Executive Summary

Parkridge 6 is a 7 story 226,000 sq.ft. commercial office building located in Reston, VA. The building is designed to a maximum height of 115'. The south face of the building is made up of sloping columns that slope outward from the ground level to the roof. The north face of the building contains an arcade created by stepped portions of additional floor area on the second floor through the fifth floor.

The foundation for Parkridge 6 is a shallow foundation system made up primarily of spread footings. The typical floor is a composite system with 3 ¼" of lightweight concrete on a 2"-20 gauge steel deck. The buildings grid consists of 3 bays in the N-S direction spaced at 37'-2", 35'-0", and 37'-2" respectively. In the E-W direction there are 10 bays with the first bay on both ends being 25'-8" and all others 25'-0".

The lateral system for Parkridge 6 is a series of braced frames. In the N-S direction there are 2 frames and in the E-W direction there are 3 frames. The bracing elements of these frames are made up of HSS sections ranging from 8x8 to 12x12.

Through this report I have found that seismic loading controls the lateral design of Parkridge 6 based on the load combinations in ASCE 7-05. In addition to the applied seismic loads the building induces other lateral loads based on the eccentricity of the frames enter of rigidity to the buildings center of gravity. The resulting torsion from this eccentricity increased the load in the frames by up to 50 Kips. The sloping columns on the south face also create a lateral load due to self weight.

The method used to distribute the lateral force to each of the frames was done using the distribution by rigidity method. In this method the rigidity of each frame is determined. Then the rigidity of each frame is divided by the sum of the rigidities in the same direction. This decimal value is the percentage of the load that will be resisted by that particular frame.

Using the determined worst case lateral loading the deflection of the building was checked using a computer model created using RISA3D. The deflection was then compared to the industry standard of H/400. Through this report I have determined that Parkridge 6 falls well within the H/400 limitation with a deflection of H/650 in the N-S direction and H/900 in the E-W direction.

A member check at the base of Frame 5 indicated that an HSS 8x8x5/8 should be used to resist the applied loading. The original design calls for an HSS 8x8x1/2 the variation in size could have been caused by inaccuracies in my torsion calculation,

inaccuracies in my calculation of resultant lateral load caused by the sloping columns, or over estimation of loads.

The overturning moment caused by the worst case lateral load was also calculated. It was determined that the buildings self weight is adequate to resist the overturning moment resulting in a net compression in the shallow foundation system.

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Introduction

The proposed Parkridge Center – Phase VI building is a 226,000 Sq. Ft., seven story commercial office building located in Reston, VA. The building is designed to a maximum height of 115'. The south face of the building slopes outward from the ground level to the roof, while on the north face of the building there are stepped portions from the second floor to the 5th floor creating an arcade at ground level. There is no sub grade portion of the building other than the foundations.

Existing Gravity System

Foundations

Parkridge 6 rests on a shallow foundation system consisting of spread footings ranging in size from 5' x 5' to 20' x 20' with depths ranging from 12" to 42". The lateral resisting elements of the building rest on mat foundations. The allowable bearing pressure is 3000 psf. The slab on grade is 4" thick and is reinforced with a 6x6-10/10 welded wire mesh.

Floor System

Each floor contains the same three by ten bay core. The south most exterior bay on each floor varies based on the slope of the columns on the south face creating larger floor area on higher level floors. Floors 2 thru 5 contain extra floor area on the north side of the building above the arcade. The North-South (N-S) spans of the core three bays are 37'-2" for the exterior bays and 35'-0" for the interior bay. The East-West (E-W) spans of the core bays are 25'-8" for the first interior bay and then 25'-0" for the remaining bays. Intermediate beams are spaced at the third points of each bay and span in the N-S direction. Typical beam sizes for the core bays are W21's for the interior girders, W18's for the exterior girders, and W16's for the intermediate beams. Each beam is cambered to 1-1/4" this was done to account for serviceability issues arising from the members chosen. Each floor above grade uses a composite deck made up of 3 ¼" Lightweight concrete on 2"-20 gage steel deck. The total floor thickness is 5 1/4". The slab itself is to be reinforced with 6x6-10/10 WWM.

Columns

Each column extends 3 floors and is spliced above the slab. The columns along the south face of the building, column line A.1, are sloped outward from the ground to the roof. Typical sizes for the sloped columns begin at a W12x65 at the roof to the 7th floor, W12x96 from the 7th floor to the 4th floor, and W12x152 from the 4th floor to the foundation. Typical sizes for the interior columns range from a W12x53 at the upper floor to a W14x233 at the base of the building.

Existing Lateral System

Five braced frames make up the lateral system for the building. There are two frames in the N-S direction and three frames in the E-W direction. The diagonal members of the frames are HSS 10x10x1/2 for the N-S frames and HSS 8x8x1/2 for the E-W frames. Frames two and three are connected by two intermediate frames at the roof. The diagonal members of the two intermediate frames are HSS 8x8x1/4. Frame three is an eccentric braced frame while all the other frames are concentrically braced.

Report Topics

The following sections can be found in this report:

- Gravity & Lateral Loads
- Lateral load Cases
- Load Distribution
- Lateral Analysis
- Conclusion

Typical Floor Plans – With Lateral Frames Highlighted

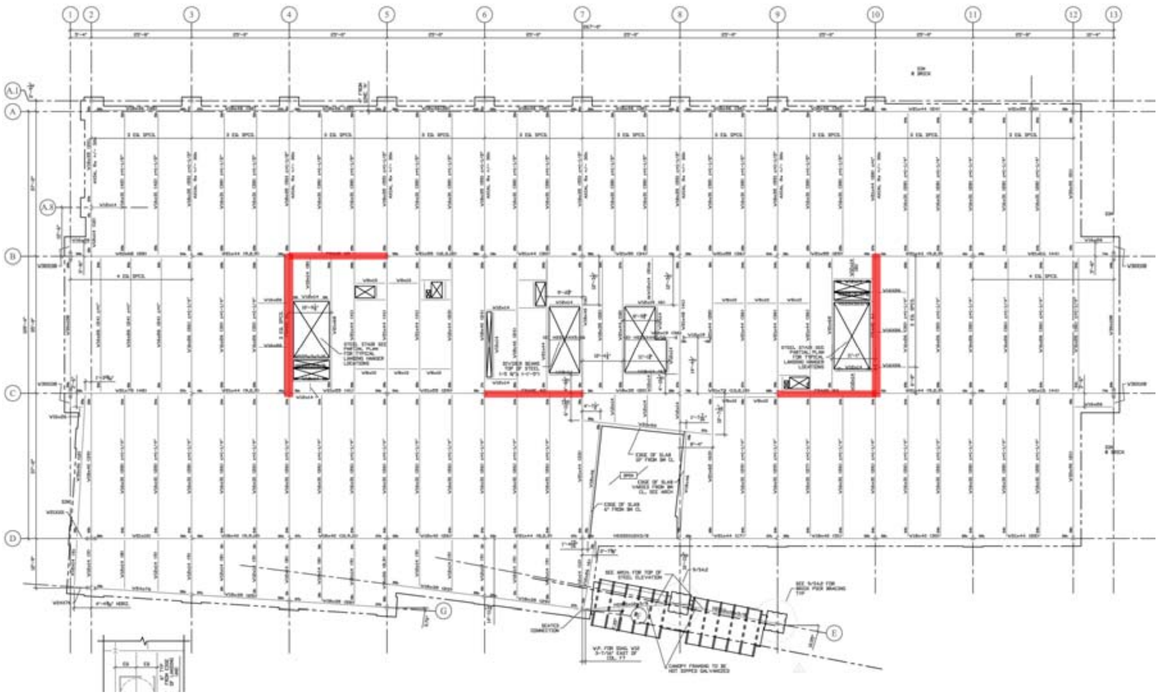


Fig 3.1 – 2nd Floor plan with highlighted frames

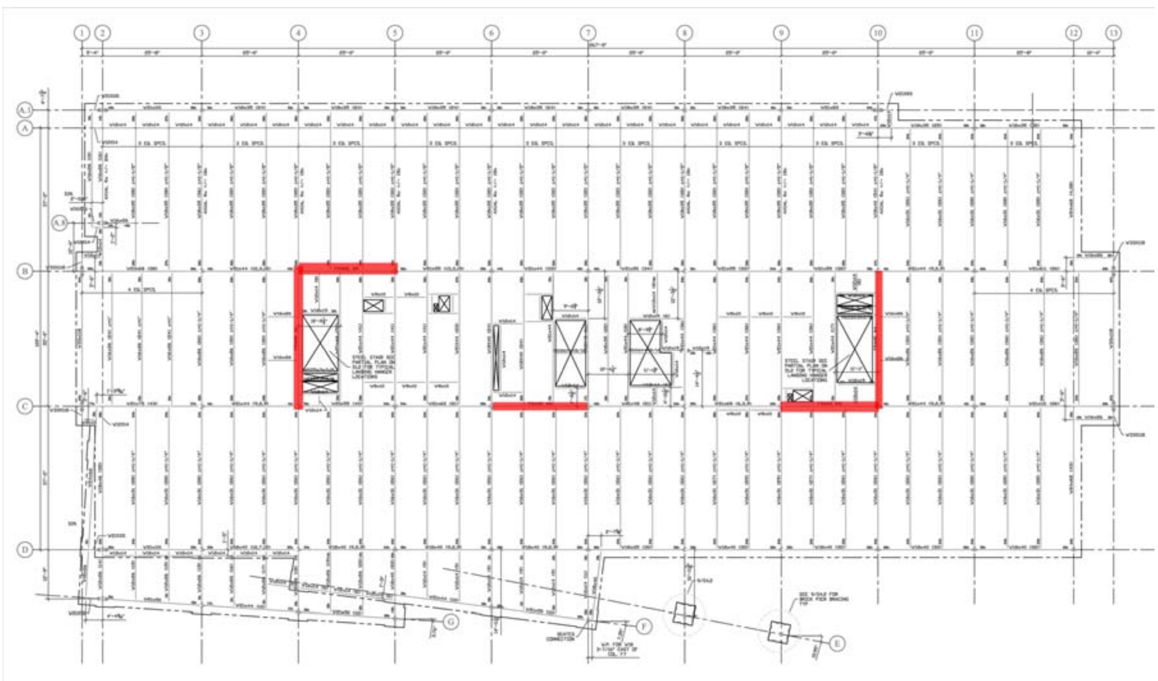


Fig 3.2 – 3rd Floor plan with highlighted frames

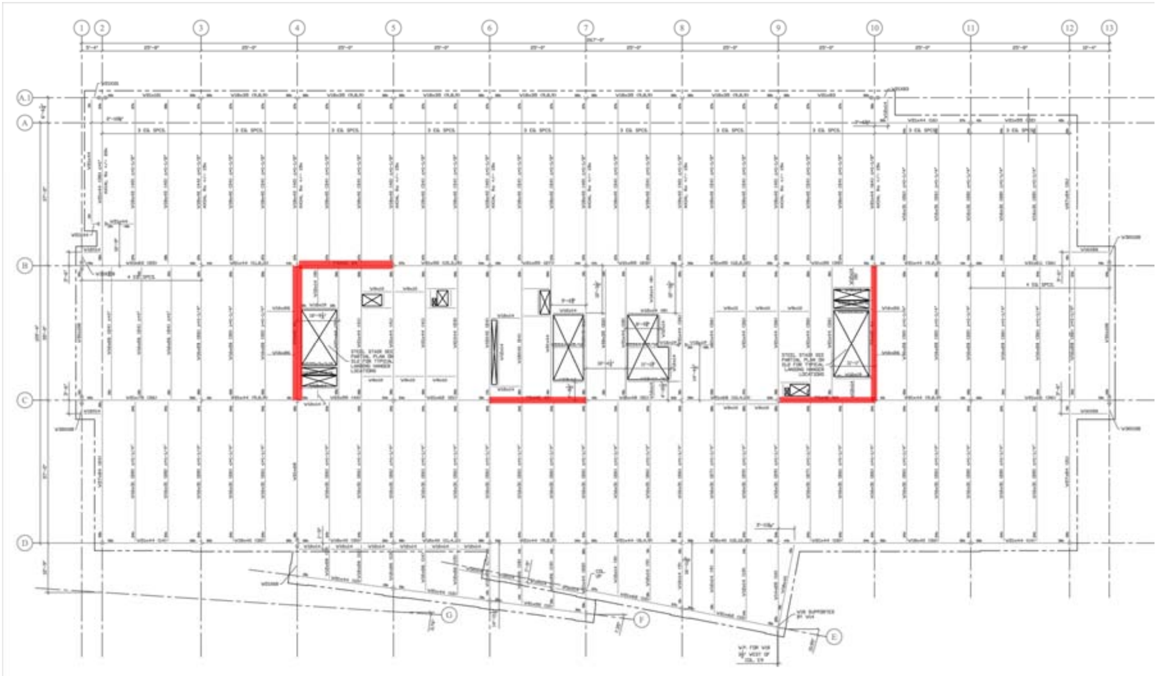


Fig 3.3 – 4th floor plan with highlighted frames

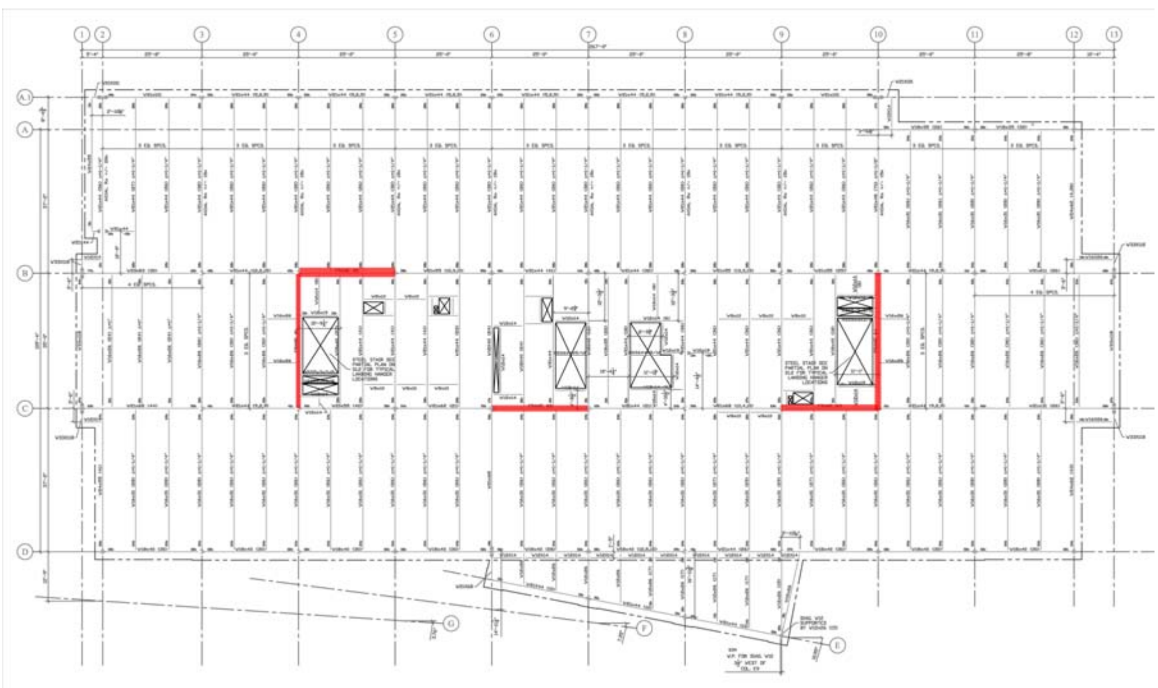


Fig 3.4 – 5th Floor plan with highlighted frames

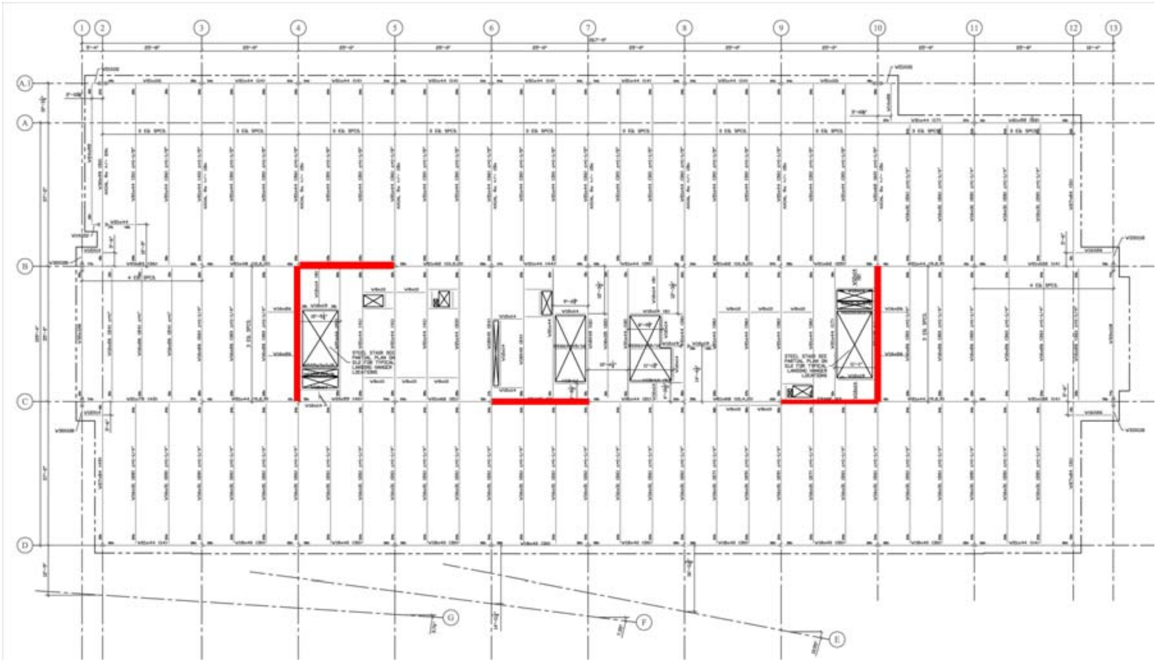


Fig 3.5 – 6th Floor plan with highlighted frames

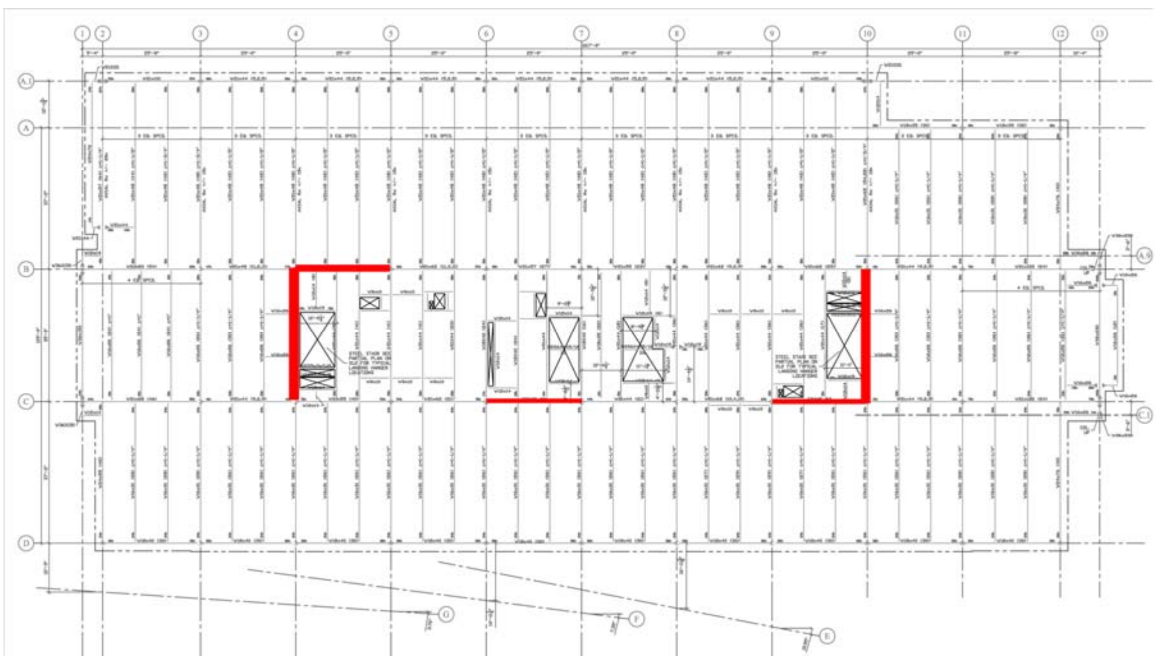


Fig 3.6 – 7th floor plan with highlighted frames

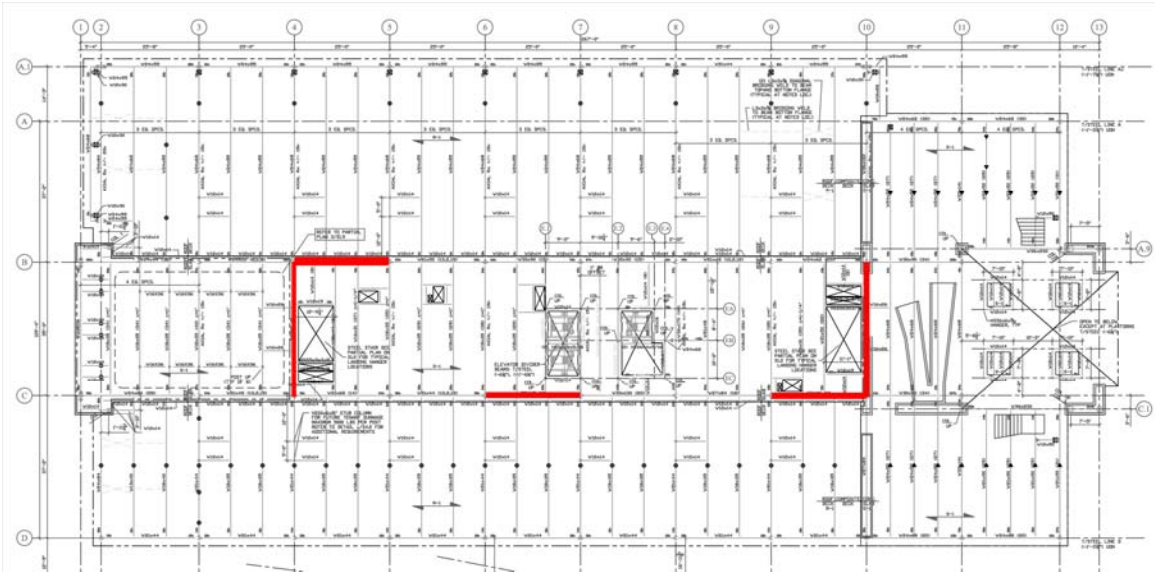


Fig 3.7 – Roof plan with highlighted frames

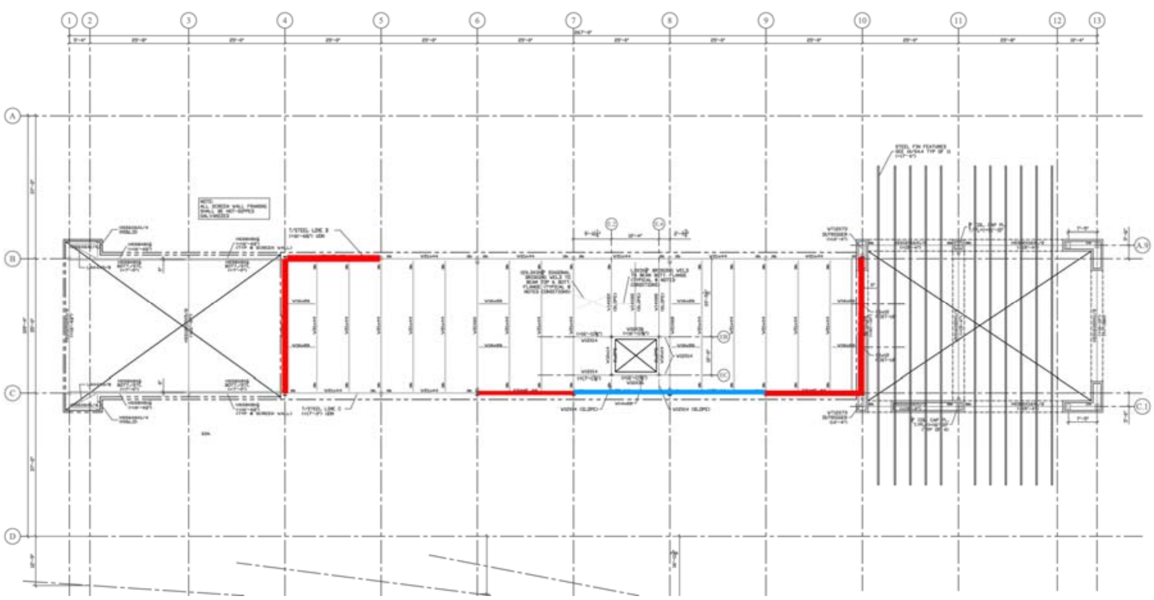


Fig 3.8 – Penthouse Roof plan with highlighted frames

Gravity Loads

Live Loads – IBC Table 1607.1	
Roof Garden	100 PSF
Offices	70 PSF
Corridors	80 PSF
Stair and Exits	100 PSF
Lobbies and First Floor Corridors	100 PSF

Table 3.1 – Live Loads

The value of live load for offices includes a 20 PSF addition for partitions. To be consistent with the original design a value of 100 PSF will be used as the live load on a typical floor.

Snow Load Chapter 7 ASCE7-05	
Pg	30 PSF
C_e	0.9
C_t	1.0
I	1.0
$P_{f,\ min}$	20 PSF
$P_{f,\ Calculated}$	18.9 PSF
P_f	20 PSF

Table 3.2 – Roof Snow Load

The roof live load will be taken to be equal to the calculated snow load of 20 psf.

Dead Loads		
Typical Floor		
Composite Floor System	41 PSF	Estimated Using United Steel Deck Catalog
Misc. (Self wt., finishes, etc.)	10 PSF	Estimated Using AISC Manual of Steel Constr.
Ponding of Concrete	10 PSF	
Roof		
Deck	2 PSF	Estimated Using United Steel Deck Catalog
Insulation	3 PSF	Estimated using AISC Manual of Steel Constr.
Roofing	20 PSF	
Curtain Wall		
Glass Curtain Wall	.215 KLF	From Building Specifications
Pre-cast Assembly	.55 KLF	From Building Specifications
Roof Garden		
	160 PSF	From Materials in Specifications

Table 3.3 – Dead Loads

Lateral Loads

Wind

(See Appendix for complete spreadsheet of wind calculation)

Total Worst Case Wind Load Each Direction	
z (ft)	P (psf)
0-15	12.503
20	13.140
25	13.650
30	14.160
40	14.924
50	15.562
60	16.071
70	16.581
80	17.091
90	17.473
100	17.728
115.17	18.212

Table 3.4 – Wind Load

Wind Shear Force at each floor (Kips)								
E-W								
Width	Floor	Height 1	Load 1	Height 2	Load 2	Height 3	Load 3	Shear
127.92	2	7.50	12.50	5.00	13.14	1.67	13.65	23.31
127.92	3	3.33	13.65	5.00	14.16	5.00	14.92	24.42
127.92	4	5.00	14.92	8.33	15.56			26.13
127.92	5	1.67	15.56	10.00	16.07	1.67	16.58	27.41
127.92	6	8.33	16.58	5.00	17.09			28.61
127.92	7	5.00	17.09	9.17	17.47			31.42
127.92	Roof	0.83	17.47	6.67	17.73			16.98
45	Penthouse Roof	9.25	18.21					7.58
N-S								
Width	Floor	Height 1	Load 1	Height 2	Load 2	Height 3	Load 3	Shear
270.00	2	7.50	12.50	5.00	13.14	1.67	13.65	49.20
270.00	3	3.33	13.65	5.00	14.16	5.00	14.92	51.55
270.00	4	5.00	14.92	8.33	15.56			55.16
270.00	5	1.67	15.56	10.00	16.07	1.67	16.58	57.86
270.00	6	8.33	16.58	5.00	17.09			60.38
270.00	7	5.00	17.09	9.17	17.47			66.32
270.00	Roof	0.83	17.47	6.67	17.73			35.84
212	Penthouse Roof	9.25	18.21					35.71

Table 3.5 – Wind Floor Shear

Seismic
(See Appendix for complete spreadsheet of seismic calculation)

Seismic Force Distribution						
Floor	w_x	h_x	k	$w_x h_x^k$	$\Sigma w_i h_i^k$	C_{vx}
Base	--	--	--	--	--	--
2	2561.24	15.00	1.00	38418.56	1030201.93	0.037
3	2692.77	28.33	1.00	76295.25	1030201.93	0.074
4	2563.19	41.67	1.00	106799.39	1030201.93	0.104
5	2570.64	55.00	1.00	141385.17	1030201.93	0.137
6	2536.08	68.33	1.00	173298.77	1030201.93	0.168
7	2645.26	81.67	1.00	216029.31	1030201.93	0.210
Roof	2638.54	96.67	1.00	255058.81	1030201.93	0.248
Penthouse Roof	198.98	115.17	1.00	22916.67	1030201.93	0.022
						1.000

Floor	F_x (Kips)
Base	770.19
2	28.72
3	57.04
4	79.84
5	105.70
6	129.56
7	161.50
Roof	190.68
Penthouse Roof	17.13
	770.19

Table 3.6 – Seismic Floor Shear

Lateral Load Cases

Per ASCE 7-05 (Section 2.3.2)
The Load combinations containing lateral loads are:

1.2D+1.6W+L+0.5(Lr or S or R)
1.2D+1.0E+L+0.2S
0.9D+1.6W+1.6H
0.9D+1.0E+1.6H

The last two combinations are to be used for overturning moment calculations. To find the controlling load case I compared 1.6W to 1.0E, doing this I found that the seismic loading controls in both directions. However in the north-south direction wind load controlled from floors 2 to 4. When looking at the building as a whole seismic design still controls as it produces larger total loads in the bracing at levels 2 to 4.

In addition to the pure lateral loads from wind and seismic loading there is an induced lateral load caused by the sloping columns on the south face. To calculate this force I summed moments about the base of the building and found the resultant shear at each story level to resist the moment caused by the vertical reaction at each sloped column. The loads to determine the induced story shear were factored by 1.2D and 1.0L.

Floor Shear To Resist Self Weight Overturning				
Sloped Columns:	**9**			
Floor	**P (Kips)**	**x (ft)**	**H (ft)**	**V (Kips)**
2	31.37	2.77	15	52.13
3	30.60	4.65	28.33	45.21
4	31.97	6.52	41.67	45.02
5	33.37	8.4	55	45.87
6	34.72	10.27	68.33	46.96
7	36.09	12.15	81.67	48.32
Roof	8.69	14.25	96.67	11.53

Table 3.7 – Factored Shear to Resist Lateral Load from Sloped Columns

Wind Load E - W (Kips)		
Story	Service	Factored (1.6*W)
2	23.31	37.29
3	24.42	39.07
4	26.13	41.81
5	27.41	43.86
6	28.61	45.77
7	31.42	50.27
Roof	16.98	27.17
Penthouse Roof	7.58	12.13

Table 3.8 – Factored Wind Load E – W

Wind Load N - S (Kips)		
Story	Service	Factored (1.6*W)
2	49.20	78.72
3	51.55	82.48
4	55.16	88.26
5	57.86	92.57
6	60.38	96.61
7	66.32	106.11
Roof	35.84	57.35
Penthouse Roof	35.71	57.14

Table 3.9 – Factored Wind N – S

Seismic E - W & N - S (Kips)		
Story	Service	Factored (1.0*E)
2	28.72	28.72
3	57.04	57.04
4	79.84	79.84
5	105.70	105.70
6	129.56	129.56
7	161.50	161.50
Roof	190.68	190.68
Penthouse Roof	17.13	17.13

Table 3.10 – Factored Seismic Each Direction

Maximum E - W Story Shear (Kips)
37.29
57.04
79.84
105.70
129.56
161.50
190.68
17.13

Maximum N - S Story Shear (Kips)
78.72
82.48
88.26
105.70
129.56
161.50
190.68
57.14

Controlling Force	
##.##	Seismic
##.##	Wind

Table 3.11 – Controlling Force by Story Level

Lateral Load Distribution

To determine the distribution of the lateral loads I modeled each frame in RISA 3D and applied a 1 Kip load at the penthouse roof level. I then determined the rigidity of each frame by the equation P=R*Δ. To determine the percentage of lateral load taken by each frame, I then took the rigidity of each frame and divided it by the sum of the rigidities in the same direction.

E - W Frames			
Frame	**Δ**	**Rigidity (1/Δ)**	**% Flr. Shear (R/ΣR)**
2	0.012	83.33	33.18%
3	0.013	76.92	30.63%
5	0.011	90.91	36.19%
	Tot.	251.17	100.00%
N - S Frames			
Frame	**Δ**	**Rigidity (1/Δ)**	**% Flr. Shear (R/ΣR)**
1	0.009	111.11	57.14%
4	0.012	83.33	42.86%
	Tot.	194.44	100.00%

Table 3.12 – Relative Frame Rigidities

Seismic - Story Shear E - W							
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Shear (K)
2	28.72		9.53	8.80		10.40	28.72
3	57.04		18.92	17.47		20.65	57.04
4	79.84		26.49	24.45		28.90	79.84
5	105.70		35.07	32.37		38.26	105.70
6	129.56		42.99	39.68		46.89	129.56
7	161.50		53.59	49.46		58.46	161.50
Roof	190.68		63.27	58.40		69.02	190.68
Penthouse Roof	17.13		5.68	5.25		6.20	17.13
N - S							
2	28.72	16.41			12.31		28.72
3	57.04	32.59			24.45		57.04
4	79.84	45.63			34.22		79.84
5	105.70	60.40			45.30		105.70
6	129.56	74.03			55.53		129.56
7	161.50	92.29			69.22		161.50
Roof	190.68	108.96			81.72		190.68
Penthouse Roof	17.13	9.79			7.34		17.13

Table 3.13 – Seismic Story Shear Distribution

N - S Floor Shear To Resist Self Weight Overturning Moment Due To Sloped Columns							
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Shear (K)
2	52.13	29.79			22.34		52.13
3	45.21	25.83			19.37		45.21
4	45.02	25.73			19.30		45.02
5	45.87	26.21			19.66		45.87
6	46.96	26.83			20.13		46.96
7	48.32	27.61			20.71		48.32
Roof	11.53	6.59			4.94		11.53

Table 3.14 – Distribution of Story Shear Caused by Sloped Columns

Lateral Analysis

The lateral analysis for this report was accomplished by using both computer models of each individual frame and hand calculations. The computer model was used in determining each frame's stiffness with an applied 1 kip load. The model was also used to determine frame deflections based on the worst case applied load.

Determination of the worst case applied lateral load was determined by the load combinations in the previous section of this report. Also the resulting torsion caused by the eccentricity of the center of rigidity of the frames from the center of mass of the building was applied along with the seismic load. Eccentricity from the geometric center of the building was also checked for the applied wind loads and was found to not control design. Torsion forces equaling less than zero were neglected as they would result in smaller loads being applied to the frames.

The computer models were produced using RISA 3D and are available upon request.

Frames 2 and 3 were analyzed as two separate frames for the determination of stiffness and distribution of loads. They were then modeled with the intermediate frames at the penthouse roof level with the applied worst case lateral loads for determination of overall deflection.

Total frame drifts were compared to H/400.

Worst Case Lateral Load (Kips)					
E - W					
Story	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
2		9.53	8.80		14.06
3		18.92	17.47		27.58
4		26.49	24.45		37.77
5		35.07	32.37		48.36
6		42.99	39.68		62.11
7		53.59	49.46		76.44
Roof		63.27	58.40		119.07
Penthouse Roof		5.68	5.25		6.20
N - S					
2	46.20			34.72	
3	58.43			45.30	
4	71.35			53.91	
5	86.61			66.25	
6	100.87			79.22	
7	119.90			94.52	
Roof	115.55			91.06	
Penthouse Roof	9.79			7.34	

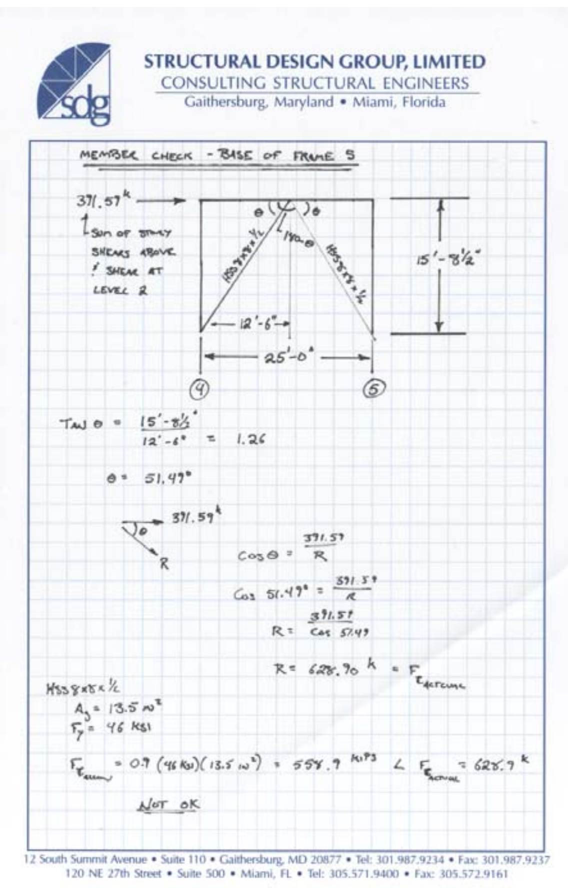
Table 3.15 – Worst Case Lateral Load with Torsion Effects

Total Frame Deflection			
Frame	Δ	H/Δ	H/400
1	2.102	657.49	3.46
2	1.401	986.47	3.46
3	1.402	985.76	3.46
4	1.62	853.11	3.46
5	2.106	656.24	3.46

Table 3.16 – Total Frame Deflections

Table 3.16 shows that each frame falls within the H/400 limitation, with a maximum drift of 2.106 in.

I have chosen to check the diagonal member at the base of frame 5 for a member check. Based on the AISC Manual of Steel Construction F_t=0.9*F_y*A_g. From my hand check of the diagonal member at the base of frame 5 an HSS9x9x1/2 or an HSS8x8x5/8 are required to resist the axial load in the member. The original design calls for an HSS8x8x1/2. The variance in member size between existing and my determined size is most likely do to inaccuracy of my torsion calculation as well as my calculation of the induced lateral load due to the sloping columns.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MEMBER CHECK - BASE OF FRAME 5

$\tan\theta = \dfrac{15'\text{-}8\tfrac{1}{2}"}{12'\text{-}6"} = 1.26$

$\theta = 51.47°$

$\cos\theta = \dfrac{371.57}{R}$

$\cos 51.47° = \dfrac{371.57}{R}$

$R = \dfrac{371.57}{\cos 51.47}$

$R = 628.90^k = F_{t_{actual}}$

HSS8x8x½
$A_g = 13.5\ in^2$
$F_y = 46\ ksi$

$F_{t_{allow}} = 0.9(46\ ksi)(13.5\ in^2) = 558.9^{kips} < F_{t_{actual}} = 628.9^k$

NOT OK

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REQUIRE A_g

$628.9 = 0.9(46)A_g$

$A_g = 15.19\ in^2$

USE EITHER
HSS 9x9x½ $A_g = 15.3$
OR
HSS 8x8x⅝ $A_g = 16.4$

REASONS FOR DIFFERENT MEMBER SIZE:

- CALCULATION OF INDUCED LATERAL LOAD FROM SLOPED COLUMNS MAY BE HIGH
- ESTIMATION OF LOADS MAY BE HIGH

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Overturning Moment and Impact on Foundation

The calculated overturning moment in the N-S direction indicates that the foundation on the north end of the building will be in compression. The overall compression is approximately 6300 lbs. The N-S direction controls as it has less area resisting the applied lateral and sloped column resultant overturning moment. The existing foundation system is capable of taking this compressive force.

Conclusions

Using the determined worst case lateral loading the deflection of the building was checked using a computer model created using RISA3D. The deflection was then compared to the industry standard of H/400. Through this report I have determined that Parkridge 6 falls well within the H/400 limitation with a deflection of H/650 in the N-S direction and H/900 in the E-W direction.

A member check at the base of Frame 5 indicated that an HSS 8x8x5/8 should be used to resist the applied loading. The original design calls for an HSS 8x8x1/2 the variation in size could have been caused by inaccuracies in my torsion calculation, inaccuracies in my calculation of resultant lateral load caused by the sloping columns, or over estimation of loads.

The overturning moment caused by the worst case lateral load was also calculated. It was determined that the buildings self weight is adequate to resist the overturning moment resulting in a net compression in the shallow foundation system.

Appendix

Wind Excel Tables

Wind Loading for Parkridge Center - Phase VI		
ASCE7-05 Chapter 6		
Method 2		
V	90	mph
Kd	0.85	MWFRS & Components and Cladding
I	0.87	Importance Factor (Occupancy Category II - Non-Hurricane Prone)
Kz, Kh	(see Table)	
Kzt	1	
G, E-W	0.859	
G, N-S	0.841	
	Enclosed	Enclosure Classification
GCpi	0.18	+/-
Wall Pressure		
Cp, E-W	0.8	windward
	-0.5	leeward
	-0.7	side wall
Cp, N-S	0.8	windward
	-0.3	leeward
	-0.7	side wall
Roof Pressure		
Cp, E-W	-0.9	0 to h/2
	-0.9	h/2 to h
	-0.5	h to 2h
	-0.3	>2h
Cp, N-S	-0.9	0 to h/2
	-0.9	h/2 to h
	-0.5	h to 2h
	-0.3	>2h
qz, qh	(see Table)	

Windward		Leeward		Sidewall		Roof	
E-W		E-W		E-W		E-W	
z	P	z	P	z	P	0 to h/2	-14.275
0-15	6.499	0-15	-6.004	0-15	-10.140	h/2 to h	-14.275
20	7.136	20	-6.004	20	-10.140	h to 2h	-6.004
25	7.646	25	-6.004	25	-10.140	>2h	-1.869
30	8.155	30	-6.004	30	-10.140		
40	8.920	40	-6.004	40	-10.140		
50	9.557	50	-6.004	50	-10.140		
60	10.067	60	-6.004	60	-10.140		
70	10.577	70	-6.004	70	-10.140		
80	11.087	80	-6.004	80	-10.140		
90	11.469	90	-6.004	90	-10.140		
100	11.724	100	-6.004	100	-10.140		
115.17	12.207	115.17	-6.004	115.17	-10.140		
N-S		N-S		N-S		N-S	
z	P	z	P	z	P	0 to h/2	-13.897
0-15	6.278	0-15	-1.742	0-15	-9.846	h/2 to h	-13.897
20	6.903	20	-1.742	20	-9.846	h to 2h	-5.794
25	7.402	25	-1.742	25	-9.846	>2h	-1.742
30	7.902	30	-1.742	30	-9.846		
40	8.651	40	-1.742	40	-9.846		
50	9.275	50	-1.742	50	-9.846		
60	9.775	60	-1.742	60	-9.846		
70	10.274	70	-1.742	70	-9.846		
80	10.773	80	-1.742	80	-9.846		
90	11.148	90	-1.742	90	-9.846		
100	11.398	100	-1.742	100	-9.846		
115.17	11.871	115.17	-1.742	115.17	-9.846		

Total Worst Case Wind Load Each Direction	
z (ft)	P (psf)
0-15	12.503
20	13.140
25	13.650
30	14.160
40	14.924
50	15.562
60	16.071
70	16.581
80	17.091
90	17.473
100	17.728
115.17	18.212

Wind Shear Force at each floor (Kips)								
E-W								
Width	Floor	Height 1	Load 1	Height 2	Load 2	Height 3	Load 3	Shear
127.92	2	7.50	12.50	5.00	13.14	1.67	13.65	23.31
127.92	3	3.33	13.65	5.00	14.16	5.00	14.92	24.42
127.92	4	5.00	14.92	8.33	15.56			26.13
127.92	5	1.67	15.56	10.00	16.07	1.67	16.58	27.41
127.92	6	8.33	16.58	5.00	17.09			28.61
127.92	7	5.00	17.09	9.17	17.47			31.42
127.92	Roof	0.83	17.47	6.67	17.73			16.98
45	Penthouse Roof	9.25	18.21					7.58
N-S								
Width	Floor	Height 1	Load 1	Height 2	Load 2	Height 3	Load 3	Shear
270.00	2	7.50	12.50	5.00	13.14	1.67	13.65	49.20
270.00	3	3.33	13.65	5.00	14.16	5.00	14.92	51.55
270.00	4	5.00	14.92	8.33	15.56			55.16
270.00	5	1.67	15.56	10.00	16.07	1.67	16.58	57.86
270.00	6	8.33	16.58	5.00	17.09			60.38
270.00	7	5.00	17.09	9.17	17.47			66.32
270.00	Roof	0.83	17.47	6.67	17.73			35.84
212	Penthouse Roof	9.25	18.21					35.71

Extra Tables for Wind Load Calculations

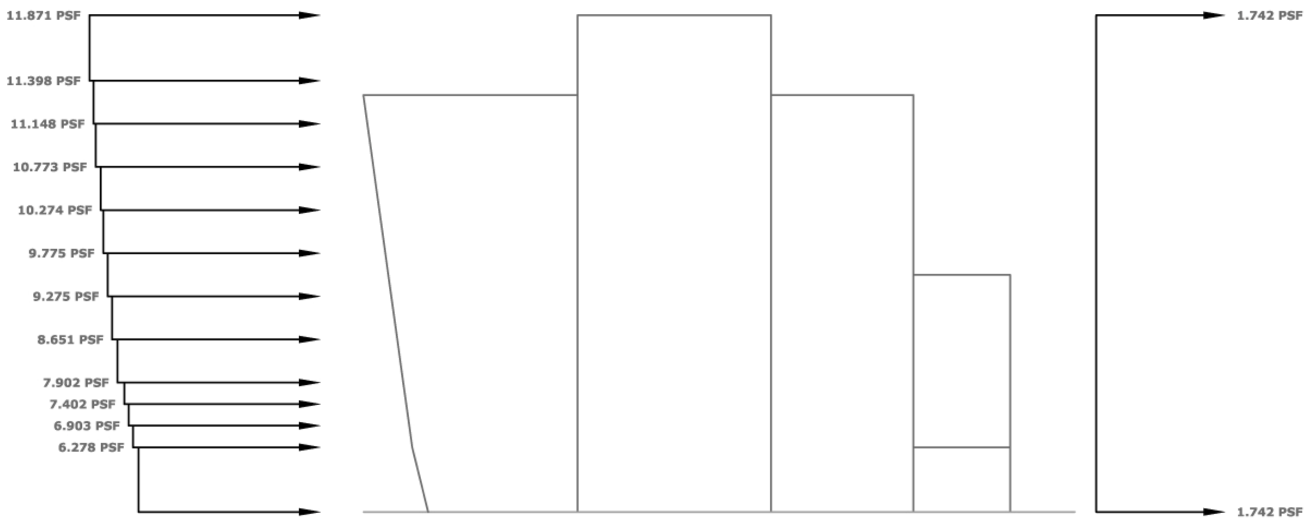
Kz, Kh	
z, ft.	C
0-15	0.85
20	0.90
25	0.94
30	0.98
40	1.04
50	1.09
60	1.13
70	1.17
80	1.21
90	1.24
100	1.26
115.17	1.30
120	1.31
140	1.36
160	1.39
180	1.43
200	1.46
250	1.53
300	1.59
350	1.64
400	1.69
450	1.73
500	1.77

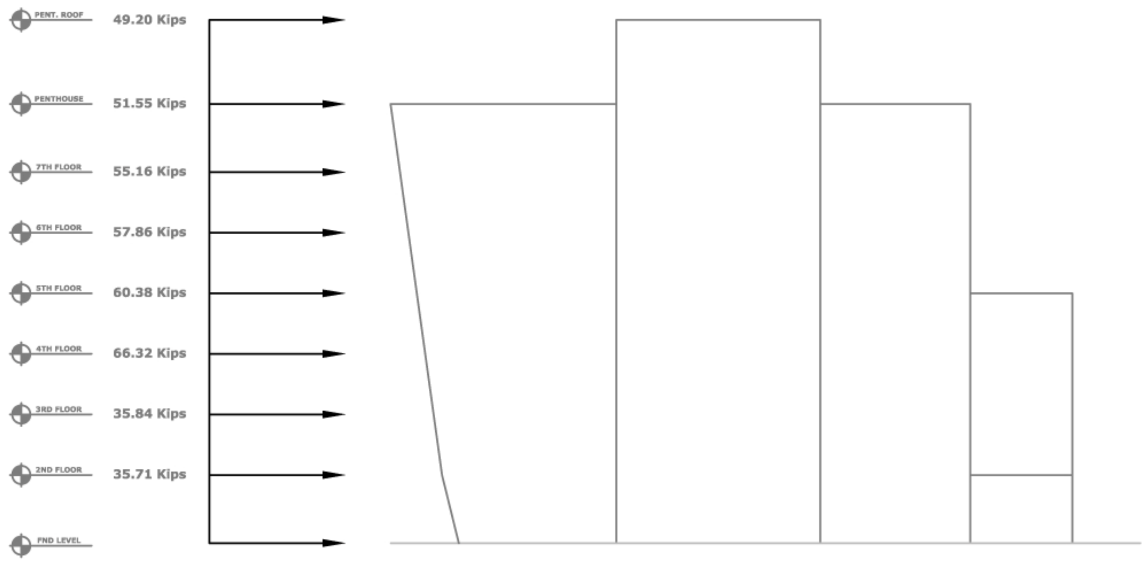
z, ft.	q	
0-15	15.771	
20	16.699	
25	17.441	
30	18.183	
40	19.297	
50	20.224	
60	20.967	
70	21.709	
80	22.451	
90	23.008	
100	23.379	
115.17	24.082	<------------ qh

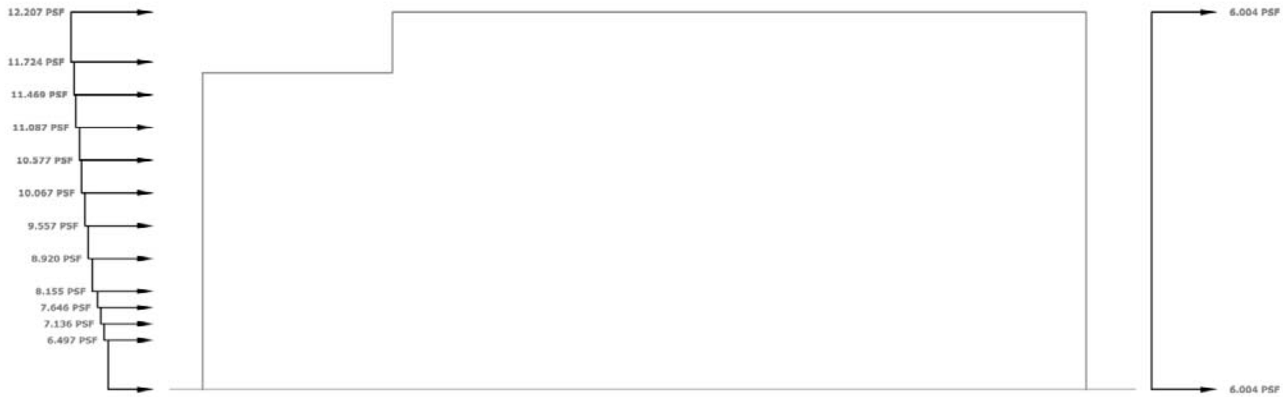
G		
B, E-W	121.43	ft
B, N-S	267	ft
h	115.17	ft
l	500	ft
z,bar	69.102	ft
z,min	15	ft
e,bar	0.2	
c	0.2	
Iz	0.176821	
gq	3.4	
gv	3.4	
Lz	579.65	
Q, E-W	0.858	
Q, N-S	0.821	
G, E-W	0.859	
G, N-S	0.841	

L/B	
L/B, E-W	0.45
L/B, N-S	2.20

Wind Loading Distribution and Story Shear Diagrams






12.207 PSF
11.724 PSF
11.469 PSF
11.087 PSF
10.577 PSF
10.067 PSF
9.557 PSF
8.920 PSF
8.155 PSF
7.646 PSF
7.136 PSF
6.497 PSF
6.004 PSF
6.004 PSF


7.58 Kips
16.98 Kips
31.42 Kips
28.61 Kips
27.41 Kips
26.13 Kips
24.42 Kips
23.31 Kips

Seismic Excel Tables

Seismic Loading
ASCE7-05
Calculation of Building Weight

Floor	Area		DL		Weight	
1	--	SF	--	KSF	--	kips
2	31705.80	SF	0.061	KSF	1934.05	kips
3	32715.30	SF	0.061	KSF	1995.63	kips
4	32211.40	SF	0.061	KSF	1964.90	kips
5	32643.40	SF	0.061	KSF	1991.25	kips
6	31963.60	SF	0.061	KSF	1949.78	kips
7	32443.76	SF	0.061	KSF	1979.07	kips
Roof	18122.80	SF	0.025	KSF	453.07	kips
Garden	6694.84	SF	0.16	KSF	1071.17	kips
Mechanical	7959.25	SF	0.14	KSF	1114.30	kips
Penthouse roof	7959.25	SF	0.025	KSF	198.98	kips
				Total:	14652.20	kips

Precast Panels

Wall	Perimeter		Height		DL		Weight	
1	765.81	LF	15.00	Ft	0.055	KSF	631.80	kips
2	855.25	LF	13.33	Ft	0.055	KSF	627.18	kips
3	950.65	LF	13.33	Ft	0.055	KSF	697.14	kips
4	815.85	LF	13.33	Ft	0.055	KSF	598.29	kips
5	790.08	LF	13.33	Ft	0.055	KSF	579.39	kips
6	799.50	LF	13.33	Ft	0.055	KSF	586.30	kips
7	807.50	LF	15.00	Ft	0.055	KSF	666.19	kips
						Total:	4386.29	kips

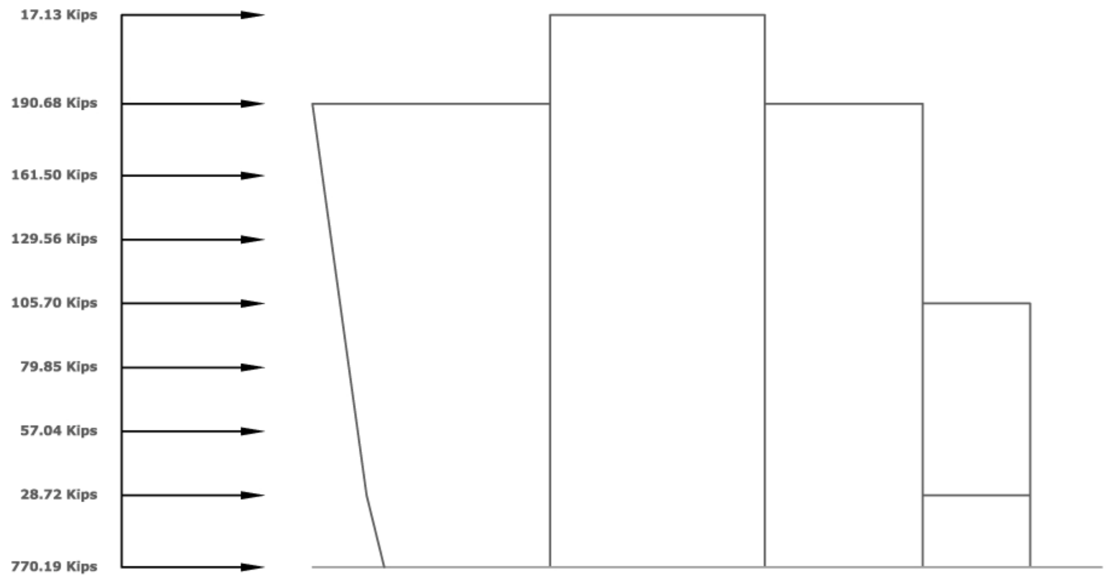
Total Building Weight:	**19038.488**	**Kips**

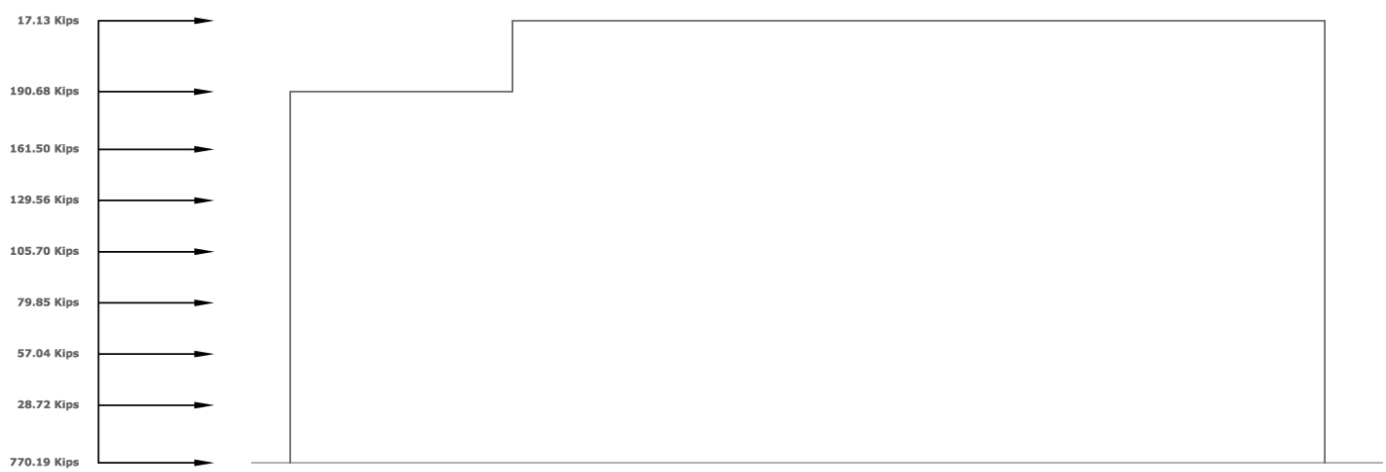
Calculation of Base Shear		
S_s	0.200	
S_1	0.080	
S_{ms}	0.320	
S_{m1}	0.192	
S_{ds}	0.213	
S_{d1}	0.128	
R	3	
Ω_0	3	
C_d	3	
I	1	
Ct	0.03	
x	0.75	
h	115.17	ft
Ta	1.05	
C_s	0.040	
C_sW	770.19	kips

Seismic Force Distribution						
Floor	w_x	h_x	k	$w_xh_x^k$	$\Sigma\, w_ih_i^k$	C_{vx}
Base	--	--	--	--	--	--
2	2561.24	15.00	1.00	38418.56	1030201.93	0.037
3	2692.77	28.33	1.00	76295.25	1030201.93	0.074
4	2563.19	41.67	1.00	106799.39	1030201.93	0.104
5	2570.64	55.00	1.00	141385.17	1030201.93	0.137
6	2536.08	68.33	1.00	173298.77	1030201.93	0.168
7	2645.26	81.67	1.00	216029.31	1030201.93	0.210
Roof	2638.54	96.67	1.00	255058.81	1030201.93	0.248
Penthouse Roof	198.98	115.17	1.00	22916.67	1030201.93	0.022
						1.000

Floor	F_x (Kips)
Base	770.19
2	28.72
3	57.04
4	79.84
5	105.70
6	129.56
7	161.50
Roof	190.68
Penthouse Roof	17.13
	770.19

Seismic Story Shear Diagrams





Calculation of Story Shear to Resist Overturning at Sloped Columns

2	**Loads (unfactored)**			
	Dead	61	PSF	
	Live	100	PSF	
	Curtain Wall	0.215	KLF	Glass
		0.55	KLF	Pre-Cast
	Load Combinations (w/o Curtain Wall)			
	1.4D	0.085	KSF	
	1.2D+1L	0.173	KSF	
	Intermediate Beam (Exterior Bay)			
	Trib Width	8.56	ft	
	Length	37.17	ft	
	Shear	27.55	Kips	
	Intermediate Beam (Interior Bay)			
	Trib Width	8.34	ft	
	Length	37.17	ft	
	Shear	26.85	Kips	
	Exterior Girder (Exterior Bay)			
	Length	25.67	ft	
	Shear	30.86	Kips	
	Exterior Girder (Interior Bay)			
	Length	25.00	ft	
	Shear	30.07	Kips	
	Exterior Girder To Column			
	Trib Width	4.28	ft	
	Length	39.94	ft	
	x	2.77	ft	
	Column R:	**30.12**	Kips	
	First Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	39.94	ft	
	x	2.77	ft	
	Column R:	**31.37**	Kips	
	Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	39.94	ft	
	x	2.77	ft	
	Column R:	**31.31**	Kips	

3	**Loads (unfactored)**			
	Dead	61	PSF	
	Live	100	PSF	
	Curtain Wall	0.215	KLF	Glass
		0.55	KLF	Pre-Cast
	Load Combinations (w/o Curtain Wall)			
	1.4D	0.085	KSF	
	1.2D+1L	0.173	KSF	
	Intermediate Beam (Exterior Bay)			
	Trib Width	8.56	ft	
	Length	41.82	ft	
	Shear	31.00	Kips	
	Intermediate Beam (Interior Bay)			
	Trib Width	8.34	ft	
	Length	41.82	ft	
	Shear	30.20	Kips	
	Exterior Girder (Exterior Bay)			
	Length	25.67	ft	
	Shear	34.31	Kips	
	Exterior Girder (Interior Bay)			
	Length	25.00	ft	
	Shear	33.43	Kips	
	Exterior Girder To Column			
	Trib Width	4.28	ft	
	Length	41.82	ft	
	x	0	ft	
	Column R:	**29.30**	Kips	
	First Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	41.82	ft	
	x	0	ft	
	Column R:	**30.60**	Kips	
	Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	41.82	ft	
	x	0	ft	
	Column R:	**30.60**	Kips	

4	**Loads (unfactored)**			
	Dead	61	PSF	
	Live	100	PSF	
	Curtain Wall	0.215	KLF	Glass
		0.55	KLF	Pre-Cast
	Load Combinations (w/o Curtain Wall)			
	1.4D	0.085	KSF	
	1.2D+1L	0.173	KSF	
	Intermediate Beam (Exterior Bay)			
	Trib Width	8.56	ft	
	Length	43.69	ft	
	Shear	32.39	Kips	
	Intermediate Beam (Interior Bay)			
	Trib Width	8.34	ft	
	Length	43.69	ft	
	Shear	31.55	Kips	
	Exterior Girder (Exterior Bay)			
	Length	25.67	ft	
	Shear	35.70	Kips	
	Exterior Girder (Interior Bay)			
	Length	25.00	ft	
	Shear	34.78	Kips	
	Exterior Girder To Column			
	Trib Width	4.28	ft	
	Length	43.69	ft	
	x	0	ft	
	Column R:	**30.61**	Kips	
	First Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	43.69	ft	
	x	0	ft	
	Column R:	**31.97**	Kips	
	Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	43.69	ft	
	x	0	ft	
	Column R:	**31.97**	Kips	

5	**Loads (unfactored)**			
	Dead	61	PSF	
	Live	100	PSF	
	Curtain Wall	0.215	KLF	Glass
		0.55	KLF	Pre-Cast
	Load Combinations (w/o Curtain Wall)			
	1.4D	0.085	KSF	
	1.2D+1L	0.173	KSF	
	Intermediate Beam (Exterior Bay)			
	Trib Width	8.56	ft	
	Length	45.6	ft	
	Shear	33.80	Kips	
	Intermediate Beam (Interior Bay)			
	Trib Width	8.34	ft	
	Length	45.6	ft	
	Shear	32.93	Kips	
	Exterior Girder (Exterior Bay)			
	Length	25.67	ft	
	Shear	37.11	Kips	
	Exterior Girder (Interior Bay)			
	Length	25.00	ft	
	Shear	36.16	Kips	
	Exterior Girder To Column			
	Trib Width	4.28	ft	
	Length	45.6	ft	
	x	0	ft	
	Column R:	**31.95**	Kips	
	First Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	45.6	ft	
	x	0	ft	
	Column R:	**33.37**	Kips	
	Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	45.6	ft	
	x	0	ft	
	Column R:	**33.37**	Kips	

6	**Loads (unfactored)**			
	Dead	61	PSF	
	Live	100	PSF	
	Curtain Wall	0.215	KLF	Glass
		0.55	KLF	Pre-Cast
	Load Combinations (w/o Curtain Wall)			
	1.4D	0.085	KSF	
	1.2D+1L	0.173	KSF	
	Intermediate Beam (Exterior Bay)			
	Trib Width	8.56	ft	
	Length	47.44	ft	
	Shear	35.17	Kips	
	Intermediate Beam (Interior Bay)			
	Trib Width	8.34	ft	
	Length	47.44	ft	
	Shear	34.26	Kips	
	Exterior Girder (Exterior Bay)			
	Length	25.67	ft	
	Shear	38.48	Kips	
	Exterior Girder (Interior Bay)			
	Length	25.00	ft	
	Shear	37.49	Kips	
	Exterior Girder To Column			
	Trib Width	4.28	ft	
	Length	47.44	ft	
	x	0	ft	
	Column R:	**33.24**	Kips	
	First Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	47.44	ft	
	x	0	ft	
	Column R:	**34.72**	Kips	
	Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	47.44	ft	
	x	0	ft	
	Column R:	**34.72**	Kips	

7	**Loads (unfactored)**			
	Dead	61	PSF	
	Live	100	PSF	
	Curtain Wall	0.215	KLF	Glass
		0.55	KLF	Pre-Cast
	Load Combinations (w/o Curtain Wall)			
	1.4D	0.085	KSF	
	1.2D+1L	0.173	KSF	
	Intermediate Beam (Exterior Bay)			
	Trib Width	8.56	ft	
	Length	49.32	ft	
	Shear	36.56	Kips	
	Intermediate Beam (Interior Bay)			
	Trib Width	8.34	ft	
	Length	49.32	ft	
	Shear	35.62	Kips	
	Exterior Girder (Exterior Bay)			
	Length	25.67	ft	
	Shear	39.87	Kips	
	Exterior Girder (Interior Bay)			
	Length	25.00	ft	
	Shear	38.85	Kips	
	Exterior Girder To Column			
	Trib Width	4.28	ft	
	Length	49.32	ft	
	x	0	ft	
	Column R:	**34.56**	Kips	
	First Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	49.32	ft	
	x	0	ft	
	Column R:	**36.09**	Kips	
	Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	49.32	ft	
	x	0	ft	
	Column R:	**36.09**	Kips	

R	**Loads (unfactored)**			
	Dead	25	PSF	
	Live	20	PSF	
	Curtain Wall	0.215	KLF	Glass
		0.55	KLF	Pre-Cast
	Load Combinations (w/o Curtain Wall)			
	1.4D	0.035	KSF	
	1.2D+0.5L	0.040	KSF	
	Intermediate Beam (Exterior Bay)			
	Trib Width	8.56	ft	
	Length	51.42	ft	
	Shear	8.80	Kips	
	Intermediate Beam (Interior Bay)			
	Trib Width	8.34	ft	
	Length	51.42	ft	
	Shear	8.58	Kips	
	Exterior Girder (Exterior Bay)			
	Length	25.67	ft	
	Shear	12.11	Kips	
	Exterior Girder (Interior Bay)			
	Length	25.00	ft	
	Shear	11.80	Kips	
	Exterior Girder To Column			
	Trib Width	4.28	ft	
	Length	51.42	ft	
	x	0	ft	
	Column R:	**21.37**	Kips	
	First Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	51.42	ft	
	x	0	ft	
	Column R:	**8.69**	Kips	
	Interior Girder to Column			
	Trib Width	8.45	ft	
	Length	51.42	ft	
	x	0	ft	
	Column R:	**8.69**	Kips	

Sloped Column Verical Reactions (Kips)			
Floor	Exterior	First Interior	Interior
2	30.12	31.37	31.31
3	29.30	30.60	30.60
4	30.61	31.97	31.97
5	31.95	33.37	33.37
6	33.24	34.72	34.72
7	34.56	36.09	36.09
Roof	21.37	8.69	8.69

Floor Shear To Resist Self Weight Overturning				
Sloped Columns:	**9**			
Floor	**P (Kips)**	**x (ft)**	**H (ft)**	**V (Kips)**
2	31.37	2.77	15	52.13
3	30.60	4.65	28.33	45.21
4	31.97	6.52	41.67	45.02
5	33.37	8.4	55	45.87
6	34.72	10.27	68.33	46.96
7	36.09	12.15	81.67	48.32
Roof	8.69	14.25	96.67	11.53

Frame Calculation Spread Sheets

Rigidity and Distribution Percentages

E - W Frames			
Frame	Δ	Rigidity (1/Δ)	% Flr. Shear (R/ΣR)
2	0.012	83.33	33.18%
3	0.013	76.92	30.63%
5	0.011	90.91	36.19%
	Tot.	251.17	100.00%
N - S Frames			
Frame	Δ	Rigidity (1/Δ)	% Flr. Shear (R/ΣR)
1	0.009	111.11	57.14%
4	0.012	83.33	42.86%
	Tot.	194.44	100.00%

Story Shears at each frame

Wind - Story Shear E - W							
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Shear (K)
2	23.31		7.73	7.14		8.44	23.31
3	24.42		8.10	7.48		8.84	24.42
4	26.13		8.67	8.00		9.46	26.13
5	27.41		9.09	8.39		9.92	27.41
6	28.61		9.49	8.76		10.35	28.61
7	31.42		10.42	9.62		11.37	31.42
Roof	16.98		5.63	5.20		6.15	16.98
Penthouse Roof	7.58		2.52	2.32		2.74	7.58
N - S							
2	49.20	28.11			21.09		49.20
3	51.55	29.46			22.09		51.55
4	55.16	31.52			23.64		55.16
5	57.86	33.06			24.80		57.86
6	60.38	34.50			25.88		60.38
7	66.32	37.90			28.42		66.32
Roof	35.84	20.48			15.36		35.84
Penthouse Roof	35.71	20.41			15.30		35.71

Seismic - Story Shear							
E - W							
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Shear (K)
2	28.72		9.53	8.80		10.40	28.72
3	57.04		18.92	17.47		20.65	57.04
4	79.84		26.49	24.45		28.90	79.84
5	105.70		35.07	32.37		38.26	105.70
6	129.56		42.99	39.68		46.89	129.56
7	161.50		53.59	49.46		58.46	161.50
Roof	190.68		63.27	58.40		69.02	190.68
Penthouse Roof	17.13		5.68	5.25		6.20	17.13
N - S							
2	28.72	16.41			12.31		28.72
3	57.04	32.59			24.45		57.04
4	79.84	45.63			34.22		79.84
5	105.70	60.40			45.30		105.70
6	129.56	74.03			55.53		129.56
7	161.50	92.29			69.22		161.50
Roof	190.68	108.96			81.72		190.68
Penthouse Roof	17.13	9.79			7.34		17.13

N - S Floor Shear To Resist Self Weight Overturning Moment Due To Sloped Columns							
Story	Shear (K)	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5	Shear (K)
2	52.13	29.79			22.34		52.13
3	45.21	25.83			19.37		45.21
4	45.02	25.73			19.30		45.02
5	45.87	26.21			19.66		45.87
6	46.96	26.83			20.13		46.96
7	48.32	27.61			20.71		48.32
Roof	11.53	6.59			4.94		11.53

Worst Case Lateral Load - Factored (Kips) E - W					
Story	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
2		9.53	8.80		14.06
3		18.92	17.47		27.58
4		26.49	24.45		37.77
5		35.07	32.37		48.36
6		42.99	39.68		62.11
7		53.59	49.46		76.44
Roof		63.27	58.40		119.07
Penthouse Roof		5.68	5.25		6.20
N - S					
2	46.20			34.72	
3	58.43			45.30	
4	71.35			53.91	
5	86.61			66.25	
6	100.87			79.22	
7	119.90			94.52	
Roof	115.55			91.06	
Penthouse Roof	9.79			7.34	

Load Combinations

Wind Load E - W (Kips)		
Story	Unfactored	Factored (1.6*W)
2	23.31	37.29
3	24.42	39.07
4	26.13	41.81
5	27.41	43.86
6	28.61	45.77
7	31.42	50.27
Roof	16.98	27.17
Penthouse Roof	7.58	12.13

Wind Load N - S (Kips)		
Story	Unfactored	Factored (1.6*W)
2	49.20	78.72
3	51.55	82.48
4	55.16	88.26
5	57.86	92.57
6	60.38	96.61
7	66.32	106.11
Roof	35.84	57.35
Penthouse Roof	35.71	57.14

Seismic E - W & N - S (Kips)		
Story	Unfactored	Factored (1.0*E)
2	28.72	28.72
3	57.04	57.04
4	79.84	79.84
5	105.70	105.70
6	129.56	129.56
7	161.50	161.50
Roof	190.68	190.68
Penthouse Roof	17.13	17.13

Maximum E - W Story Shear (Kips)
37.29
57.04
79.84
105.70
129.56
161.50
190.68
17.13

Maximum N - S Story Shear (Kips)
78.72
82.48
88.26
105.70
129.56
161.50
190.68
57.14

Controlling Force	
##.##	Seismic
##.##	Wind

Center of Rigidity

Center of Rigidity - All Floors						
	Distance from Reference		Relative Rigidity			
Element	x	y	x	y	RxY	RyX
Frame 1	50.670	54.670		0.009	0.000	0.456
Frame 2	113.170	37.170	0.012		0.446	0.000
Frame 3	188.170	37.170	0.013		0.483	0.000
Frame 4	200.670	54.670		0.012	0.000	2.408
Frame 5	63.210	72.170	0.011		0.794	0.000
		total:	0.036	0.021	1.723	2.864
			x_r=	136.384	ft	
			y_r=	47.864	ft	

Center of Mass

Floor 2 - Center of Mass							
				Distance from Reference			
Floor Area	area (sf)	unit weight (Ksf)	Weight (Kips)	x	y	Wx	Wy
1	8703.990	0.061	530.943	125.920	94.250	66856.392	50041.415
2	12150.000	0.061	741.150	128.170	54.670	94993.196	40518.671
3	4219.000	0.061	257.359	191.670	15.083	49328.000	3881.832
4	6309.500	0.061	384.880	61.771	9.000	23774.327	3463.916
5	304.600	0.061	18.581	103.920	-17.170	1930.896	-319.029
6			0.000			0.000	0.000
Façade	Length (ft)	Unit Weight (Klf)	Weight (Kips)	x	y	Wx	Wy
1	256.000	0.215	55.040	125.313	111.333	6897.200	6127.787
2	34.167	0.550	18.792	253.333	94.250	4760.596	1771.132
3	9.833	0.550	5.408	258.250	77.167	1396.702	417.345
4	45.000	0.550	24.750	263.167	54.670	6513.383	1353.083
5	9.833	0.550	5.408	258.250	32.167	1396.702	173.970
6	34.167	0.550	18.792	253.333	15.083	4760.596	283.444
7	258.833	0.550	142.358	125.313	-2.000	17839.350	-284.717
8	34.000	0.550	18.700	-4.500	13.250	-84.150	247.775
9	3.583	0.550	1.971	-5.000	32.167	-9.854	63.396
10	45.000	0.550	24.750	-6.833	54.670	-169.125	1353.083
11	4.167	0.550	2.292	-4.670	77.167	-10.703	176.855
12	34.167	0.215	7.346	-2.670	94.250	-19.614	692.352
		total:	2258.520			280153.893	109962.306
				x_{mass}=	124.043	ft	
				y_{mass}=	48.688	ft	

Floor 3 - Center of Mass							
				Distance from Reference			
Floor Area	area (sf)	unit weight (Ksf)	Weight (Kips)	x	y	Wx	Wy
1	912.096	0.061	55.638	100.667	113.500	5600.878	6314.897
2	9469.230	0.061	577.623	99.560	99.420	57508.149	57427.282
3	1618.660	0.061	98.738	229.650	94.250	22675.241	9306.081
4	12149.990	0.061	741.149	128.170	54.670	94993.117	40518.637
5	8723.880	0.061	532.157	125.670	15.080	66876.130	8024.923
6			0.00			0.00	0.00
Façade	Length (ft)	Unit Weight (Klf)	Weight (Kips)	x	y	Wx	Wy
1	210.583	0.215	45.275	100.667	115.667	4557.725	5236.857
2	4.333	0.215	0.932	205.958	113.083	191.885	105.356
3	47.375	0.550	26.056	230.354	111.333	6002.166	2900.929
4	34.167	0.550	18.792	253.333	94.250	4760.596	1771.132
5	9.833	0.550	5.408	258.250	77.167	1396.702	417.345
6	45.000	0.550	24.750	263.167	54.670	6513.383	1353.083
7	9.833	0.550	5.408	258.250	32.167	1396.655	173.964
8	34.167	0.550	18.792	253.333	15.083	4760.596	283.437
9	123.479	0.550	67.914	179.000	-2.000	12156.524	-135.827
10	50.000	0.550	27.500	102.833	-17.313	2827.917	-476.094
11	6.000	0.550	3.300	78.000	-17.000	257.400	-56.100
12	84.500	0.550	46.475	35.854	-17.104	1666.322	-794.916
13	46.438	0.550	25.541	-4.771	9.000	-121.850	229.866
14	3.583	0.550	1.971	-5.000	32.167	-9.854	63.396
15	45.000	0.550	24.750	-6.833	54.670	-169.125	1353.083
16	18.729	0.550	10.301	128.979	-8.813	1328.620	-90.778
17	2.167	0.550	1.192	-4.125	77.167	-4.916	91.971
18	38.500	0.215	8.278	-4.670	96.354	-38.656	797.572
		total:	2367.939			295125.603	134816.094
				x_{mass}=	124.634	ft	
				y_{mass}=	56.934	ft	

Floor 4 - Center of Mass							
				Distance from Reference			
Floor Area	**area (sf)**	**unit weight (Ksf)**	**Weight (Kips)**	**x**	**y**	**Wx**	**Wy**
1	1333.65	0.061	81.353	100.667	114.500	8189.500	9314.878
2	29235.320	0.061	1783.355	124.375	54.667	221804.718	97490.047
3	442.500	0.061	26.993	258.250	54.667	6970.813	1475.590
4	2820.450	0.061	172.047	113.313	-13.208	19495.127	-2272.460
5	99.372	0.061	6.062	-5.688	54.667	-34.476	331.372
6			0.00			0.00	0.00
Façade	**Length (ft)**	**Unit Weight (Klf)**	**Weight (Kips)**	**x**	**y**	**Wx**	**Wy**
1	210.583	0.215	45.275	100.667	117.667	4557.725	5327.407
2	6.333	0.215	1.362	205.958	114.083	280.432	155.335
3	47.375	0.550	26.056	230.354	111.333	6002.166	2900.929
4	34.167	0.550	18.792	253.333	94.250	4760.596	1771.132
5	9.833	0.550	5.408	258.250	77.167	1396.702	417.345
6	45.000	0.550	24.750	263.167	54.670	6513.383	1353.083
7	9.833	0.550	5.408	258.250	32.167	1396.655	173.964
8	34.167	0.550	18.792	253.333	15.083	4760.596	283.437
9	123.479	0.550	67.914	191.583	-2.000	13011.103	-135.827
10	18.729	0.550	10.301	128.729	-11.292	1326.045	-116.316
11	80.000	0.550	44.000	87.958	-15.479	3870.167	-681.083
12	8.250	0.550	4.538	48.896	-6.271	221.865	-28.454
13	51.500	0.550	28.325	23.750	-2.167	672.719	-61.380
14	34.167	0.550	18.792	-2.000	15.083	-37.584	283.437
15	4.833	0.550	2.658	-4.417	32.167	-11.741	85.511
16	45.000	0.550	24.750	-6.833	54.667	-169.125	1353.000
17	2.167	0.550	1.192	-5.729	77.167	-6.828	91.971
18	40.500	0.215	8.708	-4.625	97.417	-40.272	848.256
		total:	2426.829			304930.284	120361.175
				x_{mass}=	125.650	**ft**	
				y_{mass}=	49.596	**ft**	

Floor 5 - Center of Mass							
				Distance from Reference			
Floor Area	**area (sf)**	**unit weight (Ksf)**	**Weight (Kips)**	**x**	**y**	**Wx**	**Wy**
1	1754.820	0.061	107.044	100.667	115.500	10775.765	12363.584
2	29235.321	0.061	1783.355	124.354	54.667	221767.573	97490.050
3	442.500	0.061	26.993	258.250	54.667	6970.813	1475.590
4	1702.810	0.061	103.871	138.583	-13.208	14394.846	-1371.968
5	99.372	0.061	6.062	-5.729	54.667	-34.728	331.372
6			0.00			0.00	0.00
Façade	**Length (ft)**	**Unit Weight (Klf)**	**Weight (Kips)**	**x**	**y**	**Wx**	**Wy**
1	210.583	0.215	45.275	100.667	119.667	4557.740	5417.958
2	8.333	0.215	1.792	205.958	115.083	368.994	206.183
3	47.375	0.550	26.056	230.354	111.333	6002.166	2900.929
4	34.167	0.550	18.792	253.333	94.250	4760.596	1771.132
5	9.833	0.550	5.408	258.250	77.167	1396.702	417.345
6	45.000	0.550	24.750	263.167	54.670	6513.383	1353.083
7	9.833	0.550	5.408	258.250	32.167	1396.655	173.964
8	34.167	0.550	18.792	253.333	15.083	4760.596	283.437
9	72.125	0.550	39.669	217.271	-2.000	8618.862	-79.338
10	22.479	0.550	12.364	179.146	-13.167	2214.877	-162.791
11	80.000	0.550	44.000	137.833	-16.667	6064.667	-733.348
12	7.083	0.550	3.896	99.313	-5.625	386.887	-21.913
13	102.000	0.550	56.100	49.000	-2.000	2748.900	-112.200
14	34.167	0.550	18.792	-2.000	15.083	-37.584	283.437
15	4.833	0.550	2.658	-4.417	32.167	-11.741	85.511
16	45.000	0.550	24.750	-6.833	54.667	-169.125	1353.000
17	2.208	0.550	1.215	-5.729	77.167	-6.959	93.726
18	42.396	0.215	9.115	-4.625	98.417	-42.157	897.078
		total:	2386.155			303397.727	124415.822
				x_{mass}=	127.149	**ft**	
				y_{mass}=	52.141	**ft**	

Floor 6 - Center of Mass							
				Distance from Reference			
Floor Area	area (sf)	unit weight (Ksf)	Weight (Kips)	x	y	Wx	Wy
1	9469.230	0.061	577.623	99.560	99.420	57508.149	57427.282
2	1618.660	0.061	98.738	229.650	94.250	22675.241	9306.081
3	12149.990	0.061	741.149	128.170	54.670	94993.117	40518.637
4	8723.880	0.061	532.157	125.670	15.080	66876.130	8024.923
5			0.000			0.000	0.000
6			0.00			0.00	0.00
Façade	Length (ft)	Unit Weight (Klf)	Weight (Kips)	x	y	Wx	Wy
1	210.580	0.215	45.275	100.670	121.670	4557.804	5508.573
2	10.330	0.215	2.221	205.960	116.100	457.427	257.852
3	47.380	0.550	26.059	230.360	111.340	6002.951	2901.409
4	34.170	0.550	18.794	253.340	94.250	4761.145	1771.287
5	45.000	0.550	24.750	258.250	77.170	6391.688	1909.958
6	9.840	0.550	5.412	264.500	54.670	1431.474	295.874
7	34.170	0.550	18.794	258.250	32.170	4853.421	604.587
8	255.340	0.550	140.437	125.670	-2.170	17648.718	-304.748
9	34.170	0.550	18.794	-2.000	15.080	-37.587	283.406
10	4.840	0.550	2.662	-4.420	32.170	-11.766	85.637
11	45.000	0.550	24.750	-6.840	54.670	-169.290	1353.083
12	44.500	0.550	24.475	-4.625	99.420	-113.197	2433.305
13			0.00			0.00	0.00
14			0.00			0.00	0.00
15			0.00			0.00	0.00
16			0.00			0.00	0.00
17			0.00			0.00	0.00
		total:	2302.089			287825.426	132377.144
				x_{mass}=	125.028	ft	
				y_{mass}=	57.503	ft	

Floor 7 - Center of Mass							
				Distance from Reference			
Floor Area	**area (sf)**	**unit weight (Ksf)**	**Weight (Kips)**	**x**	**y**	**Wx**	**Wy**
1	2597.151	0.061	158.426	100.667	117.500	15948.239	18615.080
2	29235.321	0.061	1783.355	124.354	54.667	221767.573	97490.050
3	442.500	0.061	26.993	258.250	54.667	6970.813	1475.590
4	99.372	0.061	6.062	-5.729	54.667	-34.728	331.372
5			0.000			0.000	0.000
6			0.00			0.00	0.00
Façade	**Length (ft)**	**Unit Weight (Klf)**	**Weight (Kips)**	**x**	**y**	**Wx**	**Wy**
1	210.580	0.215	45.275	100.670	123.670	4557.804	5599.122
2	12.333	0.215	2.652	205.960	117.083	546.123	310.457
3	47.380	0.550	26.059	230.360	111.340	6002.951	2901.409
4	34.170	0.550	18.794	253.340	94.250	4761.145	1771.287
5	45.000	0.550	24.750	258.250	77.170	6391.688	1909.958
6	9.840	0.550	5.412	264.500	54.670	1431.474	295.874
7	34.170	0.550	18.794	258.250	32.170	4853.421	604.587
8	255.340	0.550	140.437	125.670	-2.170	17648.718	-304.748
9	34.170	0.550	18.794	-2.000	15.080	-37.587	283.406
10	4.840	0.550	2.662	-4.420	32.170	-11.766	85.637
11	45.000	0.550	24.750	-6.840	54.670	-169.290	1353.083
12	46.396	0.215	9.975	-4.625	102.458	-46.135	1022.033
13			0.00			0.00	0.00
14			0.00			0.00	0.00
15			0.00			0.00	0.00
16			0.00			0.00	0.00
17			0.00			0.00	0.00
		total:	2313.187			290580.442	133744.196
				x_{mass}=	125.619	**ft**	
				y_{mass}=	57.818	**ft**	

Roof - Center of Mass							
				Distance from Reference			
Floor Area	**area (sf)**	**unit weight (Ksf)**	**Weight (Kips)**	**x**	**y**	**Wx**	**Wy**
1	10090.014	0.025	252.250	101.938	101.417	25713.770	25582.390
2	8586.086	0.061	523.751	96.000	56.354	50280.120	29515.565
3	7565.145	0.025	189.129	98.542	16.708	18637.050	3160.024
4	6157.377	0.160	985.180	226.292	54.667	222938.097	53856.524
5	438.752	0.160	70.200	258.250	54.667	18129.233	3837.641
6			0.00			0.00	0.00
Façade	**Length (ft)**	**Unit Weight (Klf)**	**Weight (Kips)**	**x**	**y**	**Wx**	**Wy**
1	210.580	0.215	45.275	100.670	125.670	4557.804	5689.672
2	14.250	0.215	3.064	205.960	118.500	631.010	363.054
3	47.380	0.550	26.059	230.360	111.340	6002.951	2901.409
4	34.170	0.550	18.794	253.340	94.250	4761.145	1771.287
5	45.000	0.550	24.750	258.250	77.170	6391.688	1909.958
6	9.840	0.550	5.412	264.500	54.670	1431.474	295.874
7	34.170	0.550	18.794	258.250	32.170	4853.421	604.587
8	255.340	0.550	140.437	125.670	-2.170	17648.718	-304.748
9	34.170	0.550	18.794	-2.000	15.080	-37.587	283.406
10	4.840	0.550	2.662	-4.420	32.170	-11.766	85.637
11	45.000	0.550	24.750	-6.840	54.670	-169.290	1353.083
12	48.500	0.215	10.428	-4.625	103.500	-48.227	1079.246
13			0.00			0.00	0.00
14			0.00			0.00	0.00
15			0.00			0.00	0.00
16			0.00			0.00	0.00
17			0.00			0.00	0.00
		total:	2359.727			381709.610	131984.608
				x_{mass}=	161.760	**ft**	
				y_{mass}=	55.932	**ft**	

Torsion

Seismic - Torsional Eccentricity		
Floor	**ex (ft)**	**ey (ft)**
2	12.341	-0.823
3	11.750	-9.069
4	10.735	-1.732
5	9.235	-4.276
6	11.356	-9.639
7	10.765	-9.954
Roof	-25.376	-8.068

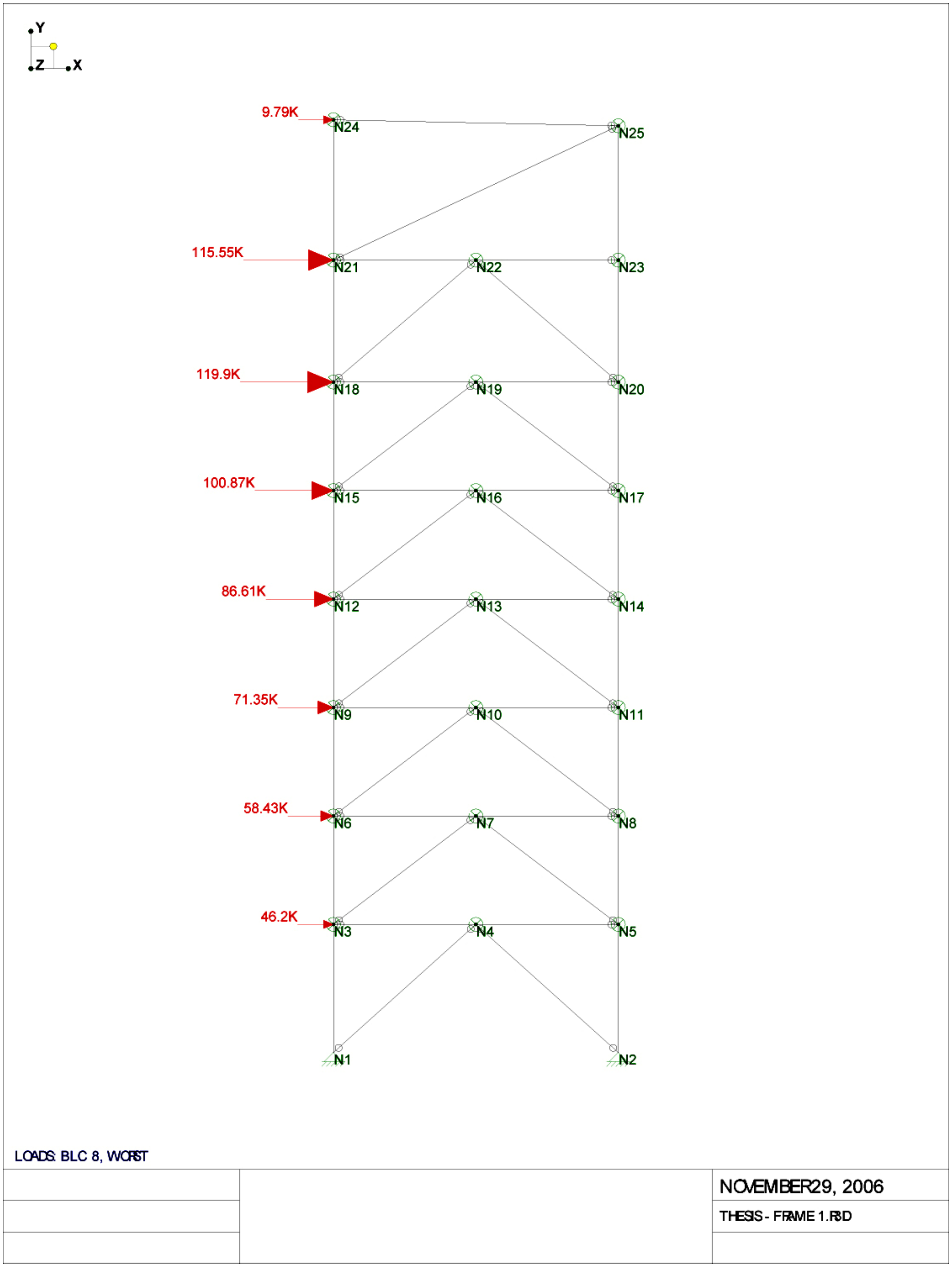
Seismic Torsional Moment					
	Mt (ft-k)				
Floor	Frame 1	Frame 2	Frame 3	Frame 4	Frame 5
2	13.513	117.606	108.559	10.135	128.297
3	295.607	222.372	205.266	221.706	242.587
4	79.006	284.372	262.497	59.254	310.224
5	258.287	323.875	298.961	193.715	353.318
6	713.584	488.163	450.612	535.188	532.542
7	918.613	576.854	532.481	688.960	629.296
Roof	879.073	1605.430	1481.935	659.305	1751.378

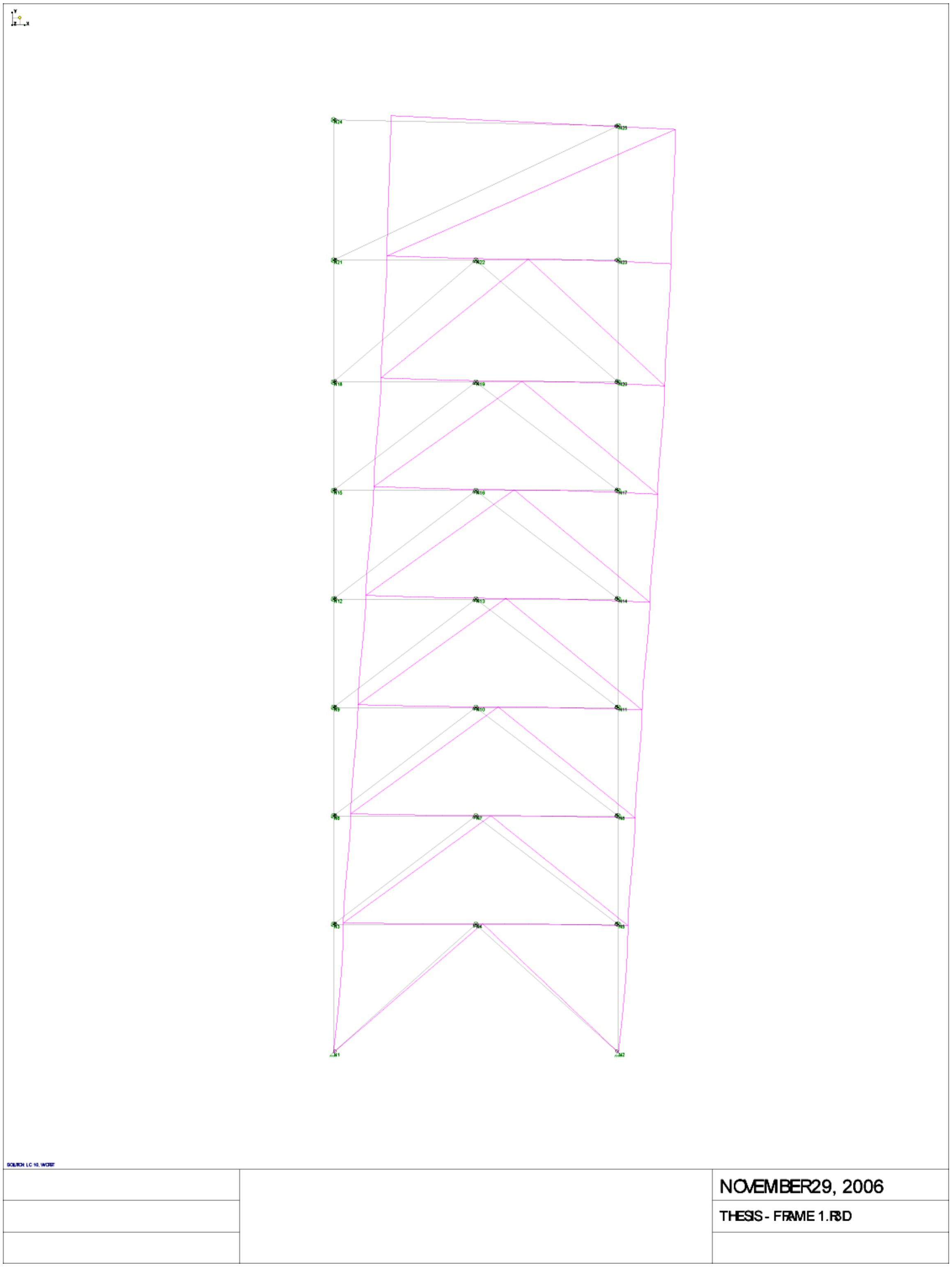
Calculation of Torsional Constant J for Each Direction			
Element	**d**	**k**	**kd^2**
Frame 1	-85.708	0.009	66.113
Frame 4	64.292	0.012	49.601
		J=	**115.714**
Frame 2	-10.688	0.012	1.371
Frame 3	-10.688	0.013	1.485
Frame 5	24.313	0.011	6.502
		J=	**9.358**

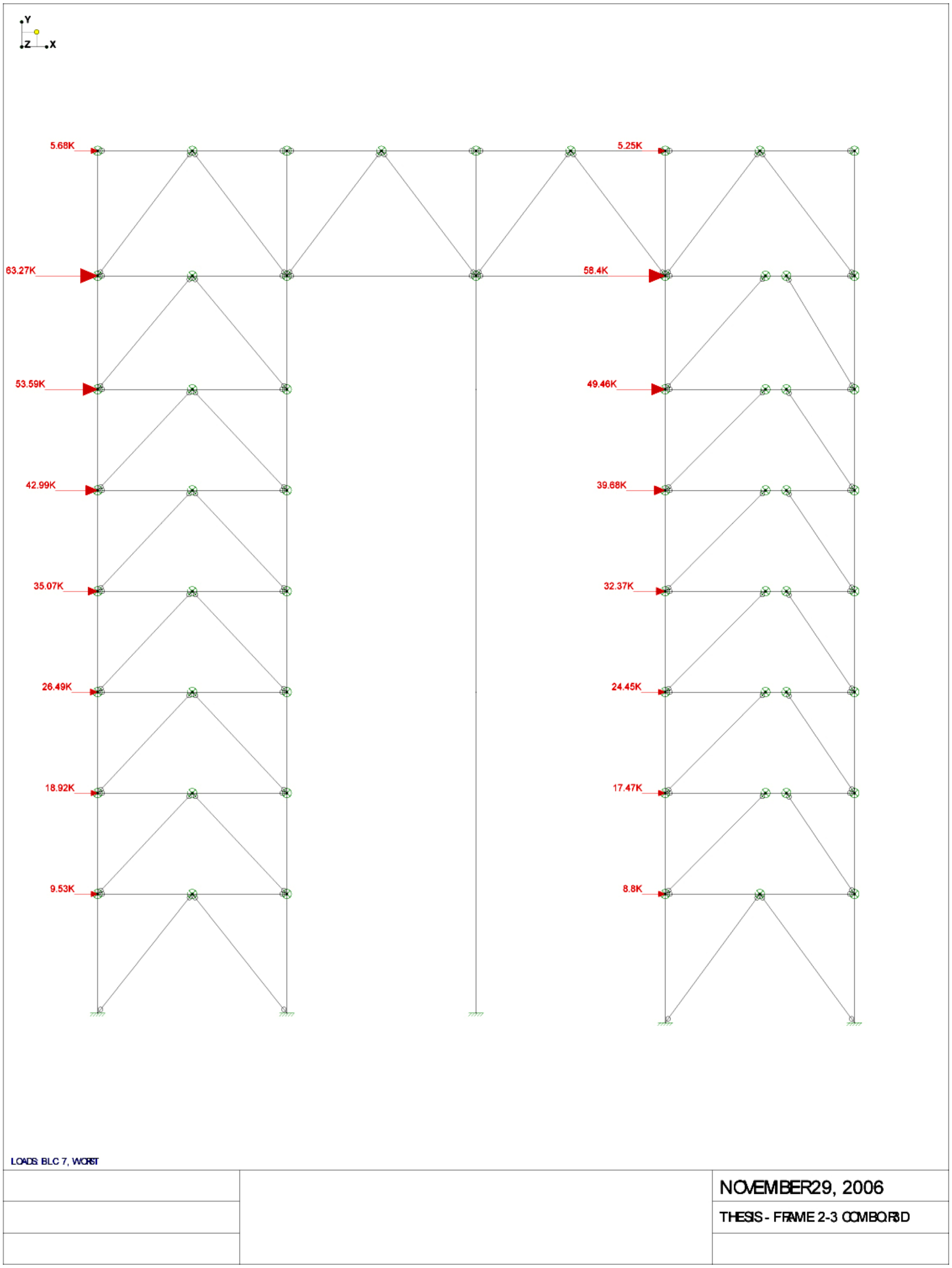
KD/J	
frame 1	-0.012
frame 2	-0.014
frame 3	-0.015
frame 4	0.007
frame 5	0.029

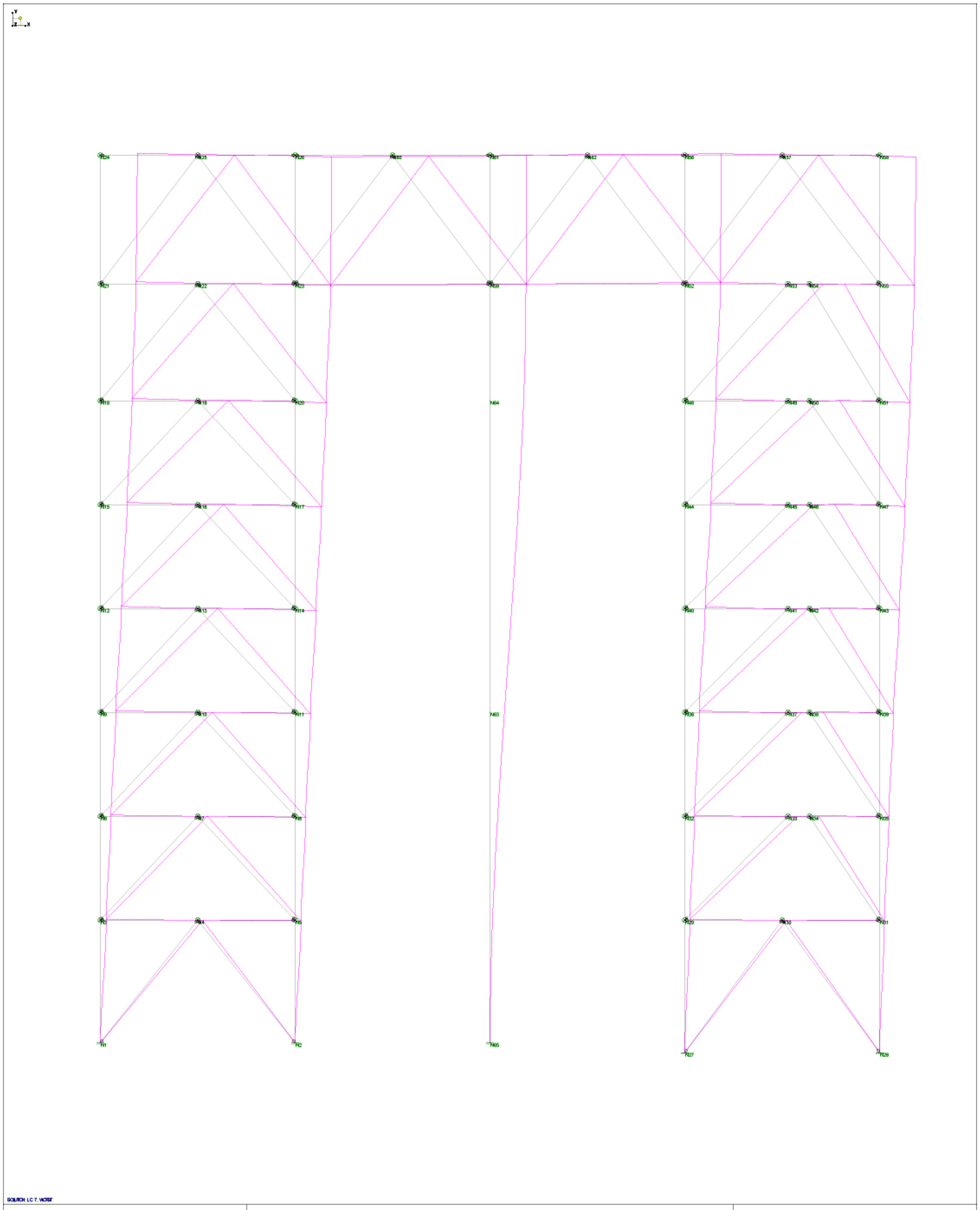
Seismic Torsional Force		
Element	**floor**	**$F_{torsion}$**
Frame 1	2	-0.158
	3	-3.449
	4	-0.922
	5	-3.014
	6	-8.326
	7	-10.718
	Roof	-10.257
Frame 2	2	-1.612
	3	-3.048
	4	-3.897
	5	-4.439
	6	-6.690
	7	-7.906
	Roof	-22.003
Frame 3	2	-1.612
	3	-3.048
	4	-3.897
	5	-4.439
	6	-6.690
	7	-7.906
	Roof	-22.003
Frame 4	2	0.068
	3	1.478
	4	0.395
	5	1.292
	6	3.568
	7	4.593
	Roof	4.396
Frame 5	2	3.667
	3	6.933
	4	8.866
	5	10.098
	6	15.220
	7	17.985
	Roof	50.054

RISA 3D – Frame Models with Worst Case Loading (Deflection Magnified by a factor of 40)

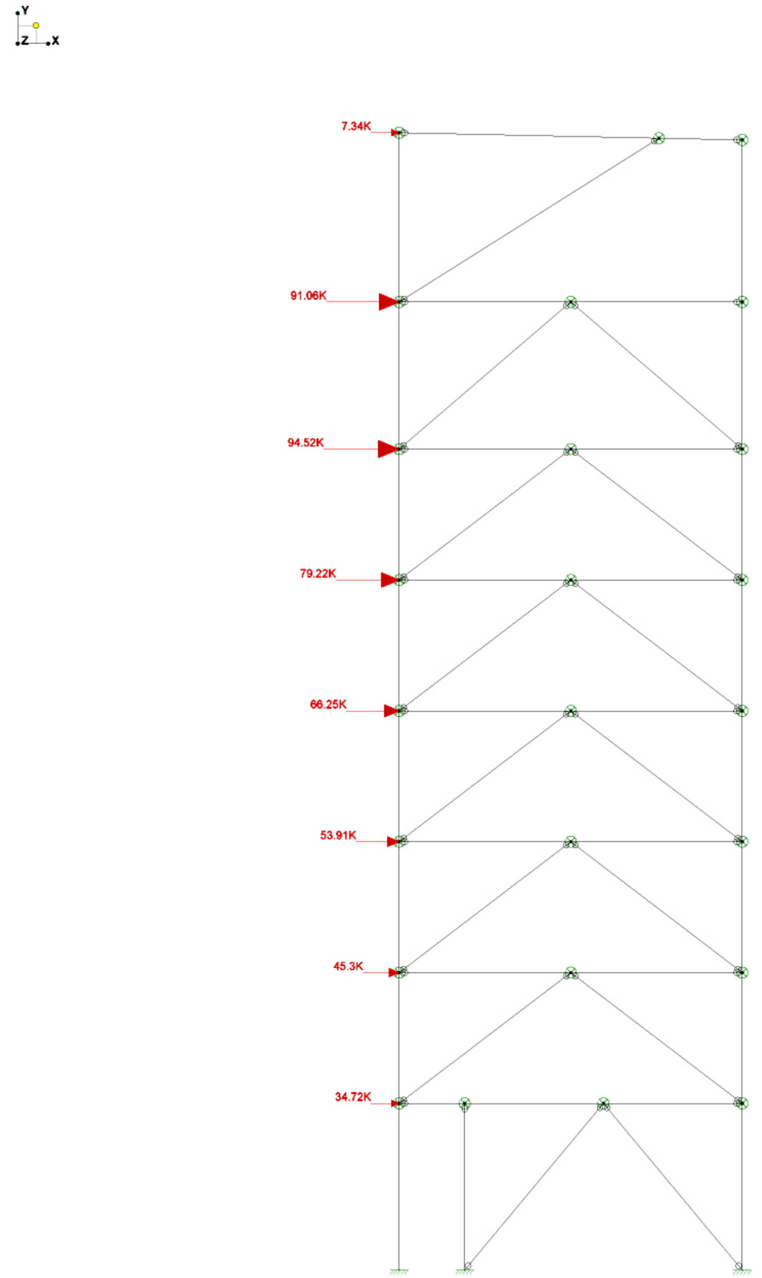



NOVEMBER 29, 2006
THESIS - FRAME 1.R3D


5.68K
5.25K
63.27K
58.4K
53.59K
49.46K
42.99K
39.68K
35.07K
32.37K
26.49K
24.45K
18.92K
17.47K
9.53K
8.8K
LOADS: BLC 7, WORST
NOVEMBER 29, 2006
THESIS - FRAME 2-3 COMBO.R3D

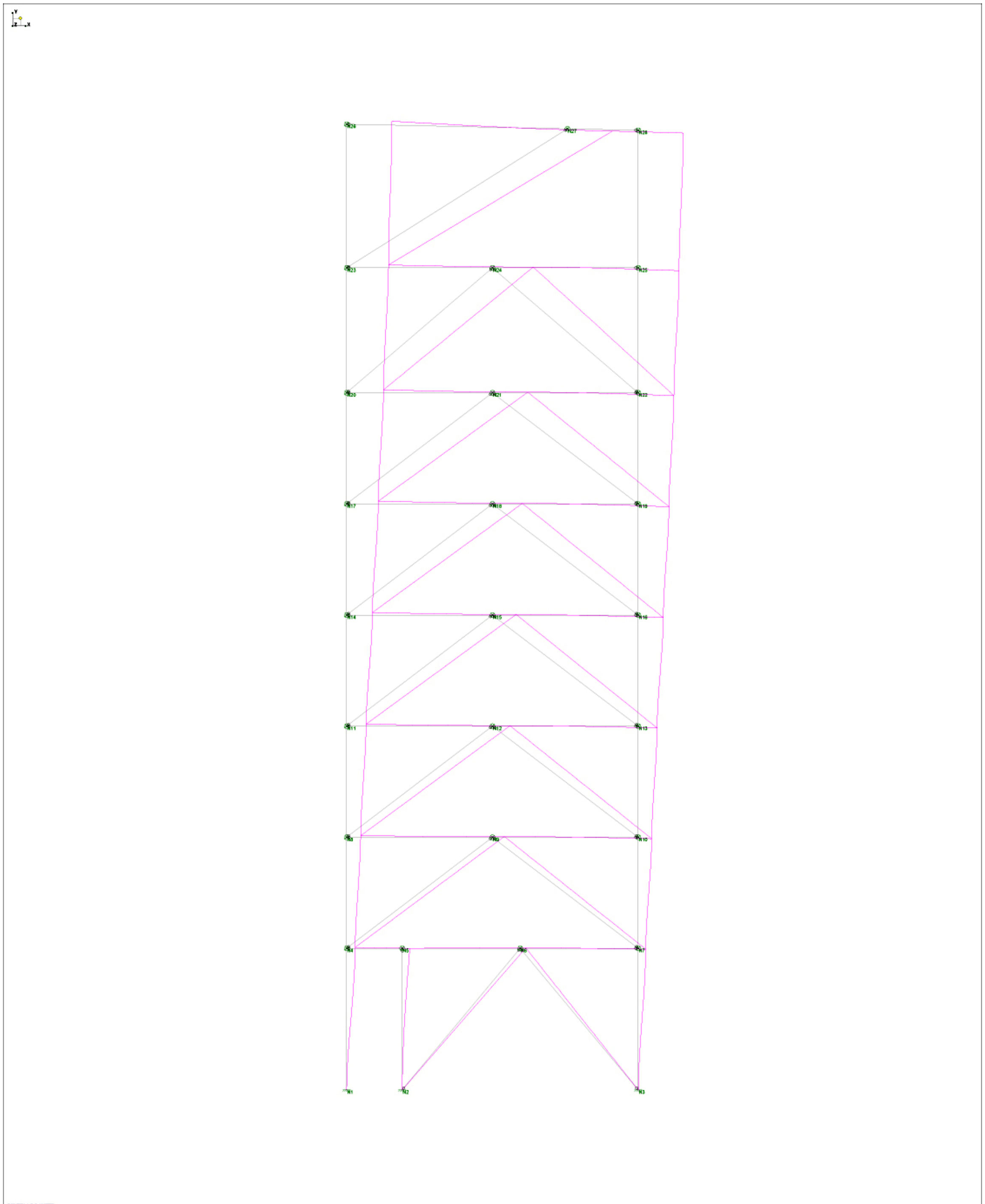


		NOVEMBER 29, 2006
		THESIS - FRAME 2-3 COMBO.R3D

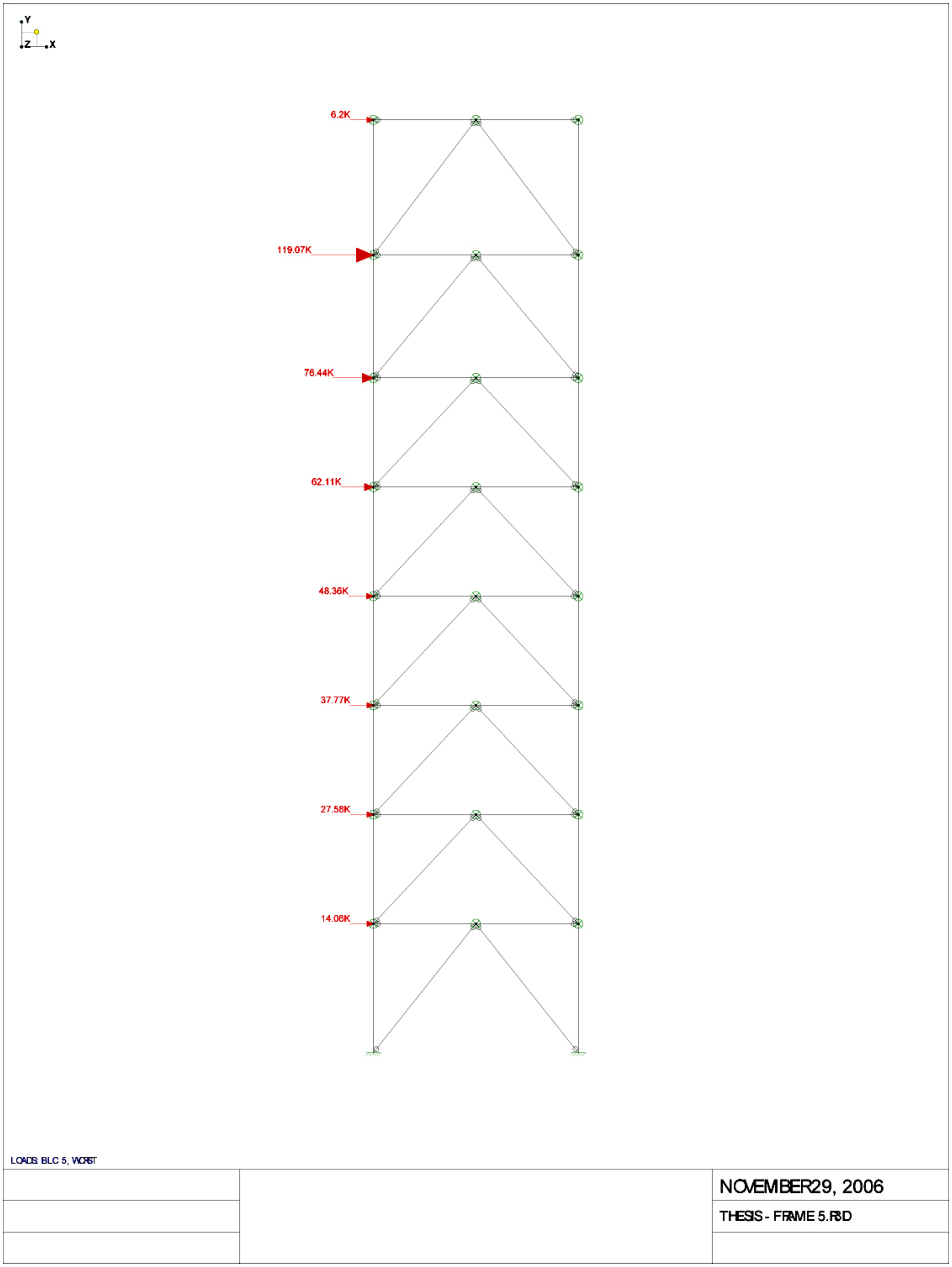


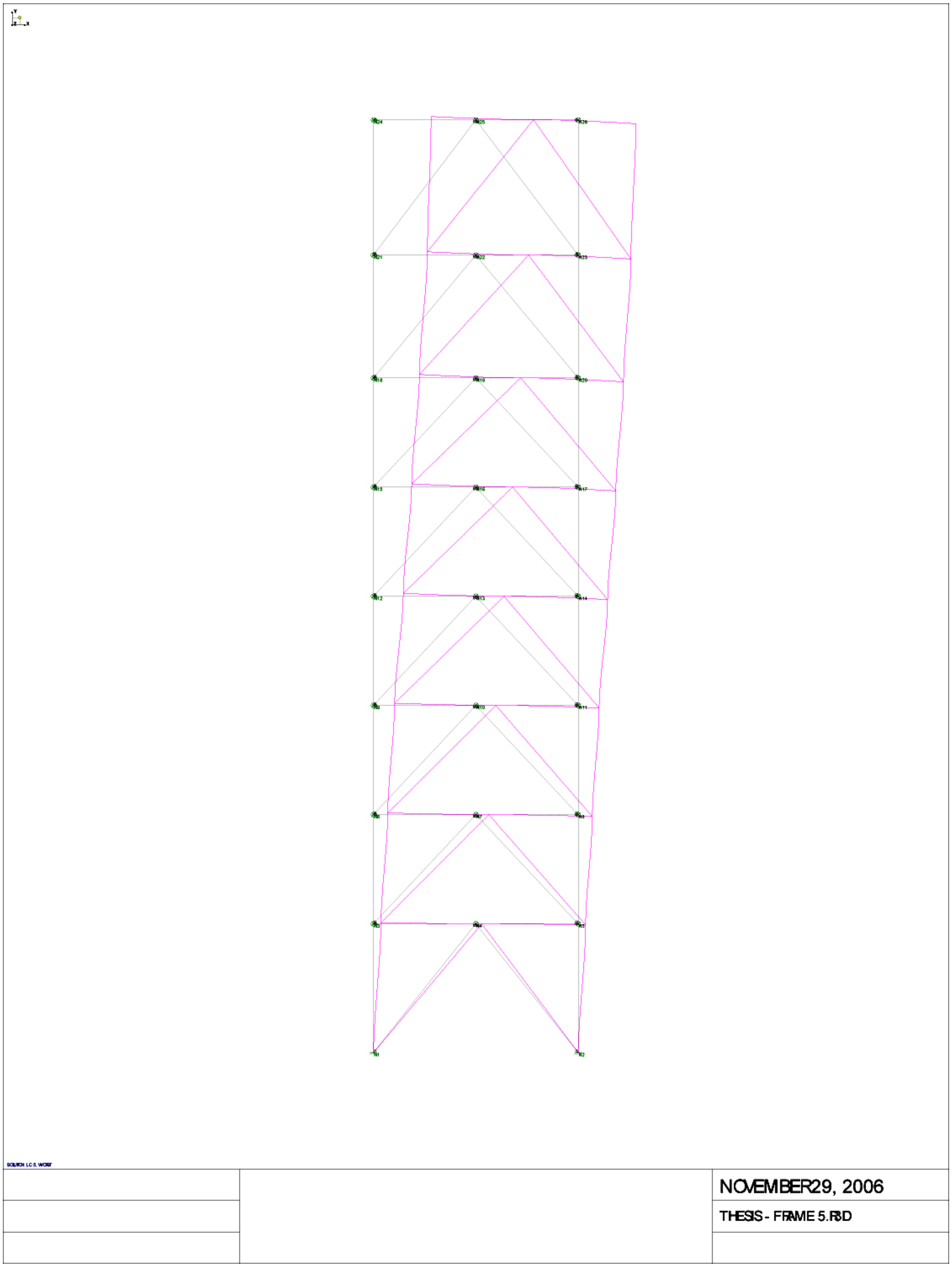
LOADS: BLC 5, WORST

		NOVEMBER 29, 2006
		THESIS - FRAME 4.R3D



		NOVEMBER 29, 2006
		THESIS - FRAME 4.R3D


Y
Z
X
6.2K
119.07K
76.44K
62.11K
48.36K
37.77K
27.58K
14.06K
LOADS: BLC 5, WORST
NOVEMBER 29, 2006
THESIS - FRAME 5.R3D


NOVEMBER 29, 2006
THESIS - FRAME 5.R3D

Overturning Moment

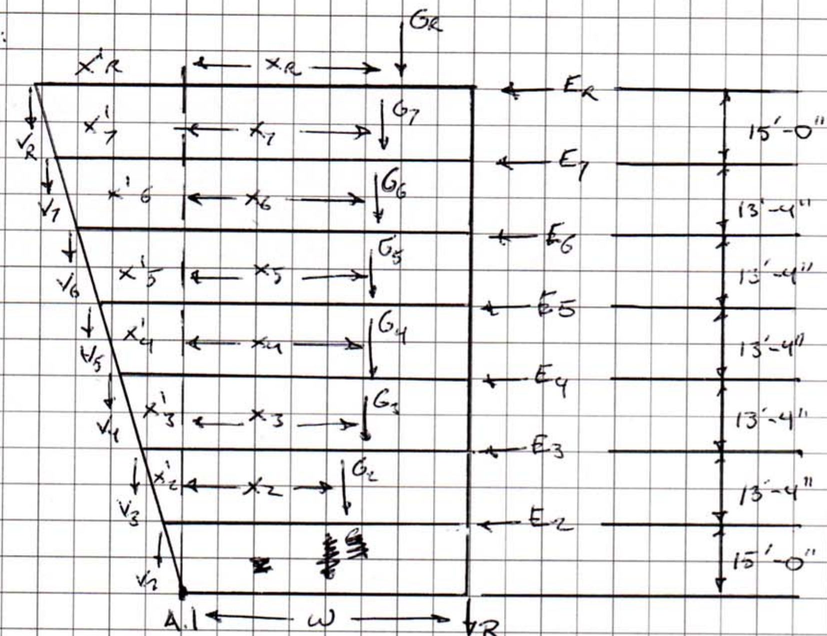
STRUCTURAL DESIGN GROUP, LIMITED
CONSULTING STRUCTURAL ENGINEERS
Gaithersburg, Maryland • Miami, Florida

JOB THESIS - TECH 3
SHEET NO. ______ OF ______
CALCULATED BY DNB III DATE ______
SCALE ______

GIVEN: CENTER OF MASS EACH FLOOR
FLOOR PLANS
LOADING

FIND: OVERTURNING MOMENT @ COLUMN LINE A.1

SOLUTION:



N-S – CRITICAL DIRECTION

LOAD COMBINATION: $0.9D + 1.0E$

WILL USE EXCEL SPREAD SHEET TO SOLVE

$$(+)\ \Sigma M_{A.1} = V_2 x'_2 + V_3 x'_3 + V_4 x'_4 + V_5 x'_5 + V_6 x'_6 + V_7 x'_7 + V_R x'_R$$

$$+$$

$$15 E_2 + (15 + 13.33) E_3 + (15 + 2(13.33)) E_4 + (15 + 3(13.33)) E_5$$

$$+$$

$$(15 + 4(13.33)) E_6 + (15 + 5(13.33)) E_7 + (2(15) + 5(13.33)) E_R$$

$$-$$

$$G_R x_R + G_7 x_7 + G_6 x_6 + G_5 x_5 + G_4 x_4 + G_3 x_3 + G_2 x_2$$

$$-$$

RW R IS ONLY UNKNOWN

120 NE 27th Street • Suite 500 • Miami, FL 33137 • Tel: 305.571.9400 • Fax: 305.572.9161
12 South Summit Avenue • Suite 110 • Gaithersburg, MD 20877 • Tel: 301.987.9234 • Fax: 301.987.9237

		G	Factor	Mr,dead (ft-K)
x2	63.42	1934.05	0.9	110386.12
x3	55.17	1995.63	0.9	99089.18
x4	62.50	1964.90	0.9	110525.37
x5	59.96	1991.25	0.9	107452.69
x6	54.60	1949.78	0.9	95819.48
x7	54.27	1979.07	0.9	96665.17
xr	56.17	2638.54	0.9	133386.11
			total:	**753324.12**

		P	Ma,dead (ft-K)
x'2	2.77	203.58	563.92
x'3	4.65	192.99	897.41
x'4	6.52	201.62	1314.57
x'5	8.40	210.44	1767.65
x'6	10.27	218.93	2248.37
x'7	12.15	227.60	2765.37
x'r	14.25	68.43	975.18
		total:	**10532.47**

		E	Ma, seismic (ft-K)
h2	15.00	28.72	430.83
h3	28.33	57.04	1615.91
h4	41.66	79.84	3326.30
h5	54.99	105.70	5812.47
h6	68.32	129.56	8851.49
h7	81.65	161.50	13186.88
hr	96.65	190.68	18429.57
		total:	**51653.44**

W	110

R	-6283.07	Kips
If negative, compression at footing opposite A.1		