
Technical Report 3

Falls Church Tower

Falls Church, VA



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Structural Option

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Executive Summary

Falls Church Tower is a fairly complex building. This is mainly due to its long, curved midsection which connected to shorter sections at either end that run almost perpendicular to the midsection. Add to this the fact that the floor area decreases as the building's height increases. For these reasons the engineers developed a complex layout for gravity and lateral load resisting systems. The gravity system consists of a flat plate system with post tensioned strands running in the North-South direction. The lateral system consists of an irregular array of columns with a variety of sizes .

Due to this complexity the building was modeled in ETABS in order to determine such factors as shear, bending, and drift. Additionally, the critical overturning moment was determined along with the resisting moment of the building.

The overturning moment was controlled by seismic forces which produced 1873116 in-K which was largely exceeded by the building's resisting moment of 61244959 in-K. Because of this overturning was not an issue.

The total story drift for both wind and seismic loads came in under the allowable drift limits. Seismic drift totaled 0.205" which is well below the 30.96" allowed. The total due to wind was 1.338' which is only 1/3 of the allowable drift of 4.08".

For shear, the controlling load case was torsion due to wind which produced a maximum shear of 22.7 Kips in the columns that frame the corridor. Additional strength checks were performed on the 12"x48" column located in the southern facade of the building to check its ability to carry simultaneous axial and bending loads.

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 an ordinary concrete moment frame.

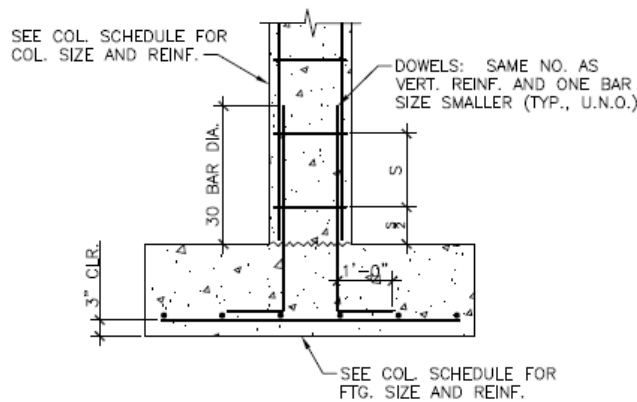


Foundation

The foundation system of Falls Church Tower was designed in accordance with the geotechnical report provided by Whitlock, Dairymples, 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



TYPICAL COLUMN FOOTING DETAIL

Figure 1-1

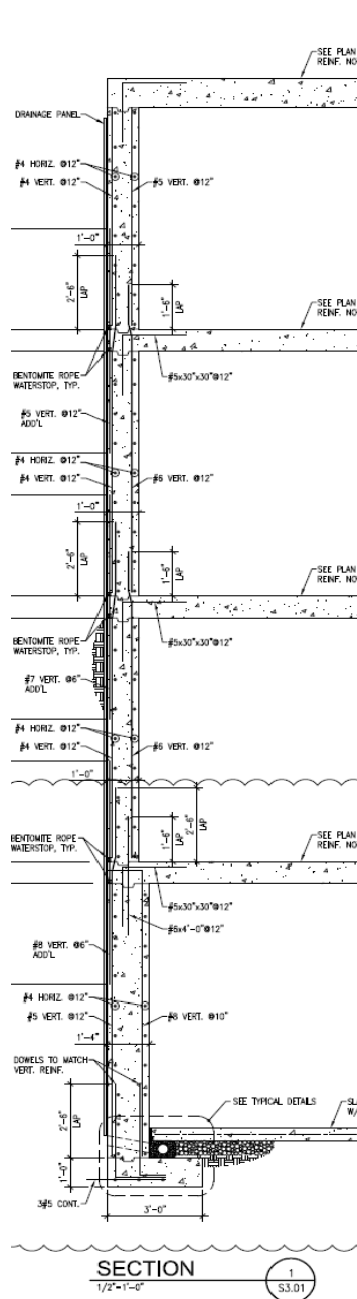


Figure 1-2

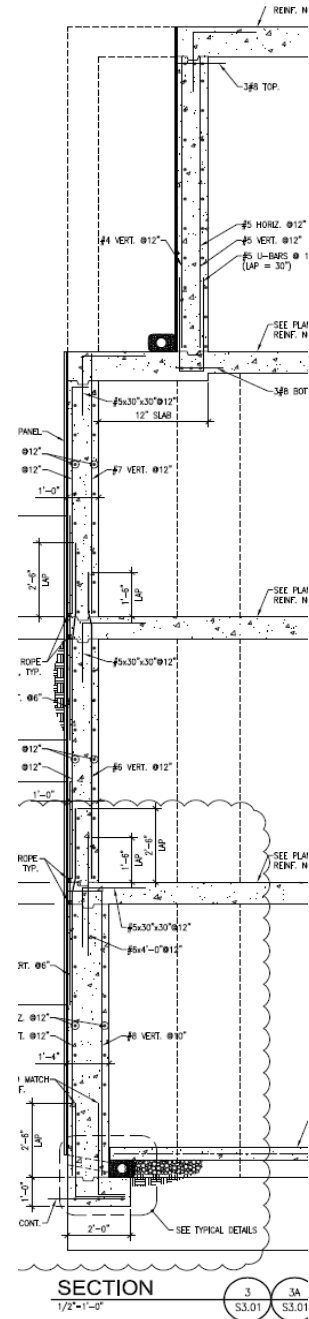


Figure 1-3

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 ksi, 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.

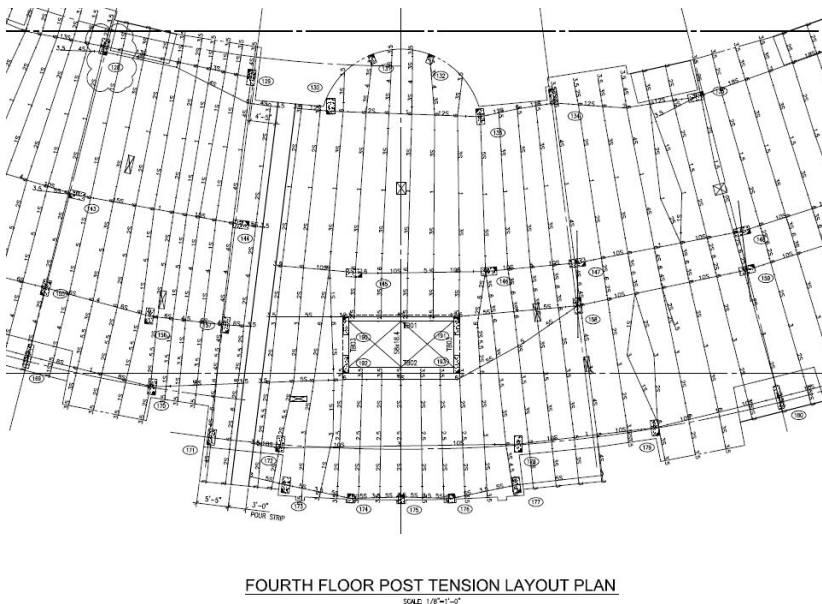


Figure 1-4
(for a larger
view refer to
Appendix A)

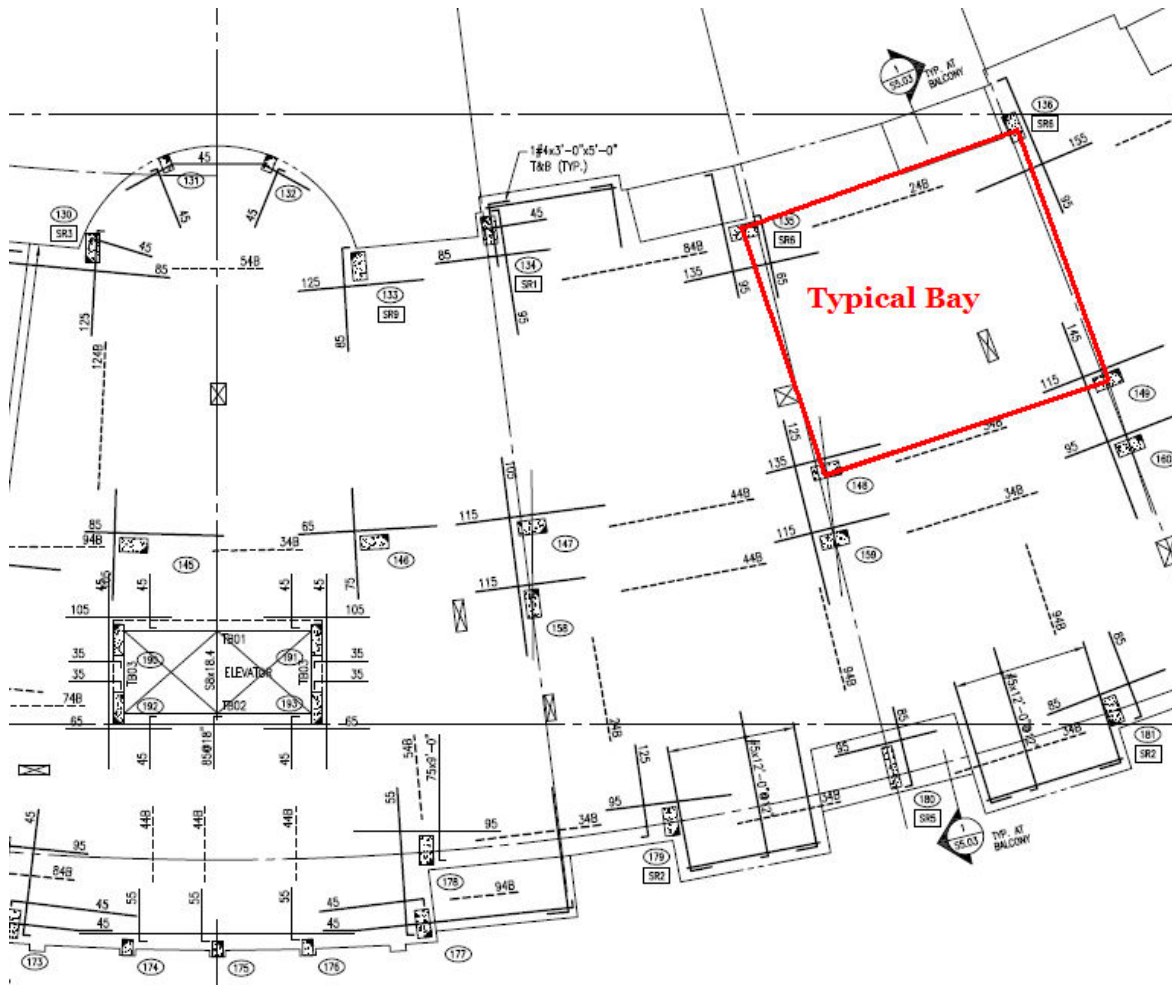


Figure 1-5

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. The irregular layout of the columns is shown in Figure 1-6

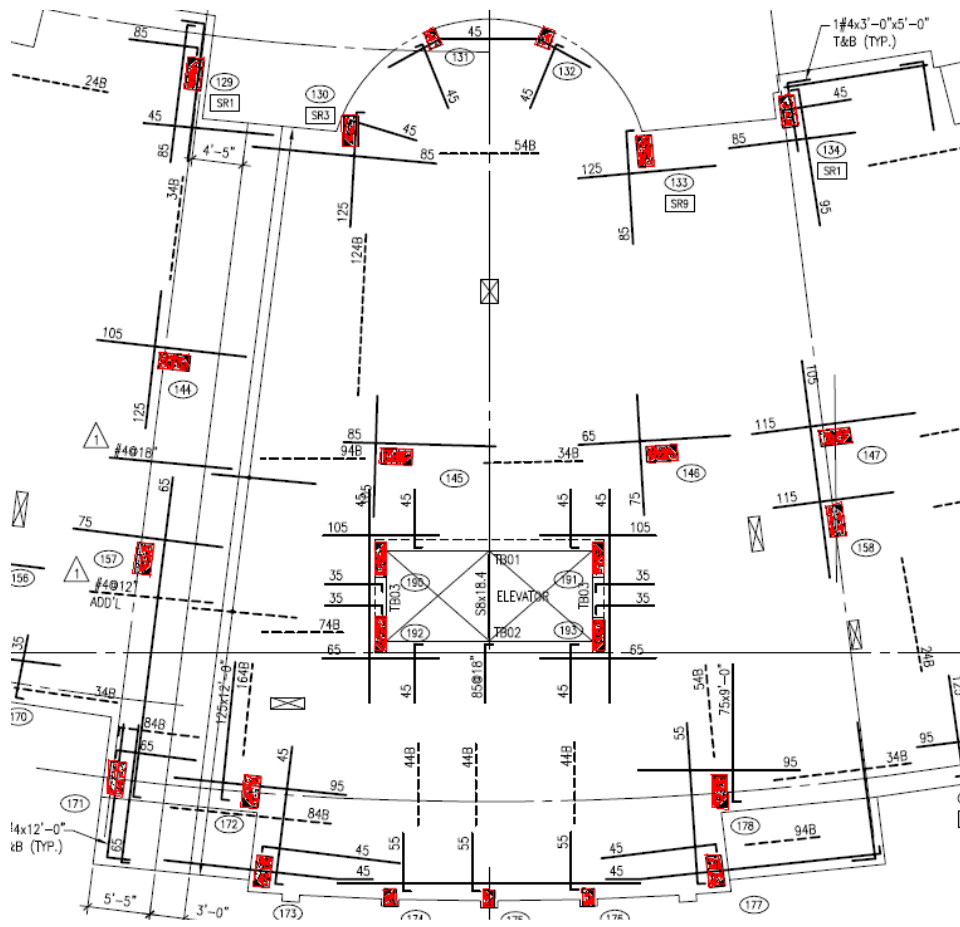


Figure 1-6 : Column Layout

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_p (psf)	Windward (psf)	Leeward (psf)	Total Pressure (psf)
B1	0.00	0.570	10.05	18.24	10.41	-11.36	21.77
1	10.00	0.570	10.05	18.24	10.41	-11.36	21.77
2	21.00	0.628	11.07	18.24	11.13	-11.36	22.49
3	30.58	0.704	12.41	18.24	12.08	-11.36	23.44
4	40.17	0.761	13.41	18.24	12.79	-11.36	24.15
5	49.75	0.809	14.26	18.24	13.39	-11.36	24.75
6	59.33	0.847	14.93	18.24	13.87	-11.36	25.23
7	68.92	0.886	15.62	18.24	14.35	-11.36	25.72
8	78.50	0.924	16.29	18.24	14.83	-11.36	26.19
9	88.08	0.954	16.81	18.24	15.20	-11.36	26.56
10	97.67	0.983	17.33	18.24	15.57	-11.36	26.93
11	107.25	1.008	17.77	18.24	15.88	-11.36	27.24
MainRoof	117.83	1.035	18.24	18.24	16.21	-11.36	27.58
Mech. Roof	126.33	1.056	18.61	18.24	-	-11.36	27.83
Pent. Roof	136.33	1.081	19.50	18.24	-	-11.36	28.46

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	10.000	21.77	39.73	957.24	0.00	0
1	11.000	21.77	79.46	917.50	10.00	794.64
2	9.583	22.49	75.49	838.04	21.00	1585.30
3	9.583	23.44	73.03	762.55	30.58	2233.19
4	9.583	24.15	75.67	689.52	40.17	3039.55
5	9.583	24.75	77.75	613.85	49.75	3868.06
6	9.583	25.23	79.47	536.10	59.33	4714.78
7	9.583	25.72	81.01	456.64	68.92	5583.17
8	9.583	26.19	78.23	375.63	78.50	6140.84
9	9.583	26.56	79.50	297.40	88.08	7002.54
10	9.583	26.93	73.35	217.90	97.67	7163.62
11	10.583	27.24	75.12	144.55	107.25	8056.97
Main Roof	18.500	27.58	53.09	69.43	117.83	6255.59
Mech. Roof	-	27.83	3.55	16.34	126.33	448.47
Penthouse Roof	-	28.46	12.79	12.79	136.33	1743.66
		Base Shear =	957.24		Overturning Moment =	58630.39

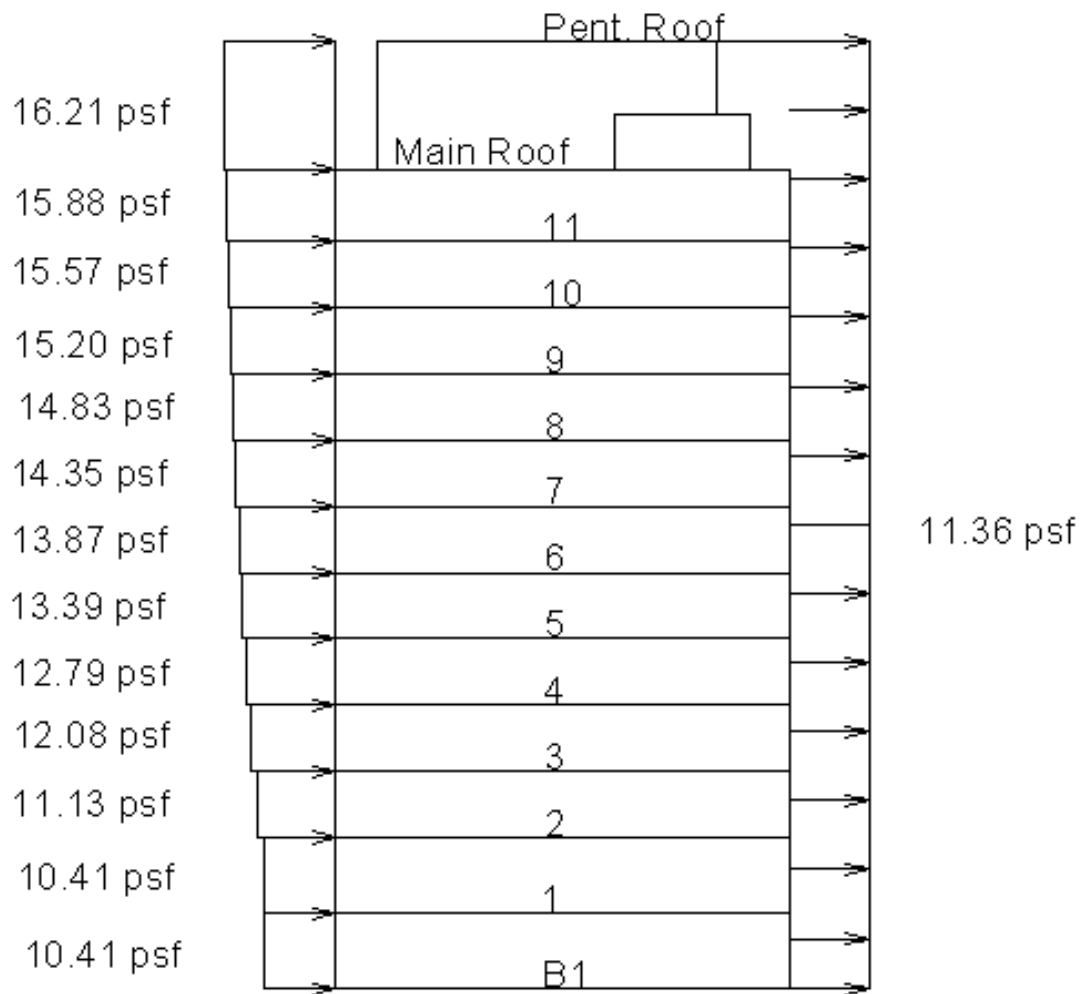


Figure 2-1: Design Wind Pressures in the N-S Direction

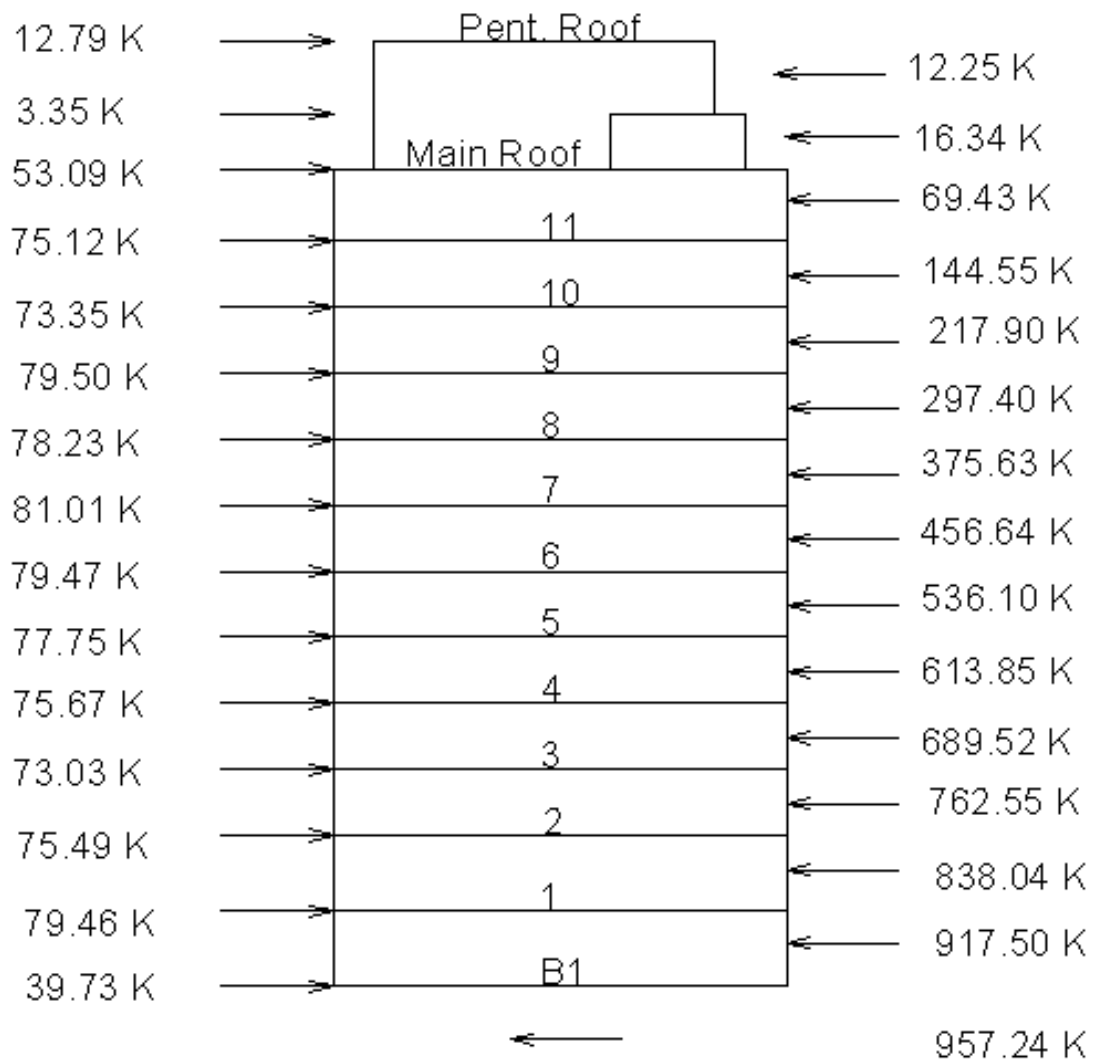


Figure 2-2: Shear Story Forces in the N-S Direction

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 forces. When comparing the base shear from seismic forces to the base shear from wind forces the seismic base shear was larger and therefore controlled in the design of the structure.

Table 3-2: 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	136.33	69000476.21	0.0064	12.31	-	1678.80
Mech. Roof	135.67	126.33	11769657.81	0.0011	2.1005	-	265.35
Main Roof	2123	117.83	1035151238.30	0.0956	184.7389	14.41	21767.78
11	2791.08	107.25	1397021879.69	0.1290	249.3203	199.15	26739.60
10	2917.01	97.67	1286315510.66	0.1188	229.5630	448.47	22421.42
9	3747.92	88.08	1645034166.21	0.1519	293.5820	678.04	25858.70
8	3772.08	78.50	1371899889.79	0.1267	244.8369	971.62	19219.70
7	4049.75	68.92	1242915821.62	0.1148	221.8177	1216.46	15287.67
6	4055.06	59.33	969892557.87	0.0895	173.0924	1438.27	10269.57
5	4055.06	49.75	722769665.97	0.0667	128.9895	1611.37	6417.23
4	4055.06	40.17	505675047.82	0.0467	90.2456	1740.36	3625.17
3	4019.16	30.58	315926849.69	0.0292	56.3821	1830.60	1724.16
2	4216.18	21.00	182692699.20	0.0169	32.6044	1886.98	684.69
1	5201.69	10.00	75155584.87	0.0069	13.4127	1919.59	134.13
			$\Sigma w h_i^k =$ 10831221046	Base Shear = 1933		Overturning Moment = 156093.97	

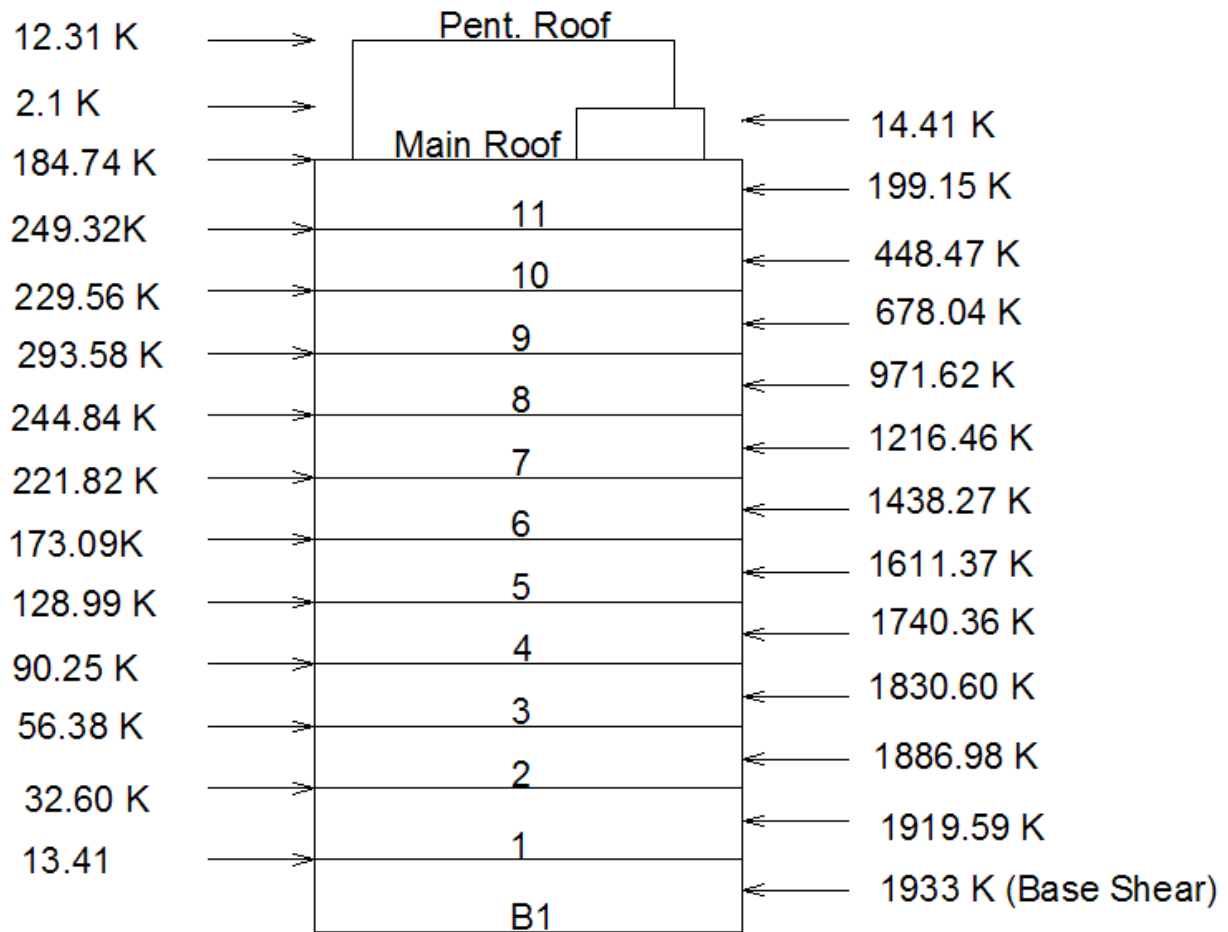
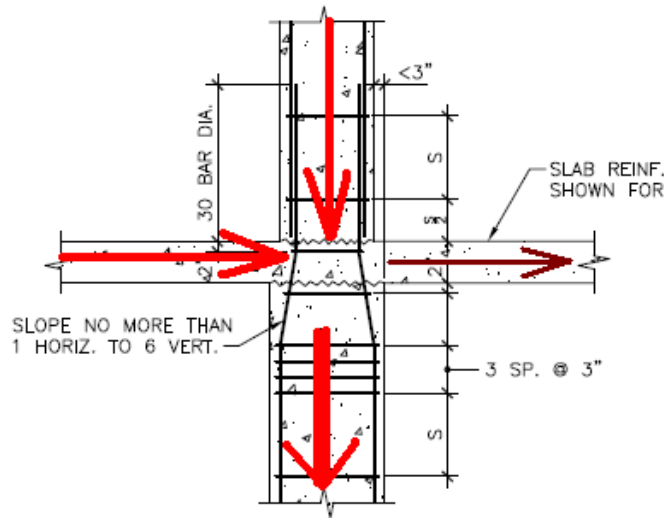


Figure 3-1: Story Shear Forces

Lateral Analysis

In order to carry out this lateral analysis the building's main lateral load and gravity load resisting elements were modeled using ETABS. From this model and the subsequent analysis the building's relative story drifts, direct shear values, torsional shear values and moments were obtained.

The main lateral load resisting system of the building is the aforementioned concrete moment frame consisting of a variety column sizes. The most typical columns found throughout the building are 16"x32", 12"x36", and 12"x48". The moment frame transfers the lateral loads on the building to the foundations as demonstrated in the following figure.



TYPICAL DETAIL OF COLUMN FRAMED AT FLOOR

Figure 4-1 : Load Path

The thin red arrows represent the transferred forces from the slab and higher floors. The thick red arrow represents the cumulative force consisting of the forces from the higher floors and a fraction of the forces from the slab. The thin dark red arrow represents the reduced force in the slab.

Overturning Moment

The critical over turning moment is a result of lateral forces on the face of a building in the direction of the buildings least depth. For a typical building this depth would be the shortest dimension. For Falls Church Tower this depth, which acts as the resisting moment arm, is not clearly defined due to the irregular shape of the building. Because of this the building's center of mass was used as the resisting moment arm.

The critical moment is determined by multiplying the story forces from seismic and wind loading by their respective heights above ground. This is then compared with the resisting moment which is determined by multiplying the building's total weight by the moment arm, which in this case is the center of gravity. If the resisting moment exceeds the overturning moment then the building is stable.

The overturning moments for Falls Church Tower can be found in Appendix E along with the calculations. The results of the calculations show that the forces produces by seismic control but are still well below the resisting moment.

Direct Shear

Direct shear is the result of lateral forces acting on the face or vertical elements of a building and being distributed to the lateral force resisting system. Direct shear is typically calculated by multiplying the story force by the relative stiffness of a lateral force resisting element.

Due to the complexity of the column layout the direct shear forces from wind and seismic loading were determined using an ETABS analysis. The results from the analysis showed that the wind and seismic loads produced different reactions in different columns. The columns that resisted the majority of the shear forces are outlined in red in the figures below.

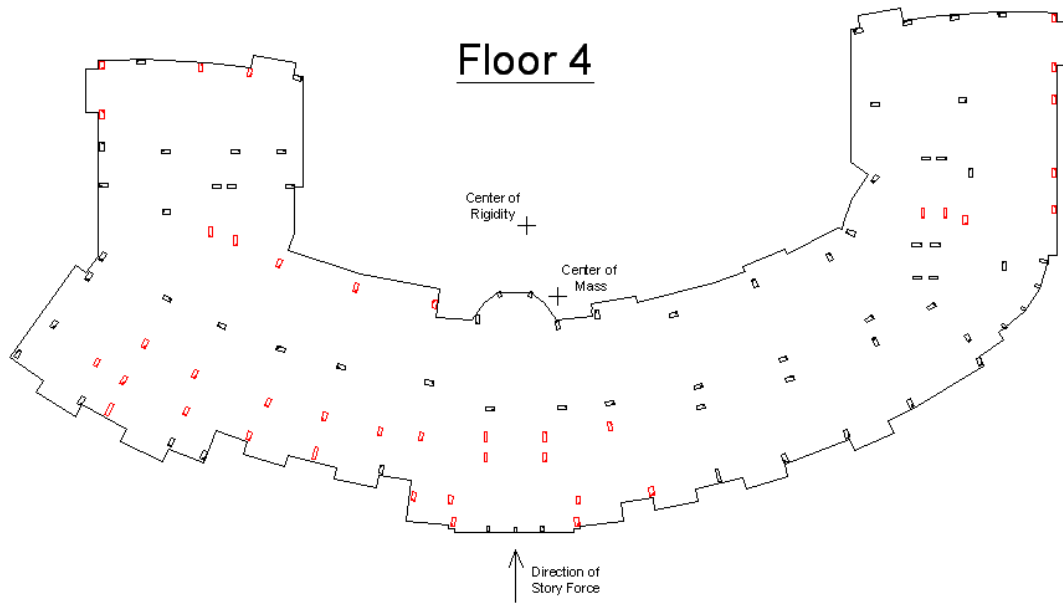


Figure 5-1 : Columns Resisting Wind Loads

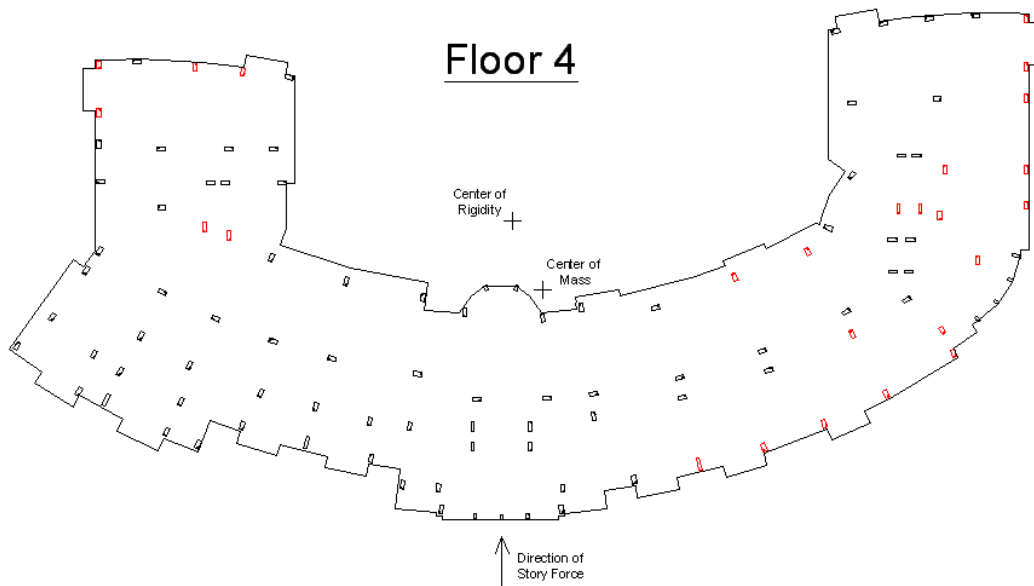


Figure 5-2 : Columns Resisting Seismic Loads

Torsion

Torsional shear is a product of lateral forces and where they are applied with respect to the center of mass and the center of rigidity of a floor or even an entire building. The centers of mass and rigidity within a structure are determined using similar methods. To find the center of mass the weight of every mass element on a floor is multiplied by the element's distance to a point of origin in both the x and y directions. These values are then summed and divided by the total mass of the floor. This method is expressed in the following equations.

$$X = \frac{\sum m_i x_i}{\sum m} , Y = \frac{\sum m_i y_i}{\sum m}$$

To find the center of rigidity you must first find the relative stiffness of all the structural elements on a floor. This is can be done in several ways but the method used for this report was to applied a known load to a fixed member and determine its total deflection using ETABS. This deflection is then divided by the applied load to find the deflection per unit load (stiffness). This is done for all the structural elements on a floor. The relative stiffness is then determined for each element by dividing that element's stiffness by the total stiffness of the floor as expressed by the following equation.

$$\text{Relative Stiffness} = \frac{R}{\sum R}$$

In the same way center of mass is determined the relative stiffness of all the elements are multiplied by their respective distance to a point of origin and then summed to produce both the x and y components of the center of rigidity. Due to the irregular shape and complex column layout of Falls Church Tower, the centers of mass and rigidity where determined using ETABS and are shown in Table 4-1.

Table 4-1

Floor	Center of Mass (in)		Center of Rigidity (in)	
	x	y	x	y
Pent. Roof	1933	451	1993	480
Mech. Roof	2445	1123	2134	904
Main Roof	2448	737	2180	897
11	2608	876	2608	876
10	2418	833	2418	833
9	2418	833	2418	833
8	2130	913	2130	913
7	2130	913	2130	913
6	2130	913	2130	913
5	2130	913	2130	913
4	2089	913	1970	1181
3	2087	911	2087	911
2	2121	918	2121	918
Average	2237	865	2188	883

The torsional shear of a building is determined by the following equation

$$T = \frac{V \cdot e \cdot d_i \cdot R_i}{J}$$

V = Story Shear

e = distance from center of mass to center of rigidity

d_i = distance from structural element to center of rigidity

R_i = relative stiffness of the element

J = $\sum(R_i d_i^2)$ = torsional moment of inertia

The torsional shear for Falls Church was determined using ETABS for the same reasons as the centers of mass and rigidity. Figure 6-1 shows the fourth floor elements that provided the greatest resistance against torsional shear for both wind and seismic loading.

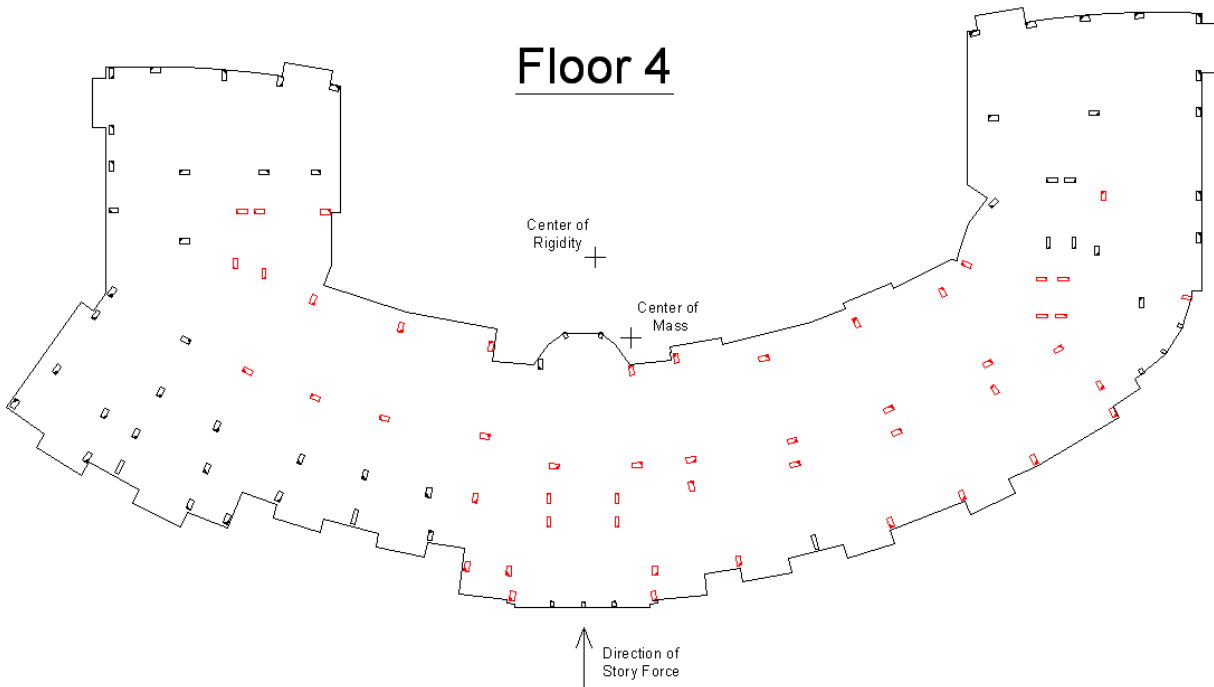


Figure 6-1 : Columns Resisting torsional Shear

Lateral Movement

Drift is a serviceability requirement that is always taken into account during building design. The degree to which engineers focus on drift depends various factors such as building location, height and shape. Drift is particularly critical for buildings in locations that warrant high seismic and wind modification factors as well as buildings that are very tall or very wide.

The lateral drift analysis for Falls Church used a displacement limit of $1/400^{\text{th}}$ of the building height for story drift due to wind, $1/600^{\text{th}}$ for total building drift due to wind, and $1/50^{\text{th}}$ for story drift due to seismic activity. The story drifts were determined using ETABS with the results showing the maximum total drift due to wind being 1.338" which is only 33% of the maximum allowable drift of 4.08". The maximum total drift due to seismic activity was 0.205" which is far below the maximum allowable drift of 30.96".

The results prove show that the building is well within the established drift limits. There is, however, one concern that has to do with the mechanical roof. The results show that the mechanical roof experienced a total story drift of 0.196" which exceeds the maximum allowable story drift of 0.17". The results of the drift analysis can be viewed in Table ?.

Table 5-1 : Story Drift Values

Floor	Seismic Drift(in)			Wind Drift(in)		
	x	y	Allowable	x	y	Allowable
Pent. Roof	0.004	0.024	4.440	0.013	0.215	0.370
Mech Roof	0.004	0.024	2.040	0.012	0.196	0.170
Main Roof	0.004	0.023	2.400	0.011	0.175	0.200
11	0.004	0.023	2.160	0.009	0.153	0.180
10	0.004	0.022	2.160	0.008	0.133	0.180
9	0.004	0.020	2.160	0.007	0.114	0.180
8	0.003	0.018	2.160	0.006	0.096	0.180
7	0.003	0.016	2.160	0.005	0.080	0.180
6	0.002	0.013	2.160	0.004	0.064	0.180
5	0.002	0.010	2.160	0.003	0.049	0.180
4	0.001	0.007	2.160	0.002	0.035	0.180
3	0.001	0.004	2.160	0.001	0.021	0.180
2	0.000	0.001	2.640	0.000	0.007	0.220
1	0.000	0.000	0.000	0.000	0.000	0.000
Total	0.036	0.205	30.960	0.081	1.338	4.08

Strength Check

Strength checks were carried out for controlling shear and simultaneous bending and axial loading. The shear check was performed for column 143 which is highlighted below in Figure 7-1. For this check the torsional shear force due to wind was the greatest and was therefore used in the check. The bending and axial check was performed for column 180 which is highlighted below in Figure 7-2. The moments produced by wind were the greatest and were therefore used in the check. The strength check calculations can be found in Appendix ?



Figure 7-1 : Column 143 (highlighted)

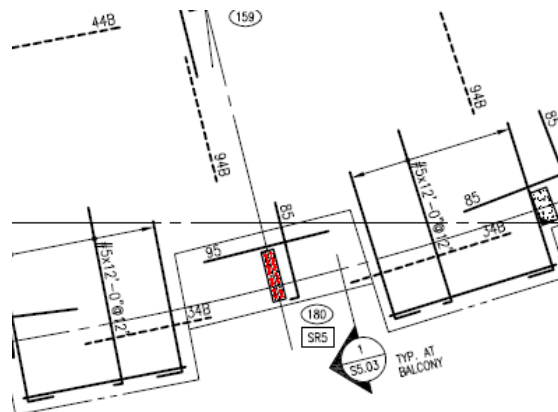


Figure 7-2 : Column 180 (highlighted)

Conclusion

Falls Church Tower is a fairly complex building with its curved facade and irregular column layout. In order to better understand how the building works as a whole when acted upon by lateral loads, it was modeled using ETABS. After running an analysis of the building it was found that wind was the controlling lateral force in lateral displacement, torsion and strength checks. Seismic proved to be the controlling factor for overturning.

For overturning the seismic loads produced a critical overturning moment of 1873116 in-K in the North-South direction. However, this value was dwarfed by the resisting moment of 61244959 in-K produced by the building's self weight.

Direct shear load analysis showed which columns took most of the wind and seismic loads. As shown in Figure ? And Figure ? these columns differ with each load case. Winds loads are resisted by columns in the southwest section of the building where as columns in the eastern section resist seismic loads.

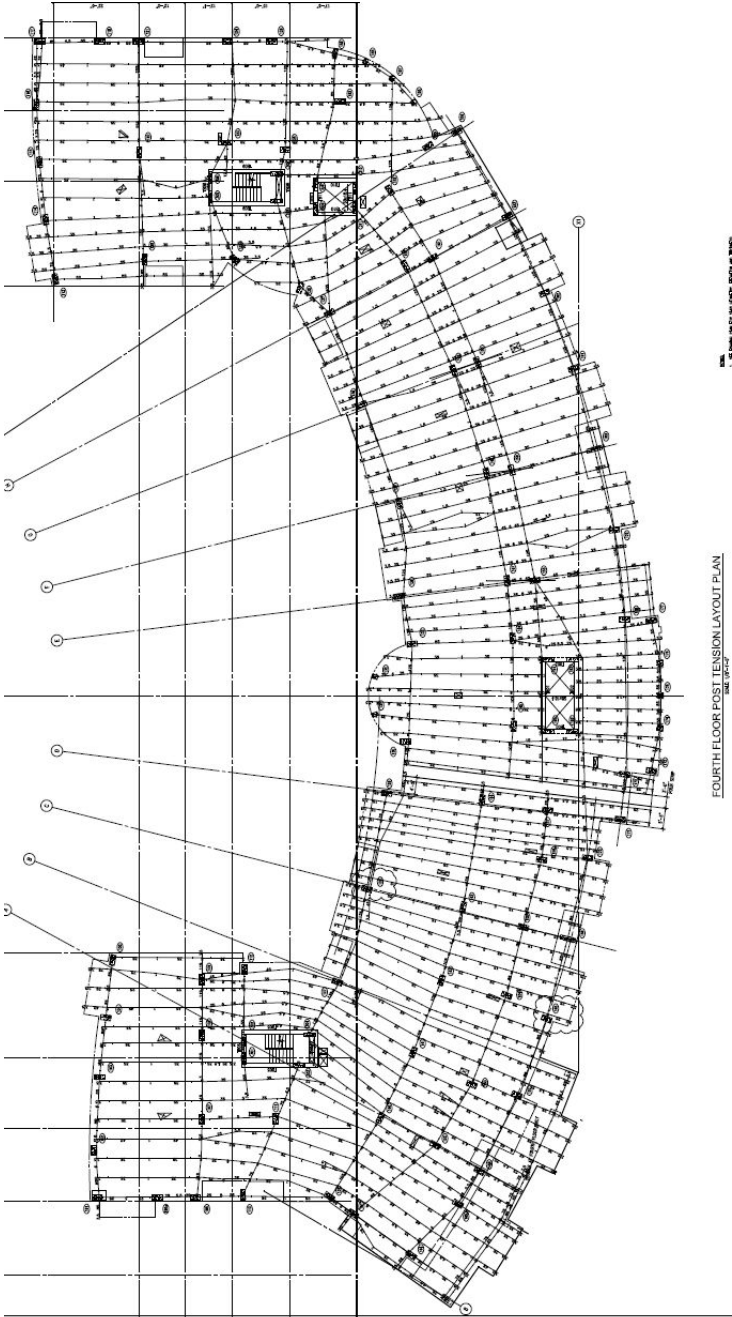
The centers of mass and rigidity had a significant difference between their locations. This caused torsional shear stresses to control in the shear strength check. The results from the check showed that the 16"x32" column was more than enough to handle the torsional shear load. Another strength test was performed on a 12"x48" column. Based on the results of the strength check the column was not able to stand up to lateral and axial loads.

The total drifts of the building came in under the allowable limits for both seismic and shear. The only concern with respect to story drift arose when the mechanical roof's story drift exceeded the allowable value for wind loading.

Overall the building performed amicably under the specified loads. Future studies will re-evaluate the concerns mentioned in this report such as the excessive drift of the mechanical roof and the strength of the 12"x48" column.

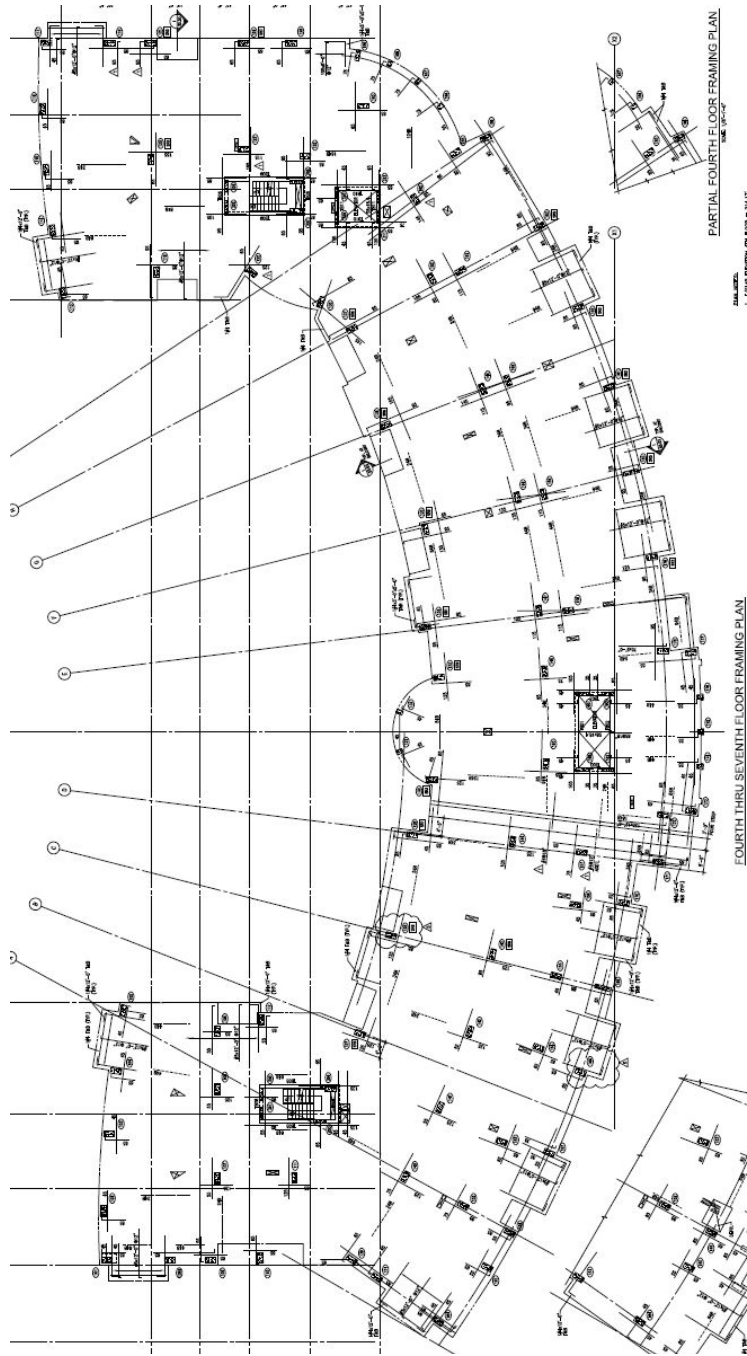
Appendix

Appendix A – Figures



Typical Post Tension Layout

Appendix A – Figures



Typical Column Layout

Appendix B – Building Weight Tables

Floor	Height(ft)	Column Size and Number												Total Volume (cf)	Total Weight (K)	
		16x32 (3,536 sf)	12x32 (2,672 sf)	24x32 (5,336 sf)	12x12 (1sf)	12x18 (1,536 sf)	12x36 (3sf)	16x24 (2,672 sf)	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			6	16				3			566.46	5098.14	764.72
B3	9	96	11			3	16				3			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	284.51
10	9.583	20,238.000	935.500	0.583	11,805.50	8,964.90	1,770.83	288.95
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 – Wind Loads

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

Location: Arlington, VA
 Residential Building:
 Topography: Homogeneous

Basic Wind Speed:	Exposure	Building Classification
Arlington, VA = 90 mph	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
 $V = 90$ 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 C – Wind Loads

2

Gust Effect Factor

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

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

$$G_e = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_v^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z} \right) \quad \begin{matrix} g_v = 3.4 \\ g_R = 3.4 \end{matrix}$$

$I_z = c \left(\frac{z}{z_{max}} \right)^{1/6}$; $\bar{z} = \begin{cases} 0.6h \\ \text{max } z_{min} \end{cases}$; $z_{min} = 30 \text{ ft}$; $h = 117.83 \text{ ft}$

$\bar{z} = \begin{cases} 70.7 \\ \text{max } 30 \end{cases} \Rightarrow \bar{z} = 70.7$

$c = 0.30$

$I_z = 0.3 \left(\frac{33}{70.7} \right)^{1/6} = 0.264$

$R = \sqrt{\frac{1}{\beta} R_u R_n R_E (0.53 + 0.47 R_L)}$; $R_n = \frac{747 N_1}{(1 + 10.3 N_1)^{5/3}}$; $N_1 = \frac{n_1 L_z}{\sqrt{z}}$; $n_1 = 0.734$; $L_z = l \left(\frac{z}{33} \right)^{1/5}$

$\beta = \text{assumed } 1.59 \text{ or } 0.015$

$R_n = \frac{747(4.21)}{(1 + 10.3(4.21))^{5/3}} = 0.057$

$N_1 = \frac{(0.734)(412.53)}{71.86} = 4.21$

$N_1 = 4.21$

$\sqrt{z} = l \left(\frac{z}{33} \right)^{1/5} \sqrt{\frac{58}{60}}$

$= 0.45 \left(\frac{70.7}{33} \right)^{1/5} \sqrt{90 \left(\frac{58}{60} \right)}$

$L_z = 71.86$

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

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

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

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

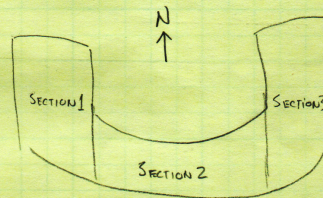
$= 0.164$

Appendix C – Wind Loads

3

Note: In order to obtain values for B and L in the calculation of R_B and R_e 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 since 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

$$R_B = R_n \text{ for } \eta = 4.6 \eta_1 B / \sqrt{L}$$

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

$$= 3.051$$

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

$$= 0.274$$

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 C – Wind Loads

4

R_L Values

Section 1

N-S

$$R_L = R_h \text{ for } \eta = 15.4 \eta_1 \frac{L}{\sqrt{Z}}$$

$$= 15.4(0.734) \left(\frac{118}{71.86} \right)$$

$$= 18.56$$

$$R_L = \boxed{0.052}$$

E-W

$$\eta = 15.4(0.734) \left(\frac{65}{71.86} \right)$$

$$= 10.22$$

$$R_L = \boxed{0.093}$$

Section 2

N-S

$$\eta = 15.4(0.734) \left(\frac{65}{71.86} \right)$$

$$= 10.38$$

$$R_L = \boxed{0.092}$$

E-W

$$\eta = 15.4(0.734) \left(\frac{174.18}{71.86} \right)$$

$$= 27.4$$

$$R_L = \boxed{0.036}$$

Section 3

N-S

$$\eta = 15.4(0.734) \left(\frac{104}{71.86} \right)$$

$$= 16.36$$

$$R_L = \boxed{0.059}$$

E-W

$$\eta = 15.4(0.734) \left(\frac{65}{71.86} \right)$$

$$= 10.22$$

$$R_L = \boxed{0.093}$$

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

Background Response

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

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

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

$$Q = \boxed{0.852}$$

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 C- Wind Loads

5

Gust Effect Factor

$$G_f = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_p^2 Q^2 + g_r^2 R^2}}{1 + 1.7 g_v I_z} \right)$$

* Largest Gust Factors controls

Section 1

$$N-S: G_f = 0.925 \left(\frac{1 + 1.7(0.264) \sqrt{(3.4)^2(0.952)^2 + (4.12)^2(0.308)^2}}{1 + 1.7(3.4)(0.264)} \right) = \boxed{0.996} *$$

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

Section 2

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

$$E-W: G_f = 0.925 \left(\frac{1 + 1.7(0.264) \sqrt{(3.4)^2(0.952)^2 + (4.12)^2(0.304)^2}}{1 + 1.7(3.4)(0.264)} \right) = \boxed{0.985} *$$

Section 3

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

$$E-W: G_f = 0.925 \left(\frac{1 + 1.7(0.264) \sqrt{(3.4)^2(0.837)^2 + (4.12)^2(0.298)^2}}{1 + 1.7(3.4)(0.264)} \right) = \boxed{0.865}$$

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)

$$G_{ci} = (\pm 0.18) =$$

Appendix C – Wind Loads

6

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

$p = (18.24)(0.886)(-0.15) + 18.24(-0.18) = -11.36 \text{ psf}$

C	Cp E-W
0.8	0.55
0.886	0.886
1.6	0.63

values for these directions

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	R_n	R_h	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

Floor	Height Above Ground (ft)	K_z	q_z (psf)	q_s (psf)	Windward (psf)	Leeward (psf)	Total Pressure (psf)
B1	0.00	0.570	10.05	18.24	10.41	-11.36	21.77
1	10.00	0.570	10.05	18.24	10.41	-11.36	21.77
2	21.00	0.628	11.07	18.24	11.13	-11.36	22.49
3	30.58	0.704	12.41	18.24	12.08	-11.36	23.44
4	40.17	0.761	13.41	18.24	12.79	-11.36	24.15
5	49.75	0.809	14.26	18.24	13.39	-11.36	24.75
6	59.33	0.847	14.93	18.24	13.87	-11.36	25.23
7	68.92	0.886	15.62	18.24	14.35	-11.36	25.72
8	78.50	0.924	16.29	18.24	14.83	-11.36	26.19
9	88.08	0.954	16.81	18.24	15.20	-11.36	26.56
10	97.67	0.983	17.33	18.24	15.57	-11.36	26.93
11	107.25	1.008	17.77	18.24	15.88	-11.36	27.24
MainRoof	117.83	1.035	18.24	18.24	16.21	-11.36	27.58
Mech. Roof	126.33	1.056	18.61	18.24	-	-11.36	27.83
Pent. Roof	136.33	1.081	19.50	18.24	-	-11.36	28.46

Design Wind Pressure

Appendix D – Seismic

Seismic Load

Spectral Response Acceleration Parameters

Soil Site Class: C

$$S_s = 0.16 \text{ (ASCE-7 Figure 22-1)}$$

$$S_1 = 0.051 \text{ (ASCE-7 Figure 22-2)}$$

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

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

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

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

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

$$T = C_t h_n^x ; C_t = 0.016$$

$$x = 0.9$$

$$h_n = 137.33 \text{ ft}$$

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

$$T_L = 8 \text{ sec (Figure 22-15)}$$

$$T_L > T$$

$$C_s = \frac{S_{DS}}{R/I} ; R = 3$$

$$I = 1$$

$$C_s = \frac{0.128}{(3/1)} = 0.0427$$

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

← B1-B3 Ex. (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 D – 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	136.33	69000476.21	0.0064	12.31	-	1678.80	
Mech. Roof	135.67	126.33	11769657.81	0.0011	2.1005	-	265.35	
Main Roof	2123	117.83	1035151238.30	0.0956	184.7389	14.41	21767.78	
11	2791.08	107.25	1397021879.69	0.1290	249.3203	199.15	26739.60	
10	2917.01	97.67	1286315510.66	0.1188	229.5630	448.47	22421.42	
9	3747.92	88.08	1645034166.21	0.1519	293.5820	678.04	25858.70	
8	3772.08	78.50	1371899889.79	0.1267	244.8369	971.62	19219.70	
7	4049.75	68.92	1242915821.62	0.1148	221.8177	1216.46	15287.67	
6	4055.06	59.33	969892557.87	0.0895	173.0924	1438.27	10269.57	
5	4055.06	49.75	722769665.97	0.0667	128.9895	1611.37	6417.23	
4	4055.06	40.17	505675047.82	0.0467	90.2456	1740.36	3625.17	
3	4019.16	30.58	315926849.69	0.0292	56.3821	1830.60	1724.16	
2	4216.18	21.00	182692699.20	0.0169	32.6044	1886.98	684.69	
1	5201.69	10.00	75155584.87	0.0069	13.4127	1919.59	134.13	
		$\Sigma w_i h_i^k =$	10831221046	Base Shear =		1933	Overtuning Moment = 156093.97	

Design Seismic Loads

Appendix E – Overturning

Nathan Eck

12-16-10

Overturning Moment (Wind)

Resisting Moment (M_r) = Build Weight \times Moment Arm; The moment arm will be taken as the N-S component of the average center of mass due to the building's irregular shape.

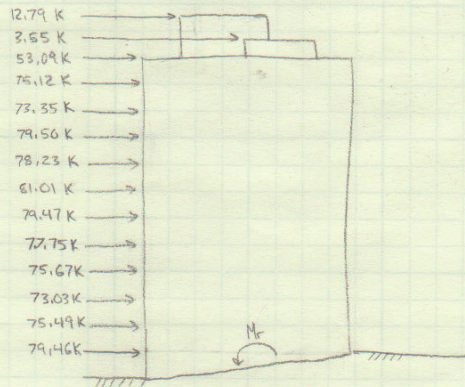
Average Center of Mass (Etabs) = 865 in

$$M_r = (70803K) \times (865 \text{ in}) = 61244595 \text{ in-K}$$

$$M_o = \sum (\text{Story Forces} \times \text{Story Height})$$

$$\begin{aligned} &= (79.46 \times 10) + (75.49 \times 21) + (73.03 \times 30.58) \\ &+ (75.67 \times 40.17) + (77.75 \times 49.75) \\ &+ (79.47 \times 59.33) + (81.01 \times 69.92) \\ &+ (78.23 \times 78.50) + (79.50 \times 88.08) \\ &+ (73.35 \times 97.67) + (75.12 \times 107.25) \\ &+ (53.09 \times 117.83) + (3.55 \times 126.33) \\ &+ (12.79 \times 136.33) \end{aligned}$$

$$M_o = 58630.39 \text{ ft-K} (12 \text{ in/ft}) = 703565 \text{ in-K} \ll 61244595 \text{ in-K} \therefore \text{okay}$$



Appendix E – Overturning

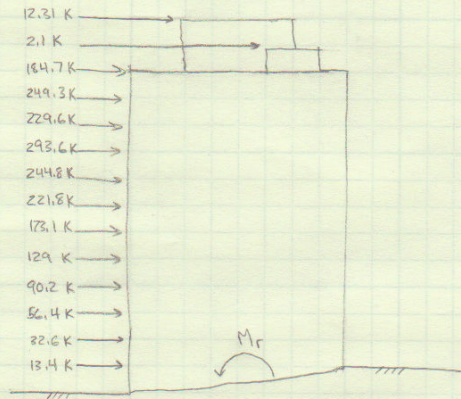
Over Turning Moment (Seismic)

$$M_r = 61244595 \text{ in-K}$$

$$M_o = \Sigma(\text{Story Forces} \times \text{Height})$$

$$= 156094 \text{ ft K} = 1873128 \text{ in K} < 61244595 \text{ in-K} \therefore \text{okay}$$

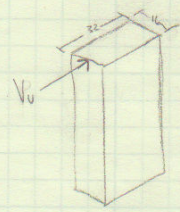
From Seismic Forces Table



Appendix F – Strength Checks

Spot Checks

Direct Shear



Column 143 - Floor 4

$\phi V_n > V_u$; $V_u = 22.77 \text{ K}$ (Max Shear from torsion)

$\phi V_n = \phi (V_c + V_s)$; $V_c = 0.17 \left(1 + \frac{N_u}{14A_g}\right) 2\sqrt{f'_c} b_w d$ (11.2.12)

$f'_c = 5000 \text{ psi}$
 $b_w = 16 \text{ in}$
 $d = 29.5 \text{ in}$

$N_u = 1156.28 \text{ K}$ (from tech 1)
 $= 5.143 \text{ MN}$
 $A_g = 0.3303 \text{ m}^2$

$V_c = 0.17 \left(1 + \frac{5813}{14(0.3303)}\right) \sqrt{5} (16)(29.5)$
 $= 378.9 \text{ K}$
 $\phi V_c = 0.75(378.9 \text{ K}) = 284.2 \text{ K} > 22.77 \text{ K} \therefore \text{ok}$

Bending and Compression

Column 180 - Floor 1

Given: Column Length = 11 ft = 132 in

$r = 0.3h = 14.4$

Column Above $\Rightarrow I_c/l_c = \frac{110592 \text{ in}^4}{108 \text{ in}} (0.7) = 716.8 \text{ in}^3$

Column Below $\Rightarrow I_c/l_c = \frac{110592 \text{ in}^4}{132 \text{ in}} (0.7) = 587.8 \text{ in}^3$

Interior Slab $\Rightarrow I_s/l_s = \frac{15734 \text{ in}^4}{236 \text{ in}} (0.25) = 16.67 \text{ in}^3$

$\Psi_A = \frac{\sum I_c/l_c}{I_s/l_s} = \frac{716.8 + 587.8}{16.67} = 99.25$

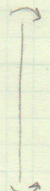
Column Above $\Rightarrow I_c/l_c = 837.8 \text{ in}^3$

Column Below $\Rightarrow I_c/l_c = \frac{110592 \text{ in}^4}{120 \text{ in}} (0.7) = 645.1 \text{ in}^3$

Interior Slab $\Rightarrow I_s/l_s = 16.67$

$\Psi_B = \frac{\sum I_c/l_c}{I_s/l_s} = \frac{837.8 + 645.1}{16.67} = 88.95$

k (from Fig. R10.10.1.1) = 1.0



Appendix F – Strength Checks

Slenderness Effects

$$\frac{kL_u}{r} = \frac{1.0(132\text{ in})}{14.4\text{ in}} = 9.17 < 34 - 12\left(\frac{3661.8}{5174.5}\right) = 25 \therefore \text{slenderness does not need to be considered}$$

$$M_c = \delta_{ns} M_2$$

$$\delta_{ns} = \frac{C_m}{(1 - P_u / 0.75 P_c)} \geq 1.0$$

$$C_m = 0.6 + 0.4(0.708) = 0.88 \Rightarrow 1.0$$

$$P_u = \text{Roof Load} + 10(\text{Floor Load}) + \Sigma \text{Column Weight}$$

Roof Load

$$10'' \text{ Slab} = 121 \text{ psf}$$

$$\text{Water Proofing} = 5.5 \text{ psf}$$

$$\text{Misc.} = 10 \text{ psf}$$

$$\text{Total DL} = 136.5 \text{ psf}$$

$$\text{Snow} = 19.25 \text{ psf}$$

$$\text{Roof} = 30 \text{ psf}$$

$$\text{Total LL} = 49.25 \text{ psf}$$

$$W_u = 1.2(136.5) + 1.6(49.25) = 242 \text{ psf}$$

$$P_u = W_u A_T = (242 \text{ psf})(334 \text{ sf}) = 80828 \text{ lbs} = 80.83 \text{ K}$$

Floor Load

$$7'' \text{ Slab} = 85 \text{ psf}$$

$$\text{Finish} = 16 \text{ psf}$$

$$\text{Misc.} = 10 \text{ psf}$$

$$\text{Total DL} = 111 \text{ psf}$$

$$\text{Total LL} = 60 \text{ psf}$$

$$W_u = 1.2(111) + 1.6(60) = 229 \text{ psf}$$

$$P_u = (229 \text{ psf})(334) = 76486 \text{ lb} = 76.49 \text{ K}$$

Column Weight

$$12 \times 22': (1)(1.83)(10)(150 \text{ pcf}) = 2745 \text{ lbs} = 2.75 \text{ K}$$

$$12 \times 48': (1)(4)(9)(150 \text{ pcf}) = 5400 \text{ lbs} = 5.4 \text{ K} (9 \text{ Floors}) = 48.6 \text{ K}$$

$$P_u = 80.83 + 10(76.49) + 2.75 + 48.6 = 897 \text{ K}$$

$$P_c = \frac{\pi^2(EI)_{\text{eff}}}{(kL_u)^2}; \text{ assume } \beta_d = 0.6 \Rightarrow (EI)_{\text{eff}} = 0.25(3000)(110592) = 99532800 \text{ k-in}^2$$

$$P_c = \frac{\pi^2(99532800)}{(132\text{ in})^2} = 56379 \text{ K}$$

Appendix F – Strength Checks

$$\delta_{ns} = \frac{C_m}{\left(1 - \frac{P_u}{0.75 P_c}\right)} = \frac{1.0}{\left(1 - \frac{897}{0.75(56379)}\right)} = \frac{1.0}{0.9787} = 1.022$$

$$M_c = \delta_{ns} M_z = 1.022(5174 \text{ k-ft}) = 5277.48$$

$$\left. \begin{array}{l} P_u = 897 \\ M_u = 5277.48 \end{array} \right\} \text{no good}$$

Given that applied axial force and moment exist so far outside the interaction curve, it is prudent that the model be re-evaluated.

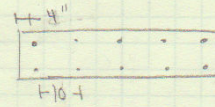
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Appendix F – Strength Checks

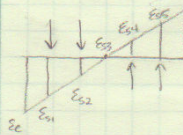
Column 180 Interaction Curve

$$\begin{aligned}
 P_o | P_o &= 0.85 f'_c [b_w h - A_s] + A_s f_y \\
 &= 0.85(6) [(12)(48) - 8(1)] + 8(1)(60) \\
 &= 3050 \text{ K}
 \end{aligned}$$

$$\begin{aligned}
 f'_c &= 6 \text{ ksi} \\
 f_y &= 60 \text{ ksi} \\
 8 \# 9 \text{ Bars} & \quad A = 1.0 \text{ in}^2
 \end{aligned}$$



Balanced Strain



$$\epsilon_y = 60 / 29000 = 0.00207$$

$$c = \frac{0.003}{0.003 + 0.00207} (48 - 4) = 26.04$$

$$\epsilon_{s1} = \frac{0.003}{26.04} (26.04 - 4) = 0.002539$$

$$f_{s1} = 60 \text{ ksi}$$

$$\epsilon_{s2} = \frac{0.003}{26.04} (26.04 + 14) = 0.001387$$

$$f_{s2} = 0.001387 (29000) = 40.23 \text{ ksi}$$

$$\epsilon_{s3} = \frac{0.003}{26.04} (26.04 - 24) = -0.000235$$

$$f_{s3} = 0.000235 (29000) = 6.82 \text{ ksi}$$

$$\epsilon_{s4} = \frac{0.003 (26.04 - 34)}{26.04} = -0.000917$$

$$f_{s4} = -0.000917 (29000) = -26.59 \text{ ksi}$$

$$f_{s5} = -60 \text{ ksi}$$

$$\begin{aligned}
 P_b &= 0.85(6)(0.85)(26.04)(12) + 2(60 + 40.23 + 6.82 - 26.59 - 60) \\
 &= 1375 \text{ K}
 \end{aligned}$$

$$\begin{aligned}
 M_b &= 1355 \left(24 - \frac{0.85(26.04)}{2} \right) + 2(60(24-4) + 40.23(24-14) + 6.82(24-24) \\
 &\quad - 26.59(24-34) - 60(24-44)) \\
 &= 17524 + 2(1200 + 402.3 + 265.9 + 1200) \\
 &= 20592 \text{ Kin} = 1716 \text{ K-ft}
 \end{aligned}$$

Appendix F – Strength Checks

Pure Bending

Assume ϵ_{s1} and ϵ_{s2} do not yield
 Assume $\epsilon_{s3}, \epsilon_{s4}, \epsilon_{s5}$ do yield

$$f_{s1} = \frac{0.003}{c} (c-4)(29000)$$

$$f_{s2} = \frac{0.003}{c} (c-14)(29000)$$

$$f_{s3} = -60$$

$$f_{s4} = -60$$

$$f_{s5} = -60$$

$$\Sigma F = 0 = 0.85(6)(12)(0.85)c + 2(1)f_{s1} + 2(1)f_{s2} + 2(1)f_{s3} + 2(1)f_{s4} + 2(1)f_{s5}$$

$$0 = 52.02c + 2\left(87 - \frac{348}{c}\right) + 2\left(87 - \frac{1218}{c}\right) - 120(3)$$

$$= 52.02c + 174 - \frac{696}{c} + 174 - \frac{2436}{c} - 360$$

$$= 52.02c + \frac{3132}{c} - 12$$

$$= 52.02c^2 - 12c - 3132$$

$$c = 7.88$$

$$f_{s1} = 42.84 \text{ ksi}$$

$$f_{s2} = -67.57$$

$$M_o = 0.85(6)(12)(0.85)(7.88)\left(24 - \frac{0.85(7.88)}{2}\right) + 2\left[42.84(24-4) - 67.57(24-14) - 60(24-24) - 60(24-24) - 60(24-44)\right]$$

$$= 8465 + 2(1981) = 12427 \text{ k-in} = 1035 \text{ k-ft}$$

c = h

$$c = 48$$

$$\epsilon_{s1} = \frac{0.003(48-4)}{48} = 0.00275$$

$$f_{s1} = 60 \text{ ksi}$$

$$\epsilon_{s2} = \frac{0.003(48-14)}{48} = 0.002125$$

$$f_{s2} = 60 \text{ ksi}$$

$$\epsilon_{s3} = \frac{0.003(48-24)}{48} = 0.0015$$

$$f_{s3} = 0.0015(29000) = 43.5 \text{ ksi}$$

$$\epsilon_{s4} = \frac{0.003(48-34)}{48} = 0.000875$$

$$f_{s4} = 0.000875(29000) = 25.38 \text{ ksi}$$

$$\epsilon_{s5} = \frac{0.003(48-44)}{48} = 0.00025$$

$$f_{s5} = 0.00025(29000) = 7.25 \text{ ksi}$$

$$P = 0.85(6)(12)(0.85)(48) + 2(60) + 2(60) + 2(43.5) + 2(25.38) + 2(7.25) = 2889 \text{ K}$$

$$M = 0.85(6)(12)(0.85)(48)\left(24 - \frac{0.85(48)}{2}\right) + 2\left[60(24-4) + 60(24-14) + 43.5(24-24) + 25.38(24-34) + 7.25(24-44)\right]$$

$$= 8989 + 2(1401) = 11791 \text{ k-in} = 982.6 \text{ k-ft}$$

Appendix F – Strength Checks

$E_f = E_{gs} = 0.005$
 $c = \frac{0.003}{0.003 + 0.005} (44) = 16.5$
 $E_{s1} = \frac{0.003(16.5 - 4)}{16.5} = 0.0023$
 $f_{s1} = 60 \text{ ksi}$
 $E_{s2} = \frac{0.003(16.5 - 14)}{16.5} = 0.000455$
 $f_{s2} = 0.000455(29000) = 13.18 \text{ ksi}$
 $E_{s3} = \frac{0.003(16.5 - 24)}{16.5} = -0.001636$
 $f_{s3} = -0.001636(29000) = -4745 \text{ ksi}$
 $E_{s4} = \frac{0.003(16.5 - 34)}{16.5} = -0.003162$
 $f_{s4} = -60 \text{ ksi}$
 $f_{s5} = -60 \text{ ksi}$

$P = 0.85(6)(12)(0.85)(16.5) + 2[60 + 13.18 - 4745 - 60 - 60]$
 $= 669.79 \text{ K}$
 $M = 0.85(6)(12)(0.85)(16.5)(24 - \frac{0.85(16.5)}{2})$
 $+ 2[60(20) + 13.18(10) - 60(-10) - 60(-20)]$
 $= 14580 - 468 = 14111 \text{ K-in} = 1175 \text{ K-ft}$

Appendix F – Strength Checks

