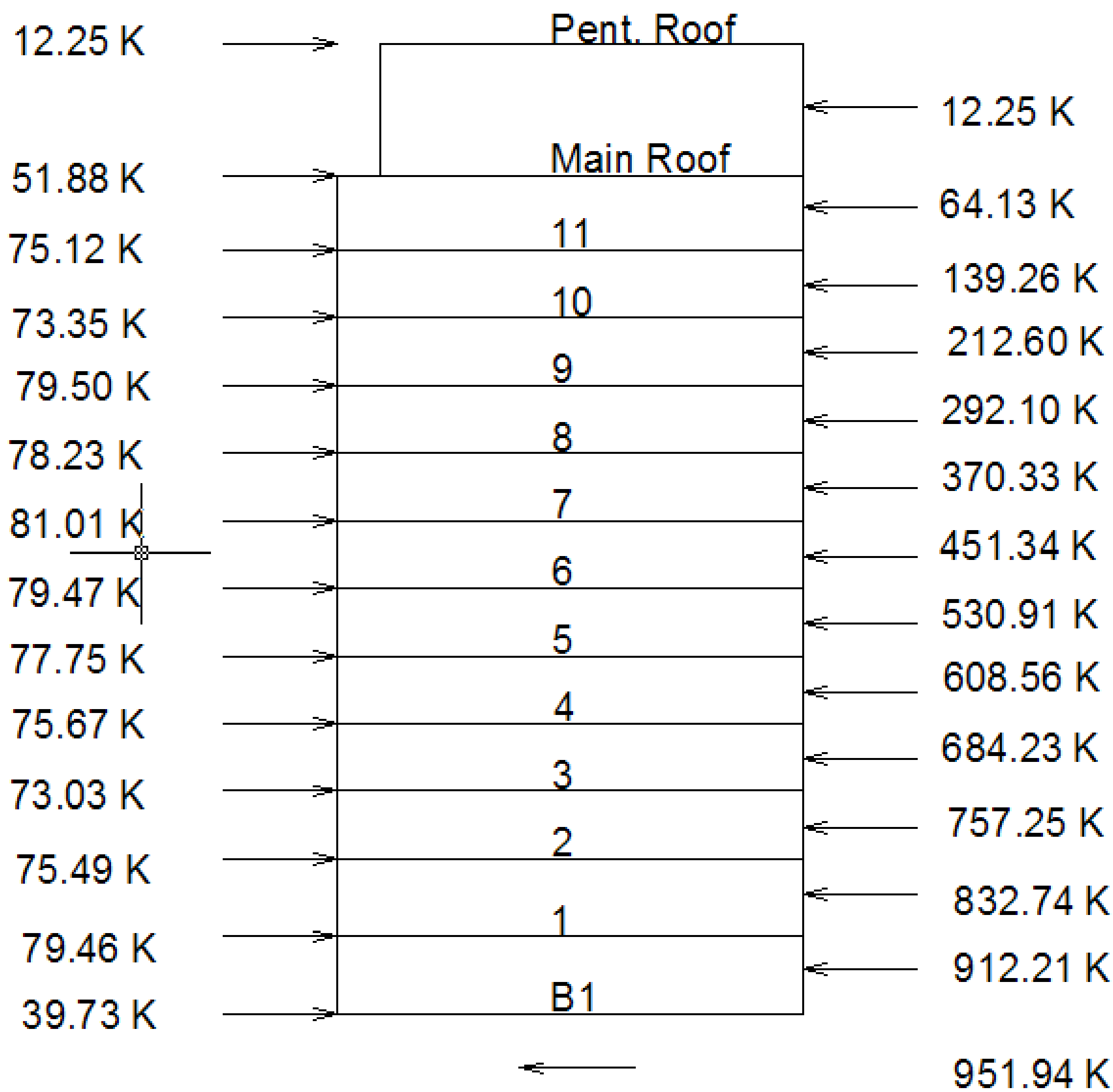


Figure 2-2: Shear Story Forces in the N-S Direction

Seismic Loads

Seismic loads for Falls Church Tower were calculated in accordance with sections 11 and 12 of ASCE 7-05. The method used to determine the seismic loads was the Equivalent Lateral Force Procedure from section 12.8 Variables used in the seismic load calculations can be found below in Table 3-1. Calculations used to determine these values can be found in Appendix E.

Table 3-1: Seismic Design Variables

Seismic Variables		
Soil Site Class	C	-
Spectral Response Acceleration (Short)	S_s	0.16
Spectral Response Acceleration (1s)	S_1	0.05
MCE Spectral Response Acceleration (Short)	S_{ms}	0.19
MCE Spectral Response Acceleration (1s)	S_{m1}	0.09
Design Spectral Acceleration (Short)	S_{DS}	0.13
Design Spectral Acceleration (1s)	S_{D1}	0.06
Fundamental Period	T	1.34 s
Long Period Transition Period	T_L	8 s
Building Period Coefficient	C_T	0.02
Period Parameter	x	0.9
Mean Roof Height	h_n	137.33 ft
Seismic Response Coefficient	C_s	0.04
Response Modification Coefficient	R	3
Importance Factor	I	1
Total Weight of Building Above Grade	W_T	45276 K
Base Shear	V	1933 K
Distribution Exponent	k	1.67

Table 3-2 provides the calculated values for shear story force, base shear, story moments, and overturning moment. Figure 3-1 illustrates the distribution of the shear story fores. When comparing the base shear from seismic forces to the base shear from wind forces the seismic base shear was larger ad therefore controlled in the design of the structure.

Table 3-2: Seismic Loads

Floor	Weight (K)	Height (f	$w_x h_x^k$	C_{vx}	F_x (K)	Story Shear (K)	Moment (ft-K)
Penthouse Roof	362.52	137.33	1347101.20	0.03	53.73	-	7379.32
Mech. Roof	135.67	131.64	469744.36	0.01	18.74	-	2466.61
Main Roof	2123	118.83	6195503.07	0.13	247.13	72.47	29366.59
11	2791.08	107.25	6863344.16	0.14	273.77	319.60	29361.88
10	2917.01	97.67	6135346.59	0.13	244.73	593.37	23902.93
9	3747.92	88.08	6633388.19	0.14	264.60	838.10	23305.77
8	3772.08	78.50	5508244.65	0.11	219.72	1102.70	17247.80
7	4049.75	68.92	4758442.36	0.10	189.81	1322.42	13081.60
6	4055.06	59.33	3709927.70	0.08	147.98	1512.23	8779.92
5	4055.06	49.75	2764660.05	0.06	110.28	1660.21	5486.38
4	4055.06	40.17	1934253.29	0.04	77.16	1770.49	3099.32
3	4019.16	30.58	1215670.53	0.03	48.49	1847.65	1482.87
2	4216.18	21.00	680809.12	0.01	27.16	1896.14	570.29
1	5201.69	10.00	243301.32	0.01	9.70	1923.30	97.05
		$\Sigma w_i h_i^k$=	**48459736.59**	**Base Shear =**	**1933**	**Overturning Momer**	**165628.33**

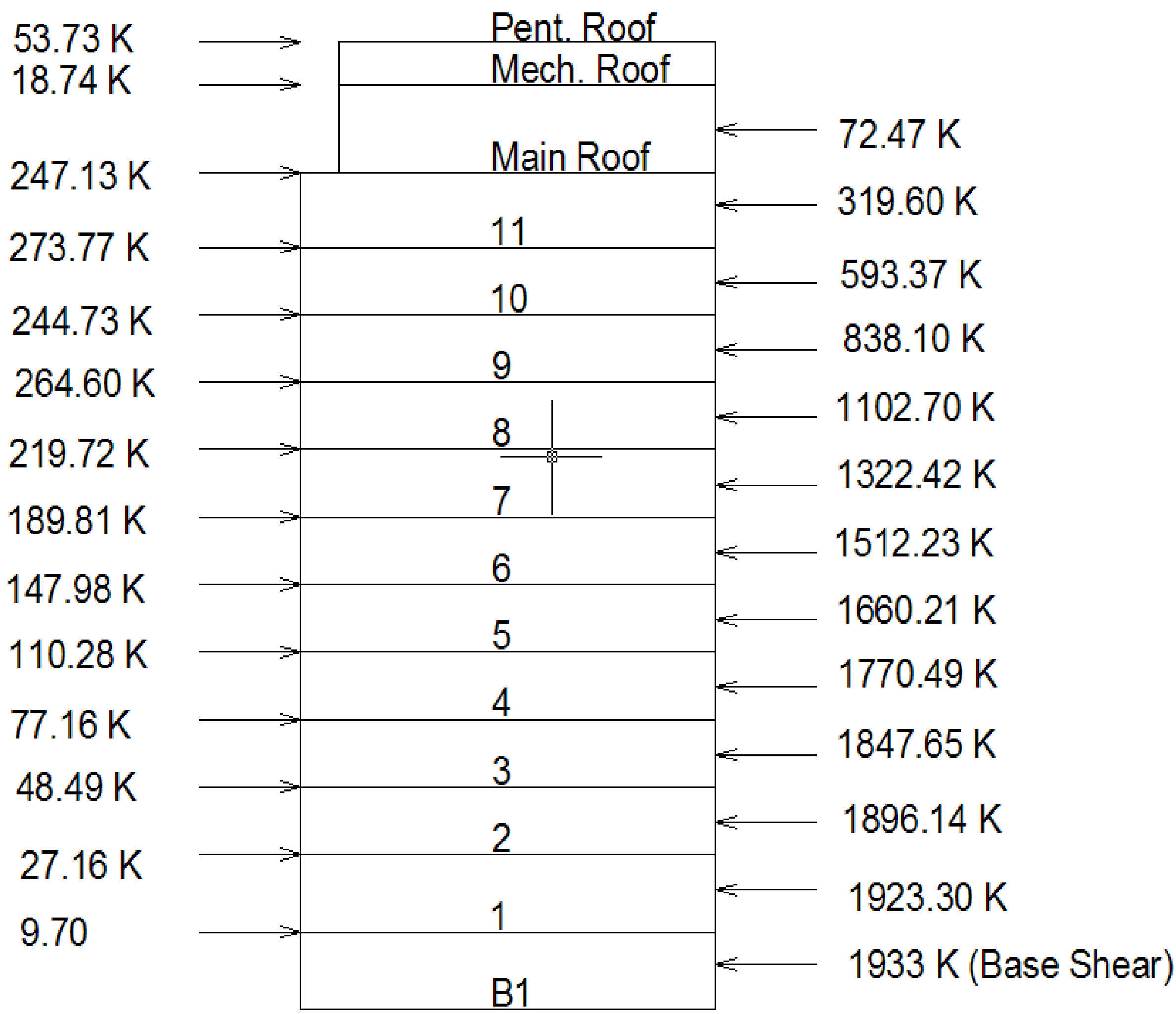


Figure 3-1: Story Shear Forces

Spot Checks

Spot checks of typical framing elements were performed in order to verify the loads used throughout this report in addition to the framing sizes by the structural engineers. These spot checks only took into account the gravity loads of the building which could lead to some variation in results seeing as the engineers also factored in lateral loads. For the purpose of this report spot checks were only done for a typical 16"x32" column of designation 120 that runs continuously from the 2^{nd} level slab to the 11^{th} level slab. A change in steel reinforcing occurs between level 2 and level 3 and was taken into account. For the time, punching shear and general loading spot checks for the post tension slabs have been forgone and will be addressed in Technical Report 2 as appropriate.

Figure 4-1 shows the location of the column in question Table 4-1 provides the calculation criteria and load values for the column. For more information regarding the spot check calculations refer to Appendix F.

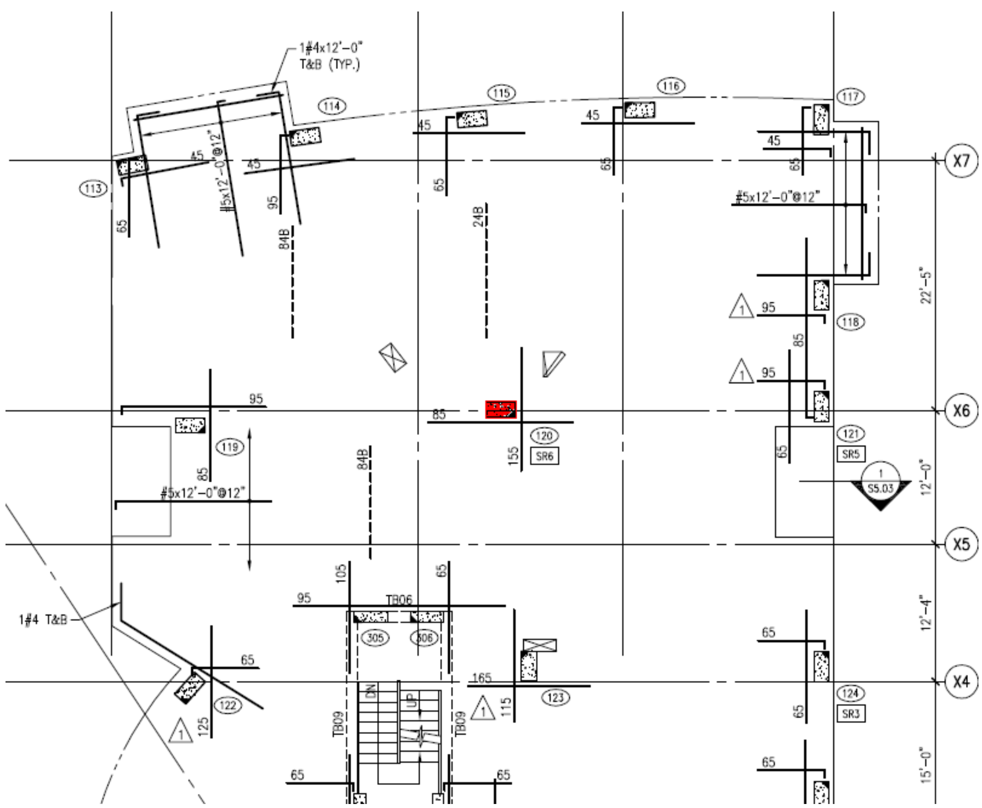


Figure 4-1: Column 120 (highlighted)

Table 4-1: Spot Check Values

Floor	Dead Load (psf)	Live Load (psf)	Total Load (psf)	Required Strength (K)	Design Strength (K
2	111.00	60.00	229.20	1486.49	1622.00
3	111.00	60.00	229.20	1321.80	1363.00
4	111.00	60.00	229.20	1156.28	1363.00
5	111.00	60.00	229.20	992.42	1363.00
6	111.00	60.00	229.20	827.74	1363.00
7	111.00	60.00	229.20	663.00	1363.00
8	111.00	60.00	229.20	498.36	1363.00
9	111.00	60.00	229.20	333.65	1363.00
10	136.50	49.25	242.00	168.98	1363.00

Conclusion

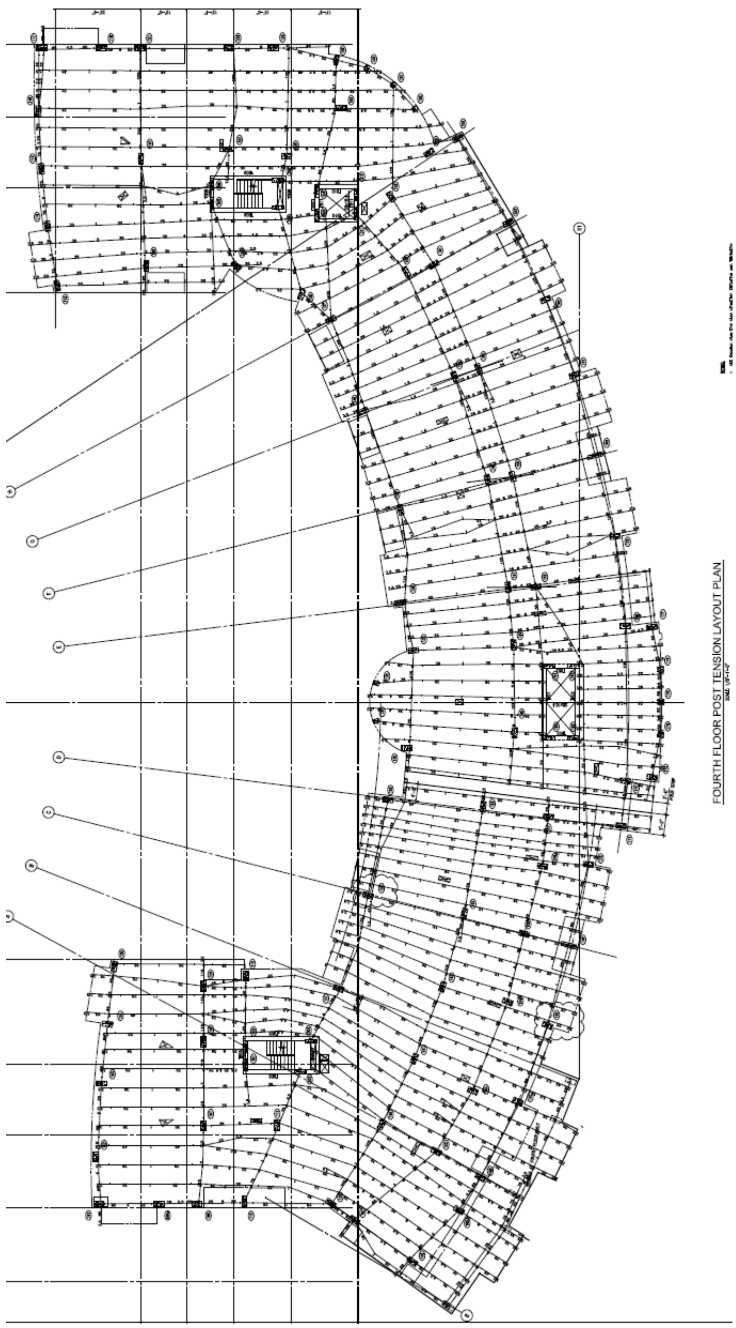
The content of the first technical report was based on the exploration of the existing conditions of the building. The beginning of the report introduced key elements of the structural system through a description of the foundation, gravity load resisting systems, and lateral load resisting systems. Basic design loads were determined using ASCE 7-05 and the values provided in the structural drawing notes.

Weight of the building was the first criteria to be determined as it is integral in calculating seismic loads and performing spot checks. This was followed by wind load calculations and seismic load calculations, the base shears of which were compared to determine the controlling load, which was determined to be the seismic loads. These results are presented in the body of the report as well as the appendix.

Spot checks were performed on a typical 16”x32” column to verify both the size and reinforcement of the column as well as the design loads used for analysis. Both the columns and the design loads proved adequate for the purposes of the report. Slab spot checks such as punching shear and serviceability will be performed in later reports that deal more directly with those topics. The results for the spot checks can be found in designated section of the report and the supporting calculations can be found in the appendix.

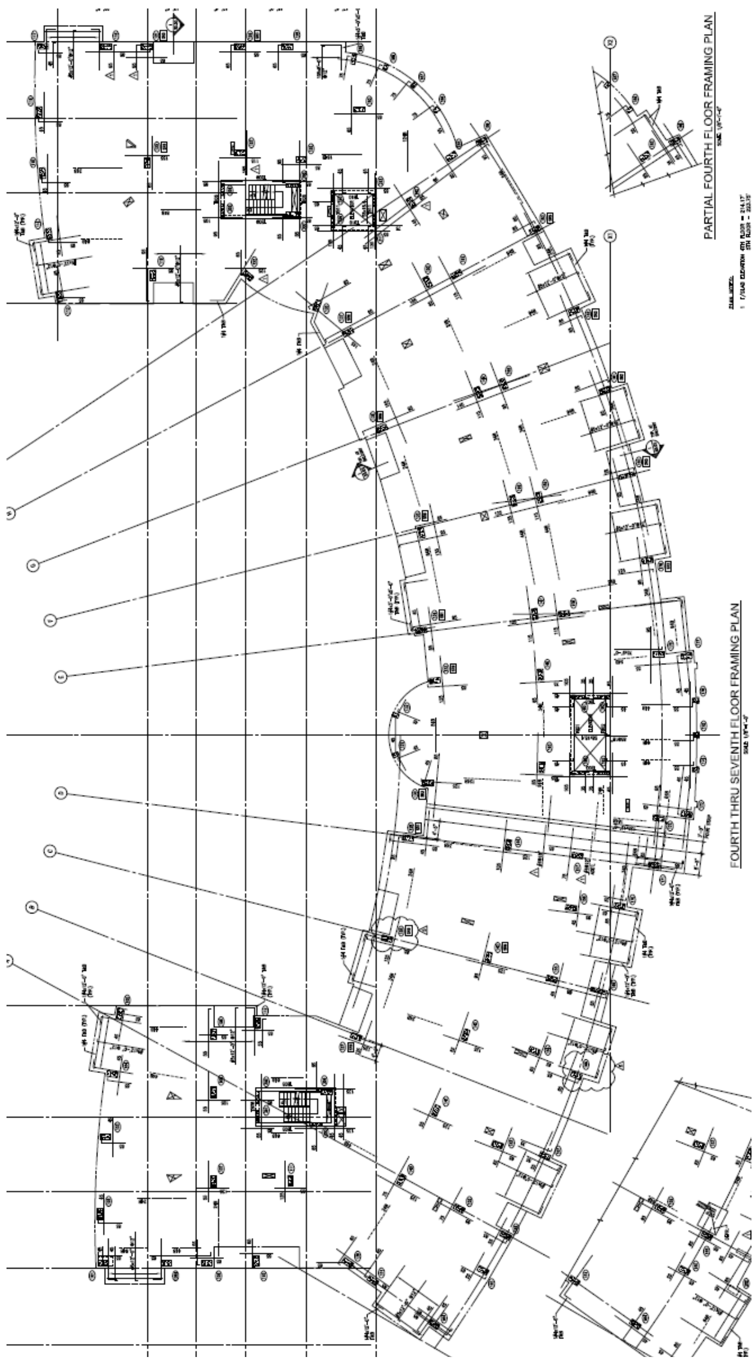
Appendix

Appendix A – Figures



Typical Post Tension Layout

Appendix A – Figures



Typical Column Layout

Appendix B – Building Weight Tables

		Column Size and Number										
Floor	Height(ft)	16x32 (3.56 sf)	12x32 (2.67sf)	24x32 (5.33sf)	12x12 (1sf)	12x18 (1.5sf)	12x36 (3sf)	16x24 (2.67sf)	12x48 (4sf)	Column Area (sf)	Total Volume (cf)	Total Weight (K)
Penthouse Roof	18.5	4				5	4			33.74	624.19	93.63
Elev. Mech. Room	8.5	4				5	6			39.74	337.79	50.67
Main Roof	10.58	4				5	12			57.74	611.06	91.66
11	9.58	38	1			8	8		1	177.95	1705.29	255.79
10	9.58	52	1			8	16	1	1	254.46	2438.49	365.77
9	9.58	61	1		1	8	16	1	2	291.5	2793.44	419.02
8	9.58	61	1		1	8	16	1	2	291.5	2793.44	419.02
7	9.58	78	1			8	16		2	348.35	3338.24	500.74
6	9.58	78	1			8	16		2	348.35	3338.24	500.74
5	9.58	78	1			8	16		2	348.35	3338.24	500.74
4	9.58	78	1			8	16		2	348.35	3338.24	500.74
3	9.58	82	1			8	16		3	366.59	3513.03	526.95
2	11	82	1			8	16		3	366.59	4032.49	604.87
1	11	84	1			14	16		3	382.71	4209.81	631.47
B1	9	118	22			12	16		3	556.82	5011.38	751.71
B2	9	118	19	5		6	16		3	566.46	5098.14	764.72
B3	9	96	11	6		3	16			455.61	4100.49	615.07
B3 Ext.	0	45	7	4			4			212.21	1909.89	286.48

Column Weights

Appendix B – Building Weight Tables

Beam #	Size	Cum. Length (ft)	Volume (cf)	Weight K
TB01	12x16	20.00	26.60	3.99
TB02	12x16	20.00	26.60	3.99
TB03	12x16	20.00	26.60	3.99
TB04	12x16	10.00	13.30	2.00
TB05	12x16	20.00	26.60	3.99
TB06	12x16	20.00	26.60	3.99
TB07	12x16	20.00	26.60	3.99
TB08	12x16	10.00	13.30	2.00
TB09	12x16	40.00	53.20	7.98
TB10	12x16	22.00	29.26	4.39
TB11	12x16	20.00	26.60	3.99
	Total Weight per Floor (1 - Main Roof)			**44.29**

Beam Weights: Level 1 – Main Roof

Appendix B – Building Weight Tables

Beam #	Size	Cum. Length (ft)	Volume (cf)	Weight K
PHB1	16x30	14.00	46.62	6.99
PHB2	16x30	22.00	73.26	10.99
PHB3	16x30	6.00	19.98	3.00
PHB4	16x30	23.00	76.59	11.49
PHB5	16x30	38.00	126.54	18.98
PHB6	16x30	25.00	83.25	12.49
PHB7	16x30	46.00	153.18	22.98
PHB8	12x12	26.00	26.00	3.90
PHB9	12x12	18.00	18.00	2.70
PHB10	12x12	16.00	16.00	2.40
PHB11	12x24	52.00	104.00	15.60
PHB12	36x12	24.00	72.00	10.80
PHB13	16x30	14.00	46.62	6.99
PHB14	16x30	21.00	69.93	10.49
PHB15	16x30	6.00	19.98	3.00
PHB16	16x24	5.00	13.35	2.00
PHB17	16x24	26.00	69.42	10.41
PHB18	16x24	4.00	10.68	1.60
PHB19	16x16	27.00	48.06	7.21
SRB1	12x16	4.00	5.32	0.80
SRB2	12x16	15.00	19.95	2.99
SRB3	12x20	17.00	28.39	4.26
SRB4	12x16	18.00	23.94	3.59
MRB1	12x16	16.00	21.28	3.19
MRB2	12x16	20.00	26.60	3.99
MRB3	12x16	10.00	13.30	2.00
MRB4	12x16	22.00	29.26	4.39
W8x15	-	573.00	-	8.60
W8x21	-	144.00	-	3.02
	Total Weight of Penthouse Roof/Mech. Roof			200.84

Beam Weights: Penthouse/Mechanical Roof

Appendix B – Building Weight Tables

Floor	Floor Height	Area (sf)	Perimeter	Slab Depth(ft)	Slab Volume (cf)	Wall Area (sf)	Slab Weight (K)	Wall Weight (K)
Penthouse Roof	18.500	2,354.000	198.000	0.500	1,177.00	1,831.50	176.55	54.95
Elev. Mech. Roof	8.500	289.000	77.000	0.500	144.50	327.25	21.68	9.82
Main Roof	10.583	17,147.000	805.830	0.583	10,002.42	6,422.80	1,500.36	192.68
11	9.583	20,134.000	920.083	0.583	11,744.83	8,817.16	1,761.73	264.51
10	9.583	20,238.000	935.500	0.583	11,805.50	8,964.90	1,770.83	268.95
9	9.583	27,052.000	1,103.583	0.583	15,780.33	9,770.27	2,367.05	293.11
8	9.583	27,052.000	1,103.583	0.583	15,780.33	10,575.64	2,367.05	317.27
7	9.583	28,776.000	1,140.500	0.583	16,786.00	10,752.52	2,517.90	322.58
6	9.583	28,776.000	1,140.500	0.583	16,786.00	10,929.41	2,517.90	327.88
5	9.583	28,776.000	1,140.500	0.583	16,786.00	10,929.41	2,517.90	327.88
4	9.583	28,776.000	1,140.500	0.583	16,786.00	10,929.41	2,517.90	327.88
3	9.583	28,193.000	1,156.917	0.583	16,445.92	11,008.07	2,466.89	330.24
2	11.000	28,992.000	1,179.917	0.583	16,912.00	12,032.91	2,536.80	360.99
1	11.000	30,708.000	1,175.830	0.750	23,031.00	12,934.13	3,454.65	388.02
B1	9.000	55,836.000	1,272.670	0.750	41,877.00	12,726.70	6,281.55	805.98
B2	9.000	53,587.000	1,079.750	0.750	40,190.25	9,717.75	6,028.54	939.41
B3	9.000	46,332.000	1,019.250	0.750	34,749.00	9,173.25	5,212.35	886.78
B3 Ext.	0.000	13,398.000	511.000	0.417	5,582.50	2,299.50	837.38	222.29
Total							**46854.99**	**6641.23**

Slab and Wall Weights

Appendix B – Building Weight Tables

Floor	Ext. Wall Weight (K)	Column Weight (K)	Slab Weight (K)	Beam Weight (K)	Area (sf)	Add. Dead Load (psf)	Total Weight (K)
Penthouse Roof	54.95	-	170.67	100.42	2354.00	15.50	362.52
Mech. Roof	9.82	-	20.95	100.42	289.00	15.50	135.67
Main Roof	171.83	190.76	1450.35	44.29	17147.00	15.50	2123.00
11	264.51	255.79	1703.00	44.29	20134.00	26.00	2791.08
10	268.95	365.79	1711.80	44.29	20238.00	26.00	2917.01
9	293.11	419.02	2288.15	44.29	27052.00	26.00	3747.92
8	317.27	419.02	2288.15	44.29	27052.00	26.00	3772.08
7	322.58	500.74	2433.97	44.29	28776.00	26.00	4049.75
6	327.88	500.74	2433.97	44.29	28776.00	26.00	4055.06
5	327.88	500.74	2433.97	44.29	28776.00	26.00	4055.06
4	327.88	500.74	2433.97	44.29	28776.00	26.00	4055.06
3	330.24	526.95	2384.66	44.29	28193.00	26.00	4019.16
2	360.99	604.87	2452.24	44.29	28992.00	26.00	4216.18
1	388.02	631.47	3339.50	44.29	30708.00	26.00	5201.69
B1	805.98	751.71	6072.17	182.00	55836.00	10.00	8370.22
B2	939.41	764.72	5827.59	167.73	53587.00	10.00	8235.32
B3	886.78	615.07	5038.61	206.70	46332.00	10.00	7210.47
B3 Ext.	222.29	286.48	809.46	33.53	13398.00	10.00	1485.75
Total Building Weight							**70802.99**

Total Building Weight

Appendix C – Snow Loads

Snow Loads

$P_f = 0.7 C_e C_t I p_g$ $\quad P_g = 25$ psf (ASCE 7-05 Figure 7-1)

$C_e = 1.0$ (Table 7-2)

$C_t = 1.1$ (Table 7-3)

$I = 1.0$

$P_f = 0.7(25)(1.0)(1.1)(1.0) = 19.25$ psf $\quad$ * the load used by the engineer = 27.5 psf

Drift

$h_b = P_f / \gamma$; $\gamma = 0.13 p_g + 14 \leq 30$

$= 0.13(25) + 14 = 17.25$

$h_b = \frac{15.75}{17.25} = 0.913$ ft

$h_c = 20.17$ ft

$h_b / h_c = \frac{0.913 \text{ ft}}{20.17 \text{ ft}} = 0.045 < 0.2$ ∴ drift does not need to be calculated

Appendix D – Wind Loads

Nathan Eck | Wind Loads | 9-27-10 | 1

Location: Arlington, VA
Residential Building:
Topography: Homogeneous

Basic Wind Speed:	Exposure	Building Classification
Arlington, VA = 90mph	B	Category II

Velocity Pressure (q_z)

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

$I = 1.00$ from Table 6-1
$K_d = 0.85$ from Table 6-4
$V = 90\text{mph}$ from Figure 6-1

Height Above Ground Level, z	K_z*
0-15	0.57
20	0.62
25	0.66
30	0.70
40	0.76
50	0.81
60	0.85
70	0.89
80	0.93
90	0.96
100	0.99
120	1.04
140	1.09
160	1.13
180	1.17
200	1.20
250	1.28
300	1.35
350	1.41
400	1.47
450	1.52
500	1.56

*K_z values per level were determined through interpolation and can be referenced on the q_z value chart

$K_{zt} = 1.00$ based on Section 6.5.7

*refer to chart for q_z values

Appendix D – Wind Loads

2

Gust Effect Factor

n_1 = natural frequency $= \frac{100}{H}$; $H = 136'4''$

$n_1 = \frac{100}{136.33} = \boxed{0.734\ Hz} < 1\ Hz$ ∴ the building is flexible

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

$g_v = \boxed{3.4}$

$g_Q = \boxed{3.4}$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}$$

$$g_R = \sqrt{2\ln(3600)(0.734)} + \frac{0.577}{\sqrt{2\ln(3600)(0.734)}}$$

$$g_R = 3.970 + 0.145 = \boxed{4.12}$$

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}$; $\bar{z} = \max\left| \begin{matrix} 0.6h \\ z_{min} \end{matrix} \right.$; $z_{min} = 30 ft$

$h = 117.83\ ft$

$\bar{z} = \max\left| \begin{matrix} 70.7 \\ 30 \end{matrix} \right. \Rightarrow \bar{z} = \boxed{70.7}$

$c = 0.30$

$$I_{\bar{z}} = 0.3\left(\frac{33}{70.7}\right)^{1/6} = \boxed{0.264}$$

$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$; $R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}}$; $N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$; $n_1 = 0.734$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{70.7}{33}\right)^{1/3} = \boxed{412.53}$

β = assumed 1.5% or 0.015

$$R_n = \frac{7.47(4.21)}{(1+10.3(4.21))^{5/3}} = \boxed{0.057}$$

$$N_1 = \frac{(0.734)(412.53)}{71.86}$$

$N_1 = \boxed{4.21}$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{70.7}{33}\right)^{1/4}(90)\left(\frac{88}{60}\right)$$

$\bar{V}_{\bar{z}} = \boxed{71.86}$

$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta})$; $\eta = 4.6 n_1\left(\frac{h}{\bar{V}_{\bar{z}}}\right)$

$= 4.6(0.734)(117.83/71.86)$

$= \boxed{5.54}$

$$= \frac{1}{5.54} - \frac{1}{2(5.54^2)}\left(1 - e^{-2(5.54)}\right)$$

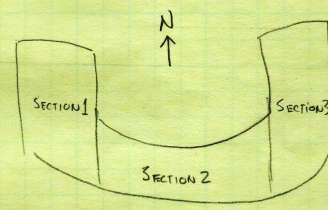
$= \boxed{0.164}$

Appendix D – Wind Loads

3

Note: In order to obtain values for B and L in the calculation of R_B and R_L the building will be divided into three sections. The values for B and L will be taken for each section, assuming they are all rectangular for ease of calculation. This will be done for both N-S and E-W directions. The three sections will be specified on floors 2-7 seeing as these floors are the most typical layout for the building.

Section	Variable	N-S	E-W
1	B_1	65'	118'
	L_1	118'	65'
2	B_2	174.18'	66'
	L_2	66'	174.18'
3	B_3	65'	104'
	L_3	104'	65'



R_B Values

Section 1

N-S

$R_B = R_h$ for $\eta_1 = 4.6 n_1 B / \bar{V}_z$

$= 4.6(0.734)(65/71.86)$

$= 3.054$

$$R_B = \frac{1}{3.05} - \frac{1}{2(3.05^2)}\left(1 - e^{-2(3.05)}\right)$$

$= 0.274$

E-W

$\eta = 4.6(0.734)(118/71.86) = 5.54$

$R_B = 0.164$

Section 2

N-S

$\eta = 4.6(0.734)(174.18)/71.86$

$= 8.18$

$R_B = 0.115$

E-W

$\eta = 4.6(0.734)(66/71.86)$

$= 3.10$

$R_B = 0.271$

Section 3

N-S

$\eta = 4.6(0.734)(65/71.86)$

$= 3.05$

$R_B = 0.274$

E-W

$\eta = 4.6(0.734)(104/71.86)$

$= 4.89$

$R_B = 0.184$

Appendix D – Wind Loads

4

R_L Values

Section 1

N-S

$R_L = R_h$ for $\eta = 15.4 n_1 L/\bar{V}_{\bar{z}}$
$= 15.4(0.734)(118/71.86)$
$= 18.56$

$R_L = \boxed{0.052}$

E-W

$\eta = 15.4(0.734)(65/71.86)$
$= 10.22$

$R_L = \boxed{0.093}$

Section 2

N-S

$\eta = 15.4(0.734)(66/71.86)$
$= 10.38$

$R_L = \boxed{0.092}$

E-W

$\eta = 15.4(0.734)(174.18/71.86)$
$= 27.4$

$R_L = \boxed{0.036}$

Section 3

N-S

$\eta = 15.4(0.734)(104/71.86)$
$= 16.36$

$R_L = \boxed{0.059}$

E-W

$\eta = 15.4(0.734)(65/71.86)$
$= 10.22$

$R_L = \boxed{0.093}$

*Refer to R value chart for R values for each section

Background Response

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

Section 1

N-S

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{65+117.83}{412.53}\right)^{0.63}}}$$

$Q = \boxed{0.852}$

E-W

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{118+117.83}{412.53}\right)^{0.63}}}$$

$Q = \boxed{0.832}$

Section 2

N-S

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{174.18+117.83}{412.53}\right)^{0.63}}}$$

$Q = \boxed{0.815}$

E-W

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{66+117.83}{412.53}\right)^{0.63}}}$$

$Q = \boxed{0.852}$

Section 3

N-S

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{65+117.83}{412.53}\right)^{0.63}}}$$

$Q = \boxed{0.852}$

E-W

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{104+117.83}{412.53}\right)^{0.63}}}$$

$Q = \boxed{0.837}$

Appendix D – Wind Loads

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Gust Effect Factor

* Largest Gust Factors controls

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

Section 1

N-S: $G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4)^2(0.852^2)+(4.12^2)(0.308^2)}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.886}$ *

E-W: $G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4)^2(0.832)^2+(4.12)^2(0.242)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.859}$

Section 2

N-S: $G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4^2)(0.815^2)+(4.12)^2(0.203)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.842}$

E-W: $G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4^2)(0.852)^2+(4.12)^2(0.304)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.885}$ *

Section 3

N-S: $G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4)^2(0.852)^2+(4.12)^2(0.309)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.886}$ *

E-W: $G_f = 0.925\left(\frac{1+1.7(0.264)\sqrt{(3.4)^2(0.837)^2+(4.12)^2(0.256)^2}}{1+1.7(3.4)(0.264)}\right) = \boxed{0.865}$

External Pressure Coefficients (Figure 6-8)

C_p: Windward = 0.8

Leeward:

	N-S	E-W
Section 1	-0.34	-0.5
Section 2	-0.5*	-0.27
Section 3	-0.38	-0.5*

* these values control for their respective directions

Internal Pressure Coefficient (Figure 6-5)

$GC_{pi} = \pm 0.18$

Appendix D – Wind Loads

Design Wind Pressures

Windward: $p = q_z G C_p - q_h (G C_{pi})$; $q_h = 10.05$

Sample Calc: $p = (10.05\text{ psf})(0.886)(0.8) - 18.24(-0.18) = 10.41\text{ psf}$

Leeward: $p = q_h G C_p - q_h (G C_{pi})$; $q_h = 18.24$

$p = (18.24)(0.886)(-0.5) + 18.24(-0.18) = -11.36\text{ psf}$

Appendix D – Wind Loads

Floor	Height Above Ground (ft)	K_z	K_{zt}	K_d	V	I	q_z (psf)
B1	0.00	0.570	1.00	0.85	90.00	1.00	10.05
1.000	10.00	0.570	1.00	0.85	90.00	1.00	10.05
2.000	21.00	0.628	1.00	0.85	90.00	1.00	11.07
3.000	30.58	0.704	1.00	0.85	90.00	1.00	12.41
4.000	40.17	0.761	1.00	0.85	90.00	1.00	13.41
5.000	49.75	0.809	1.00	0.85	90.00	1.00	14.26
6.000	59.33	0.847	1.00	0.85	90.00	1.00	14.93
7.000	68.92	0.886	1.00	0.85	90.00	1.00	15.62
8.000	78.50	0.924	1.00	0.85	90.00	1.00	16.29
9.000	88.08	0.954	1.00	0.85	90.00	1.00	16.81
10.000	97.67	0.983	1.00	0.85	90.00	1.00	17.33
11.000	107.25	1.008	1.00	0.85	90.00	1.00	17.77
Penthouse	118.83	1.035	1.00	0.85	90.00	1.00	18.24

Velocity Pressure Values

R Values						
Section	Direction	Rn	Rh	R_B	R_L	R
1	N-S	0.057	0.164	0.274	0.052	0.308
	E-W	0.057	0.164	0.164	0.093	0.242
2	N-S	0.057	0.164	0.115	0.092	0.203
	E-W	0.057	0.164	0.271	0.036	0.304
3	N-S	0.057	0.164	0.274	0.059	0.309
	E-W	0.057	0.164	0.184	0.093	0.256

R Values

Design Wind Pressure P (psf)							
Floor	Height Above Ground (ft)	K_z	q_z(psf)	q_h(psf)	Windward (psf)	Leeward (psf)	Total Pressure (psf)
B1	0.000	0.570	10.05	18.24	10.41	-11.36	21.77
1.000	10.000	0.570	10.05	18.24	10.41	-11.36	21.77
2.000	21.000	0.628	11.07	18.24	11.13	-11.36	22.49
3.000	30.580	0.704	12.41	18.24	12.08	-11.36	23.44
4.000	40.170	0.761	13.41	18.24	12.79	-11.36	24.15
5.000	49.750	0.809	14.26	18.24	13.39	-11.36	24.75
6.000	59.330	0.847	14.93	18.24	13.87	-11.36	25.23
7.000	68.920	0.886	15.62	18.24	14.35	-11.36	25.72
8.000	78.500	0.924	16.29	18.24	14.83	-11.36	26.19
9.000	88.080	0.954	16.81	18.24	15.20	-11.36	26.56
10.000	97.670	0.983	17.33	18.24	15.57	-11.36	26.93
11.000	107.250	1.008	17.77	18.24	15.88	-11.36	27.24
Penthouse	118.830	1.035	18.24	18.24	16.21	-11.36	27.58

Design Wind Pressure

Appendix E – Seismic

Seismic Load

Spectral Response Acceleration Parameters

Soil Site Class: C

$S_s = 0.16$ (ASCE-7 Figure 22-1)

$S_1 = 0.051$ (ASCE-7 Figure 22-2)

$S_{MS} = S_s F_a$; $F_a = 1.2$ (ASCE-7 Table 11.4-1)

$S_{MS} = (0.16)(1.2) = \boxed{0.192}$

$S_{M1} = S_1 F_v$; $F_v = 1.7$ (ASCE-7 Table 11.4-2)

$S_{M1} = (0.051)(1.7) = \boxed{0.087}$

Design Spectral Acceleration Parameters

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.192) = 0.128$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.087) = 0.058$

Seismic Base Shear

$T = C_T h_n^x$; $C_T = 0.016$, $x = 0.9$, $h_n = 137.33\text{ft}$

$T = (0.016)(137.33)^{0.9} = 1.34$ sec

$T_L = 8$sec (Figure 22-15)

$T_L > T$

$C_s = \frac{S_{DS}}{R/I}$; $R = 3$, $I = 1$ $\qquad C_s = \frac{0.128}{(3/1)} = 0.0427$

$V = C_s W_T$; $W_T = 70497\text{K} - (1476 + 7190 + 8210 + 8345) = 45276\text{ K}$

B1-B3 Ext. (subgrade)

$V = (0.0427)(45276) = 1933\text{ K}$

Vertical Distribution of Seismic Forces

$F_x = C_{vx} V$; $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$; $k = 1.67$

(Values for $\sum_{i=1}^{n} w_i h_i^k$, $w_x h_x^k$, C_{vx}, and F_x are given in seismic loading chart

Appendix E – Seismic Loads

Floor	Weight (K)	Height (ft)	$w_x h_x^k$	C_{vx}	F_x (K)	Story Shear (K)	Moment (ft-K)
Penthouse Roof	362.52	137.33	1347101.20	0.03	53.73	-	7379.32
Mech. Roof	135.67	131.64	469744.36	0.01	18.74	-	2466.61
Main Roof	2123	118.83	6195503.07	0.13	247.13	72.47	29366.59
11	2791.08	107.25	6863344.16	0.14	273.77	319.60	29361.88
10	2917.01	97.67	6135346.59	0.13	244.73	593.37	23902.93
9	3747.92	88.08	6633388.19	0.14	264.60	838.10	23305.77
8	3772.08	78.50	5508244.65	0.11	219.72	1102.70	17247.80
7	4049.75	68.92	4758442.36	0.10	189.81	1322.42	13081.60
6	4055.06	59.33	3709927.70	0.08	147.98	1512.23	8779.92
5	4055.06	49.75	2764660.05	0.06	110.28	1660.21	5486.38
4	4055.06	40.17	1934253.29	0.04	77.16	1770.49	3099.32
3	4019.16	30.58	1215670.53	0.03	48.49	1847.65	1482.87
2	4216.18	21.00	680809.12	0.01	27.16	1896.14	570.29
1	5201.69	10.00	243301.32	0.01	9.70	1923.30	97.05
		$\Sigma w_i h_i^k$ =	**48459736.59**	**Base Shear =**	**1933**	**Overturning Moment =**	**165628.33**

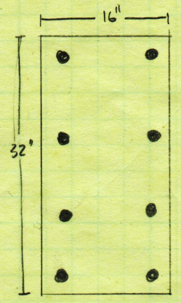
Design Seismic Loads

Appendix F – Spot Checks

1

Spot Check: Typical Column 120 (Floors 2 – 10)

Level 10



Rebar: 8#9 $A_s = 8(1 in^2) = 8 in^2$

$f'_c = 5000$ psi $A_g = 16 in \times 32 in = 512 in^2 = 3.56 ft^2$

$f_y = 60{,}000$ psi

$A_T = 28.5' \times 24.5' = 698.25$ sf

$W_u = 1.2DL + 1.6LL$; DL: 10" Slab = 121 psf
Waterproofing = 5.5 psf
Misc. = 10 psf
Total DL = $\boxed{136.5 \text{ psf}}$

LL: Snow = 19.25 psf
Roof = 30 psf
Total LL = $\boxed{49.25 \text{ psf}}$

$W_u = 1.2(136.5) + 1.6(49.25) = 242$ psf

$P_u = w_u A_T = (242 \text{ psf})(698.25 \text{ sf}) = 168977 \text{ lbs} = 168.98$ K

ACI 318-08: Equation 10-2

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

$= 0.8(0.65)[0.85(5)(512-8) + 60(8)]$

$= 1363 \text{ K} > 168.98\text{K} = P_u$ ∴ okay

Level 9 DL: 7" Slab = 85 psf LL: Private Rooms = 60 psf
Finish = 16 psf
Misc = 10 psf
Total DL = 111 psf

$w_u = 1.2(111) + 1.6(60) = 229.2$ psf

$P_u = \frac{(229.2 \text{ psf})(698.25 \text{ sf})}{1000} + 168.98\text{K} + 4.65 = 333.65\text{K} < 1363 \text{ K} = \phi P_{N\,MAX}$ ∴ okay

↳ column weight (K)

Level 8

$P_u = \frac{2(229.2)(698.25)}{1000} + 168.98 + 2(4.65) = 498.36 \text{ K} < 1363 \text{K} = \phi P_{N\,MAX}$ ∴ okay

Level 7 $P_u = \frac{3(229.2)(698.25)}{1000} + 168.98 + 3(4.65) = 663.0 \text{ K} < 1363 \text{ K} = \phi P_{N\,MAX}$ ∴ okay

Level 6

$P_u = \frac{4(229.2)(698.25)}{1000} + 168.98 + 4(4.65) = 827.74 \text{ K} < 1363 \text{ K} = \phi P_{N\,MAX}$ ∴ okay

Appendix F – Spot Checks

2

Spot Check: Typical Column (120) (Floors 2-10)

Level 5|

$P_u = \frac{5(229.2)(698.25)}{1000} + 168.98 + 5(4.65) = 992.42\,K < 1363\,K = \phi P_{NMAX} \therefore$ okay

Level 4|

$P_u = \frac{6(229.2)(698.25)}{1000} + 168.98 + 6(4.65) = 1156.28\,K < 1363\,K = \phi P_{NMAX} \therefore$ okay

Level 3|

$P_u = \frac{7(229.2)(698.25)}{1000} + 168.98 + 7(4.65) = 1321.8\,K < 1363\,K = \phi P_{NMAX} \therefore$ okay

Level 2| Rebar = 12#11 $A_s = 12(1.41) = 16.92\ in^2$

$\phi P_{NMAX} = 0.8(0.65)[0.85(5)(512 - 16.92) + 60(16.92)] = 1622\,K$

$P_u = \frac{8(229.2)(698.25)}{1000} + 168.98 + 8(4.65) = 1486.49\,K < 1622 = \phi P_{NMAX} \therefore$ okay

Level 1|

$P_u =$

Appendix F – Spot Checks

Floor	Dead Load (psf)	Live Load (psf)	Total Load (psf)	Required Strength (K)	Design Strength (K
2	111.00	60.00	229.20	1486.49	1622.00
3	111.00	60.00	229.20	1321.80	1363.00
4	111.00	60.00	229.20	1156.28	1363.00
5	111.00	60.00	229.20	992.42	1363.00
6	111.00	60.00	229.20	827.74	1363.00
7	111.00	60.00	229.20	663.00	1363.00
8	111.00	60.00	229.20	498.36	1363.00
9	111.00	60.00	229.20	333.65	1363.00
10	136.50	49.25	242.00	168.98	1363.00