The Residences of Sherman Plaza
Evanston, IL

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Structural Technical Report 3
Lateral System Analysis and Confirmation Design

Executive Summary

Sherman Plaza is a luxury condominium, located in the heart of downtown Evanston, IL. This 25 story condominium includes a health club, rooftop gardens, and two floors of retail space. The structural system of the building is a reinforced, cast-in-place concrete superstructure. The floor system is made up of two-way flat plates on reinforced concrete columns with deep edge beams surrounding the building's perimeter. The building rests on a foundation of belled-caissons that extend to hardpan at approximately 70 feet below grade. The edge beams also serve as perimeter moment frames to support the lateral loads. These moment frames act in combination with reinforced concrete shear walls to form the lateral resisting system of Sherman Plaza.

The lateral loads are distributed to this combined shear wall and frame system by computing the stiffnesses of each of the members. The elements with the highest stiffness will receive the most load. These loads will then be used to complete a strength check of critical members, the story drifts, the total drift and the overturning moment. The load distribution calculation was performed at floors 25, 22, 14, and 6, and therefore, the design checks were performed for lateral elements on these floors only.

A shear wall check was performed by analyzing the wall as a deep cantilever beam. The allowable shear force was calculated and compared to the actual shear force, calculated by the load distribution described above. Each of the shear walls that were checked was found to be sufficient. The moment frames were analyzed using Visual Analysis, a finite element design and analysis software. The software was used to determine the moments on the frame due to the distributed lateral load. The beams

were then checked for the flexural strength due to this moment. Each beam was found to be sufficient.

The overturning moment was computed by multiplying the story shear forces by the story height. The resisting moment was then found to be the total building weight multiplied by the distance to the center of mass. The resisting moment was found to be much larger than the overturning moment. Therefore, overturning does not need to be considered when designing the foundations.

The story drift and total drift was found using an ETABS model. The drift values were compared to the allowable drift value of H/600. This criteria is higher than the industry standard of H/400, but Sherman Plaza was designed to drift no more than H/600. The 25th floor was found to drift 0.227 inches, which was slightly higher than the allowable drift of 0.217 inches. Most of the drift values, however, were less than the allowable drift. The total building drift was found to be 3.1 inches, which was less than the allowable drift of 5.21 inches.

Introduction

Sherman Plaza is a complex reinforced, cast-in-place concrete structure that uses a combination of shear walls and moment frames as its lateral resisting system. This report will provide a more in-depth analysis of the lateral system, as a continuation of the lateral calculations from Structural Technical Report 1. The calculated wind and seismic loads from Report 1 will be used to determine the controlling load combination, which will then be used to provide an analysis and a confirmation design study to determine if the system meets the allowable code values.

This report will also include the lateral load distribution and a check of strength, drift, story drift and overturning of the building. The story shear loads will be distributed according to the relative stiffness of each lateral member, with the stiffer members receiving the most load. The check will include a spot check of critical members, a comparison of drift values with allowable code values, and a discussion of overall building torsion issues. This information will be used in the proposal for the building redesign.

Building Description

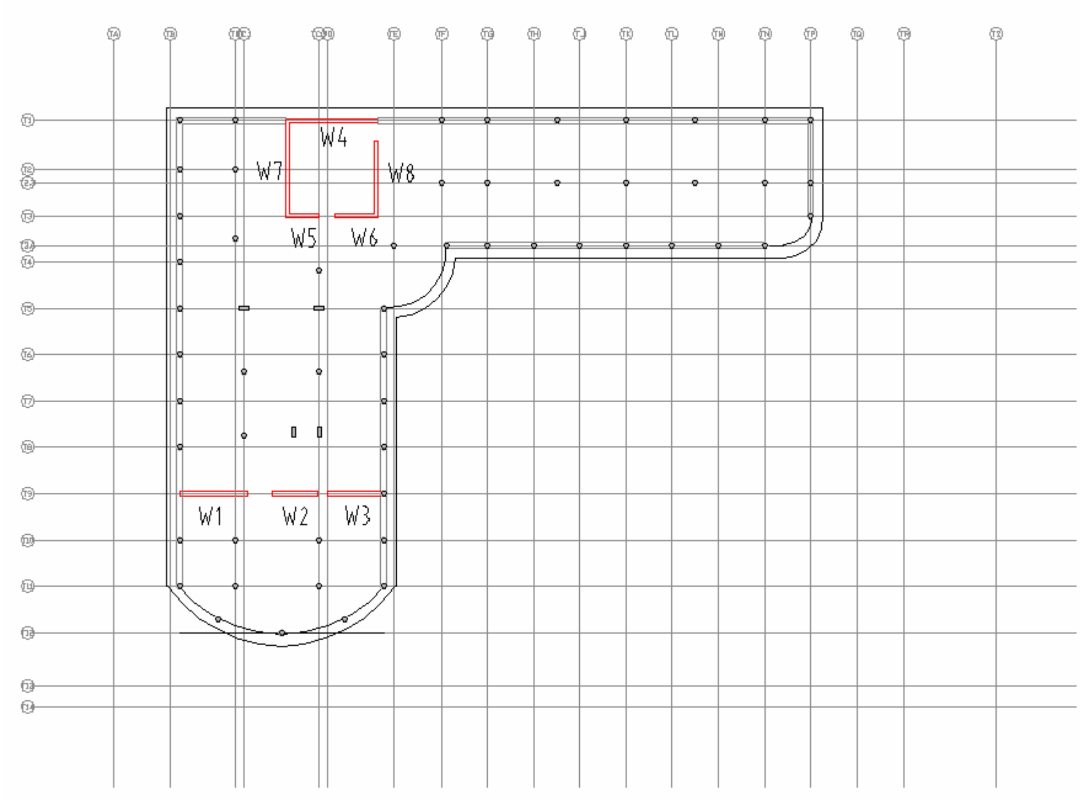
Sherman Plaza is a 25 story condominium, located in Evanston, IL. The residences include 253 condominiums, lofts and penthouses. The building also accommodates a 54,000 square foot health club, garage parking, a ½ acre rooftop garden and 152,000 square feet of retail space on the bottom two floors. The rooftop gardens are located on the third, sixth and seventh floor roofs as the building steps back. The eighth floor to the twenty-second floor makes up the condominium’s L-shaped tower and follows a typical floor plan. The top three levels are the penthouse floors, which follow a different column grid than the rest of the building. These floors also step back to accommodate the private terraces.

The structural system for the Sherman Plaza residential tower consists of reinforced cast-in-place concrete columns, shear walls, slabs and beams. A typical floor slab for both the lower retail floors and the upper residential stories is 8 inch thick two-way reinforced concrete plate. The column sizes range from a 36”x36” square column at the ground floor to a 20”x20” square column at the roof level. ACI 318-95 was the primary design code used for the structural concrete design. The building is supported by a foundation of belled caissons that extend to hardpan at approximately 70 feet below grade, with an allowable bearing pressure of 30 ksf.

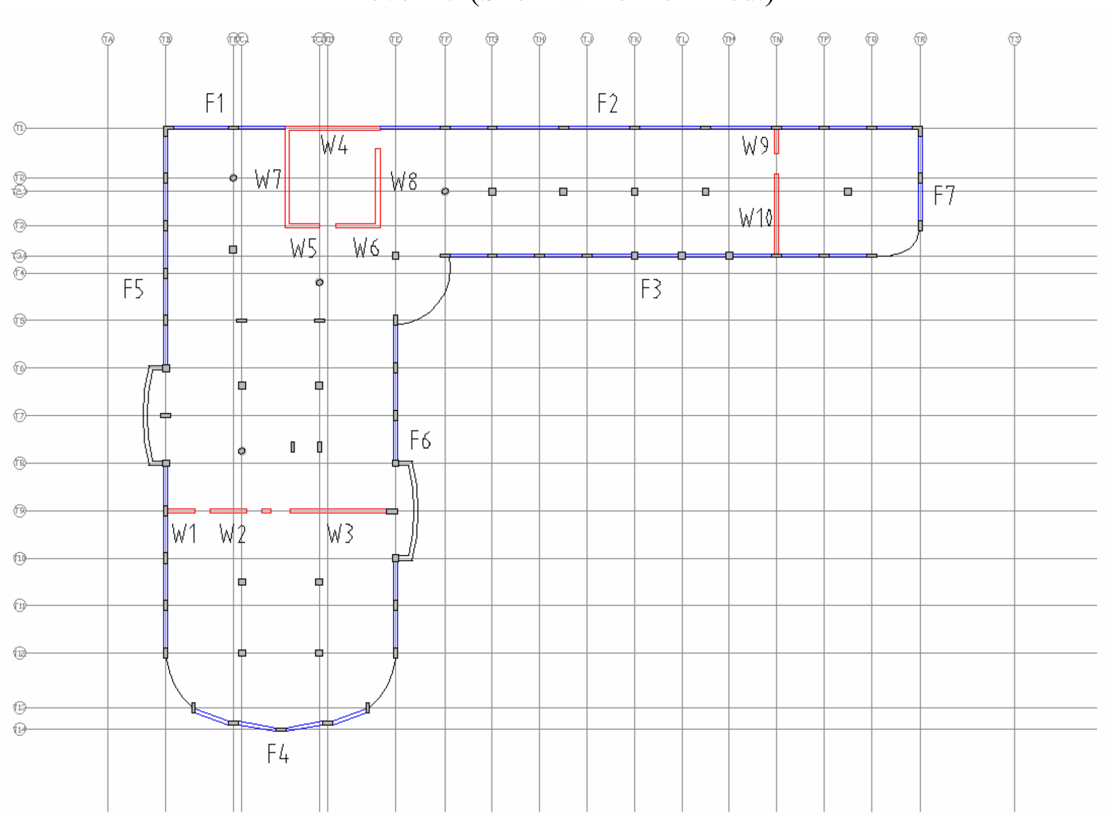
Lateral Resisting System

The lateral support for the building is made up of a combination of reinforced concrete shear walls and perimeter moment frames for the first twenty-two stories. There are shear walls located around the elevator core, near the intersection of the L-shape of the building. There is also a shear wall in each arm of the L-shape. The elevator core shear walls are 18” thick for the first six floors, 16” thick for floors 7 to 22, and 12” thick for the last three floors. The shear walls located in the L-shape’s arms are 18” thick for the first six floors, 15” for floors 7 to 12, and 12” thick for the remaining floors. The reinforcement for the shear walls is #5@12”, in general. The moment frames are made up of deep edge beams around the building’s perimeter. A typical perimeter beam is a 13”x34” beam with 4 #7 reinforcement bars on top and bottom. The typical perimeter columns are 13”x36” with 8#7 bars.

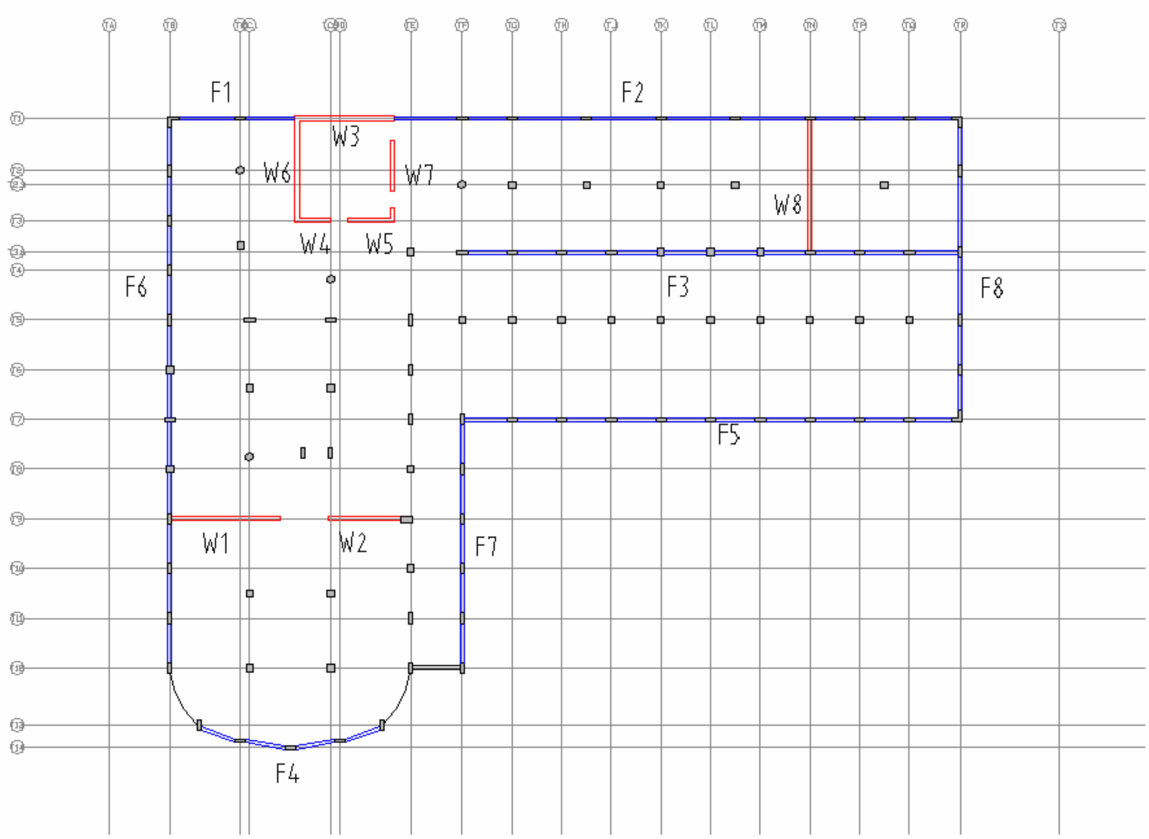
The top three floors of Sherman Plaza are penthouse levels and have a different column grid than the rest of the building. Therefore, the moment frames do not continue up to these floors. Instead, it is assumed that the shear walls on this level will take all the lateral load. Due to the large size and complexity of this building, a load distribution will not be performed at every floor. The distribution calculations will be completed for levels 25, 22, 14 and 6. On the 25th floor, the load will be distributed only to the shear walls. The 22nd and 14th floors have the same basic layout, but the walls and frames will have different stiffnesses at these levels, so the distribution will change. At the 6th floor, the shape of the shear walls changes, and there will be an additional moment frame to consider due to the area where the building steps back. See the plans below for the layout of the shear walls and frames of each level.



Level 25 (Shear walls are in red.)



Typical Floor Plan: Levels 8-22
(Shear walls are in red and moment frames are in blue.)

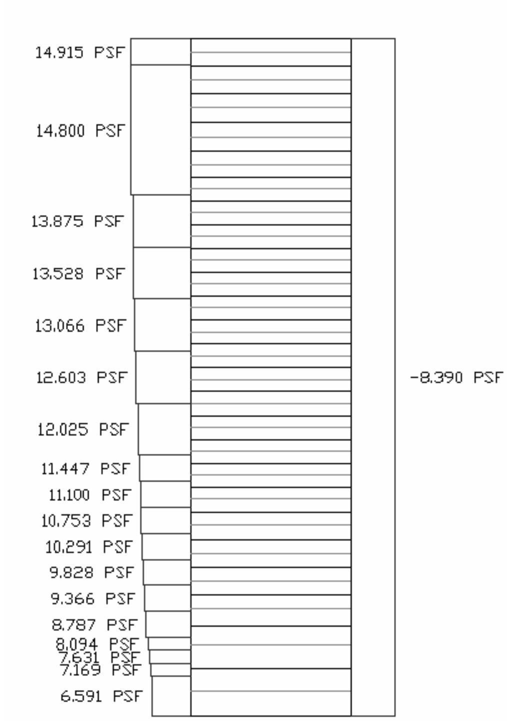


Level 6
(Shear walls in red and moment frames in blue.)

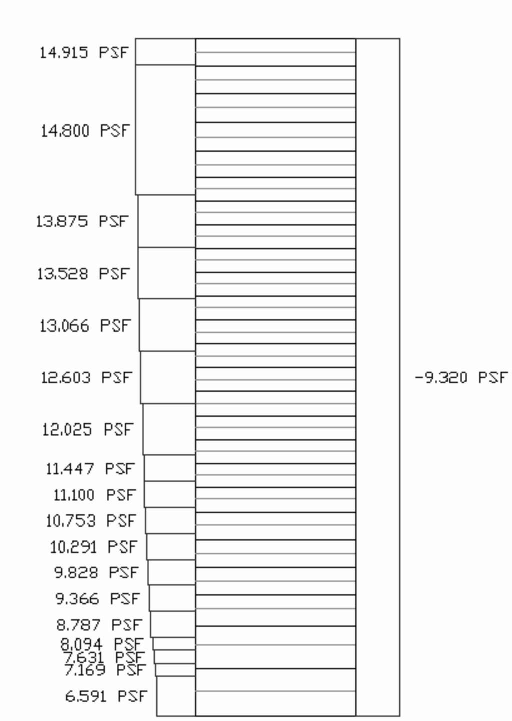
Lateral Loads and Load Combinations

Lateral Load Calculations

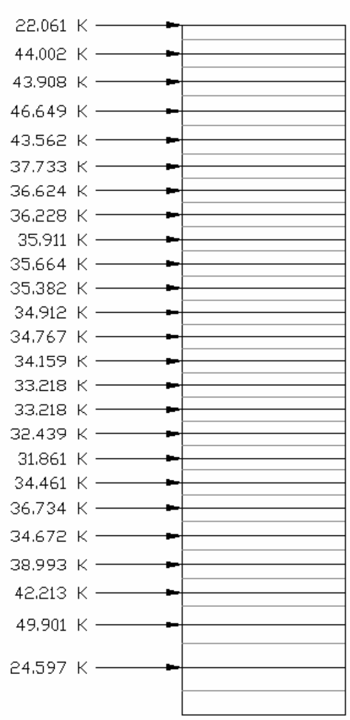
The actual lateral system of Sherman Plaza was designed to resist wind forces, according to ASCE 7-98, and a seismic analysis was not performed, because it is not required by the City of Chicago Building Code. A complete calculation of both wind and seismic loads, however, was performed in Structural Technical Report 1. These loads were calculated using ASCE 7-02 and the results of that calculation can be seen in the Load Combinations Table in Appendix A. The wind pressure and story force diagrams can also be seen below. Refer to Structural Technical Report 1 for the full calculations and design assumptions.



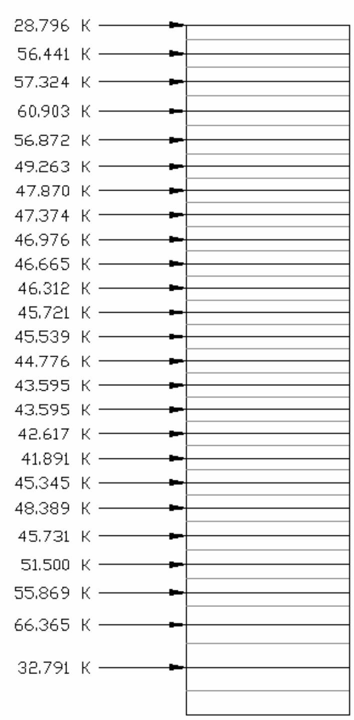
North-South Wind Pressure Diagram



East-West Wind Pressure Diagram



North-South Story Forces



East-West Story Forces

Load Combinations

The load factors and load combinations were found in ASCE 7-02, Section 2. The controlling load combination is determined in the table in Appendix A. The load cases are:

- Case 1: 1.4D
- Case 2: 1.2D + 1.6L + 0.5S
- Case 3: 1.2D + 1.6S + L
- Case 4: 1.2D + 1.6W + L + 0.5S
- Case 5: 1.2D + 1.0E + L + 0.2S
- Case 6: 0.9D + 1.6W + 1.6H
- Case 7: 0.9D + 1.0E + 1.6H

As seen in the load combination table, load case 2 controls, but for this report, load case 4 will be used for the lateral analysis. Therefore, the wind shear forces control and the seismic forces will not be considered.

Load Distribution

The cast-in-place concrete structure of Sherman Plaza acts as a rigid diaphragm. The columns and slabs, in addition to the moment frames and shear walls, receive part of the lateral forces. However, the slabs are only 8" thick and therefore, won't provide as much lateral resistance as the moment frames that have beams that are 34" deep, in general. In this analysis, it will be assumed that the shear walls and frames will take all the lateral load.

The lateral loads were distributed to the moment frames and shear walls at each level by relative stiffnesses. The stiffness of each member was found by first finding the members' deflection and taking the stiffness as the inverse of deflection. The direct shear force was found by proportioning the shear by stiffness, with the members with the highest stiffness receiving the most load. In addition, the force due to the torsional moment of each floor was calculated by locating the center of rigidity of each floor. The calculations of the center of rigidity can be found in Appendix B. The eccentric shear force was found by distributing the total shear force according to the members' stiffness and distance to the center of rigidity. The total shear force to each member is the sum of the concentric and eccentric forces.

The procedure for distributing the lateral loads to the shear walls was taken from the PCI Handbook. First, the ratio of the wall height to length was calculated, and it was found that this value was between 0.3 and 3.0, meaning that the deflection of the wall will be the sum of the deflections due to shear and flexure. Next, the stiffnesses of each wall was found by taking, $k = 1/\Delta$.

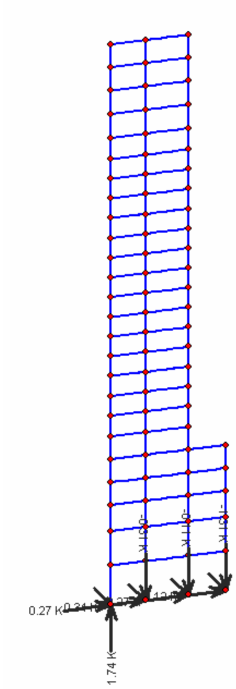
- Flexure: $\Delta = \dfrac{Ph^3}{3EI}$

- Shear: $\Delta = \dfrac{2.78Ph}{AwE}$

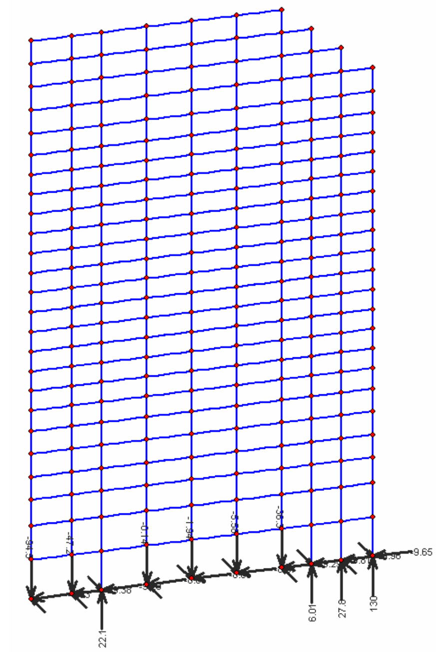
The stiffness of the moment frames was found by modeling each frame in Visual Analysis, a finite element design and analysis software. The frame was loaded with a unit load of one kip at the level being analyzed to determine the deflection of the frame at that level. The stiffness was then found as the inverse of this deflection. The frames from Visual Analysis can be seen in the figures below. The complete calculation of the shear forces can be found in Appendix B.

Building Torsion

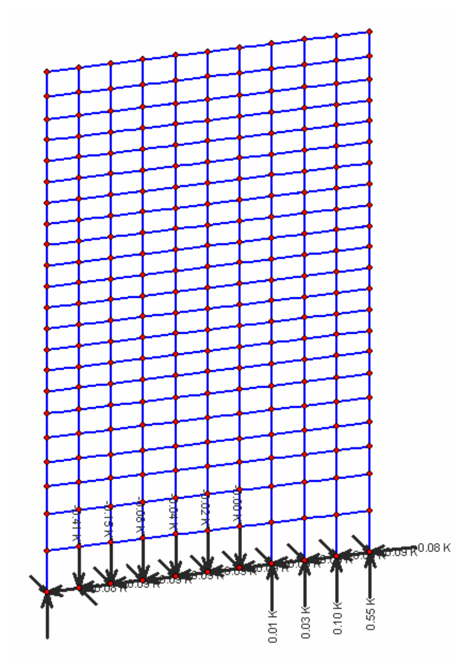
Due to the asymmetrical shape of Sherman Plaza, the torsional forces due to the eccentricity of the building needed to be considered. For each of the calculated levels, the center of rigidity of the floor was found. In most cases, the center of rigidity was fairly close to the building's center of mass. In some cases, the center of rigidity was actually smaller of the center of mass. In these cases, the eccentricity was taken as 5% of the building length. The eccentric forces due to building torsion were therefore taken into account and added to the total shear force taken by each of the lateral resisting elements.



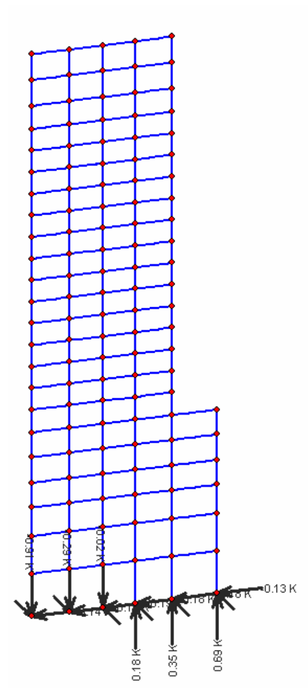
Moment Frame 1



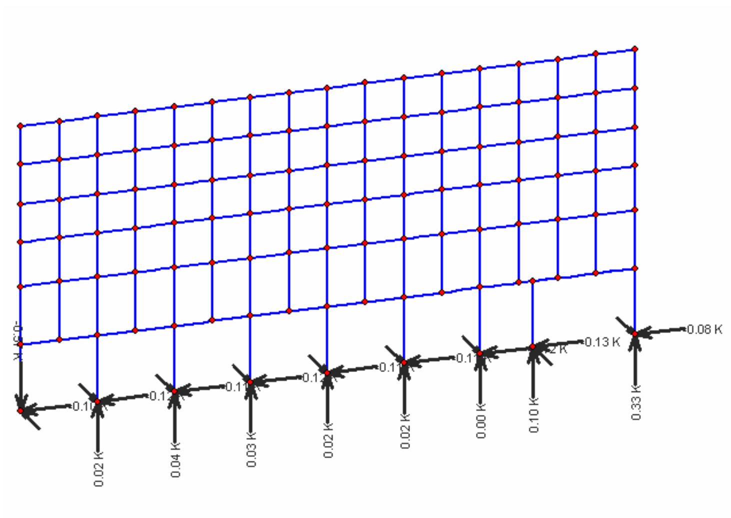
Moment Frame 2



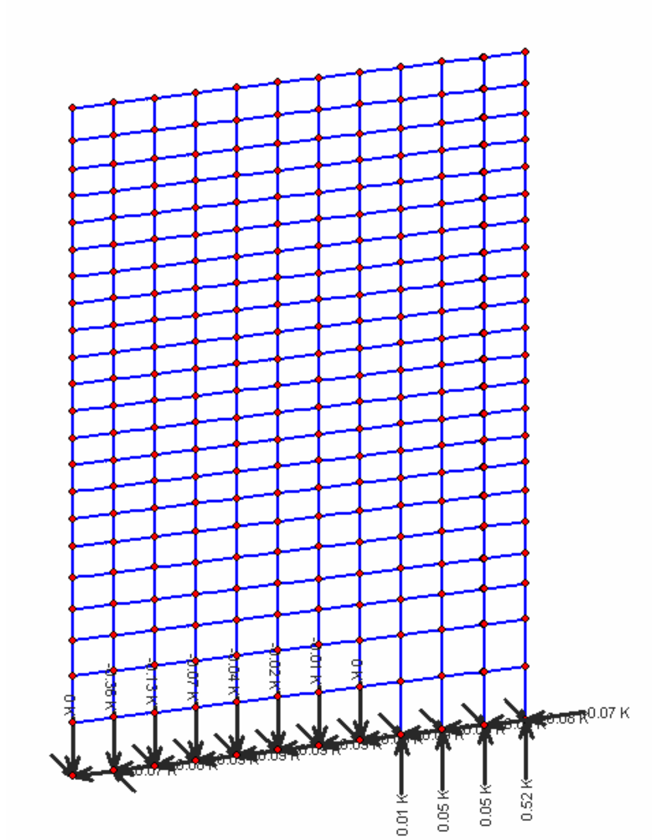
Moment Frame 3



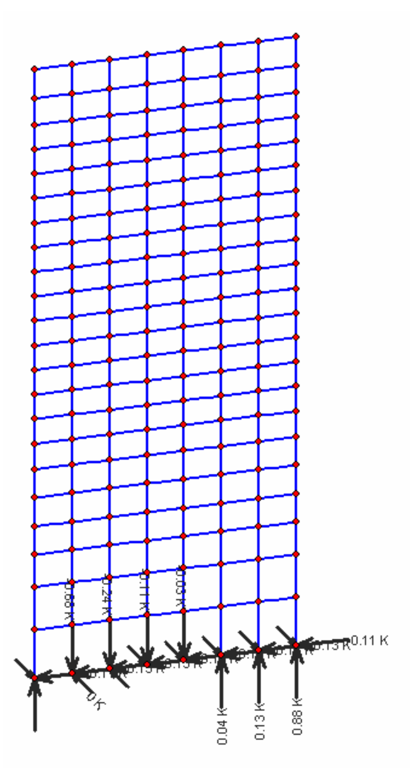
Moment Frame 4



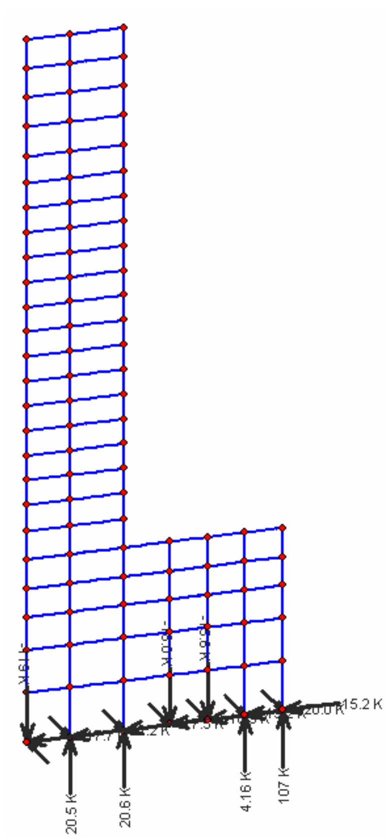
Moment Frame 5



Moment Frame 6



Moment Frame 7



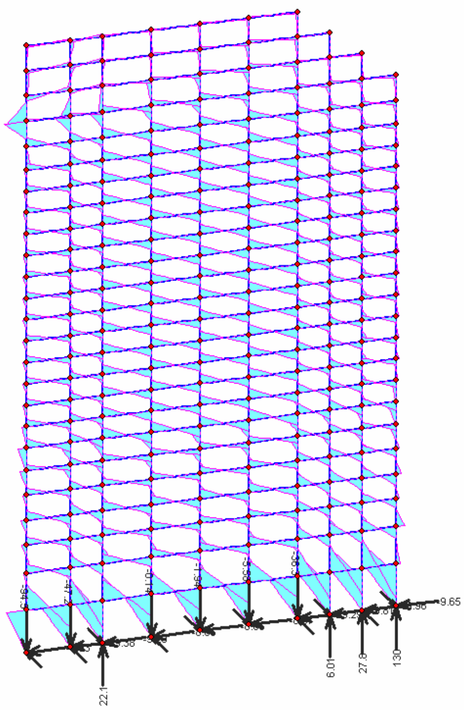
Moment Frame 8

Lateral System Calculations

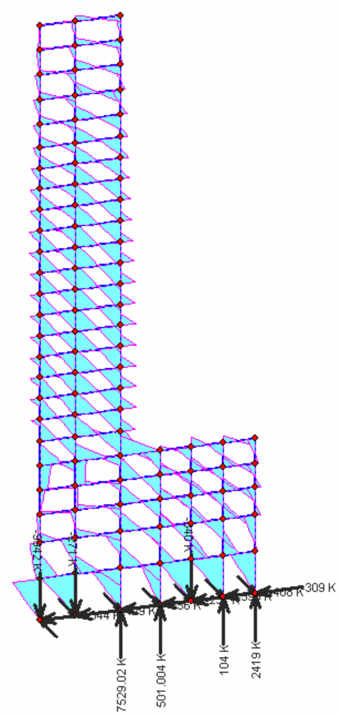
Strength Check of Critical Members

A strength check of critical lateral members was performed for each of the levels at which the load distribution was calculated. For each floor, the critical shear wall and moment frame was determined by choosing the element with the highest load. This check of the critical element at each floor will be sufficient, because it is assumed that the other elements will be able to take their lower loads. Only the shear walls were checked on the 25th floor, because it was assumed that the moment frames would not take a significant amount of load.

The shear capacity of each of the critical shear walls was calculated, treating the wall as a deep cantilevered beam. The actual dimensions and reinforcement of the walls was used in the calculations, and each of the walls was found to be adequate. The moment frames were then checked using Visual Analysis. The calculated load on the frame was applied, and the moments were found. A check of the beam with the highest moment was performed. Again, the actual dimensions and reinforcement was used in the check, and each beam was found to be sufficient. Refer to Appendix C for the full calculations of the shear wall and moment frame design checks.



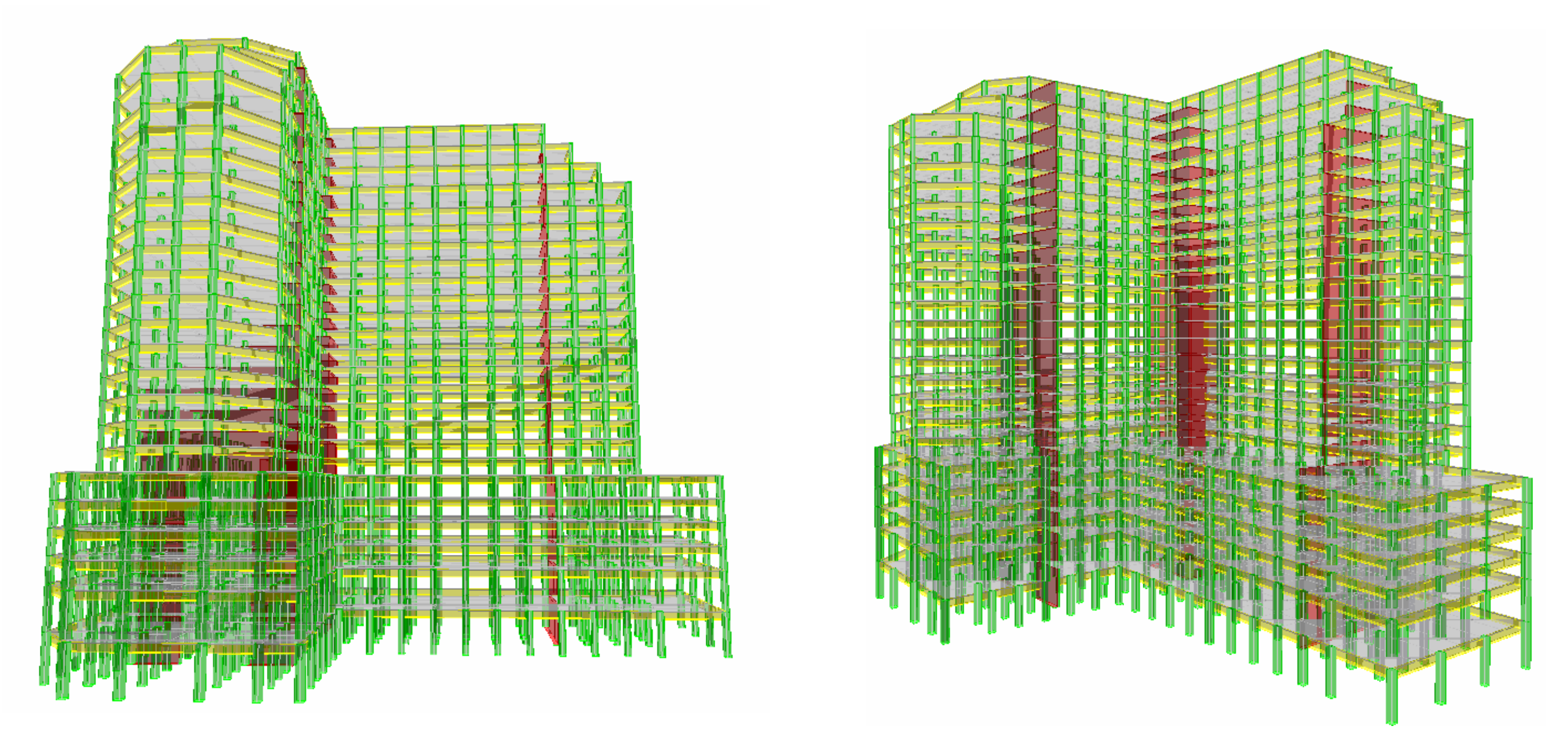
Frame 2 Moment Diagram



Frame 8 Moment Diagram

Story Drift and Total Drift

The values for the story drifts and the total building drift were found using an ETABS model. Each of the floors was modeled, and the dead, live and wind loads applied. The story drifts were found and added together to find the total drift. While the industry standard of design is an allowable drift of H/400, Sherman Plaza was designed to move laterally a maximum of H/600. .



3D View of ETABS Model

The drifts were tabulated for the East-West wind direction only, because that is the critical direction. The drift at the 25th floor was found to be 0.227 in, and the allowable drift according to the H/600 standard is 0.2167 in. The 25th story drift is therefore slightly higher than the allowable drift, but most of the floors have a smaller drift than the allowable. The total building drift was found to be 3.1 in., and the allowable drift was 5.21 in. The drift value is therefore less than the imposed total drift criteria and is adequate for this design. Refer to Appendix C for the full drift calculations.

Edit View

Diaphragm CM Displacements

Story	Diaphragm	Load	UX	UY	UZ	RX	RY	RZ
STORY25	D1	WINDEW	0.2272	0.0262	0.0000	0.00000	0.00000	-0.00007
STORY24	D1	WINDEW	0.2203	0.0233	0.0000	0.00000	0.00000	-0.00007
STORY23	D1	WINDEW	0.2153	0.0205	0.0000	0.00000	0.00000	-0.00006
STORY22	D1	WINDEW	0.2046	0.0164	0.0000	0.00000	0.00000	-0.00006
STORY21	D1	WINDEW	0.1936	0.0155	0.0000	0.00000	0.00000	-0.00006
STORY20	D1	WINDEW	0.1855	0.0152	0.0000	0.00000	0.00000	-0.00006
STORY19	D1	WINDEW	0.1771	0.0149	0.0000	0.00000	0.00000	-0.00005
STORY18	D1	WINDEW	0.1686	0.0146	0.0000	0.00000	0.00000	-0.00005
STORY17	D1	WINDEW	0.1599	0.0142	0.0000	0.00000	0.00000	-0.00005
STORY16	D1	WINDEW	0.1512	0.0138	0.0000	0.00000	0.00000	-0.00005
STORY15	D1	WINDEW	0.1424	0.0134	0.0000	0.00000	0.00000	-0.00004
STORY14	D1	WINDEW	0.1335	0.0130	0.0000	0.00000	0.00000	-0.00004
STORY13	D1	WINDEW	0.1246	0.0126	0.0000	0.00000	0.00000	-0.00004
STORY12	D1	WINDEW	0.1157	0.0121	0.0000	0.00000	0.00000	-0.00004
STORY11	D1	WINDEW	0.1068	0.0116	0.0000	0.00000	0.00000	-0.00004
STORY10	D1	WINDEW	0.0981	0.0111	0.0000	0.00000	0.00000	-0.00003
STORY9	D1	WINDEW	0.0894	0.0105	0.0000	0.00000	0.00000	-0.00003
STORY8	D1	WINDEW	0.0809	0.0099	0.0000	0.00000	0.00000	-0.00003
STORY7	D1	WINDEW	0.0727	0.0094	0.0000	0.00000	0.00000	-0.00003
STORY6	D1	WINDEW	0.0663	0.0041	0.0000	0.00000	0.00000	-0.00002
STORY5	D1	WINDEW	0.0550	0.0037	0.0000	0.00000	0.00000	-0.00002
STORY4	D1	WINDEW	0.0439	0.0031	0.0000	0.00000	0.00000	-0.00002
STORY3	D1	WINDEW	0.0333	0.0026	0.0000	0.00000	0.00000	-0.00001
STORY2	D1	WINDEW	0.0235	0.0021	0.0000	0.00000	0.00000	-0.00001
STORY1	D1	WINDEW	0.0093	0.0010	0.0000	0.00000	0.00000	0.00000

ETABS Output of Story Drift in the E-W Wind Direction

Overturning Moment

The overturning moment was calculated in the East-West wind direction, because it is the critical direction with the highest applied wind loads. The North-South direction, therefore, will not control. The overturning moment was calculated by multiplying the story shear forces with the story height. This moment was then compared to the resisting moment to determine if overturning should be considered in the design of the building's foundations. The resisting moment was found by multiplying the total building weight by the distance from the building corner to the center of mass. The resisting moment was found to be much larger than the overturning moment. Therefore, overturning will not be a consideration when designing the building foundations. Refer to Appendix C for the full overturning moment calculations.

Summary and Conclusion

The lateral system for Sherman Plaza is a combination of shear walls and moment frames that were analyzed by computing each element's relative stiffness. The shear forces were distributed according to these stiffnesses, with the stiffest members receiving the most load. Due to the large size and complexity of the building, the load distributions were calculated only for floors 25, 22, 14 and 6. These loads were then used to complete a strength check of critical members, the overturning moment, story drift, and total drift.

A critical shear wall and moment frame was chosen for each of the computed floors and checked for adequacy. The critical members had the highest shear forces on that floor, and it will therefore be conservative to assume that the design will be sufficient for the rest of the members on the floor. The shear wall was analyzed as a deep cantilever beam, and the allowable shear force was found and compared to the actual shear force on the wall. Each of the critical walls was found to be sufficient. The moment frames were analyzed using Visual Analysis, a finite element design and analysis software. The frames were loaded with the computed loads, and a critical beam was chosen by determining which had the highest applied moment. The beams were then checked, and each was found to be adequate.

The overturning moment was computed by multiplying the story shear forces by the story height. The resisting moment was then found to be the total building weight multiplied by the distance to the center of mass. The resisting moment was found to be much larger than the overturning moment. Therefore, overturning does not need to be considered when designing the foundations.

The story drift and total drift was found using an ETABS model. The drift values were compared to the allowable drift value of H/600. This criteria is higher than the industry standard of H/400, but Sherman Plaza was designed to drift no more than H/600. The 25th floor was found to drift 0.227 inches, which was slightly higher than the allowable drift of 0.217 inches. Most of the drift values, however, were less than the allowable drift. The total building drift was found to be 3.1 inches, which was less than the allowable drift of 5.21 inches.

Appendix A

Loads and Load Combinations

Load Combinations												
Level	DL	LL	SL	EL	WL	1	2	3	4	5	6	7
	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)	(kips)
Roof	234	1248	312.000	40.950	28.796	327.6	2433.6	2028	1730.8736	1632.15	256.6736	251.55
25	259.5	1384	0.000	42.325	56.441	363.3	2525.8	1695.4	1785.7056	1737.725	323.8556	275.875
24	280.5	1496	0.000	42.555	57.324	392.7	2730.2	1832.6	1924.3184	1875.155	344.1684	295.005
23	297	1584	0.000	41.773	60.903	415.8	2890.8	1940.4	2037.8448	1982.173	364.7448	309.073
22	297	1584	0.000	38.196	56.872	415.8	2890.8	1940.4	2031.3952	1978.596	358.2952	305.496
21	297	1584	0.000	35.551	49.263	415.8	2890.8	1940.4	2019.2208	1975.951	346.1208	302.851
20	297	1584	0.000	32.981	47.870	415.8	2890.8	1940.4	2016.992	1973.381	343.892	300.281
19	297	1584	0.000	30.488	47.374	415.8	2890.8	1940.4	2016.1984	1970.888	343.0984	297.788
18	297	1584	0.000	28.073	46.976	415.8	2890.8	1940.4	2015.5616	1968.473	342.4616	295.373
17	297	1584	0.000	25.739	46.665	415.8	2890.8	1940.4	2015.064	1966.139	341.964	293.039
16	297	1584	0.000	23.485	46.312	415.8	2890.8	1940.4	2014.4992	1963.885	341.3992	290.785
15	297	1584	0.000	21.314	45.721	415.8	2890.8	1940.4	2013.5536	1961.714	340.4536	288.614
14	297	1584	0.000	19.228	45.539	415.8	2890.8	1940.4	2013.2624	1959.628	340.1624	286.528
13	297	1584	0.000	17.228	44.776	415.8	2890.8	1940.4	2012.0416	1957.628	338.9416	284.528
12	297	1584	0.000	15.316	44.776	415.8	2890.8	1940.4	2012.0416	1955.716	338.9416	282.616
11	297	1584	0.000	13.494	43.595	415.8	2890.8	1940.4	2010.152	1953.894	337.052	280.794
10	297	1584	0.000	11.765	43.595	415.8	2890.8	1940.4	2010.152	1952.165	337.052	279.065
9	297	1584	0.000	10.132	42.617	415.8	2890.8	1940.4	2008.5872	1950.532	335.4872	277.432
8	297	1584	0.000	8.597	41.891	415.8	2890.8	1940.4	2007.4256	1948.997	334.3256	275.897
7	484.5	2584	0.000	11.686	45.345	678.3	4715.8	3165.4	3237.952	3177.086	508.602	447.736
6	484.5	2584	0.000	9.109	48.389	678.3	4715.8	3165.4	3242.8224	3174.509	513.4724	445.159
5	484.5	2584	0.000	6.793	45.731	678.3	4715.8	3165.4	3238.5696	3172.193	509.2196	442.843
4	484.5	2584	0.000	4.870	51.500	678.3	4715.8	3165.4	3247.8	3170.27	518.45	440.92
3	721.5	3848	0.000	4.781	55.869	1010.1	7022.6	4713.8	4803.1904	4718.581	738.7404	654.131
2	721.5	3848	0.000	1.771	66.365	1010.1	7022.6	4713.8	4819.984	4715.571	755.534	651.121

Appendix B

Lateral Loads Distribution to Shear Walls and Moment Frames

Level 25														
Shear Wall #	Length	Thickness	H/L	I	Aw	Deflection due to		Defl.	k	Concentric	Dist. To	k * D^2	kD/ΣkD^2	Torsional	Total
	(ft.)	(in.)		(in.^4)	(in.^2)	Flexure	Shear	(in.)		Shear	C.R. (ft.)			Shear (k)	Shear (k)
						(in.)	(in.)			(kips)					
N-S															
1	29.1667	12	0.37	42875147	350.0004	0.049798489	0.0052064	0.0550049	18.180187	32.32806798	8.75	1391.920557	0.000925004	2.7908275	35.118895
2	9.25	12	1.15	1367631	111	1.561179552	0.0164167	1.5775963	0.6338757	1.127159904	8.75	48.53111053	3.22515E-05	0.0973058	1.2244657
3	12.667	12	0.84	3512085.3	152.004	0.607934431	0.0119882	0.6199227	1.6131045	2.868427645	8.75	123.5033101	8.20744E-05	0.2476265	3.1160542
4	26.8333	12	0.40	33386124	321.9996	0.063952245	0.0056592	0.0696114	14.365455	25.54469953	104.5	156874.3616	0.00872917	26.336756	51.881455
5	9.1667	12	1.16	1331014.5	110.0004	1.604127919	0.0165659	1.6206938	0.6170197	1.097186426	75.75	3540.497504	0.000271781	0.8199894	1.9171759
6	13	12	0.82	3796416	156	0.562403475	0.0116811	0.5740846	1.7419035	3.097458513	75.75	9995.151116	0.000767263	2.3149058	5.4123643
								SUM =	37.151545			171973.9652			
E-W															
7	28.75	12	0.37	41063625	345	0.05199535	0.0052819	0.0572773	17.458936	55.1228623	9.8333	1688.170717	0.035927551	65.013832	120.13669
8	23.25	12	0.46	21717639	279	0.0983126	0.0065314	0.104844	9.5379811	30.1141377	18	3090.305865	0.035928534	65.015611	95.129749
								SUM =	26.996918			4778.476582			
Height (ft.) =	260.5														
Ec (ksi) =	4768.9622														
N-S Shear (k) =	66.063														
E-W Shear (k) =	85.237														
N-S Tor. Mom. =	3017.0972	(ft.-kips)													
E-W Tor. Mom. =	1809.5815	(ft.-kips)													

Level 22															
Shear Wall #	Length	Thickness	H/L	I	Aw	Deflection due to		Defl.	k	Concentric	Dist. To	k * D^2	kD/ΣkD^2	Torsional	Total
	(ft.)	(in.)		(in.^4)	(in.^2)	Flexure	Shear	(in.)		Shear	C.R. (ft.)			Shear (k)	Shear (k)
						(in.)	(in.)			(kips)					
N-S															
1	8.1667	12	1.12	941203.52	98.0004	1.299222046	0.0154418	1.3146638	0.7606507	0.461968323	60.43	2777.732872	2.68995E-05	0.0726678	0.5346361
2	11.0833	12	0.83	2352615.8	132.9996	0.51977564	0.0113782	0.5311539	1.8826936	1.14342204	60.43	6875.19215	6.65791E-05	0.1798606	1.3232827
3	31	12	0.30	51478848	372	0.023754074	0.004068	0.0278221	35.942671	21.82917136	60.43	131254.9018	0.001271068	3.4337351	25.262906
4	26.8333	16	0.34	44514831	429.3328	0.027470224	0.0035248	0.030995	32.263274	19.5945524	52.32	88316.91917	0.00098783	2.6685788	22.263131
5	9.1667	16	1.00	1774686	146.6672	0.689041527	0.0103179	0.6993594	1.4298799	0.868413297	23.57	794.3624792	1.97226E-05	0.0532799	0.9216931
6	13	16	0.71	5061888	208	0.241576339	0.0072755	0.2488518	4.0184557	2.440540897	23.57	2232.432557	5.54274E-05	0.1497348	2.5902757
								SUM =	76.297625			1708811.109			
E-W															
7	28.75	16	0.32	54751500	460	0.022334226	0.0032898	0.025624	39.025901	61.60458822	55.193	118883.325	0.001921779	10.392903	71.997491
8	23.25	16	0.39	28956852	372	0.042229465	0.004068	0.0462975	21.59945	34.09595128	27.36	16168.69173	0.000527261	2.8514038	36.947355
9	7.5	12	1.22	729000	90	1.677410658	0.0168144	1.6942251	0.5902403	0.931727697	89.307	4707.603644	4.70306E-05	0.2543398	1.1860675
10	24.1667	12	0.38	24389101	290.0004	0.050138477	0.0052183	0.0553567	18.064646	28.51606375	89.307	144078.9257	0.0014394	7.7842163	36.30028
								SUM =	79.280238			1120813.912			
Height (ft.) =	216.333														
Ec (ksi) =	4768.9622														
N-S Shear (k) =	200.182														
E-W Shear (k) =	260.336														

Frame #	Defl.	k	Concentric	Dist. To	k * D^2	kD/ΣkD^2	Torsional	Total
	(in.)		Shear (k)	C.R. (ft.)			Shear (k)	Shear (k)
N-S								
1	0.0369	27.100271	16.45889	52.32	74183.805	0.00082975	2.2415334	18.700423
2	0.0077	129.87013	78.874421	52.32	355504.21	0.003976335	10.741894	89.616315
3	0.0337	29.673591	18.021752	14.653	6371.2288	0.00025445	0.6873857	18.709138
4	0.015	66.666667	40.488869	124.93	1040500.3	0.004873954	13.166773	53.655642
				SUM =	1708811.1			
E-W								
5	0.03	33.333333	52.618548	91.11	276701.07	0.002709638	14.653613	67.272162
6	0.0484	20.661157	32.614803	23.276	11193.64	0.000429071	2.3204004	34.935203
7	0.0316	31.64557	49.954318	131.723	549080.66	0.003719127	20.112889	70.067207
				SUM =	1120813.9			
N-S Total k =	329.60828							
E-W Total k =	164.9203							
N-S Tor. Mom. =	2701.4561	(ft.-kips)						
E-W Tor. Mom. =	5407.9597	(ft.-kips)						

Level 14														
Shear Wall #	Length	Thickness	H/L	I	Aw	Deflection due to		Defl.	k	Concentric	Dist. To	k * D^2	kD/ΣkD^2	Torsional	Total
	(ft.)	(in.)		(in.^4)	(in.^2)	Flexure	Shear	(in.)		Shear	C.R. (ft.)			Shear (k)	Shear (k)
						(in.)	(in.)			(kips)					
N-S															
1	8.1667	12	1.12	941203.52	98.0004	0.375252138	0.0102073	0.3854594	2.5943068	2.040858566	60.43	9473.850314	4.79703E-05	0.3155255	2.356384
2	11.0833	12	0.83	2352615.8	132.9996	0.150125932	0.0075212	0.1576471	6.3432801	4.990056584	60.43	23164.29463	0.000117291	0.7714841	5.7615407
3	31	12	0.30	51478848	372	0.00686085	0.002689	0.0095499	104.71339	82.37468925	60.43	382390.7685	0.001936214	12.73548	95.110169
4	26.8333	16	0.34	44514831	429.3328	0.007934179	0.0023299	0.0102641	97.426806	76.64256704	52.32	266694.4233	0.001559713	10.25904	86.901607
5	9.1667	16	1.00	1774686	146.6672	0.199014716	0.0068203	0.205835	4.8582592	3.821837916	23.57	2698.981149	3.5038E-05	0.2304628	4.0523007
6	13	16	0.71	5061888	208	0.069774091	0.0048092	0.0745833	13.407825	10.54750898	23.57	7448.648668	9.66978E-05	0.6360312	11.18354
								SUM =	229.34386			3268146.106			
E-W															
7	28.75	16	0.32	54751500	460	0.006450757	0.0021746	0.0086254	115.93716	199.2836914	55.193	353175.5749	0.002656081	35.094183	234.37787
8	23.25	16	0.39	28956852	372	0.012197066	0.002689	0.0148861	67.176791	115.4697841	27.36	50286.50357	0.000762904	10.080076	125.54986
9	7.5	12	1.22	729000	90	0.484483724	0.0111146	0.4955984	2.0177629	3.468320548	89.307	16093.15265	7.4798E-05	0.9882893	4.4566099
10	24.1667	12	0.38	24389101	290.0004	0.014481413	0.0034494	0.0179308	55.770018	95.86275076	89.307	444807.1798	0.002067383	27.315853	123.1786
								SUM =	240.90173			2409158.764			

Height (ft.) =	143
Ec (ksi) =	4768.9622
N-S Shear (k) =	487.404
E-W Shear (k) =	636.055

Frame #	Defl.	k	Concentric	Dist. To	k * D^2	kD/ΣkD^2	Torsional	Total
	(in.)		Shear (k)	C.R. (ft.)			Shear (k)	Shear (k)
N-S								
1	0.0189	52.910053	41.622655	52.32	144835.05	0.000847041	5.571427	47.194082
2	0.0055	181.81818	143.03058	52.32	497705.89	0.002910741	19.145449	162.17603
3	0.0312	32.051282	25.213723	14.653	6881.7439	0.000143705	0.9452191	26.158942
4	0.0081	123.45679	97.119527	124.93	1926852.5	0.004719329	31.041467	128.16099
				SUM =	3268146.1			
E-W								
5	0.0283	35.335689	60.738304	91.11	293322.69	0.001336331	17.656641	78.394945
6	0.0447	22.371365	38.454005	23.276	12120.183	0.00021614	2.8558099	41.309815
7	0.014	71.428571	122.77814	131.723	1239353.5	0.003905424	51.601467	174.37961
				SUM =	2409158.8			

N-S Total k =	619.58017	
E-W Total k =	370.03736	
N-S Tor. Mom. =	6577.517	(ft.-kips)
E-W Tor. Mom. =	13212.771	(ft.-kips)

Level 6															
Shear Wall #	Length	Thickness	H/L	I	Aw	Deflection due to		Defl.	k	Concentric	Dist. To	k * D^2	kD/ΣkD^2	Torsional	Total
	(ft.)	(in.)		(in.^4)	(in^2)	Flexure	Shear	(in.)		Shear	C.R. (ft.)			Shear (k)	Shear (k)
						(in.)	(in.)			(kips)					
N-S															
1	31	18	0.30	77218272	558	0.000488211	0.0008504	0.0013386	747.05662	158.4733629	38.233	1092019.199	0.002449193	32.900429	191.37379
2	35.8333	18	0.26	119260167	644.9994	0.000316105	0.0007357	0.0010518	950.76904	201.686943	38.233	1389798.323	0.003117056	41.871938	243.55888
3	31	18	0.30	77218272	558	0.000488211	0.0008504	0.0013386	747.05662	158.4733629	74.517	4148243.531	0.004773534	64.123696	222.59706
4	9.1667	18	1.00	1996521.8	165.0006	0.018882235	0.0028758	0.021758	45.960021	9.749514023	45.767	96268.69969	0.00018037	2.4229381	12.172452
5	13	18	0.71	5694624	234	0.006620067	0.0020278	0.0086479	115.63519	24.52973059	45.767	242211.5873	0.00045381	6.0961007	30.625831
								SUM =	2606.4775			11661887.53			
E-W															
6	28.75	18	0.32	61595438	517.5	0.000612039	0.0009169	0.001529	654.0369	309.4392696	90.455	5351399.935	0.005135052	86.055617	395.49489
7	14.25	18	0.64	7500316.5	256.5	0.005026294	0.0018499	0.0068762	145.42844	68.80540016	62.622	570299.799	0.000790472	13.247096	82.052496
8	37.6667	18	0.24	138518480	678.0006	0.000272157	0.0006999	0.000972	1028.7819	486.7393664	54.045	3004929.869	0.004826017	80.87666	567.61603
								SUM =	1828.2472			11520994.34			

Height (ft.) =	67.8333
Ec (ksi) =	4768.9622
N-S Shear (k) =	757.652
E-W Shear (k) =	991.039

Frame #	Defl.	k	Concentric	Dist. To	k * D^2	kD/ΣkD^2	Torsional	Total
	(in.)		Shear (k)	C.R. (ft.)			Shear (k)	Shear (k)
N-S								
1	0.0093	107.52688	22.809712	74.517	597073.47	0.000687074	9.2295829	32.039295
2	0.0037	270.27027	57.33252	74.517	1500752.2	0.00172697	23.198681	80.531201
3	0.0272	36.764706	7.7989089	36.85	49923.621	0.000116172	1.560552	9.359461
4	0.0042	238.09524	50.50722	102.733	2512873.6	0.002097451	28.175416	78.682636
5	0.0032	312.5	66.290726	10.233	32723.215	0.000274211	3.6835168	69.974243
				SUM =	11661888			
E-W								
6	0.0248	40.322581	19.077501	126.372	643946.87	0.000442292	7.4121392	26.489641
7	0.0383	26.109661	12.353056	58.539	89472.964	0.000132665	2.2232634	14.576319
8	0.005	200	94.624407	96.461	1860944.9	0.001674526	28.062486	122.68689
				SUM =	11520994			

N-S Total k =	3571.6346	
E-W Total k =	2094.6794	
N-S Tor. Mom. =	13433.17	(ft.-kips)
E-W Tor. Mom. =	16758.469	(ft.-kips)

Center of Rigidity Calculations

Center of Rigidity				
Level 25				
Shear Wall #	Stiffness, k	X,Y (ft.)	(X,Y)*k	
N-S				
1	18.18	42	763.56	
2	0.634	42	26.628	
3	1.613	42	67.746	
4	0.617	126.5	78.0505	
5	1.742	126.5	220.363	
SUM =	22.786		1156.3475	
E-W				
6	17.459	35.9167	627.06967	
7	9.538	63.75	608.0475	
SUM =	26.997		1235.1172	
Xcr (ft.) =	45.7501635			
Ycr (ft.) =	50.7481568			

Levels 8-22				
Shear Wall #	Stiffness, k	X,Y (ft.)	(X,Y)*k	Dist. To
				C.R. (ft.)
N-S				
1	0.437	64.5	28.1865	60.429952
2	1.085	64.5	69.9825	60.429952
3	21.564	64.5	1390.878	60.429952
4	19.154	177.25	3395.0465	52.320048
5	0.823	148.5	122.2155	23.570048
6	2.323	148.5	344.9655	23.570048
E-W				
7	23.279	35.9167	836.10486	55.19285
8	12.717	63.75	810.70875	27.35955
9	0.339	180.4167	61.161261	89.30715
10	10.658	180.4167	1922.8812	89.30715
Frame #				
N-S				
1	27.1	177.25	4803.475	52.320048
2	129.87	177.25	23019.458	52.320048
3	29.674	139.5833	4141.9948	14.653348
4	66.667	0	0	124.92995
E-W				
5	33.333	0	0	91.10955
6	20.661	67.8333	1401.5038	23.27625
7	31.646	222.833	7051.7731	131.72345
SUM N-S k =	298.697			
SUM E-W k =	132.633			
SUM N-S k*d =	37316.2018			
SUM E-W k*d =	12084.133			
Xcr (ft.) =	91.1095503			
Ycr (ft.) =	124.929952			

Levels 6				
Shear Wall #	Stiffness, k	X,Y (ft.)	(X,Y)*k	Dist. To
				C.R. (ft.)
N-S				
1	747.057	64.5	48185.177	38.233064
2	950.769	64.5	61324.601	38.233064
3	747.0566	177.25	132415.78	74.516936
4	45.96	148.5	6825.06	45.766936
5	115.635	148.5	17171.798	45.766936
E-W				
7	654.037	35.9167	23490.851	90.455379
8	145.428	63.75	9271.035	62.622079
9	1028.782	180.4167	185609.45	54.044621
Frame #				
N-S				
1	107.527	177.25	19059.161	74.516936
2	270.27	177.25	47905.358	74.516936
3	36.765	139.5833	5131.78	36.850236
4	238.095	0	0	102.73306
5	312.5	92.5	28906.25	10.233064
E-W				
6	40.323	0	0	126.37208
7	26.11	67.8333	1771.1275	58.538779
8	200	222.833	44566.6	96.460921
SUM N-S k =	3571.6346			
SUM E-W k =	2094.68			
SUM N-S k*d =	366924.965			
SUM E-W k*d =	264709.067			
Xcr (ft.) =	126.372079			
Ycr (ft.) =	102.733064			

Appendix C

Shear Wall Design Checks

Level	25
Wall #	7
Reinforcement:	#4@12"
Shear Force (k) =	120.14
f'c (ksi) =	5
fy (ksi) =	60
Thickness (in.) =	12
Length (ft.) =	28.75
Av (in^2) =	0.2
Spacing (in.) =	12
Height (ft.) =	10.8333

Vn, max = 25370.91 kips

Vc = 5074.18 kips
Vs = 276.00 kips

Vn = Vc + Vs

Vn = 5350.18 kips

Factored Vu = 192.224 kips

ΦVn = 4012.64 kips > Vu = 192.22 kips

Level	22
Wall #	7
Reinforcement:	#5@12"
Shear Force (k) =	72
f'c (ksi) =	5
fy (ksi) =	60
Thickness (in.) =	16
Length (ft.) =	28.75
Av (in^2) =	0.31
Spacing (in.) =	12
Height (ft.) =	9.1667

Vn, max = 21467.84 kips

Vc = 4293.57 kips
Vs = 427.80 kips

Vn = Vc + Vs

Vn = 4721.37 kips

Factored Vu = 115.2 kips

ΦVn = 3541.03 kips > Vu = 115.2 kips

Level	14
Wall #	7
Reinforcement:	#5@12"
Shear Force (k) =	234.4
f'c (ksi) =	5
fy (ksi) =	60
Thickness (in.) =	16
Length (ft.) =	28.75
Av (in^2) =	0.31
Spacing (in.) =	12
Height (ft.) =	9.1667

Vn, max = 21467.84 kips

Vc = 4293.57 kips
Vs = 427.80 kips

Vn = Vc + Vs

Vn = 4721.37 kips

Factored Vu = 375.04 kips

ΦVn = 3541.03 kips > Vu = 375.04 kips

Level	6
Wall #	8
Reinforcement:	#5@12"
Shear Force (k) =	567.6
f'c (ksi) =	5
fy (ksi) =	60
Thickness (in.) =	18
Length (ft.) =	28.75
Av (in^2) =	0.31
Spacing (in.) =	12
Height (ft.) =	11

Vn, max = 25761.31 kips

Vc = 5152.26 kips
Vs = 427.80 kips

Vn = Vc + Vs

Vn = 5580.06 kips

Factored Vu = 908.16 kips

ΦVn = 4185.05 kips > Vu = 908.16 kips

Moment Frame Design Checks

Level	22
Frame #	2
Reinforcement:	6#9
Moment (ft-k) =	198
f'c (ksi) =	5
fy (ksi) =	60
Width (in) =	36
Length (ft.) =	14
As (in^2) =	6
d (in) =	34

d = 31.00 in.

a = 2.73 kips

Mn = 10668.16 ft-kips

ΦMn = 9601.34 ft-kips > Mu = 198 ft-kips

Level	14
Frame #	7
Reinforcement:	4#7
Moment (ft-k) =	560
f'c (ksi) =	5
fy (ksi) =	60
Width (in) =	13
Length (ft.) =	14.75
As (in^2) =	2.4
d (in) =	34

d = 31.00 in.

a = 1.09 kips

Mn = 4385.31 ft-kips

ΦMn = 3946.77 ft-kips > Mu = 560 ft-kips

Level	6
Frame #	8
Reinforcement:	2#7
Moment (ft-k) =	403
f'c (ksi) =	5
fy (ksi) =	60
Width (in) =	14
Length (ft.) =	14.75
As (in^2) =	1.2
d (in) =	24

d = 21.00 in.

a = 0.81 kips

Mn = 1482.96 ft-kips

ΦMn = 1334.66 ft-kips > Mu = 403 ft-kips

Overturning Moment Calculations

Overturning Moment Calculations				
Level	Height	Story	Moment	
	(ft.)	Shear (k)	(ft-kips)	
Roof	260.5	28.796	7501.358	
25	249.6667	56.441	14091.438	
24	239	57.324	13700.436	
23	228.3333	60.903	13906.183	
22	216.3333	56.872	12303.307	
21	207.1667	49.263	10205.653	
20	198	47.81	9466.38	
19	188.8333	47.374	8945.7888	
18	179.6667	46.976	8440.0229	
17	170.5	46.665	7956.3825	
16	161.3333	46.312	7471.6678	
15	152.1667	45.721	6957.2137	
14	143	45.539	6512.077	
13	133.8333	44.776	5992.5198	
12	124.6667	44.776	5582.0762	
11	115.5	43.595	5035.2225	
10	106.3333	43.595	4635.6002	
9	97.1667	42.617	4140.9533	
8	88	41.891	3686.408	
7	78.8333	45.345	3574.696	
6	67.8333	48.389	3282.3856	
5	56.8333	45.731	2599.0436	
4	46.5	51.5	2394.75	
3	36.1667	55.869	2020.5974	
2	18.6667	66.365	1238.8155	
		SUM =	171640.98	
Total Overturning Moment =			171640.98	ft- kips

- Only the East-West direction will be considered- this is the critical direction.
- Only the controlling wind loads will be used. Seismic will not be considered.

- Total Overturning Moment = 171,641 ft-kips
- Resisting Moment = Building Weight * Distance to the Center of Mass
    - Total Building Weight = 68888.25 kips
        - (see Technical Report 1 for calculations)
    - Distance to the Center of Mass = 70.337 ft.
        - (approximate of the center of mass taken at Level 8)

- Resisting Moment = 68888.25 * 70.337 = 505,433 ft-kips > 171,641 ft-kips

- Overturning will not be a problem, because the resisting moment is much greater than the overturning moment.

Story Drift and Total Drift Calculations

Story Drift and Total Drift Values			
Level	Height	Story Drift	Allowable Drift
	(ft.)	(in.)	(in.)
Roof	260.5	0.2272	0.216666
25	249.6667	0.2203	0.213334
24	239	0.2153	0.213334
23	228.3333	0.2046	0.24
22	216.3333	0.1936	0.183332
21	207.1667	0.1855	0.183334
20	198	0.1771	0.183334
19	188.8333	0.1686	0.183332
18	179.6667	0.1599	0.183334
17	170.5	0.1512	0.183334
16	161.3333	0.1424	0.183332
15	152.1667	0.1335	0.183334
14	143	0.1246	0.183334
13	133.8333	0.1157	0.183332
12	124.6667	0.1068	0.183334
11	115.5	0.0981	0.183334
10	106.3333	0.0894	0.183332
9	97.1667	0.0809	0.183334
8	88	0.0727	0.183334
7	78.8333	0.0663	0.22
6	67.8333	0.055	0.22
5	56.8333	0.0439	0.206666
4	46.5	0.0333	0.206666
3	36.1667	0.0235	0.35
2	18.6667	0.0093	0.373334
	SUM =	3.0987	
Total Drift =	3.0987	in.	
Total Allowable Drift =		5.21	in.