Overlook Towers

Anthony Perrotta

Technical Assignment 3

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Advisor: Dr. Ali Memari

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Executive Summary

Overlook Towers is located south of Washington Dulles International Airport in Herndon, VA. The complex will have two nine story office buildings and two five story parking decks. This report will focus on one office building. Long interior spans are used to reduce the number of columns and make the office space more versatile for the tenants. The exterior walls are made of architectural precast concrete panels. Structural steel and a lightweight composite concrete deck make up the structural system. This report will focus on the steel braced frame lateral supporting system, taking into account wind and seismic forces.

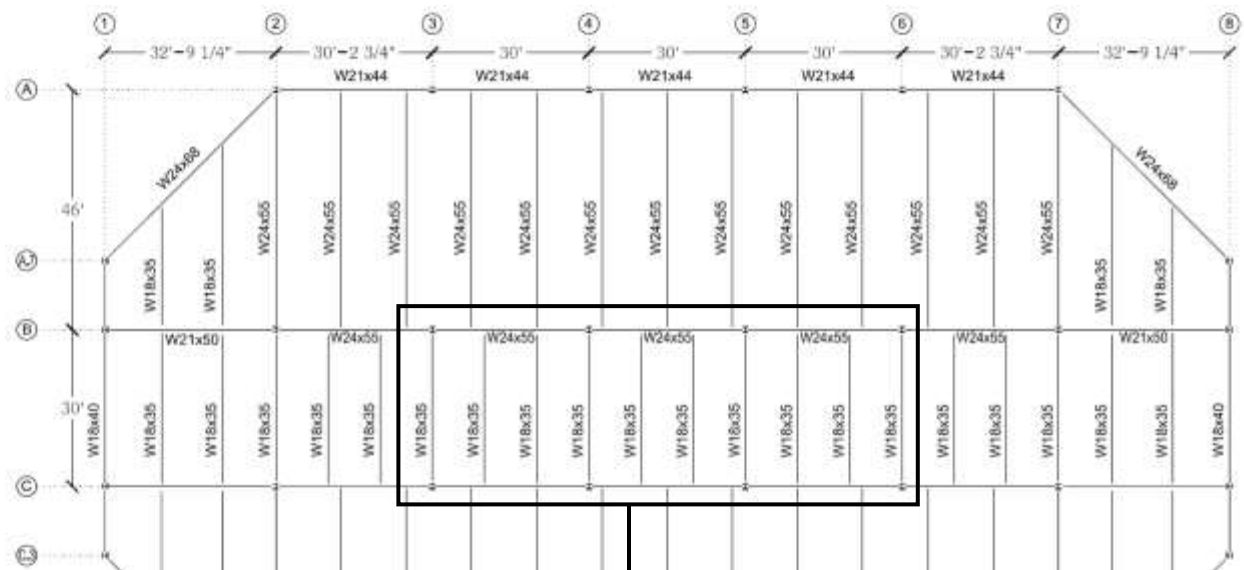
Wind and seismic forces will be calculated using ASCE7-05. The building has a height of 127' and an overall footprint of 127' by 221'. Areas of interest include overall strength of the system, total drift and story drift, overturning moments and what impact the lateral forces have on the foundation. Spot checks of the current lateral bracing system are also included.

Seismic loads were determined to have a greater effect on the design on the lateral system than the wind forces. Seismic base sheer was found to be 617 kips. Building weight alone was found to be sufficient to resist both the wind and seismic overturning moments. No special considerations will be need in the design of the foundation. Checks performed for member strengths and deflections were found to be adequate according to ASCE7-05. Tabulated values can be found in the appendices.

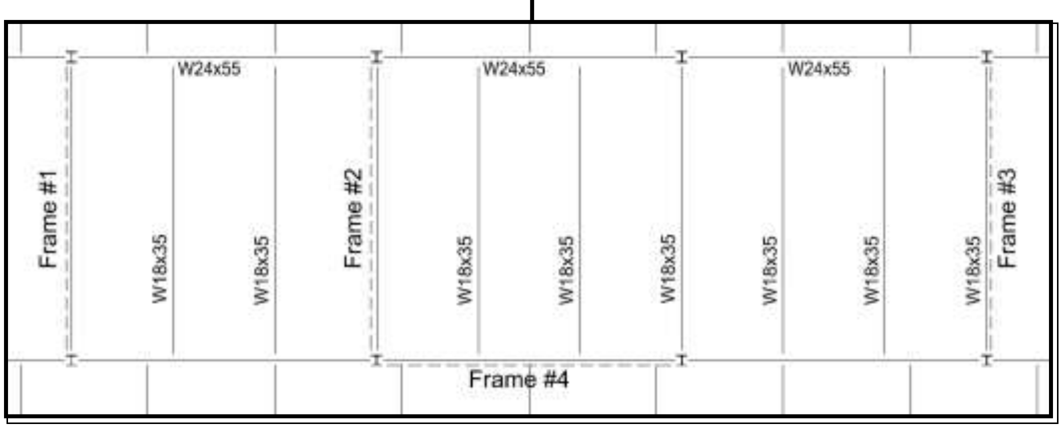
Existing Conditions

Building Summary

Overlook Towers is a nine story, 260,000 square foot steel office building. The floor plan is open, which requires a 46' beam span. This allows for a more versatile use of the office space and the use of moveable partitions. The overall dimensions are 219'-2" by 125'-2". The structural system is a 6 ¼" lightweight composite deck and beam system with A992 W shaped members. Architectural precast panels and large windows make up the building envelope. At a height of nine stories plus the mechanical penthouse, Overlook Towers stands approximately 140'. The main lateral bracing system is placed at various spots within the center of the building. The elevators and stairways are located inside of this central core. Each braced frame is noted on the framing plan below. Four different lateral braced frames are incorporated into the design, which are described in more detail on the following page.



Partial Framing Plan (typ.)



Lateral Bracing

Existing Lateral System

Below are the layouts of the four braced frames in Overlook Towers. Each floor is spaced at 13'-6" and 15'-8" to the second floor. The controlling factor of wind and seismic forces will be in the north-south direction, thus three of the four frames are oriented in this direction. The columns and beams are W shapes with HSS for the diagonal bracing. Lateral forces are distributed through the frame via 5/8" gusset plates and the steel tubing. Forces are then transferred to the ground through concrete spread footings. Footing sizes range from 5'-6" square to 13'-6" square at a depth of two through six feet. A 3'-6" grade beam also runs along the perimeter of the building.

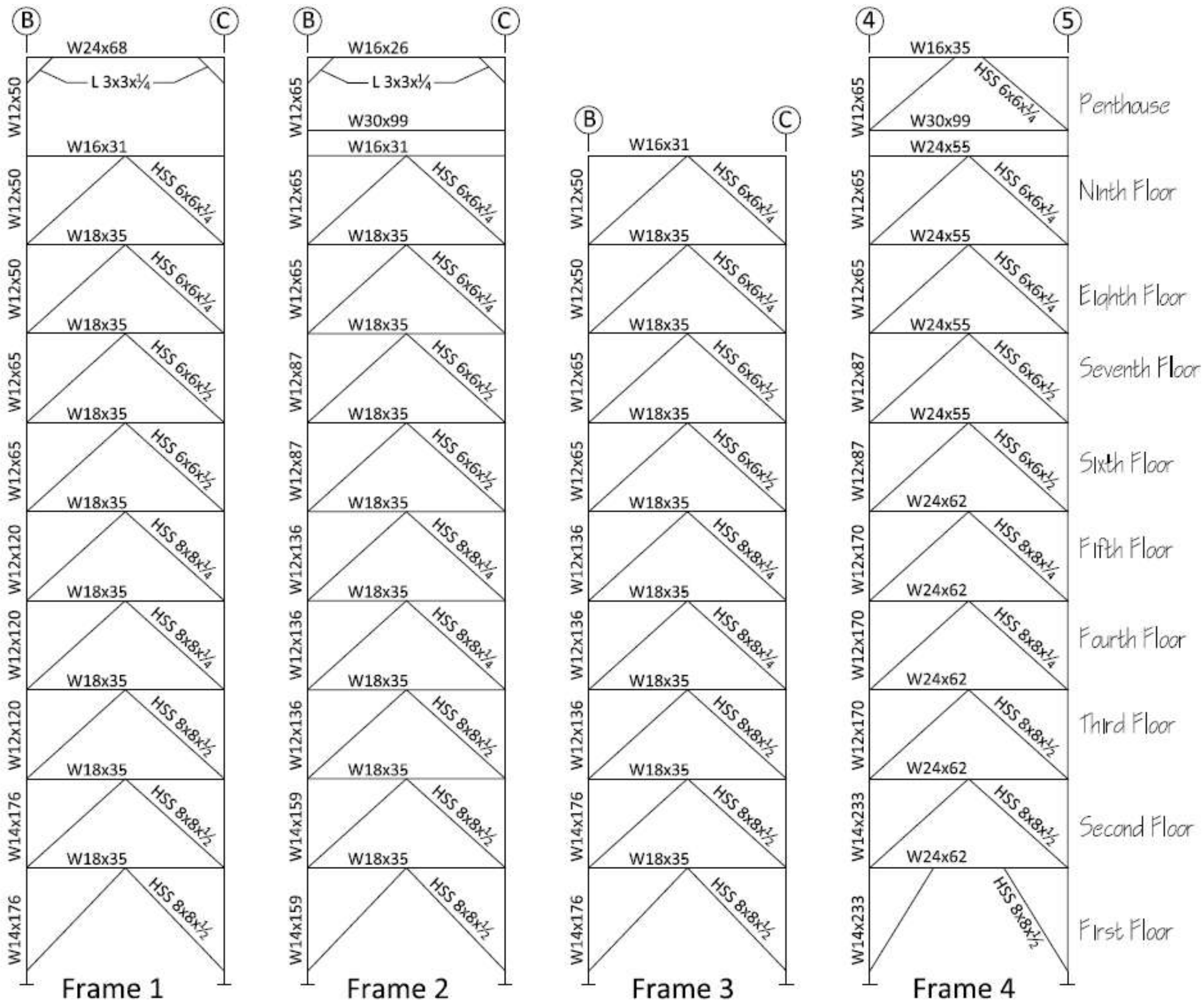


Figure 1 Lateral Bracing

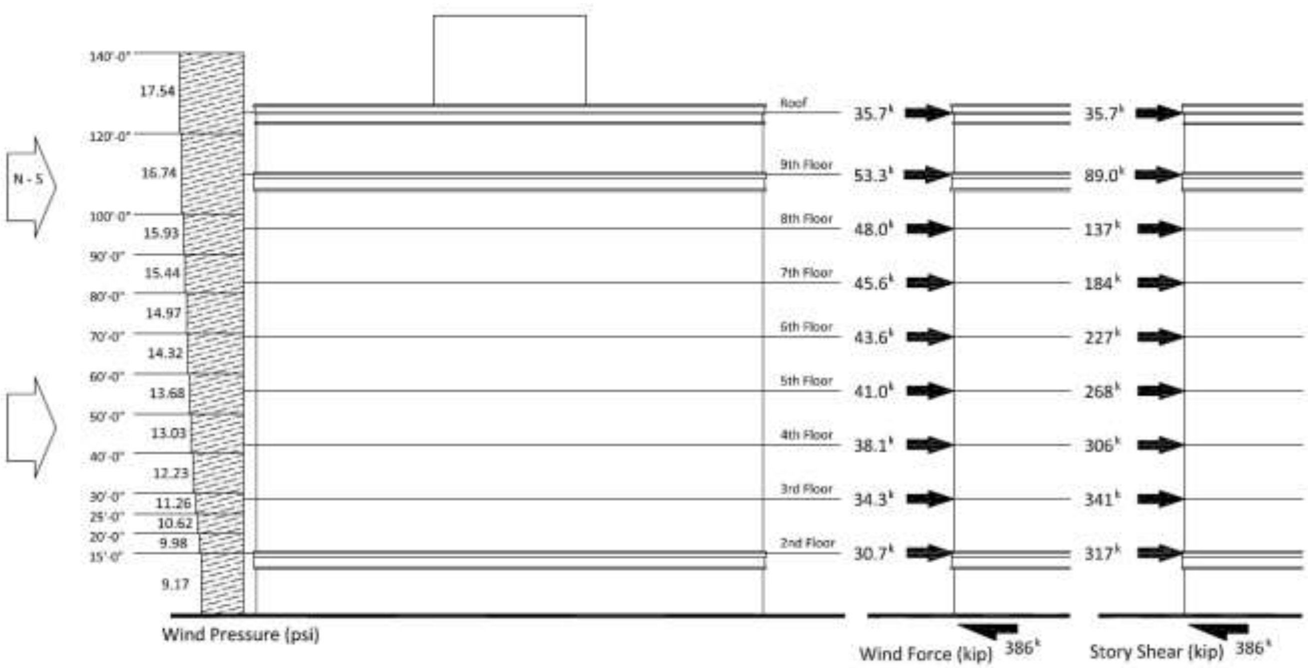
Wind and Seismic Loading

General design information from ASCE7-05:

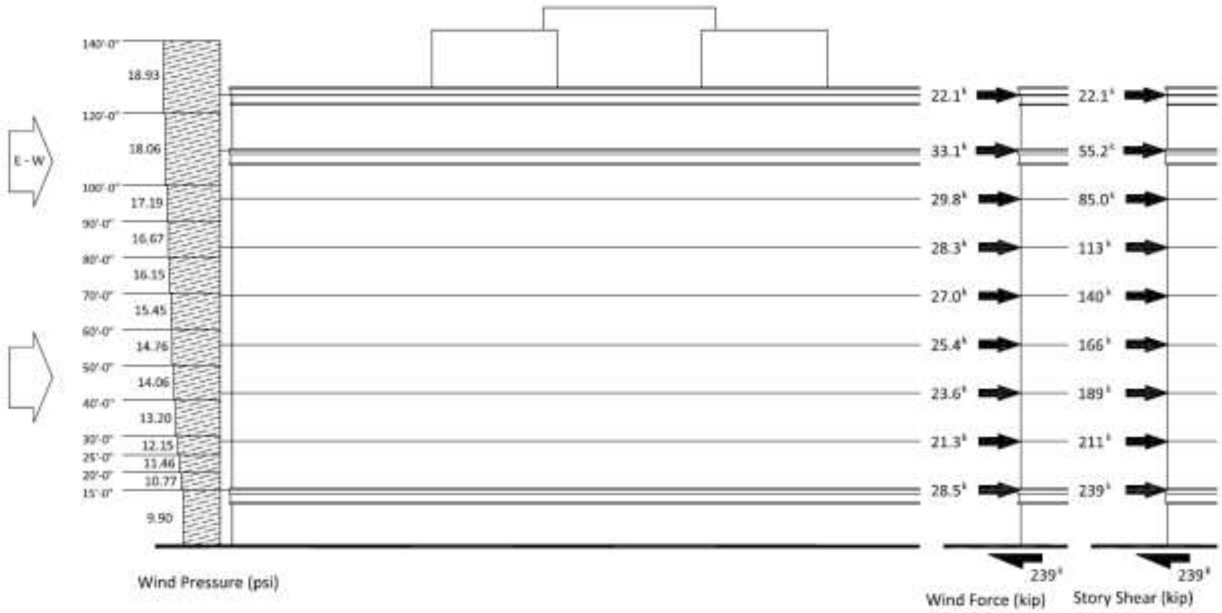
Wind Loading		Seismic Loading	
Wind Speed	90 mph	Seismic Use Group	I
Importance Factor	1.0	Importance Factor	1.0
Wind Exposure	B	Design Category	B
Enclosure Class.	0.18		

Overlook towers is a fully enclosed building. No special considerations were needed for the calculation of wind loading. The longest side of the building will be receiving the largest wind force, so the N-S direction will be the determining factor in the design. Refer to Appendix A for detailed calculations. The two drawings below summarize the calculations into wind pressure, the force at each level and finally the story shear. Total base shear was calculated to be 386^k and a base moment of 27,232'^{k}. The building weight alone is more than enough to counterbalance the overturning force due to the wind.

The allowable story drift according to the building code is $l/400$ or about 0.405" per story. This deflection limit allows for minimal damage of walls, partitions and finishes. Exceeding a drift of 0.405" may result in damage to non-structural components of the building.



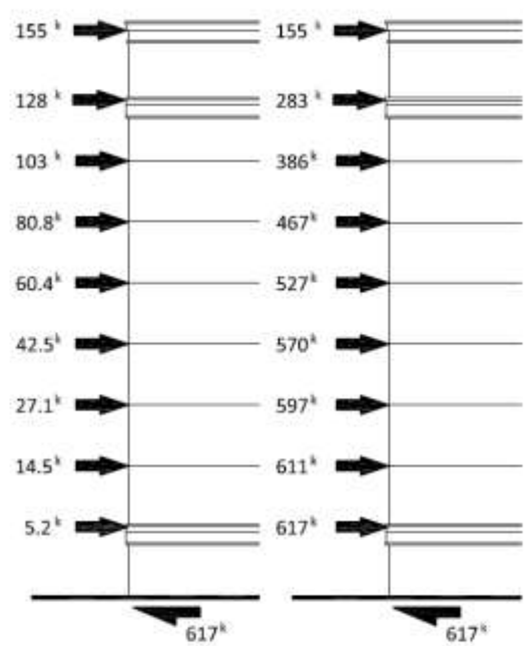
Summary of Wind Forces (N-S Direction)



Summary of Wind Forces (E-W Direction)

The equivalent lateral force procedure will be used for seismic calculations. Detailed calculations of seismic forces can be found in Appendix B. With an occupancy category I, table 12.12-1 of ASCE7-05 states that the allowable story drift is $0.020h_{fx}$ or 3.24". Total seismic base shear is calculated to be 617'^{k}. The controlling direction for seismic force is also in the N-S direction. These values are tabulated in Appendix B with a summary of the forces to the left. The overturning moment was checked and the building weight was found to be adequate to balance these forces. Member checks are performed on the following page. The top two floor of braced frame three will be checked.

Story drift calculations were performed by computer analysis using RAM Advise. The overall drift was found to be XXXX", which is within the maximum allowable drift of 3.24".



Seismic force Summary

Conclusion

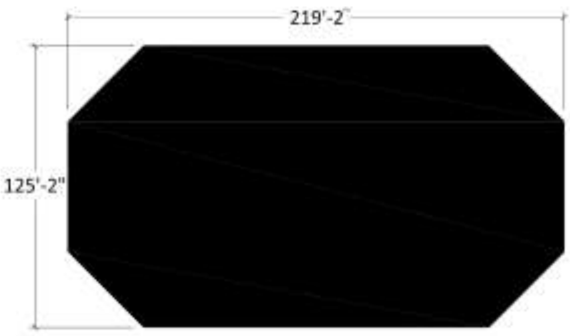
Through member checks and computer analysis, member strengths and deflections proved to be adequate for the design. Below is a table highlighting key values found throughout the report. Overturning was not an issue because the dead weight of the building was enough to counter balance moments due to the wind and seismic force.

	Wind Force			Seismic	
Floor	**N-S Shear**	**E-W Shear**	**Moment**	**Shear**	**Moment**
R	35.7 kips	22.1 kips	4480'k	154.9 kips	19157'k
9	53.3 kips	33.1 kips	5872'k	128.2 kips	14119'k
8	48.0 kips	29.8 kips	4540'k	103.4 kips	9998'k
7	45.6 kips	28.3 kips	3792'k	80.8 kips	6721'k
6	43.6 kips	27.0 kips	3038'k	60.4 kips	4211'k
5	41.0 kips	25.4 kips	2303'k	42.5 kips	2384'k
4	38.1 kips	23.6 kips	1626'k	27.1 kips	1154'k
3	34.3 kips	21.3 kips	1000'k	14.5 kips	422.9'k
2	30.7 kips	28.5 kips	481'k	5.2 kips	81.9'k
SUM	**386 kips**	**239 kips**	**27232'k**	**617 kips**	**58249'k**

Appendix

Appendix A – Wind Calculations

Analytical Procedure (ASCE&-05):



Dimensions:		Design Values:
h = 127'		Exposure B
B = 127'		V = 90 mph
L = 220'		I = 1.0
Constants:		
K_d = 0.85		G = 0.83
K_z = (see chart)		GC_p = 0.664
K_{zt} = 1.0		GC_{pi} = ±0.18

$$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right) = 0.925\left(\frac{1+1.7(3.4)0.261(0.828)}{1+1.7(3.4)0.261}\right) = 0.83$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{127+127}{422.96}\right)^{0.63}}} = 0.828$$

Where: $I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{76.2}\right)^{1/6} = 0.261$ & $L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\epsilon} = 320\left(\frac{76.2}{33}\right)^{\frac{1}{3.0}} = 422.96$

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \quad \rightarrow \textit{see chart below}$$

Wind Pressure

			Windward			Leeward	
Height (Z_t)	K_z	q_z	N-S	E-W	q_h	N-S	E-W
0-15	0.57	10.0466	9.173 psi	9.897 psi	5.2877	4.828 psi	5.209 psi
20	0.62	10.9279	9.977 psi	10.765 psi	5.2877	4.828 psi	5.209 psi
25	0.66	11.6329	10.621 psi	11.460 psi	5.2877	4.828 psi	5.209 psi
30	0.7	12.3379	11.265 psi	12.154 psi	5.2877	4.828 psi	5.209 psi
40	0.76	13.3955	12.230 psi	13.196 psi	5.2877	4.828 psi	5.209 psi
50	0.81	14.2767	13.035 psi	14.064 psi	5.2877	4.828 psi	5.209 psi
60	0.85	14.9818	13.678 psi	14.759 psi	5.2877	4.828 psi	5.209 psi
70	0.89	15.6868	14.322 psi	15.453 psi	5.2877	4.828 psi	5.209 psi
80	0.93	16.3918	14.966 psi	16.148 psi	5.2877	4.828 psi	5.209 psi
90	0.96	16.9206	15.448 psi	16.669 psi	5.2877	4.828 psi	5.209 psi
100	0.99	17.4493	15.931 psi	17.190 psi	5.2877	4.828 psi	5.209 psi
120	1.04	18.3306	16.736 psi	18.058 psi	5.2877	4.828 psi	5.209 psi
140	1.09	19.2119	17.540 psi	18.926 psi	5.2877	4.828 psi	5.209 psi

Overturning Moment

		N-S			E-W
N-S:	R:	35.7^{k} (125.5') = $4480'^{k}$	**E-W:**	R:	22.1^{k} (125.5') = $2774'^{k}$
	9:	53.3^{k} (110.17') = $5872'^{k}$		9:	33.1^{k} (110.17') = $3647'^{k}$
	8:	48.0^{k} (96.67') = $4640'^{k}$		8:	29.8^{k} (96.67') = $2881'^{k}$
	7:	45.6^{k} (83.16') = $3792'^{k}$		7:	28.3^{k} (83.16') = $2353'^{k}$
	6:	43.6^{k} (69.67') = $3038'^{k}$		6:	27.0^{k} (69.67') = $1881'^{k}$
	5:	41.0^{k} (56.16') = $2303'^{k}$		5:	25.4^{k} (56.16') = $1426'^{k}$
	4:	38.1^{k} (42.67') = $1626'^{k}$		4:	23.6^{k} (42.67') = $1007'^{k}$
	3:	34.3^{k} (29.16') = $1000'^{k}$		3:	21.3^{k} (29.16') = $621'^{k}$
	2:	30.7^{k} (15.67) = $481'^{k}$		2:	28.5^{k} (15.67) = $447'^{k}$
SUM:		$27{,}232'^{k}$			$17{,}037'^{k}$

Building Weight: $15{,}354^{k}$

N-S Overturning Check: $15{,}354^{k}$ (63.16') = $969{,}758'^{k}$ >> $27{,}232'^{k}$ **O.K.**

Drift:

		Horizontal	Vertical	Horizontal	Resultant	Rotational		
Node	L/C	X in	Y in	Z in	in	rX rad	rY rad	rZ rad
1	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000	0.000	0.000
2	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000	0.000	0.000
3	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000	0.000	0.000
4	1 LOAD CAS	0.700	0.003	0.000	0.700	0.000	0.000	-0.004
5	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000	0.000	0.000
6	1 LOAD CAS	0.692	-0.003	0.000	0.692	0.000	0.000	-0.004
7	1 LOAD CAS	1.379	0.004	0.000	1.379	0.000	0.000	-0.003
8	1 LOAD CAS	0.699	0.004	0.000	0.699	0.000	0.000	0.002
9	1 LOAD CAS	1.373	-0.004	0.000	1.373	0.000	0.000	-0.003

Structure1.std - Beam Relative Displacement Detail:

All Relative Displacement | Max Relative Displacements

Beam	L/C	Dist ft	x in	y in	z in	Resultant in
1	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000
		5.045	0.100	0.111	0.000	0.149
		10.090	0.114	0.127	0.000	0.171
		15.135	0.072	0.080	0.000	0.107
		20.180	0.000	0.000	0.000	0.000
2	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000
		7.500	-0.000	-0.126	0.000	0.126
		15.000	-0.000	-0.001	0.000	0.001
		22.500	-0.000	0.124	0.000	0.124
		30.000	0.000	0.000	0.000	0.000
3	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000
		7.500	0.000	-0.085	0.000	0.085
		15.000	-0.000	0.001	0.000	0.001
		22.500	0.000	0.087	0.000	0.087
		30.000	0.000	0.000	0.000	0.000
4	1 LOAD CAS	0.000	0.000	0.000	0.000	0.000
		3.375	-0.091	0.000	0.000	0.091
		6.750	-0.077	0.000	0.000	0.077
		10.125	-0.025	0.000	0.000	0.025
		13.500	0.000	0.000	0.000	0.000

Appendix B – Seismic Load

S_s = 0.20g	Site Class:	C
S_1 = 0.08g	Seismic Use Group:	I
S_{DS} = 0.16	Importance Factor:	1.0
S_{D1} = 0.09	Design Category:	B

Calculations based on the equivalent lateral force method ASCE7-05.

$$T_a = C_t h_n^x = 0.028(127')^{0.8} = 1.36\, sec. > 0.563 = T_s$$

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.160}{3.0/1.0} = 0.053$$

$$C_s > 0.044 S_{DS} I = 0.044(0.16)1.0 = 0.00704$$

$$C_s \le \frac{S_{D1}}{T^2(R/I)} = \frac{0.09}{0.864^2(3.0/1.0)} = 0.0402$$

$$\therefore\ C_s = 0.0402$$

Base Shear: $V = C_s * W = 0.0402(15{,}354^k) = 617^k$

Vertical Distribution: $F_x = C_{vX} V \;\; where: C_{vX} = \frac{w_x h_x^k}{\sum_{i=1}^{9} w_x h_x^k}$ See Below: K= 1.64

Seismic Calculations

Floor	Weight (kips)	Height (ft)	H^k	C_{vX}	F_x	Moment
1	1893.14	15.67’	91.18	0.00848	5.2 kips	81.9
2	1901.84	29.17’	252.6	0.0235	14.5 kips	422.9
3	1901.84	42.67’	471.4	0.04384	27.1 kips	1154
4	1901.84	56.17’	739.9	0.06881	42.5 kips	2384
5	1901.84	69.67’	1053.4	0.09797	60.4 kips	4211
6	1901.84	83.17’	1408.5	0.13099	80.8 kips	6721
7	1901.84	96.67’	1802.5	0.16763	103.4 kips	9998
8	1901.84	110.2’	2233.5	0.20771	128.2 kips	14119
9	1901.84	123.67	2699.7	0.25107	154.9 kips	19157
∑	**17107.8**		**10752.7**		**617.0 kips**	**58249’^{k}**

0.

Appendix C –6 Snow Load

$p_f = 0.7(C_eC_tIp_g) = 0.7(30) = 21$ psf roof load

$C_e = 1.0$ $C_t = 1.0$ $I = 1.0$ $p_g = 30$ psf