Hilton Hotel at BWI Airport

Linthicum Heights, MD

Technical Report 3: Lateral System Analysis and Confirmation Design

Thomas Sabol

Structural

Advisor: Dr. Ali M. Memari

The Pennsylvania State University

November 21, 2006

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EXECUTIVE SUMMARY

The purpose of this technical report is to analyze the lateral resistance system of the Hilton Hotel at BWI Airport under seismic and wind loads.

Building Description

The Hilton Hotel at BWI Airport, located in Linthicum Heights, MD, is a 280 guestroom, 203,300 s.f. hotel that elevates from the ground 11 stories plus a mechanical penthouse topping off at 131'-0" from grade. BWI Hilton Hotel's typical structural floor system is a 7-1/2" thick flat plate post-tensioned concrete system transferring load to rectangular reinforced concrete columns. Columns transfer gravity load to reinforced concrete spread footings.

Lateral System Description

BWI Hilton Hotel's lateral resistance system comprises of 12 reinforced concrete shear walls, 11 of which span the height of the building. Shear walls are located around two stair wells and central elevator shafts. All walls are 1'-0" thick specified with a 28-day f'c = 4000 psi, but vary in length from 10'-0" to 29'-2-1/2". Shear walls transfer load to reinforced concrete mat foundations specified with an f'c = 3000 psi.

Conclusions

Lateral resisting shear walls were analyzed under seismic and wind loads determined using ASCE7-05. Loads were distributed using simplified hand calculations performed using excel spreadsheets. Shear walls were more than sufficient in strength for shear and overturning moment. Deflections of walls were well within the industry standard of *H/400*. Design of walls may be governed by Architectural constraints and constructability rather than by strength.

1.1 INTRODUCTION

Hilton Hotel at BWI Airport, located in Linthicum Heights, MD, elevates from the ground 11 stories plus a mechanical penthouse topping off at 131'-0" from ground level. The 203,300 s.f. of hotel comprises of 280 guestrooms, a ballroom/ assembly room, pool with an indoor/ outdoor sundeck, restaurant, and an 80-car parking garage below grade.

The structural system of the 'BWI Hilton' varies throughout the building. Cast-in-place reinforced concrete is the primary structural entity. Typical guest room floors (floors 4-11) resist gravity loads with a two-way post tensioned flat plate system. Gravity loads are resisted on floors ground through 3 by two-way mild reinforced concrete slab with drop panels. The penthouse roof system is a two-way post-tension concrete slab with drop beams. Reinforced concrete columns varying in size carry gravity loads to spread footing foundations. The double-heighted ballroom, adjacent assembly room, and main entrance spaces are all enclosed by a structural steel system. Corrugated metal roof decks transfer gravity loads to steel members, which in turn transfer load to W-shape steel columns that frame into concrete piers.

ASCE7-05 was used to compute the wind and seismic loading on the 'BWI Hilton', even though the building was designed prior to this code utilizing ASCE7-02. ASCE7-05 was used to determine loads because of its relevance to new design, which will be implemented next semester.

This report consists of a lateral resistance system description, load cases used in determination of the design of lateral resistance system, seismic and wind loads acting on the building, a distribution of these loads on the lateral system, analysis of system, and member checks.

2.1 LATERAL LOAD RESISTANCE SYSTEM

Twelve reinforced concrete shear walls comprise of the lateral load resistance system. Eleven of which span the building height and are located in three locations: 3 walls around two stairwells located near either edge of the north and south sides, and 5 walls are located around an elevator core in the center of the building. The twelfth shear wall is located on the North side of the building and only spans vertically from foundation to the second floor. Shear walls are 1'-0" thick and are specified to reach a 28-day f'c = 4000 psi. Shear walls transfer load to reinforced concrete mat foundations which 36" thick under elevator shear walls, and 32" thick under stairwell shear walls. Concrete is specified to reach a 28-day f'c = 3000 psi. Figure 1 shows the 11 shear wall locations on a typical floor plan.

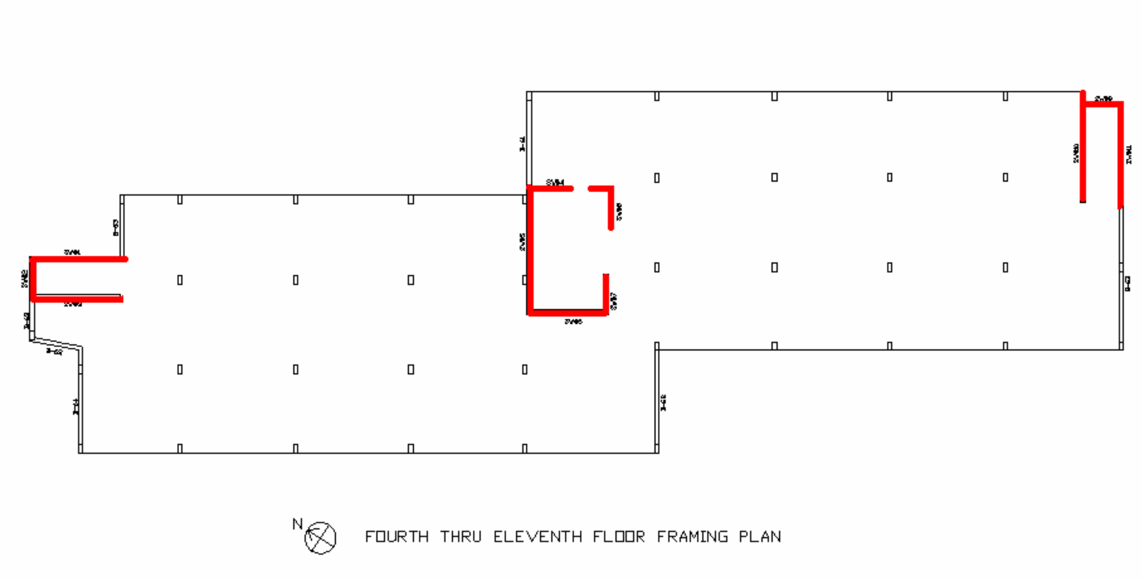


Figure 2.1: Shear walls on a typical floor plan.

3.1 LATERAL LOADS

Lateral Loads were computed using excel spread sheets. Electronic copies of the spread sheets can be obtained upon request. Procedures and equations for wind and seismic loads are referenced to ASCE7-05 Chapters 6, 11, 12 and 19.

3.2 LOAD CASES

The following load combinations should considered when combining factored loads using strength design. The most common governing combination for gravity load design would be case 2 which would simplify to 1.2D + 1.6L. When a member carries both lateral and gravity loads case 4 or 5 would usually govern depending on whether wind or seismic is the controlling lateral load.

2.3.2 Basic Combinations ASCE7-05

1) 1.4(D + F)

2) 1.2(D + F + T) +1.6(L + H) + 0.5(L_r OR S OR R)

3) 1.2D + 1.6(L_r or S or R) + (L or 0.8W)

4) 1.2D + 1.6W + L + 0.5(L_r or S or R)

5) 1.2D + 1.0E + L + 0.2S

6) 0.9D + 1.6W + 1.6H

7) 0.9D + 1.0E + 1.6H

3.3 SEISMIC LOADS

Assumptions:

The seismic Site Classification was taken directly from the Geotechnical Report. Self-weight was calculated by performing quantity takeoffs of the structure, façade, and roofing. Structure weight quantities were calculated by the square footage of the concrete slabs, multiplying by the thickness and then multiplying by the weight of concrete per cubic foot (150 lb/ft^3). A superimposed dead load of 10 psf was applied to floor systems to account for partitions and MEP equipment. Columns were also counted and quantified as well as beams and drop panels to obtain an accurate weight. The weight of the façade was taken as the weight of the concrete panels over the square feet of the elevations. A glass to concrete panel ratio was taken and then multiplied to the area for an approximation of concrete panels per elevation. Roof areas were calculated and then a weight per square foot was used to calculate the entire roof weight. The adjacent steel structure weight, for simplification, was assumed to be 10 psf and then multiplied by the area to obtain the weight. The factors used in the seismic calculations are broken down and compared to the Engineer of Record's in Table 3.1.

Conditions:

Factors	Engineer of Record ASCE7-02		Experimental Data ASCE7-05	
Seismic Use Group	I		II	
Importance Factor	1.0		1.0	
Seismic Design Cat.	B		B	
Mapped Spectral Response Accel.	S_S	S_1	S_S	S_1
	0.187	0.063	0.15	0.053
Design Spectral Response Factors	S_{DS}	S_{D1}	S_{DS}	S_{D1}
	0.20	0.10	0.16	0.085
Site Classification	D		D	
Seismic Response Coefficient (C_s)	0.017		0.022	
Response Modification Factor (R)	5		5	
Design Base Shear (V)	695^K		693^K	

Table 3.1: Seismic Factors

Results:

Seismic loads calculations yielded a base shear of 693$^{K.}$ Compared to the Engineer of Record's calculated base shear of 695^K, this is accurate. In the first Technical report a base shear of 779^K was determined. Discrepancy of base shears between Technical report 1 and Technical report 3 can be contributed to the self weight calculation counting the weight of the ground floor in Technical report 1. After discussion with faculty, it was learned that the ground floor (on grade) weight does not factor into the calculations due to base shear acting at lateral resistance members just above the grade. Portions of the structure located from ground to foundations move with the movement of the earth.

Table 3.2 breaks down the pressures, shears, and overturning moment at each level.

LEVEL	W_x (K)	H_x (ft)	$W_xH_x^{1.135}$	C_{VX}	F_X (k)	V_X (k)	Overturning Moment (ft*K)
Penthouse	2551.04	131	2551.04	0.17	114.74	114.74	15030.74
11	2983.38	114	2983.38	0.17	114.60	229.34	13064.40
10	2029.76	103	2029.76	0.10	69.49	298.83	7157.18
9	2029.76	94	2029.76	0.09	62.64	361.46	5887.93
8	2029.76	85	2029.76	0.08	55.88	417.34	4749.46
7	2029.76	76	2029.76	0.07	49.21	466.55	3740.00
6	2029.76	67	2029.76	0.06	42.65	509.20	2857.62
5	2029.76	58	2029.76	0.05	36.21	545.41	2100.16
4	2029.76	49	2029.76	0.04	29.90	575.31	1465.22
3	4604.86	40	4604.86	0.08	53.88	629.19	2155.28
2	7305.66	31	7305.66	0.09	64.01	693.20	1984.29
Base	0.00	18	0.00	0.00	0.00	693.20	0.00

Table 3.2: Seismic Calculations

3.4 WIND LOADS

Assumptions:

Exposure B Case 2 values were used for finding the K_h values from Table 6-3 in Chapter 6 of ASCE7-05. K_h values were conservatively used for simplification of wind loads at varying floor heights, e.g. the floor to floor height of the 1st floor is 18 ft. A K_h value of 0.62 (K_h value at 20 ft) was used in the computation instead of breaking the loading up into two K_h values per floor (0-15 ft and 20 ft loading). The width and lengths used in the calculations were taken from the base length and width of the building.

Conditions:

Factors	Engineer of Record ASCE7-02		Experimental Data ASCE7-05	
Basic Wind Speed	90 mph		90 mph	
Building Category	II		II	
Site Exposure	B		B	
Importance Factor	1.0		1.0	
External Pressure Coefficient GC_P	Windward	Leeward	Windward	Leeward
	+ 0.68	- 0.43	+ 0.73	- 0.64
Internal Pressure Coefficient GC_{Pi}	+/- 0.18		+/- 0.18	

Table 3.3: Wind Factors

Results:

The wind load calculations yielded a pressure of 19.22 psf in the N-S direction and a pressure 22.69 psf in the E-W direction at the top of the building. A wind loading diagram of the building can be found in the E-W direction in Appendix A. The windward GC_P of +0.73 was calculated compared to the original design value of +0.64. This value is within 10% of the original design. Discrepancies may be found in the calculation gust factor calculations where certain values might have been assumed differently by either party. Table 3.4 breaks down the pressures, shears, and overturning moment at each level.

Level	hx	Pressures				Shears (k)		Overturning Moment	
		NS windward	NS leeward	EW windward	EW leeward	N/S	E/W	N/S	E/W
Penthouse	129.67	13.98	-5.24	13.96	-8.73	73.38	103.85	9515.57	13466.57
11	114	13.34	-5.24	13.32	-8.73	123.19	174.71	5677.81	8077.977
10	103	13.34	-5.24	13.32	-8.73	163.94	232.69	4197.23	5971.519
9	94	12.70	-5.24	12.68	-8.73	203.28	288.98	3698.28	5291.428
8	85	12.31	-5.24	12.30	-8.73	241.78	344.26	3272.47	4698.912
7	76	11.93	-5.24	11.91	-8.73	279.43	398.53	2861.84	4124.584
6	67	11.42	-5.24	11.40	-8.73	315.96	451.45	2447.56	3545.878
5	58	10.90	-5.24	10.89	-8.73	351.37	503.03	2053.52	2991.422
4	49	10.39	-5.24	10.38	-8.73	385.65	553.26	1679.74	2461.219
3	40	9.75	-5.24	9.74	-8.73	418.52	601.80	1314.96	1941.793
2	31	9.75	-5.24	9.74	-8.73	466.01	671.92	1472.03	2173.729
1	18	7.95	-5.24	7.94	-8.73	523.88	759.58	1041.70	1577.854

Table 3.4: Wind Calculations

4.1 SIMPLIFICATION OF ANALYSIS

Calculations to determine the center of rigidity and the center of mass were performed using excel spread sheets. Several simplifications of the building were assumed to perform a lateral system analysis with out a computer generated building model. Since 11 of the building's shear walls are located in the area of the building that extrudes vertically 131 ft, the typical floor plan of levels 4-11 were used. For this reason while determining the center of rigidity and center of mass the typical floor plan of these levels were used and considered to be the same from ground up to top of the building.

The building was simplified to as if there were no adjacent structure on the lower levels. The figure to the right tries to illustrate this.

Figure 4.1: Hotel without adjacent structure

For simplification of calculations the 12th shear wall was not considered in the lateral system investigations due to its lack of location on the typical floor plan, and only extending vertically 2 floors. Shear walls were treated as acting as deep rectangular beams in each direction. By this I mean the shear walls did not act as together as C-shapes with adjacent perpendicular walls acting as flanges. The adjacent steel structure that encompasses the ballroom area, assembly and main entrance areas was considered negligible for the lateral load calculations. This steel system most likely utilizes the main structure as an abutment to resist lateral loads in the East-West direction.

5.1 SHEAR WALL RIGIDITY

Shear wall rigidities were calculated using the relative rigidities of each shear wall. Relative rigidities were found using $1/\Delta$. A point load of 100^K at the top of the shears walls was the assumed case loading used. The height to length ratios for each shear wall were well over 3, therefore deflections due to shear were ignored and deflections due to flexure were used. Equation 5.1 below was used to calculate the deflections of each wall.

Equation 5.1 $$\Delta = \frac{Ph^3}{3EI}$$

Where *P* is the point load, *h* is the height, *E* is the modulus of elasticity, and *I* is the moment of inertia. The shear wall relative rigidities can be seen in Table 5.1.

Shear Wall	Relative Rigidity	
	Rx	Ry
SW 1	0.490	-
SW 2	-	0.048
SW 3	0.536	-
SW 4	0.385	-
SW 5	-	1.199
SW 6	0.330	-
SW 7	-	0.045
SW 8	-	0.048
SW 9	0.063	-
SW 10	-	1.283
SW 11	-	0.791

Table 5.1: Relative Rigidities

Center of rigidity (COR) location was found using the relative rigidities of each wall and using a zero reference point near the North-West corner of the building. The 0,0 coordinate is circled in red. The COR calculations yielded a COR location of 45'-3" in the E-W (x) direction, and 196'-5" in the N-S (y) direction, which is located by a blue cross on the drawing.

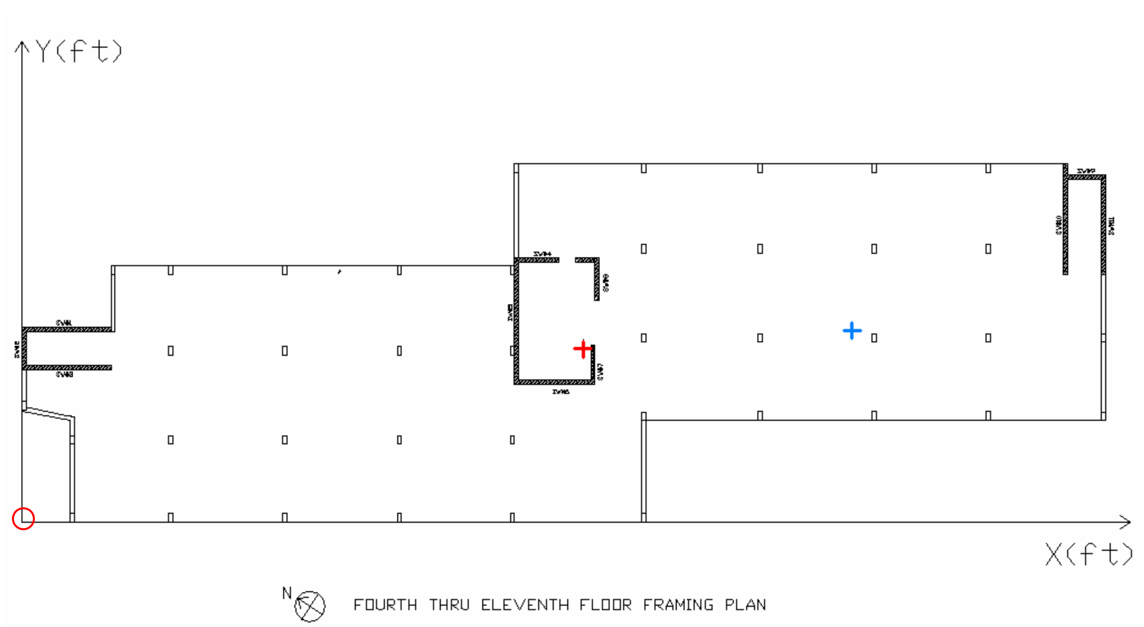


Figure 5.1: Locations of COM (red cross) and COR (blue cross)

Seismic resultant force acts at the center of mass (COM) of the building. Calculations to find the COM were conducted in a similar matter as the calculations to find the COR by using the same 0,0 coordinate shown on the drawing. Masses considered in the calculation were the floor system, shear walls, and façade of the typical floor plan shown above. Excel spread sheet calculations can be found in Appendix B. The red cross in Figure 5.1 shows the location of the COM.

6.1 DISTRIBUTION OF SHEAR FORCES

6.1.2 Direct Shear

Distributions of direct shear forces were computed by taking the ratio of the relative rigidity of the individual walls to the total relative rigidities of the walls that direction. Equation 6.1 shows the direct shear force distributed to a wall.

Equation 6.1 $$F_i = V\frac{R_i}{\sum Ri}$$

Where V is the base shear on the wall and R_i is the relative rigidity of the wall.

6.1.3 Shear Due to Torsion

Eccentricities of the resultant shear forces, wind and seismic, result in torsion acting on the building. Torsion from seismic forces is caused by the eccentricity of the COM and the COR. Torsion from wind forces is caused by the eccentricity of the COR and the geometric center of the building. These torsional moments can be resolved into shear forces acting on the shear walls. Equation 6.2 was used in determining the resultant shear forces due to torsion in each wall in each direction.

Equation 6.2 $$F_i = \frac{VeR_iC_{sw}}{\sum RC_{sw}^2}$$

Where V is the base shear acting on the building in that direction, R_i is the relative rigidity of the wall; C_{sw} is the perpendicular distance from the shear wall to the center of rigidity. Torsion resulting from wind and seismic was calculated. Tables 6.1 and 6.2 show the results of direct shear and shear due to torsion on each wall at the base of the building.

Shear Due to Seismic Forces				
	E-W		N-S	
Shear Wall	Direct Shear (k)	Torsion Shear (k)	Direct Shear (k)	Torsion Shear (k)
SW #1	0.00	0.01	188.26	0.34
SW #2	9.77	0.89	0.00	26.12
SW #3	0.00	0.45	205.99	13.02
SW #4	0.00	0.61	148.00	17.91
SW #5	243.49	9.00	0.00	263.09
SW #6	0.00	0.38	126.89	11.23
SW #7	9.06	0.26	0.00	7.55
SW #8	9.77	0.27	0.00	8.02
SW #9	0.00	0.22	24.07	6.30
SW #10	260.56	6.16	0.00	179.96
SW #11	160.57	4.48	0.00	130.89

Table 6.1: Base shears due to seismic forces

Shear Due to Wind Forces				
	E-W		N-S	
Shear Wall	Direct Shear (k)	Torsion Shear (k)	Direct Shear (k)	Torsion Shear (k)
SW #1	0.00	0.02	142.27	0.27
SW #2	10.71	1.30	0.00	21.10
SW #3	0.00	0.65	155.67	10.52
SW #4	0.00	0.89	111.85	14.46
SW #5	266.80	13.07	0.00	212.51
SW #6	0.00	0.56	95.90	9.07
SW #7	9.92	0.38	0.00	6.10
SW #8	10.71	0.40	0.00	6.48
SW #9	0.00	0.31	18.19	5.09
SW #10	285.50	8.94	0.00	145.37
SW #11	175.94	6.50	0.00	105.73

Table 6.2: Base shears due to wind forces

Larger shears due torsion in the N-S direction can be accounted for by the eccentricity of the COM to the COR of 64.2 ft. for seismic, and an eccentricity of 68.6 ft. between the COM to the geometric center of the building for wind.

7.1 BUILDING DRIFT

Building drift was taken as the deflection of the shear walls at the top of the walls in each direction. These deflections were then compared to an industry standard drift limitation of *H/400*. Even though previously mentioned flexure controlled deflection, a combination of shear and flexure were used in calculating the total deflection. The resulting combination derived yields Equation 7.1 shown below.

Equation 7.1

$$\Delta_{Top} = \frac{1.5V}{bE}\left[\left(\frac{H}{d}\right) + \left(\frac{H}{d}\right)^3\right]$$

Where V is the base shear, b is the thickness of the shear wall, H is the height of the shear wall, E is the modulus of elasticity of the concrete, and d is the depth of the wall. Table 7.1 summarizes the deflections in each direction from wind and seismic shear forces compared to *H/400*. Deflections are well below the drift standard.

H/400 (in)	Δtop E-W (in)		Δtop N-S (in)	
	Seismic	Wind	Seismic	Wind
3.93	0.781	0.856	1.47	1.11

Table 7.1: Summary of deflections

8.1 SPOT CHECKS

Spots checks for shear capacity of shear walls as well as compressive strength of the walls were calculated for overturning moment. Calculations were performed referencing the *Seismic Design Handbook, Naeim 2001.*

A nominal shear strength capacity was calculated to determine adequacy against base shear forces. Shear wall #2 which spans East-West, and shear wall #6 which spans North-South were checked. Applicable load combinations were applied to shear forces. A boundary element calculation was computed to determine adequacy of shear wall #2's boundary elements due to overturning moment and axial load. While checking the tension side of the boundary element, the concrete was neglected and steel reinforcement was only considered. See calculations in the Appendix C "Shear Wall Capacity".

Overturning moment of the shear walls was not considered at the foundation due to large mat foundations. Steel in tension was capable of handling overturning moment at ground level.

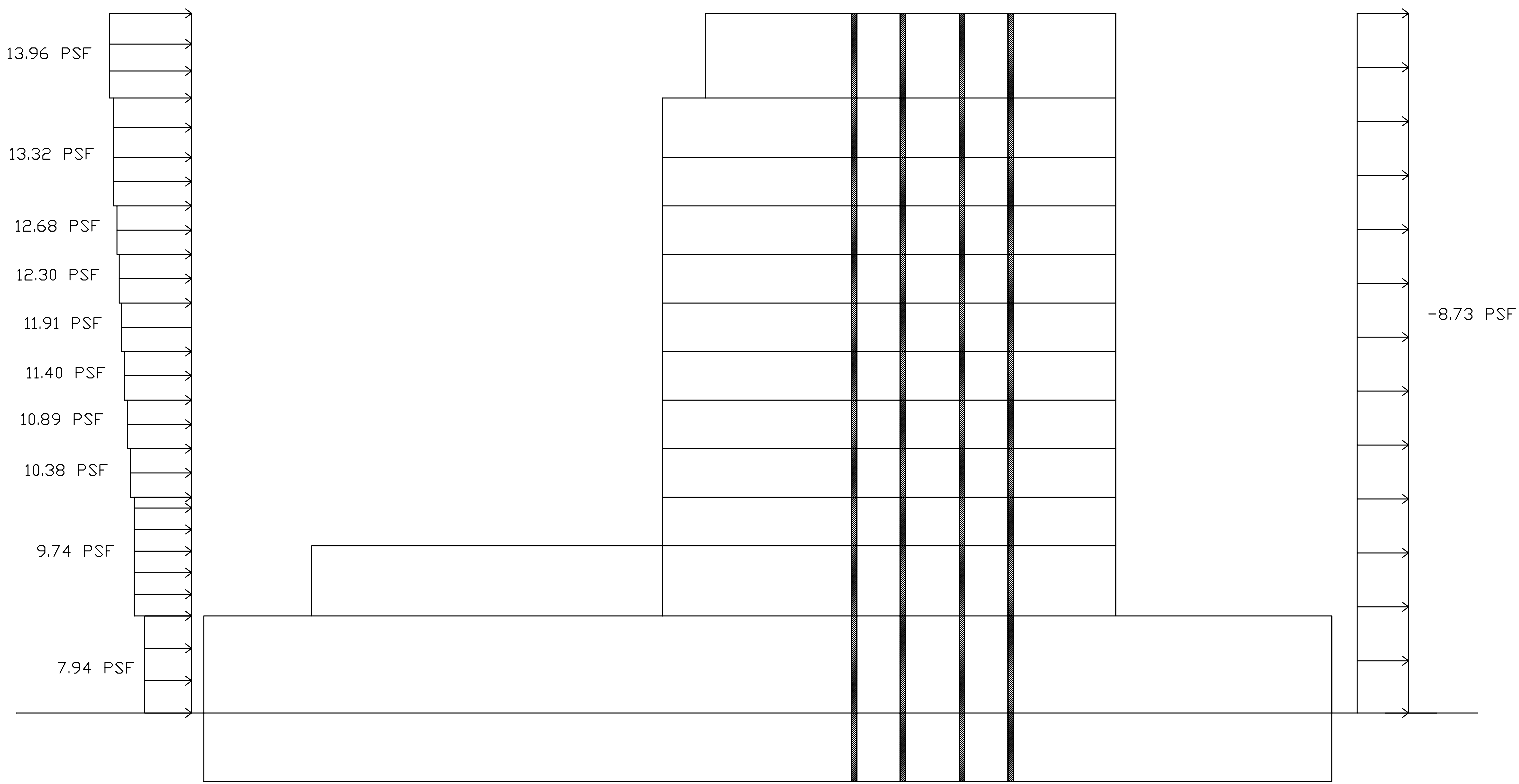
9.1 CONCLUSIONS

Lateral loads acting on the building were controlled partially by both seismic and wind. Seismic forces governed in the North-South direction with a base shear of 693^{K}, while wind loading governed in the East-West direction with a base shear of 760^{K}. Lateral forces were distributed to shear walls by relative rigidities of each wall. Loads causing deflections in both directions were well within the drift limitation of *H/400*.

Spot check calculations yielded shear strength capacity in excess of needed strength. Design of shear walls may not have been governed by strength but by several reasons. Architectural constraints placed on the structural engineer may have controlled the strength design, such as location and size of the stairs and elevator. Constructability of the walls may have played a hand in the design. Width of walls may have been increased to 12" for ease rebar placement, as well as maintaining a consistent width of entire wall for ease and cost of forms and labor.

APPENDIX A

E-W WIND PRESSURES ON BUILDING



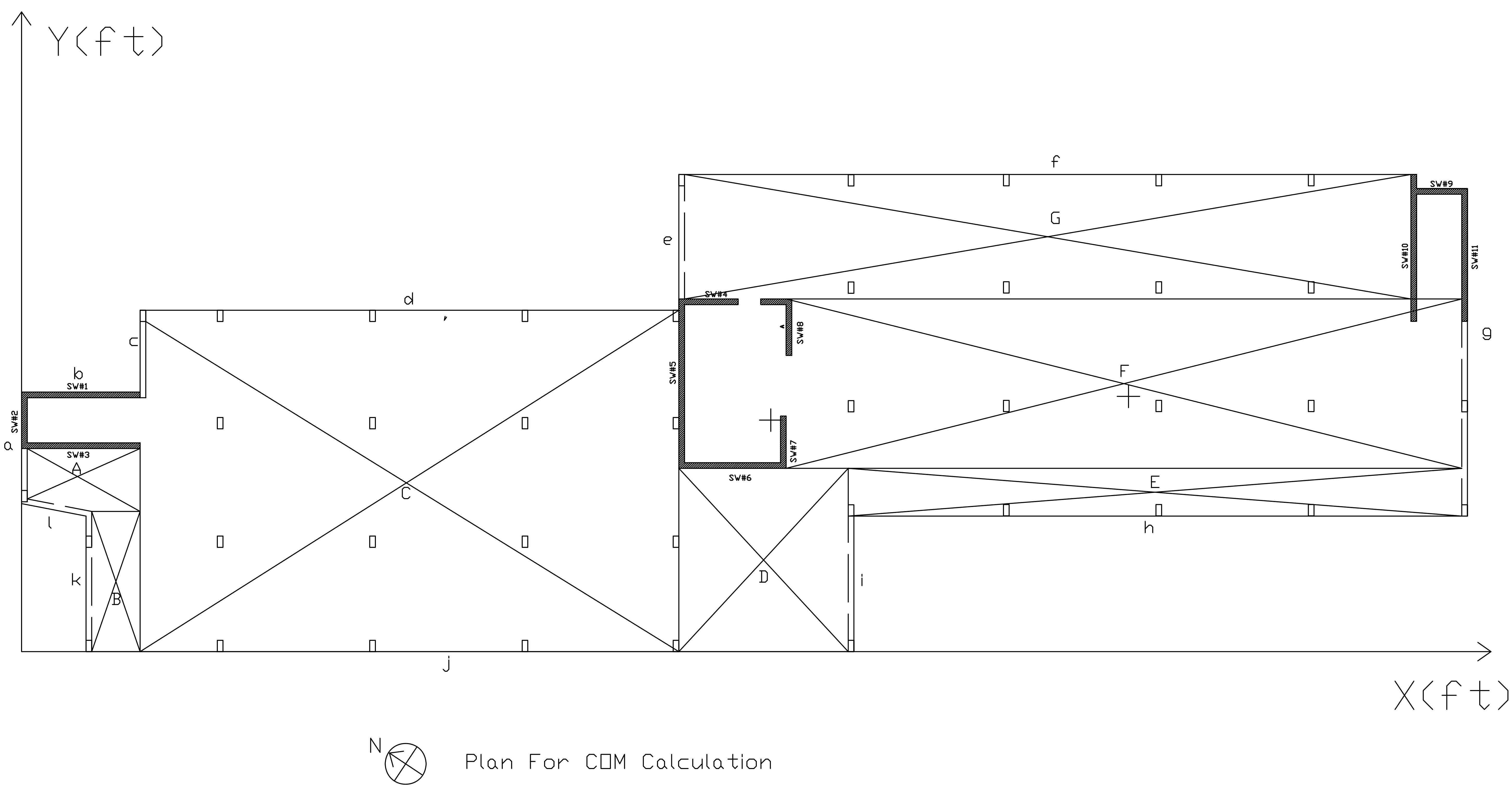
APPENDIX B

EXCEL SPREAD SHEET CALCULATIONS

Center of Mass

Element	Wall Length (ft)	Wall Height per floor (ft)	Unit Weight (k/sf)	W weight (kips)	Distance from zero reference x (ft)	y (ft)	Wx (ft-kips)	Wy (ft-kips)
Façade a	17.75	9	0.005	0.799	0	36	0.00	28.76
" b	21	9	0.005	0.945	10.5	46	9.92	43.47
" c	14.5	9	0.005	0.653	21	53.25	13.70	34.75
" d	95.5	9	0.005	4.298	68.75	60.5	295.45	260.00
" e	24	9	0.005	1.080	116.5	75.5	125.82	81.54
" f	139.75	9	0.005	6.289	186.25	84.5	1171.28	531.40
" g	60.5	9	0.005	2.723	256	54.25	696.96	147.70
" h	108.75	9	0.005	4.894	201.75	24	987.31	117.45
" i	24	9	0.005	1.080	147.5	12	159.30	12.96
" j	136	9	0.005	6.120	79.5	0	486.54	0.00
" k	24	9	0.005	1.080	11.5	12	12.42	12.96
Façade l	11.75	9	0.005	0.529	5.75	25	3.04	13.22
SW #1	21.67	9	0.15	29.255	11.0	45.5	321.80	1331.08
SW #2	10	9	0.15	13.500	0.5	41.0	6.75	553.50
SW #3	22.33	9	0.15	30.146	11.0	36.0	331.60	1085.24
SW #4	20	9	0.15	27.000	126.5	62.0	3415.50	1674.00
SW #5	29.21	9	0.15	39.434	117.0	47.5	4613.72	1873.09
SW #6	19	9	0.15	25.650	126.0	33.0	3231.90	846.45
SW #7	9.75	9	0.15	13.163	135.0	37.0	1776.94	487.01
SW #8	10	9	0.15	13.500	136.0	57.5	1836.00	776.25
SW #9	10	9	0.15	13.500	251.0	81.5	3388.50	1100.25
SW #10	26	9	0.15	35.100	246.5	71.5	8652.15	2509.65
SW #11	23.29	9	0.15	31.442	255.5	70.25	8033.30	2208.77
SW #12	-	-	-	-	-	-	-	-
	Fl Area Length (ft)	Fl Area Width (ft)						
Floor Area (A)	20	10.75	0.094	20.21	9.75	31.00	197.05	626.51
Floor Area (B)	8.5	24.75	0.094	19.78	16.75	12.50	331.24	247.19
Floor Area (C)	95.5	60.5	0.094	543.11	68.00	29.00	36931.38	15750.15
Floor Area (D)	30	35.5	0.094	100.11	131.50	16.00	13164.47	1601.76
Floor Area (E)	108.75	8.5	0.094	86.89	200.75	28.25	17443.42	2454.68
Floor Area (F)	119.75	30	0.094	337.70	195.25	47.50	65934.95	16040.51
Floor Area (G)	128.75	22	0.094	266.26	181.75	73.50	48391.85	19569.74
TOTAL				1676.22			221964.25	72020.02

x_{mass} 132.4 ft
y_{mass} 43.0 ft



Plan For COM Calculation

Assume a 100^K virtual load for Deflection calcs

Relative Rigidities

Shear Wall	E (psi)	I (in^4)	h (in)	d (in)	h / d	A_w (in^2)	I_{EQ} (in^4)	Deflection due to			Rigidity R
								Flexure	Shear	Interaction	
SW 1	3604996.5	17584113.2	1572.0	260.04	6.05	3120.48	17062741.4	2.043	IGNORE	IGNORE	0.490
SW 2	3604996.5	1728000.0	1572.0	120.00	13.10	1440.00	1716828.6	20.787	IGNORE	IGNORE	0.048
SW 3	3604996.5	19240214.4	1572.0	267.96	5.87	3215.52	18635568.6	1.867	IGNORE	IGNORE	0.536
SW 4	3604996.5	13824000.0	1572.0	240.00	6.55	2880.00	13473316.4	2.598	IGNORE	IGNORE	0.385
SW 5	3604996.5	43066384.1	1572.0	350.52	4.48	4206.24	40801133.2	0.834	IGNORE	IGNORE	1.199
SW 6	3604996.5	11852352.0	1572.0	228.00	6.89	2736.00	11580326.9	3.031	IGNORE	IGNORE	0.330
SW 7	3604996.5	1601613.0	1572.0	117.00	13.44	1404.00	1591766.8	22.427	IGNORE	IGNORE	0.045
SW 8	3604996.5	1728000.0	1572.0	120.00	13.10	1440.00	1716828.6	20.787	IGNORE	IGNORE	0.048
SW 9	3604996.5	1728000.0	1440.0	120.00	12.00	1440.00	1714703.1	15.978	IGNORE	IGNORE	0.063
SW 10	3604996.5	30371328.0	1368.0	312.00	4.38	3744.00	28704065.6	0.779	IGNORE	IGNORE	1.283
SW 11	3604996.5	21829923.0	1440.0	279.48	5.15	3353.76	20948755.9	1.265	IGNORE	IGNORE	0.791
SW 12	-	-	-	317.52		3810.24	-	-	-		

Center of Rigidity

Shear Wall	Distance from Zero refernce		Relative Rigidity		$R_x y$	$R_y x$
	N-S (x) (ft)	E-W (y) (ft)	R_x	R_y		
SW 1	11.0	45.5	0.490	-	22.3	-
SW 2	0.5	41.0	-	0.048	-	0.02
SW 3	11.0	36.0	0.536	-	19.3	-
SW 4	126.5	62.0	0.385	-	23.9	-
SW 5	117.0	47.5	-	1.199	-	140.3
SW 6	126.0	33.0	0.330	-	10.9	-
SW 7	135.0	37.0	-	0.045	-	6.0
SW 8	136.0	57.5	-	0.048	-	6.5
SW 9	251.0	81.5	0.063	-	5.1	-
SW 10	246.5	71.5	-	1.283	-	316.3
SW 11	255.5	70.25	-	0.791	-	202.0
SW 12	-	-	-	-	-	-
TOTAL			1.8	3.4	81.4	671.1

Xr 196.6 ft

Yr 45.2 ft

Deflection Due to Seismic							
Shear Wall	V (lb)	E (psi)	H (in)	b (in)	d (in)	Δ@Top E-W	Δ@Top N-S
SW #1	188256.61	3604997	1572	12	260.04		1.4816
SW #2	9769.68	3604997	1572	12	120.00	0.7660	
SW #3	205986.93	3604997	1572	12	267.96		1.4840
SW #4	148000.61	3604997	1572	12	240.00		1.4757
SW #5	243486.63	3604997	1572	12	350.52	0.7994	
SW #6	126892.02	3604997	1572	12	228.00		1.4724
SW #7	9055.12	3604997	1572	12	117.00	0.7658	
SW #8	9769.68	3604997	1572	12	120.00	0.7660	
SW #9	24068.20	3604997	1440	12	120.00		1.4521
SW #10	260555.24	3604997	1368	12	312.00	0.8012	
SW #11	160568.02	3604997	1440	12	279.48	0.7902	
SW #12		3604997		12	317.52	0.0000	

Deflection Due to Wind							
Shear Wall	V (lb)	E (psi)	H (in)	b (in)	d (in)	Δ@Top E-W	Δ@Top N-S
SW #1	142273.00	3604997	1572	12	260.04		1.1197
SW #2	10705.17	3604997	1572	12	120.00	0.8393	
SW #3	155672.51	3604997	1572	12	267.96		1.1215
SW #4	111849.94	3604997	1572	12	240.00		1.1152
SW #5	266801.54	3604997	1572	12	350.52	0.8760	
SW #6	95897.34	3604997	1572	12	228.00		1.1128
SW #7	9922.19	3604997	1572	12	117.00	0.8391	
SW #8	10705.17	3604997	1572	12	120.00	0.8393	
SW #9	18189.30	3604997	1440	12	120.00		1.0974
SW #10	285504.56	3604997	1368	12	312.00	0.8779	
SW #11	175943.12	3604997	1440	12	279.48	0.8659	
SW #12		3604997	-	12	317.52	#VALUE!	

APPENDIX C

SW #2 E-W Direction (Wind Controls)

From Structural Drawings:
$f'_c = 4000psi$
reinforcement: ASTM A615 Grade 60 => $f_y = 60^{ksi}$
#4 @ 12" each way, each face
Wall thickness = 1'-0"
$l = 10'$
$h = 298.67'$

Nominal Shear Capacity

$$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$$

$$\frac{h_w}{l_w} = \frac{298.67}{10} = 29.9 > 2.0 \therefore \alpha_c = 2.0$$

$$A_{cv} = (12)(10' \times 12) = 1440 \, in^2$$

2 curtains of Reinforcement (#4 bars)

$$A_{sl} = 2(0.20) = 0.40 \, in^2$$

$$\rho = \frac{A_{sl}}{12 \times t} = \frac{0.40}{(12)(12)} = 0.0028$$

$$V_n = (1440)\left[2.0\sqrt{4000} + (0.0028)(60000)\right]/1000$$
$$= 424.1^K$$

$$\phi V_n = (0.6)(424.1^K) = 254^K$$

Wind Base Shear => 760^K

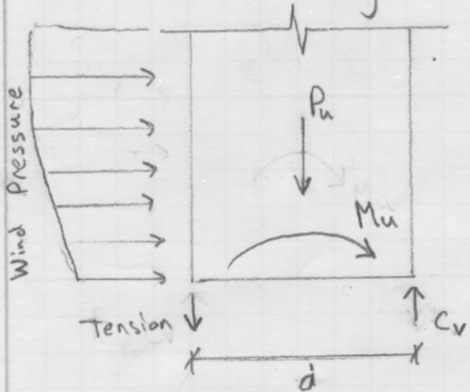
$$1.\cancel{2}D^{.90} + 1.6W \Rightarrow 1.6(760) = 1216^K$$

Conservative to Ignore self wt.

$$V_{u\,sw\#2} = 1216^K\left(\frac{0.048}{3.4}\right) = 17.2^K$$

$$\boxed{\phi V_n = 254^K > 17.2^K}$$

Overturning Boundary Capacity



$d = 10'$

Overturning moment due to wind

$M = 13{,}466.6^{'K}$

Moment on SW#2 $\Rightarrow 13466.6^{'K}\left(\frac{0.048}{3.4}\right)$
$= 190^{'K}$

Self weight of wall
$P = 448^{K}$

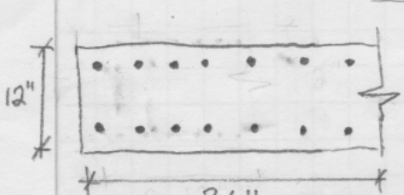
$1.2D + 1.6W \Rightarrow 1.2(448) + 1.6(190)$
$= 537.6^{K} + 304.2^{'K}$

$$C_v = \frac{P_u}{2} + \frac{M_u}{d} = \frac{537.6^{K}}{2} + \frac{304.2^{'K}}{10'}$$
$$= 299.2^{k} \approx 300^{K} \equiv P_{u\,B.E.}$$

Boundary Element Capacity

20 - #9's on Tension side
14 - #9's compressive side

<u>Compression side</u>



$A_{st} = 14(1.0) = 14 in^2$

$\rho_{st} = \frac{14}{(12)(36)} = 0.0324$

$\rho_{min} = 0.01 < \rho_{st} = 0.0324 < \rho_{max} = 0.06$
$\therefore$ ok

Axial Load capacity

$\phi P_n(max) = 0.8\phi[0.85f'_c(A_g - A_{st}) + f_y A_{st}]$
$= 0.8(0.70)[0.85(4)(43-14) + (60)(14)]$

$\phi P_n = 525.6^{k} > P_{u\,B.E.} = 300^{k} \quad \therefore$ ok

<u>Tension Side</u>

Ignore Concrete
$A_{st} = 20 in^2$

$0.8\phi f_y A_{st} = (0.8)(0.9)(60)(20)$
$= 864^{k} > 300^{k}$

$\therefore$ Reinforcing Bars can with stand overturning Tension

SW # 6 N-S Direction (Seismic Controls)

$f'_c = 4000\,psi$
reinforcement: ASTM A615 Grade 60 => $f_y = 60ksi$
#4 @ 12" each way, each face
wall thickness = 1'-0"
$l = 19'$
$h = 298.67'$

Nominal Shear Capacity

$$V_n = A_{cv}(\alpha_c\sqrt{f'_c} + \rho_t f_y)$$

$$\frac{h_w}{l_w} = \frac{298.67}{19} = 15.7 > 2.0 \quad \therefore \alpha_c = 2.0$$

$$A_{cv} = (12)(19' \times 12) = 2736\,in^2$$

2 curtains of reinforcement (#4 bars)

$$A_{se} = 2(0.20) = 0.40\,in^2$$

$$\rho = \frac{A_{se}}{12 \times t} = \frac{0.40}{(12)(12)} = 0.0028$$

$$V_n = (2736)\left[2.0\sqrt{4000} + (0.0028)(60000)\right]/1000$$
$$= 805.7^K$$

$$\phi V_n = (0.6)(805.7^K) = 483^K$$

Seismic Base Shear => 693^k

$$\cancel{1.2}^{\,.70}D + 1.0E \Rightarrow 693^K$$

Conservative to Ignore self wt.

$$V_{u_{sw\#6}} = 693^K\left(\frac{0.33}{1.8}\right) = 127^K$$

$$\boxed{\phi V_n = 483^K > V_u = 127^K}$$