The Residences of Sherman Plaza
Evanston, IL

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Structural Technical Report 1
Structural Concepts / Structural Existing Conditions Report

Executive Summary

The structural existing conditions of Sherman Plaza documented in this report have been compiled to provide an overview of the building's structural system, including information about the required loading, design criteria and assumptions. This report is divided into the following main sections: the structural system description, design codes and assumptions, and calculations.

Sherman Plaza, located in downtown Evanston, IL, is a 25 story condominium that includes a 54,000 square foot health club, a half acre rooftop garden, 152,000 square feet of retail space, and a new adjoining 1,585 car parking garage. The main structural system of the residential tower is reinforced cast-in-place concrete columns, shear walls, beams and slabs. The building is supported by a foundation of belled caissons. Lateral loads are resisted by both shear walls and perimeter moment frames.

The overarching design code for Sherman Plaza is BOCA 1996. Other standards used in the design of the building that are referenced in BOCA 1996 are ASCE 7-98, AISC LRFD 2nd Edition 1994, and ACI 318-95. The building was originally designed using older versions of the code, but the new calculations will utilize the newest version.

The existing conditions of the building were analyzed by calculating the gravity and lateral loads and by performing spot checks on typical structural elements. The wind loads and seismic loads were calculated using ASCE 7-02. For this report, the moment frames and shear walls will be considered separately in the lateral element check. A full lateral load distribution will be completed for Technical Assignment 3. A portal method analysis was used to analyze a perimeter moment frame, and a shear wall was analyzed by distributing the direct and eccentric forces to each wall. The

gravity loads were tabulated in an Excel spreadsheet. Spot checks for gravity loads were performed on typical floor framing members.

The calculated checks of the gravity and lateral elements produced very similar results to their actual design. A typical two-way slab was chosen to analyze and it was found that it required 13#6 top bars in the short span column strip and 13#6 top bas in the long span column strip. The actual design called for 12#6 bars in the long span and 10#6 bars in the short span. The rest of the slab required the minimum design value of #5@12" bars. The calculated reinforcement is very similar to the actual design. Next, a perimeter beam was checked for flexural and torsional strength. It was found that the beam required 3#7 top and bottom bars, which is smaller than the actual beam design of 4#7 top and bottom bars. This beam, however, will also take lateral load, because it is a part of the perimeter moment frame. Therefore, it will probably require more reinforcement to take this lateral load.

The portal method was used to analyze one of the perimeter moment frames, and it was found that the perimeter beam required 4#8 top and bottom bars. This design is slightly higher than the actual design of 4#7 top and bottom bars. The calculated design, however, was conservative, because the moment frames were made to take all the lateral load, without consideration of the shear walls. A shear wall was also checked by distributing the lateral loads according to wall rigidity. The wall was found to require #5@12" reinforcement, which was the same as the actual design of the wall.

Building Description

Introduction

The main structural system of the Residences of Sherman Plaza is a reinforced cast-in-place concrete superstructure. This 25 story condominium is a complex building comprised of concrete columns, beams, two-way slabs, and shear walls. This report provides a description of the existing physical conditions of the structure of the building. It is intended to give a detailed introduction the building's structural system, including information about the required loading, design criteria and assumptions.

This report includes descriptions of the building's foundation, floor framing, column grid, and lateral resisting system. The building's design codes are listed for gravity and lateral conditions. In addition, elements of the structure are analyzed by spot checks of typical floor framing under gravity loads and a check of a lateral element. The gravity and lateral loads for the entire building have also been calculated. This information will be used in further study of this building and in a proposal for a building redesign.

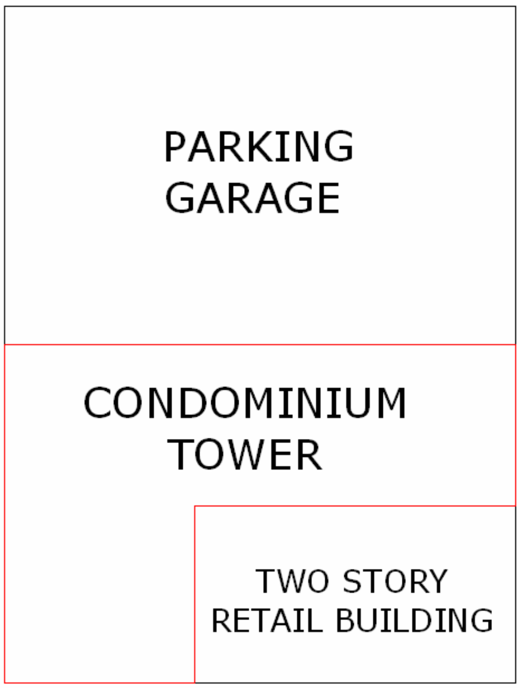
Architecture

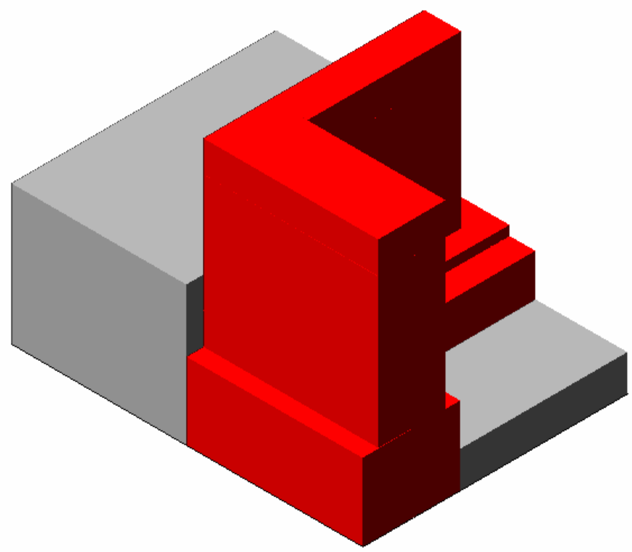
The Residences of Sherman Plaza are located in downtown Evanston, IL and provide residents with luxurious condominiums and many amenities, including a 54,000 square foot health club, a half acre rooftop garden, 152,000 square feet of retail space, and a new adjoining 1,585 car parking garage. The parking garage is structurally separate from the condominium tower and therefore, will not affect the structure of the tower. The parking garage will, therefore, be excluded from this study.

The building has a two story rectangular base containing the retail spaces and is topped by at 23 story L-shaped condominium tower. The building steps back on the third, sixth and seventh floors and the roofs of these floors are covered by an intensive garden. The top three stories are also stepped back and have large cast-in-place concrete "eyebrows" covering the balconies.

Structural System

Project Scope
For this report, only the L-shaped condominium tower will be considered in the building analysis. The parking garage and the two story retail building are structurally separate from the condominiums and will therefore not impact the analysis of the tower.



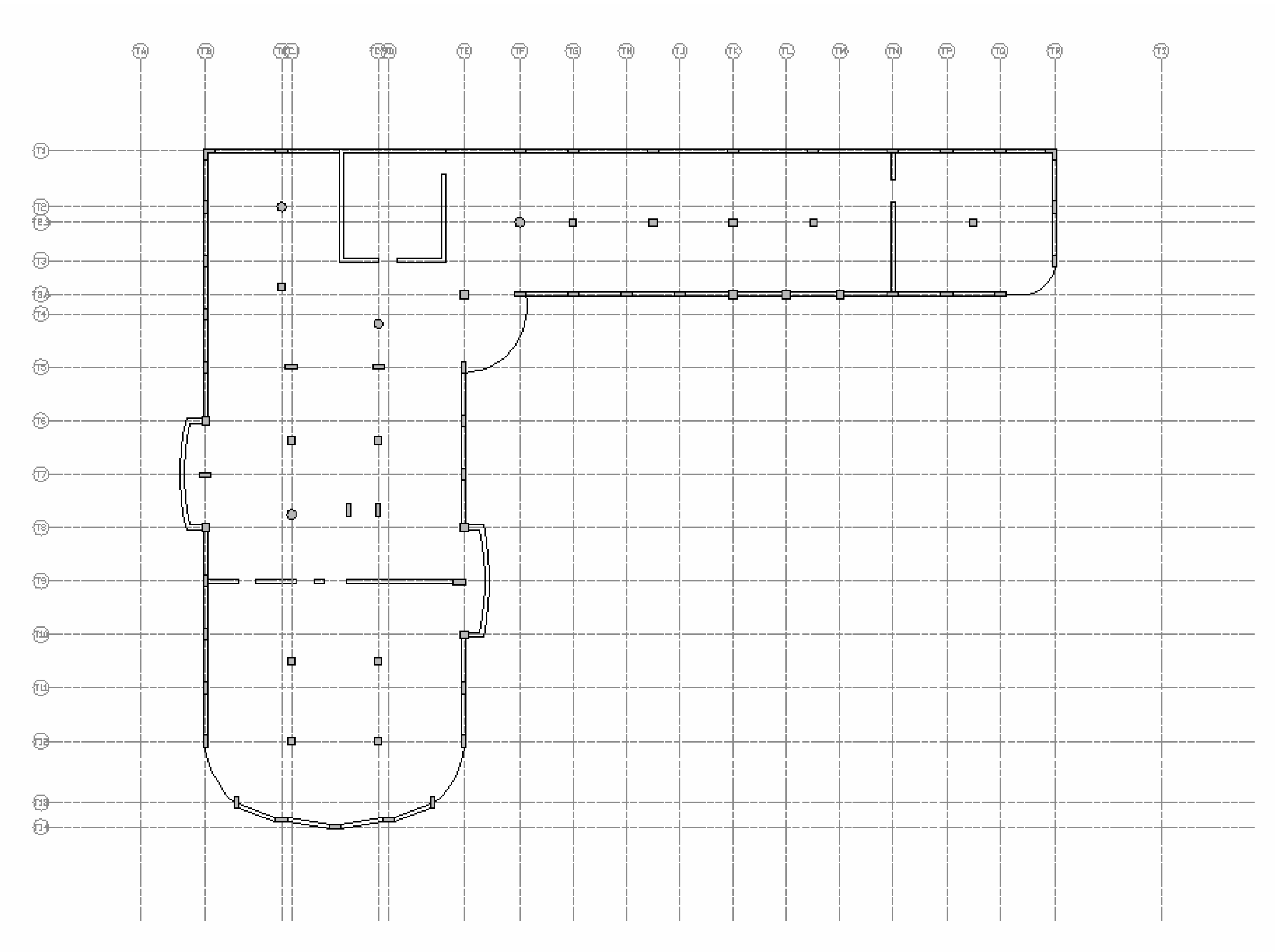


Plan View and 3D View of Sherman Plaza (the area to be considered in this report is in red.)

Floor System
The primary floor system of Sherman Plaza is reinforced concrete two-way slabs. There are some one-way slabs in irregular areas. The slab thickness of every floor is 8" with the exception of the first retail floor, which has a slab thickness of 9". The building is surrounded by perimeter edge beams, and there are interior edge beams surrounding slab openings for stairs and elevator shafts. The third and seventh floor framing has additional beams to account for the large loads due to the green roofs on those levels as the building steps back. The twenty-third floor framing also has large transfer girders to account for the change in the column locations for the penthouse levels.

The slab reinforcement remains fairly constant from floor to floor on the stories above the two retail floors. The bay sizes, however, differ throughout the plan, which causes the reinforcement size to change throughout a floor. The slab is required to have a minimum of #6@12" top reinforcement at column strip intersections, #5@12" bottom reinforcement at middle strip intersections, and #5@12" top and bottom reinforcement at intersections of the column strip and middle strip. The typical floor of the building begins on level 8, and this floor plan is continued up to floor 22. The last three floors differ, because they are penthouse levels.

Column Grid
In general, the columns are lined up along a grid, but the spacing of the columns varies. Most bays are either 14'x14' or 21'x21' square bays. Column sizes on the ground floor vary from 18"x54" on the building perimeter to 36"x36" as a typical interior column size. Column sizes differ on the upper floors and vary between a 20" diameter circular column, a 24"x24" square interior column and a 13"x36" on the perimeter.



Typical Floor Plan: Levels 8-22

Mechanical Levels/Roof
The building is topped by a two story mechanical penthouse. The columns in the mechanical levels fall along the same column grid as the lower floors. The slab is also reinforced as the typical concrete two-way slab, with #5@12" top and bottom

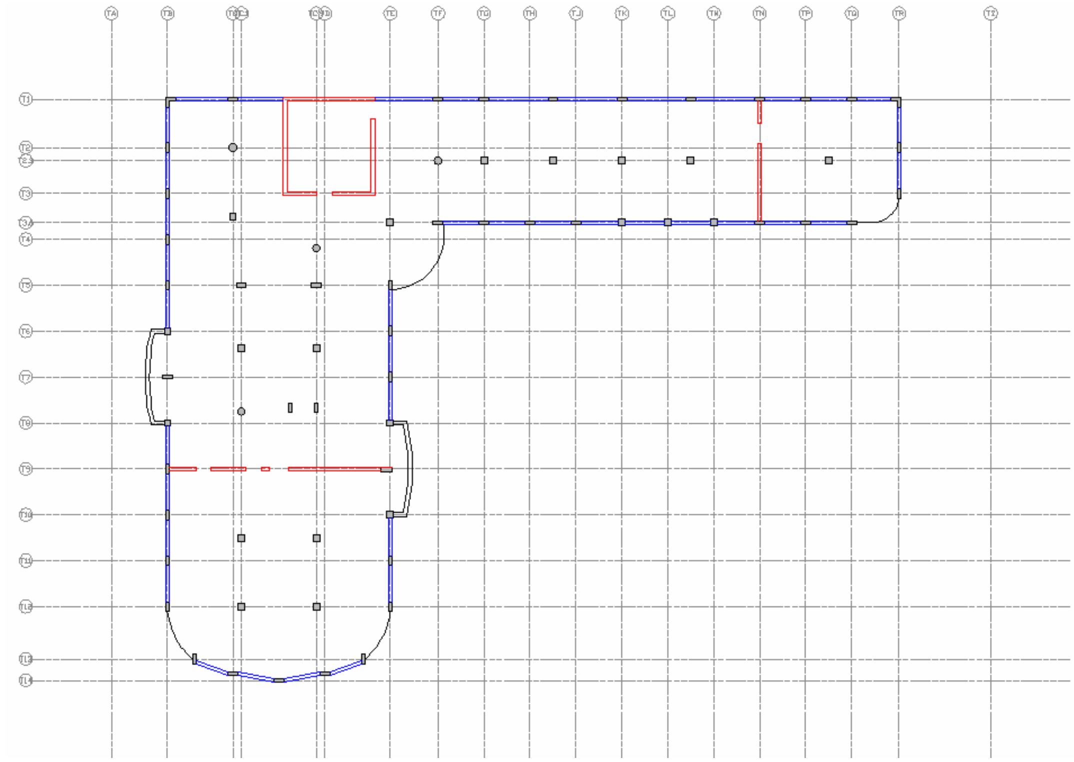
continuous reinforcement. In addition, all mechanical, plumbing, and electrical equipment pads must be reinforced with a minimum of one layer of 6x6-W4.0xW4.0 W.W.F. The roof framing also has spandrel beams between columns to support additional mechanical equipment.

Foundation

The foundation of Sherman Plaza consists of reinforced concrete belled-caissons, extending to hardpan at approximately 70 feet below grade. All the caissons will bear on hardpan soil strata with a minimum allowable bearing capacity of 30 ksf, except where the drawings indicate a minimum of 50 ksf. The largest caissons have a 15’-6” bell diameter and a 6’-0” shaft diameter in size and are spaced at 28’-0”, in general. The sizes vary down to a 6’-0” bell diameter and 2’-6” shaft diameter, spaced at either 14’-0” or 21’-0”, in general. Above the caissons is a 5” slab-on-grade with one layer of 6x6-W2.1xW2.1 W.W.F. Grade beams are located underneath the building’s shear walls.

Lateral Resisting System

The lateral support for the building is made up of a combination of reinforced concrete shear walls and perimeter moment frames. 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”x36” beam with 4 #7 reinforcement bars on top and bottom.



Typical Plan with Lateral Elements (shear walls are in red and moment frames are in blue)

Materials

Structural Concrete

Location	Concrete Compressive Strength (28-day)	Concrete Type
Drilled Piers (Caissons)	5000 psi	145 pcf Normal Weight
Foundation Walls and Piers/Pilasters	5000 psi	145 pcf Normal Weight
Shear Walls & Link Beams	5000-7000 psi	145 pcf Normal Weight
Caisson Caps & Grade Beams	5000 psi	145 pcf Normal Weight
Framed Slabs and Beams	5000 psi	145 pcf Normal Weight
Columns	5000-8000 psi	145 pcf Normal Weight
Slabs-on-Grade	4000 psi	145 pcf Normal Weight
Composite Steel Deck	4000 psi	145 pcf Normal Weight
Structural Curbs and Barrier Walls	4000 psi	145 pcf Normal Weight
Architectural Fills and Equipment Pads	4000 psi	145 pcf Normal Weight

Structural Steel

Condition	Grade and Yield Strength
Wide Flanges	ASTM A992 (Fy = 50 ksi)
Other Rolled Shapes	ASTM A572, Grade 50 (Fy=50 ksi)
Plate Girders, Misc. Plates, Base Plates	ASTM A36 (Fy = 36 ksi)
Connection Angles Sized on the Drawings	ASTM A36 (Fy = 36 ksi)
Connection Materials Sized by the Steel Fabricator	ASTM A36 or A572, Grade 50
Miscellaneous	ASTM A36 (Fy = 36 ksi)

Design Codes

Although construction for Sherman Plaza began in December 2004, the building actually began to be designed about five years earlier. The project was originally cancelled due to lack of funding but was later restarted. Therefore, the building was designed using older versions of the codes. The new calculations of the existing conditions, however, will use the most current versions of the code. Although seismic loading was not considered when designing the building, this report will check the seismic loading.

Building Code: BOCA 1996
Minimum Design Loads: ASCE 7-98
Structural Steel Design: AISC LRFD Second Edition, 1994
Structural Concrete Design: ACI 318-95
Masonry Design: ACI 530-95/ASCE 5-95/TMS 402-95
Seismic Design: Exempted
Wind Design: ASCE 7-98

Gravity Loads

The gravity loads given below are the loads used in the actual design of Sherman Plaza by the structural engineers. These loads follow the provisions of ASCE 7-98.

Code Minimum Live Loads (PSF)

1.	Minimum Roof Load:	20
2.	Parking Floor:	50
3.	Public Spaces:	100
4.	Corridors (1st Floor):	100
5.	Corridors (above 1st Floor):	80
6.	Residential	40
7.	Kitchen/Dining	100
8.	Lobbies	100
9.	Retail	100
10.	Light Storage	125
11.	Heavy Storage	250
12.	Canopies	75
13.	Sidewalks, Driveways, & Yards	250
14.	Stairs and Exit Ways	100
15.	Balconies/Terraces (Exterior)	100
16.	Partitions	20
17.	CMU Partitions	Actual Load to be Calculated.

Ceiling, Mechanical, Electrical and Plumbing 10

Green Roof:

1.	Plants and Soil	200
2.	Tree Planter Box	500

Lateral Loads

Structural Wind Loads

The following components were designed according to ASCE 7-98:

1. Main Wind Force Resisting System
2. Components and Cladding
3. Roof
4. Canopies, balconies, and parapets.

Seismic Loads

Provisions for seismic design currently do not apply in Evanston, and therefore were not used in the actual design of this building. This report, however, will perform a seismic analysis based on ASCE 7-02.

Snow Loads

Flat Roof Snow Load (Pf)	30 psf

Drifts on Roof: See ASCE 7-98, Section 7

Calculations

Superimposed Loads

The following loads will be used in the analysis of Sherman Plaza in this report. The superimposed live load for the typical residential floor will be assumed to be 80 psf to account for the 40 psf residential loading and 100 psf loading for kitchens, dining rooms, balconies and stairways, and an additional 20 psf for partitions. The superimposed dead load will account for the mechanical equipment, ceiling and floor finishes and other miscellaneous dead loads.

Dead Load =	15 psf
Live Load =	80 psf

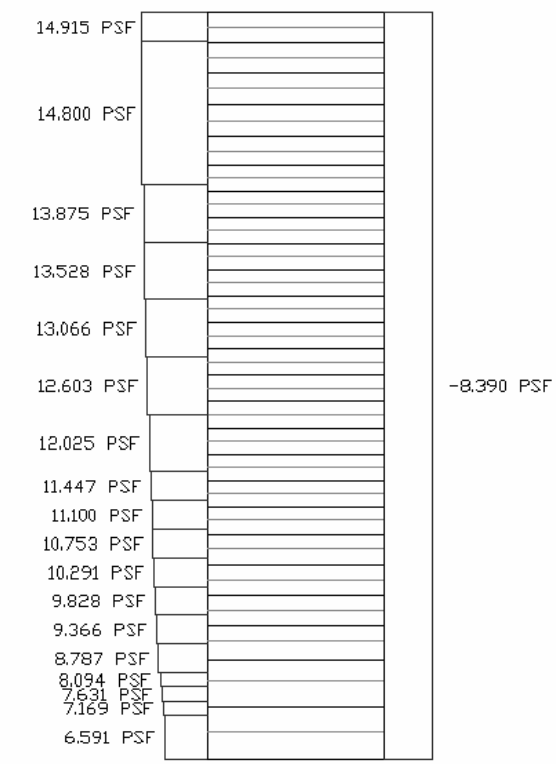
Total Superimposed Service Load = **95 psf**
Total Superimposed Factored Load = **146 psf**

Wind Load Calculations

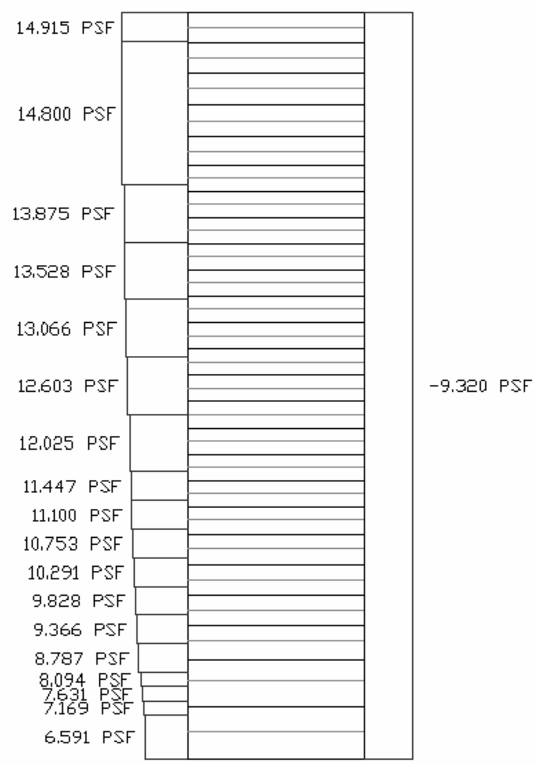
The wind loads for Sherman Plaza were calculated using ASCE 7-02. Refer to the Appendix for the full calculations and assumptions for the wind load.

Assumptions

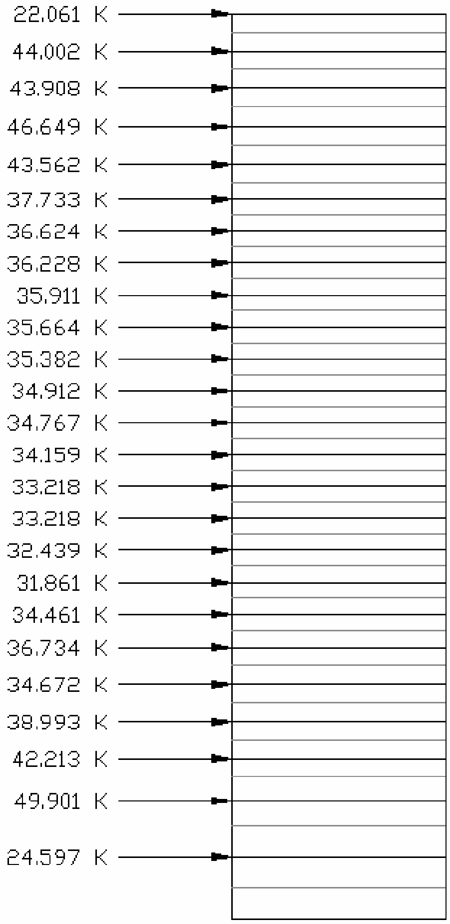
Basic Wind Speed (Evanston, IL):	90 mph
Occupancy Category:	Category II
Exposure Category:	Category B
Enclosure:	Enclosed
Rigidity:	Each story acts as a rigid diaphragm.
Shape:	Approximated as a rectangle.



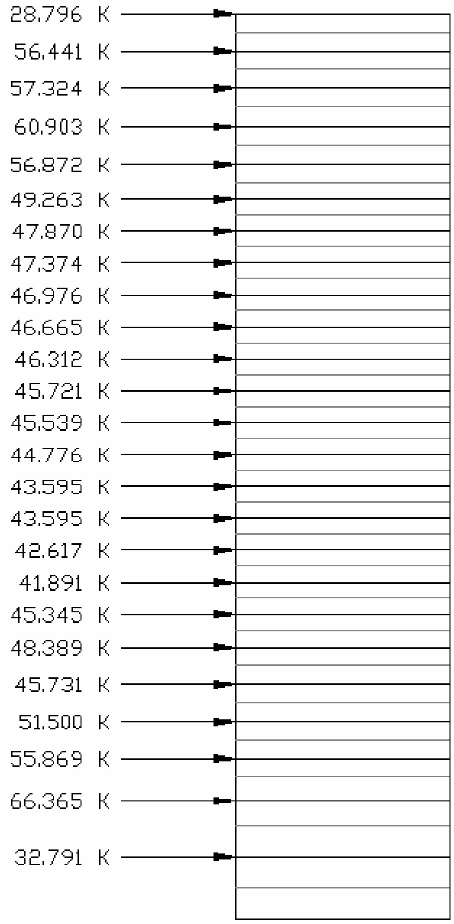
North-South Wind Pressure Diagram
Windward and Leeward Pressures



East-West Wind Pressure Diagram
Windward and Leeward Pressures



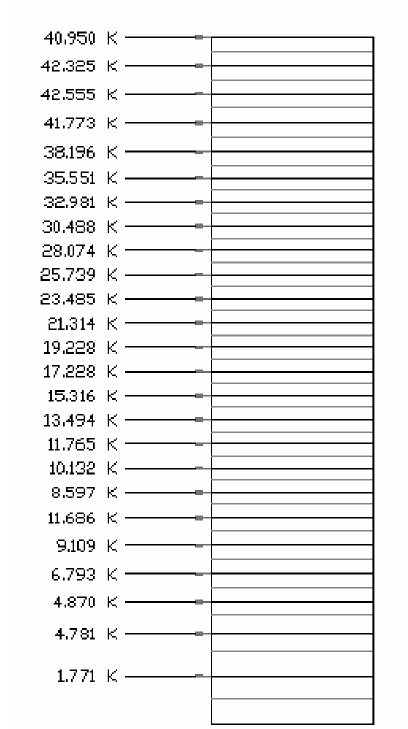
North-South Story Forces



East-West Story Forces

Seismic Load Calculations

The seismic forces on Sherman Plaza were calculated using ASCE 7-02. Refer to the Appendix for the full calculations and seismic assumptions.



Seismic Story Forces

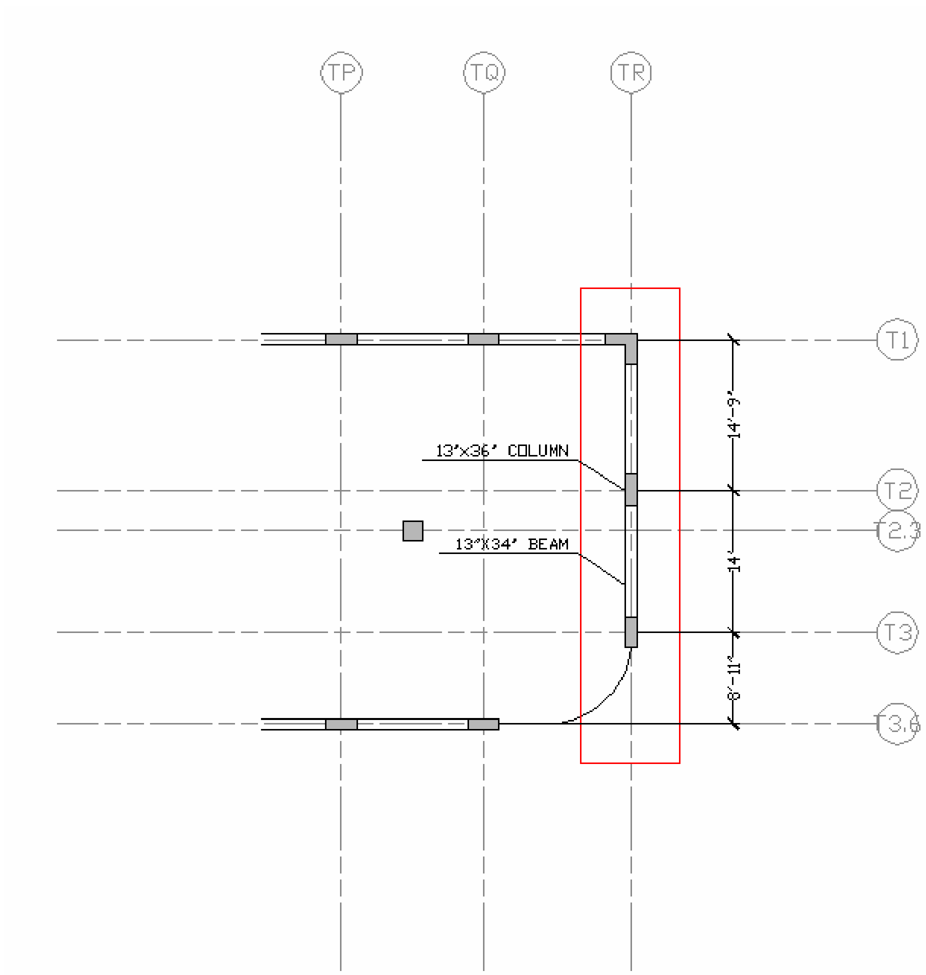
Lateral Element Analysis

The lateral elements in Sherman Plaza will be designed for wind loads only, because the seismic loads will not control. The wind loads will receive a load factor of 1.6, while the seismic loads will receive a factor of 1.0. A comparison between the factored seismic and wind story forces can be seen in Appendix A, and for every story, the wind forces are considerably larger than the seismic forces.

Since the frames and shear walls both resist the lateral forces, the forces need to be distributed to each element according to its stiffness. The elements with the highest stiffness will receive the most load. For this report, a simplified method will be used, and the shear walls and frames will be analyzed separately. The moment frame will be analyzed using the portal method, and the shear walls will be analyzed by distributing the lateral loads based on the relative rigidities of each wall. The lateral resisting elements will be considered in more detail in Technical Assignment 3, and a full analysis of the distribution of loads will be performed. The values found in this report will be conservative, however, because the load is not being distributed to all lateral resisting elements.

The portal method was used to analyze one of the perimeter moment frames in the building, as seen in the figure below. The final design of the beams included the lateral and gravity loads, and it was determined that the beam required 4#8 top and bottom bars. This calculated design is very close to the actual beam design, which called for 4 #7 bars, top and bottom, and 2#4 side bars. The side bars are necessary when designing this beam for torsion, because the reinforcement must be spaced less than 12” apart. It makes sense that the calculated design is higher than the actual design, because this frame will receive less load when the lateral forces are distributed to both the shear walls and the frames. Refer to the Appendix for the full calculations and assumptions of the moment frame.

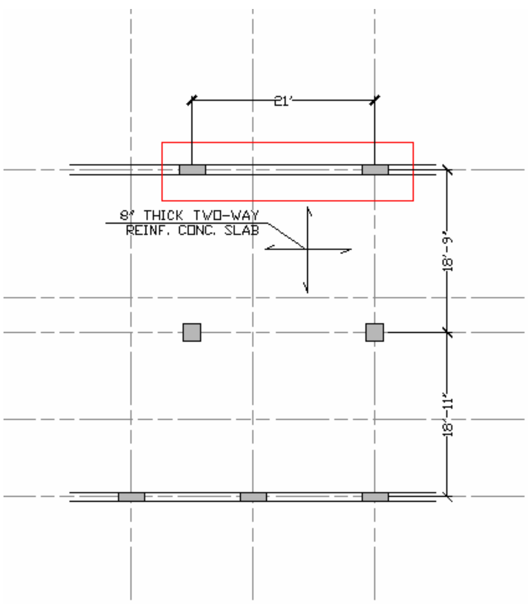
A shear wall analysis was also performed by finding the total shear force on each wall due to concentric and eccentric forces. The direct forces were found by computing the rigidity of each wall and proportioning the shear force out according to which wall was the most rigid. The wall with the highest rigidity receives the most load. The eccentric forces were calculated by finding the building’s center of rigidity, based on the individual wall rigidities. The eccentric force was then found by multiplying a ratio based on the wall rigidity and distance to the center of rigidity by the building’s torsional moment due to the eccentricity. Shear wall #2 on the eighth floor was found to carry the most load, so a check of that wall was performed. It was found that the wall required #5@12” reinforcement, which is the same as the actual design of the wall.



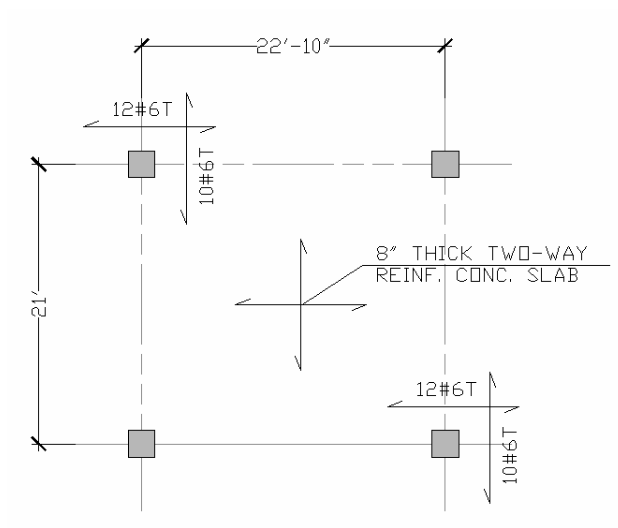
Frame Used for Portal Frame Analysis

Floor Framing Analysis

Spot checks have been performed for typical framing members and lateral resisting elements to verify that the design assumptions made in this report were accurate. The spot checks were performed on typical members on the 8th floor, where the building's typical floor plan begins. A typical interior bay was chosen for the analysis of the two-way slab, and a typical exterior beam was chosen for analysis. Refer to the Appendix for the full calculations and assumptions.



Typical Exterior Beam



Typical Bay Used for Slab Check

Beam/Slab Check

The two-way slab on the 8th floor was checked for a typical bay. It was found that the slab required 12#6 top bars in the column strip and #5@12" reinforcement at the column strip midspan of the short direction. In the long span direction, 13#6 top bars are needed at the column strip, and #5@12" bars are sufficient for the column strip midspan top and bottom. The middle strip required the minimum reinforcement of #5@12" that is required by the building's actual design. The calculated reinforcement is slightly higher than the actual design. The actual design called for 12#6 bars in the long span direction and 10#6 bars in the short direction. The calculated values are slightly higher, which means that they are more conservative.

Next, a perimeter beam was checked for flexural and torsional strength. It was found that the beam required 3#7 bottom bars, 3#7 top bars and #4 stirrups @ 12". Side bars were also required. The actual design of the beam, however, found that it required 4 #7 bottom bars, 4 #7 top bars, #4 stirrups @ 12”, and 2#4 side bars. It can be seen that the design reinforcement is somewhat larger than the calculated reinforcement. This difference is due to the fact that a large portion of the moment on the beam is due to the lateral forces, since this beam is used as a part of the perimeter moment frame as well. Therefore, it makes sense that the reinforcement is smaller when only designed for flexure. Refer to the portal frame analysis for the combined lateral and flexural moment design.

Appendix A

Wind Load Calculation using ASCE 7-02	
Basic Wind Speed, V = 90 mph	Figure 6-1
Wind Directionality Factor, Kd = 0.85	Table 6-4
Importance Factor, I = 1.0	Table 6-1
Exposure Category: B	Section 6.5.6.3
Total Building Height: 260.5 feet	
Velocity Pressure Exposure Coefficient, Kz	Table 6-3
Topographic Factor, Kzt = 1.0	Section 6.5.7
Gust Effect Factor, G = 0.82	Section 6.5.8
Enclosure Classification: Enclosed	Section 6.5.9
Internal Pressure Coefficient, GCpi = +/- 0.18	Figure 6-5
External Pressure Coefficient, Cp	Figure 6-6
See Calculations on Next Page	
Velocity Pressure, qz	Section 6.5.10
Design Wind Load, p	Section 6.5.12

Design Wind Pressure								
			N-S Wind, p (psf)		Total MWFRS	E-W Wind, p (psf)		Total MWFRS
Height (ft)	Kz	qz (psf)	Windward	Leeward	(psf)	Windward	Leeward	(psf)
0-15	0.57	10.047	6.591	-8.39	14.981	6.591	-9.32	15.911
20	0.62	10.928	7.169	-8.39	15.559	7.169	-9.32	16.489
25	0.66	11.633	7.631	-8.39	16.021	7.631	-9.32	16.951
30	0.7	12.338	8.094	-8.39	16.484	8.094	-9.32	17.414
40	0.76	13.395	8.787	-8.39	17.177	8.787	-9.32	18.107
50	0.81	14.277	9.366	-8.39	17.756	9.366	-9.32	18.686
60	0.85	14.982	9.828	-8.39	18.218	9.828	-9.32	19.148
70	0.89	15.687	10.291	-8.39	18.681	10.291	-9.32	19.611
80	0.93	16.392	10.753	-8.39	19.143	10.753	-9.32	20.073
90	0.96	16.921	11.100	-8.39	19.490	11.100	-9.32	20.420
100	0.99	17.449	11.447	-8.39	19.837	11.447	-9.32	20.767
120	1.04	18.331	12.025	-8.39	20.415	12.025	-9.32	21.345
140	1.09	19.212	12.603	-8.39	20.993	12.603	-9.32	21.923
160	1.13	19.917	13.066	-8.39	21.456	13.066	-9.32	22.386
180	1.17	20.622	13.528	-8.39	21.918	13.528	-9.32	22.848
200	1.2	21.151	13.875	-8.39	22.265	13.875	-9.32	23.195
250	1.28	22.561	14.800	-8.39	23.190	14.800	-9.32	24.120
260.5	1.29	22.737	14.915	-8.39	23.305	14.915	-9.32	24.235
Internal Building Pressure, p = qi*Gcpi =+/- 4.09 psf								

North- South Wind Story Shear Forces														
Level	Tributary Height (ft)	Total Height (ft)	P1 (psf)	H1 (ft)	P2 (psf)	H2 (ft)	P3 (psf)	H3 (ft)	P3 (psf)	H4 (ft)	Story Dist. Load (plf)	Cum. Dist. Load (plf)	Story Shear (kips)	Cum. Shear (kips)
Roof	5.333	260.171	23.305	5.333							124.286	5340.997	22.061	22.061
25	10.667	254.838	23.19	5.833	23.305	4.833					247.900	5216.711	44.002	66.063
24	10.667	244.171	23.19	10.667							247.368	4968.811	43.908	109.971
23	11.333	233.504	23.19	11.333							262.812	4721.443	46.649	156.620
22	10.583	222.171	23.19	10.583							245.420	4458.631	43.562	200.182
21	9.167	211.588	23.19	9.167							212.583	4213.211	37.733	237.915
20	9.167	202.421	22.265	6.75	23.19	2.4167					206.332	4000.629	36.624	274.539
19	9.167	193.254	22.265	9.167							204.103	3794.297	36.228	310.768
18	9.167	184.087	21.918	5.083	22.265	4.083					202.317	3590.193	35.911	346.679
17	9.167	174.92	21.918	9.167							200.922	3387.876	35.664	382.343
16	9.167	165.753	21.456	3.4167	21.918	5.75					199.337	3186.954	35.382	417.725
15	9.167	156.586	21.456	9.167							196.687	2987.617	34.912	452.637
14	9.167	147.419	20.993	1.75	21.456	7.4167					195.870	2790.929	34.767	487.404
13	9.167	138.252	20.993	9.167							192.443	2595.059	34.159	521.563
12	9.167	129.085	20.993	9.167							192.443	2402.616	34.159	555.721
11	9.167	119.918	20.415	9.167							187.144	2210.173	33.218	588.939
10	9.167	110.751	20.415	9.167							187.144	2023.029	33.218	622.157
9	9.167	101.584	19.837	7.5833	20.415	1.5833					182.753	1835.885	32.439	654.596
8	9.167	92.417	19.49	6.75	19.837	2.4167					179.498	1653.132	31.861	686.457
7	10.083	83.25	19.143	6.833	19.49	3.25					194.147	1473.634	34.461	720.918
6	11	73.167	18.681	7.833	19.143	3.167					206.954	1279.488	36.734	757.652
5	10.667	62.167	18.218	8.5	18.681	2.167					195.335	1072.533	34.672	792.324
4	10.333	51.5	17.756	10.833	18.218	1.5					219.678	877.199	38.993	831.317
3	13.917	41.167	16.484	2.75	17.177	10	17.756	1.1667			237.817	657.521	42.213	873.530
2	18	27.25	14.981	5.75	15.559	5	16.021	5	16.484	2.25	281.130	419.704	49.901	923.430
1	9.25	9.25	14.981	9.25							138.574	138.574	24.597	948.027
Building Tributary Width = 177.5'														

East-West Wind Story Shear Forces

Level	Tributary Height (ft)	Total Height (ft)	P1 (psf)	H1 (ft)	P2 (psf)	H2 (ft)	P3 (psf)	H3 (ft)	P3 (psf)	H4 (ft)	Story Dist. Load (plf)	Cum. Dist. Load (plf)	Story Shear (kips)	Cum. Shear (kips)
Roof	5.333	260.171	24.235	5.333							129.245	5580.318	28.796	28.796
25	10.667	254.838	24.12	5.833	23.305	4.833					253.325	5451.072	56.441	85.237
24	10.667	244.171	24.12	10.667							257.288	5197.747	57.324	142.560
23	11.333	233.504	24.12	11.333							273.352	4940.459	60.903	203.463
22	10.583	222.171	24.12	10.583							255.262	4667.107	56.872	260.336
21	9.167	211.588	24.12	9.167							221.108	4411.845	49.263	309.598
20	9.167	202.421	23.195	6.75	24.12	2.4167					214.857	4190.737	47.870	357.469
19	9.167	193.254	23.195	9.167							212.629	3975.880	47.374	404.842
18	9.167	184.087	22.848	5.083	23.195	4.083					210.842	3763.252	46.976	451.818
17	9.167	174.92	22.848	9.167							209.448	3552.410	46.665	498.483
16	9.167	165.753	22.386	3.4167	22.848	5.75					207.862	3342.963	46.312	544.794
15	9.167	156.586	22.386	9.167							205.212	3135.100	45.721	590.516
14	9.167	147.419	21.923	1.75	22.386	7.4167					204.395	2929.888	45.539	636.055
13	9.167	138.252	21.923	9.167							200.968	2725.492	44.776	680.831
12	9.167	129.085	21.923	9.167							200.968	2524.524	44.776	725.606
11	9.167	119.918	21.345	9.167							195.670	2323.556	43.595	769.202
10	9.167	110.751	21.345	9.167							195.670	2127.887	43.595	812.797
9	9.167	101.584	20.767	7.5833	21.345	1.5833					191.278	1932.217	42.617	855.414
8	9.167	92.417	20.42	6.75	20.767	2.4167					188.023	1740.939	41.891	897.305
7	10.083	83.25	20.073	6.833	20.42	3.25					203.524	1552.916	45.345	942.650
6	11	73.167	19.611	7.833	20.073	3.167					217.184	1349.393	48.389	991.039
5	10.667	62.167	19.148	8.5	19.611	2.167					205.255	1132.208	45.731	1036.770
4	10.333	51.5	18.686	10.833	19.148	1.5					231.147	926.953	51.500	1088.269
3	13.917	41.167	17.414	2.75	18.107	10	18.686	1.1667			250.759	695.806	55.869	1144.138
2	18	27.25	15.911	5.75	16.489	5	16.951	5	17.414	2.25	297.870	445.047	66.365	1210.504
1	9.25	9.25	15.911	9.25							147.177	147.177	32.791	1243.295

Building Tributary Width = 222.8'

Portal Frame Analysis Shear and Moment Values				
Level	Story Shear	Cum. Shear	Story Moments	Cum. Mom.
	(kips)	(kips)	(ft-k)	(ft-k)
Roof	2.766	2.766	14.750	14.750
25	5.421	8.187	57.827	72.578
24	5.506	13.693	58.732	131.310
23	5.850	19.543	66.295	197.605
22	5.463	25.005	57.811	255.416
21	4.732	29.737	43.376	298.791
20	4.598	34.335	42.149	340.941
19	4.550	38.885	41.712	382.653
18	4.512	43.397	41.362	424.014
17	4.482	47.879	41.088	465.102
16	4.448	52.328	40.777	505.880
15	4.392	56.719	40.257	546.137
14	4.374	61.093	40.097	586.234
13	4.301	65.394	39.425	625.659
12	4.301	69.695	39.425	665.083
11	4.187	73.882	38.385	703.469
10	4.187	78.069	38.385	741.854
9	4.093	82.163	37.524	779.378
8	4.024	86.186	36.885	816.263
7	4.355	90.542	43.916	860.178
6	4.648	95.190	51.125	911.303
5	4.392	99.582	46.854	958.158
4	4.947	104.529	51.113	1009.270
3	5.366	109.895	74.682	1083.953
2	6.374	116.269	114.739	1198.692
1	3.150	119.419	29.134	1227.826
Building Tributary Width = 222.8'				

WIND LOAD CALCULATIONS:

GUST EFFECT FACTOR: G

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

Table 6-2:

$\bar{z}_{min} = 30$ ft. , $c = 0.3$

$\therefore\ \bar{z} = 0.6h = 0.6(260.5) = 156.3$ ft

$g_Q = g_v = 3.4$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{156.3}\right)^{1/6} = 0.231$$

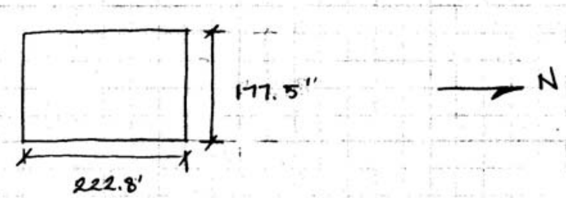
Table 6-2:

$\ell = 320$ ft

$\bar{\epsilon} = 1/3.0$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{156.3}{33}\right)^{1/3} = 537.4$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{177.5+260.5}{537.4}\right)^{0.63}}} = 0.802$$



$$G = 0.925\left(\frac{1+1.7(3.4)(0.231)(0.802)}{1+1.7(3.4)(0.231)}\right) = 0.820$$

EXTERNAL PRESSURE COEFFICIENT, C_p:

N-S: $\frac{L}{B} = \frac{222.8}{177.5} = 1.26$ windward = 0.8
leeward = -0.45
side = -0.7

E-W: $\frac{L}{B} = \frac{177.5}{222.8} = 0.80$ windward = 0.8
leeward = -0.5
side = -0.7

VELOCITY PRESSURE, q_z:

$q_z = 0.00256\, K_z K_{zt} K_D V^2 I$

$= 0.00256(1.0)(0.85)(90)^2(1.0)\, K_z$ * see excel spreadsheet

$q_h = q_z$ @ $h = 260.5'$ ∴ $q_h = 22.737$ psf

DESIGN WIND LOAD: P

$p_w = q_z GC_p = 0.820(0.8)\, q_z$ * see spreadsheet

$p_l = q_h GC_p$

N-S: $p_l = 22.737(0.820)(-0.45) = -8.39$ psf

E-W: $p_l = 22.737(0.820)(-0.5) = -9.32$ psf

INTERNAL PRESSURE:

$p = q_i GC_{pi}$

$q_i = q_z = 22.737$ psf

$GC_{pi} = \pm 0.18$ from Figure 6-5

$p = 22.737(\pm 0.18) = \pm 4.09$ psf

Seismic Load Calculation using ASCE 7-02	
Occupancy Category, II	Table 1-1
Seismic Use Group, I	Table 9.1.3
Site Classification: Assume Site Class D	
Acceleration:	
Ss = 0.180	Map 9.4.1.1a
S1 = 0.062	Map 9.4.1.1b
Site Coefficient, Fa = 1.6	Table 9.4.1.2.4a
Site Coefficient, Fv = 2.4	Table 9.4.1.2.4b
Sms = Fa*Ss = 0.288	
Sm1 = Fv*S1 = 0.149	
Design Spectral Response:	
Sds = 2/3 Sms = 0.192	
Sd1 = 2/3 Sm1 = 0.0992	
Seismic Design Category:	
B	Table 9.4.2.1a
B	Table 9.4.2.1b
Response Modification Coefficient, R = 7	Table 9.5.2.2
System Overstrength Factor, Wo = 2.5	
Deflection Amplification Factor, Cd = 6	
(for dual system with moment frame and reinforced concrete	
shear walls)	
Use Equivalent Lateral Force Analysis, Section 9.5.5	Table 9.5.2.5.1
Seismic Response Coefficient, Cs = 0.0275	Section 9.5.5.2.1
Occupancey Importance Factor, I = 1.0	Table 9.1.4
Approximate Period Paramters, Ct =0.02	Table 9.5.5.3.2
x =0.75	

Seismic Force Distribution							
Level	Total DL	Floor Area	Wx	hx	Wx*hx^k	Cvx (k)	Fx (k)
	(psf)	(ft^2)	(kips)	(ft.)			
Roof	115	15600	1794	260.5	18164843.11	0.0760874	40.950254
25	115	17300	1989.5	249.6667	18774477.23	0.078641	42.324594
24	115	18700	2150.5	239	18876573.64	0.0790687	42.554757
23	115	19800	2277	228.3333	18529805.16	0.0776162	41.773013
22	115	19800	2277	216.3333	16943283.85	0.0709707	38.196409
21	115	19800	2277	207.1667	15769630.03	0.0660546	35.550561
20	115	19800	2277	198	14629649.32	0.0612795	32.980624
19	115	19800	2277	188.8333	13523880.39	0.0566477	30.487813
18	115	19800	2277	179.6667	12452894.15	0.0521617	28.073415
17	115	19800	2277	170.5	11417264.39	0.0478237	25.738724
16	115	19800	2277	161.3333	10417642.53	0.0436366	23.485207
15	115	19800	2277	152.1667	9454726.369	0.0396032	21.314439
14	115	19800	2277	143	8529236.971	0.0357266	19.228045
13	115	19800	2277	133.8333	7641989.774	0.0320101	17.227863
12	115	19800	2277	124.6667	6793871.046	0.0284576	15.31589
11	115	19800	2277	115.5	5985824.238	0.0250729	13.494255
10	115	19800	2277	106.3333	5218920.93	0.0218606	11.765372
9	115	19800	2277	97.1667	4494350.807	0.0188256	10.131924
8	115	19800	2277	88	3813425.805	0.0159734	8.5968678
7	115	32300	3714.5	78.8333	5183762.032	0.0217133	11.686111
6	115	32300	3714.5	67.8333	4040478.184	0.0169244	9.1087276
5	115	32300	3714.5	56.8333	3013227.536	0.0126216	6.7929259
4	115	32300	3714.5	46.5	2160415.869	0.0090494	4.8703739
3	115	48100	5531.5	36.1667	2120894.403	0.0088838	4.781278
2	127.5	48100	6132.75	18.6667	785389.7977	0.0032898	1.7705582
		Totals:	68888.25	3564.8333	238736457.6	1	538.2

SEISMIC BASE SHEAR:

$V = C_s W$

$C_s = \frac{S_{DS}}{R/I} = \frac{0.192}{7/1.0} = 0.0275$

W = total dead load * see excel spreadsheet

$W = 69{,}000^k$

$V = 0.0275(69{,}000) = 1889.5$

$C_{s\,max} = \frac{S_{D1}}{T(R/I)}$

$T_a = C_t h_n^{x} = 0.02(260.5)^{0.75} = 1.297$

Table 9.5.5.3.2 $C_t = 0.02$
$x = 0.75$

$T = T_a C_u = 1.297(1.4) = 1.816$

$C_u = 1.4$

$C_{s\,max} = \frac{0.0992}{1.816(7/1.0)} = 0.0078$ ← CONTROLS

$C_{s\,min} = 0.044\, S_{DS}\, I = 0.044(0.192) = 0.0084$

∴ SEISMIC BASE SHEAR:

$V = 69{,}000(0.0078) = 538.2^k$

VERTICAL DISTRIBUTION OF SEISMIC FORCES:

$F_x = C_{vx} V$

$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$

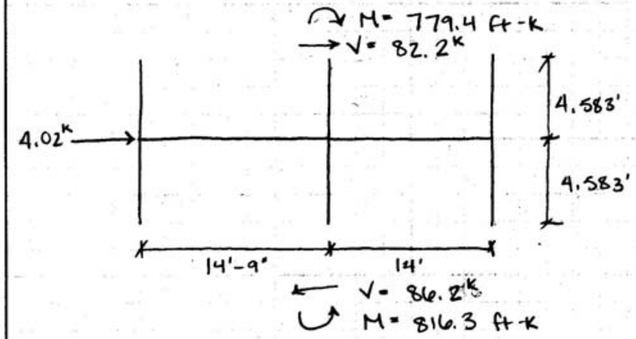
$K = 1.058$

* see excel spreadsheet for seismic force distribution.

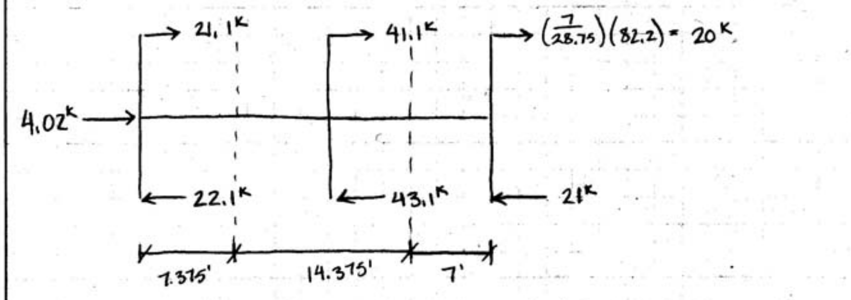
Factored Seismic and Wind Story Force Comparison				
Level	Seismic Story Forces (kips)	Wind Story Forces (N-S) (kips)	Factored Seismic (kips)	Factored Wind (kips)
Roof	40.950	28.796	40.950	46.0736
25	42.325	56.441	42.325	90.3056
24	42.555	57.324	42.555	91.7184
23	41.773	60.903	41.773	97.4448
22	38.196	56.872	38.196	90.9952
21	35.551	49.263	35.551	78.8208
20	32.981	47.870	32.981	76.592
19	30.488	47.374	30.488	75.7984
18	28.073	46.976	28.073	75.1616
17	25.739	46.665	25.739	74.664
16	23.485	46.312	23.485	74.0992
15	21.314	45.721	21.314	73.1536
14	19.228	45.539	19.228	72.8624
13	17.228	44.776	17.228	71.6416
12	15.316	44.776	15.316	71.6416
11	13.494	43.595	13.494	69.752
10	11.765	43.595	11.765	69.752
9	10.132	42.617	10.132	68.1872
8	8.597	41.891	8.597	67.0256
7	11.686	45.345	11.686	72.552
6	9.109	48.389	9.109	77.4224
5	6.793	45.731	6.793	73.1696
4	4.870	51.500	4.870	82.4
3	4.781	55.869	4.781	89.3904
2	1.771	66.365	1.771	106.184

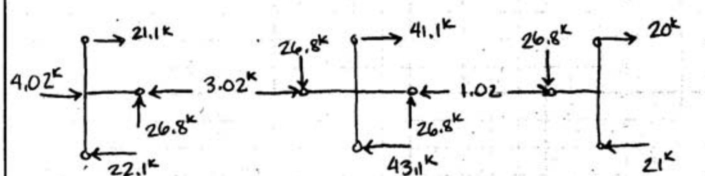
PORTAL FRAME ANALYSIS:

LEVEL 8:



* see excel spreadsheet for calculations of story forces

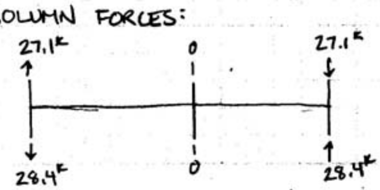




$\circlearrowleft^{+}\ \Sigma M_{joint} = 0$

$$A_y(7.38) - 21.1(4.583) - 22.1(4.583) = 0$$
$$A_y = 26.8^k$$

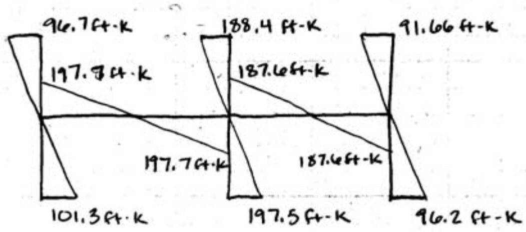
COLUMN FORCES:



$$\frac{779.4}{28.75} = 27.1^k$$

$$\frac{816.3}{28.75} = 28.4^k$$

MOMENTS:



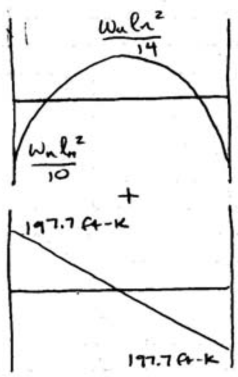
DESIGN LOAD, INCLUDING WIND:

$1.2D + 0.5L + 1.6W$

DL = 20 PSF
SLAB = 100 PSF
BRICK = 357.5 PLF
SELF = 460.4 PLF

TOT DL = $120(11.75) + 357.5 + 460.4$
$= 2227.9$ PLF
LL = $80(11.75) = 940$ PLF

$W_u = 1.2(2.23) + 0.5(0.94) = 3.15$ KLF



$$M_{pos} = \frac{3.15(11.75)^2}{14} = 31.1 \text{ ft-k}$$

$$M_{neg} = \frac{3.15(11.75)^2}{10} = 43.5 \text{ ft-k}$$

$$M_{pos} = 1.6(197.7) + 31.1 = 347.4 \text{ ft-k}$$

$$M_{neg} = 1.6(197.7) + 43.5 = 359.8 \text{ ft-k}$$

$$A_s = \frac{359.8(12)}{60(0.9)(31.5)} = 2.54 \text{ in}^2$$

try 4 #8, $A_s = 3.16 \text{ in}^2$

$$a = \frac{60(3.16)}{0.85(5)(13)} = 3.43$$

$$\phi M_n = 60(0.9)(3.16)\left(31.5 - \frac{3.43}{2}\right) = 5082 \text{ in-k} = 424 \text{ ft-k}$$

∴ use 4 #8 top and bottom reinforcement

Appendix B

Gravity Loads									
Level	Slab Thick. (in.)	Slab Wt. (psf)	Super. DL (psf)	Total DL (psf)	LL (psf)	Factored Load (psf)	Floor Area (ft^2)	Floor Load (kips)	Cum. Load (kips)
Roof	8	100	15	115	80	266	15600	4149.6	4149.6
25	8	100	15	115	80	266	17300	4601.8	8751.4
24	8	100	15	115	80	266	18700	4974.2	13725.6
23	8	100	15	115	80	266	19800	5266.8	18992.4
22	8	100	15	115	80	266	19800	5266.8	24259.2
21	8	100	15	115	80	266	19800	5266.8	29526
20	8	100	15	115	80	266	19800	5266.8	34792.8
19	8	100	15	115	80	266	19800	5266.8	40059.6
18	8	100	15	115	80	266	19800	5266.8	45326.4
17	8	100	15	115	80	266	19800	5266.8	50593.2
16	8	100	15	115	80	266	19800	5266.8	55860
15	8	100	15	115	80	266	19800	5266.8	61126.8
14	8	100	15	115	80	266	19800	5266.8	66393.6
13	8	100	15	115	80	266	19800	5266.8	71660.4
12	8	100	15	115	80	266	19800	5266.8	76927.2
11	8	100	15	115	80	266	19800	5266.8	82194
10	8	100	15	115	80	266	19800	5266.8	87460.8
9	8	100	15	115	80	266	19800	5266.8	92727.6
8	8	100	15	115	80	266	19800	5266.8	97994.4
7	8	100	15	115	80	266	32300	8591.8	106586.2
6	8	100	15	115	80	266	32300	8591.8	115178
5	8	100	15	115	80	266	32300	8591.8	123769.8
4	8	100	15	115	80	266	32300	8591.8	132361.6
3	8	100	15	115	100	298	48100	14333.8	146695.4
2	9	112.5	15	127.5	100	313	48100	15055.3	161750.7

GRAVITY LOAD CALCULATIONS:

slab thickness = 8"-9"

f'_c = 5000 psi

Normal weight concrete = 150 pcf

Slab wt = $8"/12\,(150) = 100$ psf

$= 9"/12\,(150) = 112.5$ psf

Super. DL = 15 PSF

$DL_{tot} = 15 + 100 = 115$ PSF

LL = 80 PSF

$w_u = 1.2(115) + 1.6(80) = 266$ PSF

LIVE LOAD REDUCTION:

$$L = L_o\left(0.25 - \frac{15}{\sqrt{K_{LL}A_T}}\right.$$

$K_{LL} = 1.0$ from ASCE 7-02 Table 4-2

$A_T = 21(22.833) = 479.5\ ft^2$

$K_{LL}A_T = 479.5\ ft^2 > 400\ ft^2$

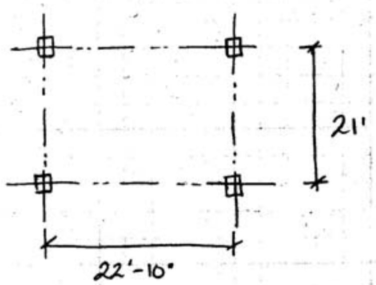
∴ live load reduction not permitted by ASCE 7-02, 4.8.1

Appendix C

APPENDIX C | SPOT CHECKS | 1

FLOOR FRAMING SPOT CHECKS:

SLAB CHECK:



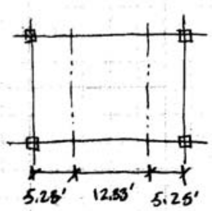
8" thick two-way reinforced flat plate

24" x 28" columns

$C.S. = \frac{1}{4}(21) = 5.25'$

short span direction:

$M.S. = 12.33'$



Superimposed Loads:

DL = 15 PSF
LL = 80 PSF

check min. slab thickness:

$$t = \frac{l_n}{33} = \frac{22.833 - 28/12}{33} = 7.5" < 8" \therefore OK \checkmark$$

self weight of slab = $8"/12\,(150) = 100$ PSF

$$W_u = 1.2(115) + 1.6(80) = 266 \text{ PSF}$$

$$M_o = \frac{W_u l_2 l_n^2}{8} = \frac{0.266\,(22.83)(19)^2}{8} = 274.0 \text{ ft-k}$$

		Total Mu (ft-k)	Total Width (ft)	Mom./Foot Width (ft-k/ft)
INT. SUPPORT (-) 65%	C.S. (75%)	133.58	10.5	12.72
	M.S. (25%)	44.53	12.33	3.61
MIDSPAN (+) 35%	C.S. (60%)	57.54	10.5	5.48
	M.S. (40%)	38.36	12.33	3.11

Shrinkage and Temperature:

min. reinforcement $= 0.0018 A_g = 0.0018(8)(12) = 0.1728 \text{ in}^2$

SPOT CHECKS

try #6@ 10" , $A_s = 0.528 in^2$

$d = 8 - 3/4 - 0.5 - 0.375 = 6.375"$

$a = \frac{0.53(60)}{0.85(5)(10)} = 0.745$

$\phi M_n = 0.9(0.528)(60)(6.375 - \frac{0.745}{2}) = 14.26$ k-ft/ft > 12.72 ft-k/ft

try #6@ 11" , $A_s = 0.48$

$a = \frac{0.48(60)}{0.85(5)(11)} = 0.616$

$\phi M_n = 0.9(0.48)(60)(6.375 - \frac{0.616}{2}) = 13.1$ k-ft/ft > 12.72 ft-k/ft

C.S. INT. SUPPORT REINF. USE $\frac{10.5'(12)}{11"} = 12$ bars

USE 12 #6T C.S.

check minimum reinforcement for C.S. midspan

try #5@ 12 , $A_s = 0.31 in^2$

$a = \frac{0.31(60)}{0.85(5)(12)} = 0.365$

$\phi M_n = 0.9(0.31)(60)(6.375 - \frac{0.365}{2}) = 8.64$ ft-k/ft > 6.695 ft-k/ft

∴ USE #5 @ 12"

long span direction:

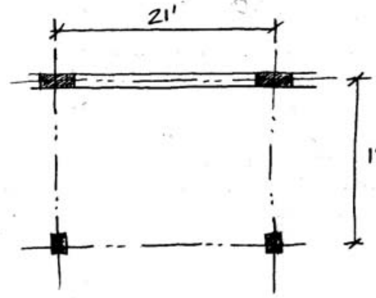
M.S. = 10.5'

$M_o = \frac{0.266(21)(20.5)^2}{8} = 293.4$ ft-k

		Total Mu (ft-k)	Total Width (ft)	Mom./Foot Width (ft-k/ft)
INT. SUPPORT	C.S. (75%)	143.03	10.5	13.62
(-) 65%	M.S. (25%)	47.68	10.5	4.54
MIDSPAN	C.S. (60%)	61.61	10.5	5.87
(+) 35%	M.S. (40%)	41.08	10.5	3.91

∴ from previous calculations:
C.S. INT. SUPPORT USE #6@10" ⇒ 13#6T
MIDSPAN C.S. USE #5@12" T+B

PERIMETER EDGE BEAM CHECK:



BEAM: 13" x 34"
COLUMNS: 13" x 36"

LL = 80 PSF
DL = 15 PSF

$\therefore w_u = 266$ PSF

$$M_o = \frac{w_u \ell_2 \ell_n^2}{8} = \frac{0.266(21)(17.21)^2}{8} = 206.81 \text{ ft-k}$$

uniformly distributed moment along edge beam:

$30\% \, M_o = 62.04$ ft-k

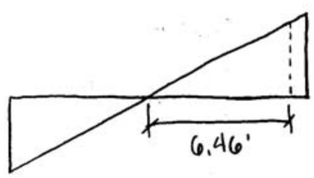
$\therefore$ total torsional moment = 62.04 ft-k

$$M_u = \frac{62.04}{21} = 2.954 \text{ ft-k/ft}$$

assume $d = 34 - 1.5 - 0.5 - 0.5 = 31.5"$

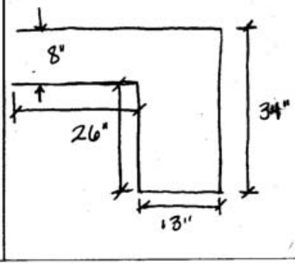
$T_u = 2.954(10.5) = 31.02$ ft-k

Design $T_u = 2.954(6.46) = 19.08$ ft-k



torsional moment reduction:

$$\phi 4 \sqrt{f'_c}\left(\frac{A_{cp}^2}{P_{cp}}\right) = 0.75(4)\sqrt{5000}\left(\frac{650^2}{146}\right) = 51.16 \text{ ft-k}$$



$A_{cp} = 34(13) + 8(26)$
$= 650 \text{ in}^2$

$P_{cp} = 2(34) + 2(39)$
$= 146"$

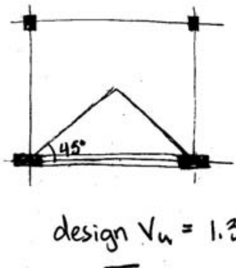
$\therefore$ USE $T_u = 19.08$ ft-k

check shear:

$$\alpha_1 = \frac{\frac{1}{12}(13)(34)^3}{\frac{1}{12}(21)(8)^3(12)} = 3.66$$

$$\alpha_1 \, l_2/l_1 = 3.66\left(\frac{21}{18.75}\right) = 4.1 > 1.0$$

$\therefore$ use tributary area @ 45° from supports



trib. area = $10.5^2 = 110.25 \text{ in}^2$

$$w_u = \frac{0.266\,(110.25)}{21} = 1.397 \text{ KLF}$$

$$V_u = \frac{w_u l}{2} = \frac{1.397(21)}{2} = 14.66^K$$

design $V_u = 1.397(6.46) = 9.025^K$

$V_c = 2\sqrt{f'_c}\, b_w d = 2\sqrt{5000}\,(13)(31.5) = 57.9^K$

torsional reinforcement spacing:

shear: $\frac{d}{2} = 15.75"$

torsion: $\frac{P_h}{8} = \frac{80}{8} = 10"$ ← CONTROLS.

$x_o = 13 - 3.5 = 9.5"$ $\quad y_o = 34 - 3.5 = 30.5$

$P_h = 2(9.5 + 30.5) = 80$

Req'd. $T_n = \frac{T_u}{\phi} = \frac{19.08}{0.75} = 25.44$ ft-k

$$A_t = \frac{T_n s}{2A_o f_y} = \frac{25.44(10)(12)}{2(246.3)(60)} = 0.103 \text{ in}^2$$

$A_{oh} = 9.5(30.5) = 289.75$ $\quad A_o = 0.85(289.75) = 246.3$

$2A_t = 2(0.103) = 0.206 \text{ in}^2$ $\quad \therefore$ use #4, $A_s = 0.4 \text{ in}^2$

min. reinforcement:

$$\frac{50 b_w s}{f_y} = \frac{50(13)(10)}{60} = 0.108 \text{ in}^2 \quad \therefore \text{OK} \checkmark$$

for torsion + shear: use #4 @ 10" closed stirrups

Longitudnal Reinforcement:

Torsion: $A_l = p_h \cot^2\theta \frac{A_t}{s} = 80(1.0)\frac{(0.103)}{10} = 0.824 \text{ in}^2$

Flexure:

$$M_o = \frac{0.266(21.0)(18.167)^2}{8} = 230.5 \text{ ft-k}$$

		Total Mu (ft-k)	Tot. Width (ft)	Mom./ft. width (ft-k/ft)
EXT. SUPPORT (-) 65%	CS (97%)	145.3	10.5	13.84
	MS (3%)	4.49	10.5	0.428
MIDSPAN (+) 35%	CS (60%)	48.41	10.5	4.61
	MS (40%)	32.27	10.5	3.07

$A_{s,min} = \frac{3\sqrt{f'_c}\, b_w d}{f_y} = \frac{3\sqrt{5000}\,(13)(31.5)}{60000} = 1.45 \text{ in}^2$ ← controls

$= \frac{200\, b_w d}{f_y} = \frac{200(13)(31.5)}{60{,}000} = 1.37 \text{ in}^2$

try 3#7 s, $A_s = 1.8 \text{ in}^2$

$a = \frac{1.8(60)}{0.85(5)(13)} = 1.955$

$\phi M_n = 0.9(1.8)(60)(31.5 - \frac{1.955}{2}) = 247 \text{ ft-k} > 13.84 \text{ ft-k/ft}$

torsional reinforcement must be spaced < 12" apart

∴ side bars are required

COLUMN SPOT CHECK:

* column check due to gravity loads only

$P = f_c(A_g + (n-1)A_{st})$

trib. area $= 21'(18.83') = 395.5\ ft^2$

typ floor: $w_u = 1.2(115) + 1.6(80) = 266$ PSF

roof: $w_u = 1.2(115) + 1.6(20) = 170$ PSF

$P_u = 395.5(170) + 395.5(266)(16) = 1750.5^k$

$A_g = 24(24) = 576\ in^2$

$n = \frac{E_s}{E_c} = \frac{29\times10^6}{4.77\times10^6} = 6.08$

$E_c = 57{,}000\sqrt{f'_c} = 57{,}000\sqrt{7000} = 4769000$

$A_{st} = 8\ in^2$ for 8 #9 bars

$P = 7000(576 + (5.08)(8)) = 4316.5^k > 1750.5^k$

∴ this column design is more than adequate for gravity loads.

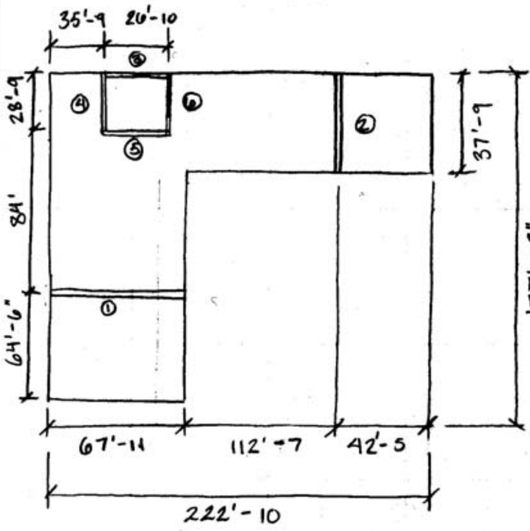
Appendix D

Shear Wall Lateral Load Distribution - Wall Rigidities												
					Wall Rigidities (R/E)							
Level	hx	Story	Cum. Shear		Wall 1	Wall 2	Wall 3	Wall 4	Wall 5	Wall 6	Sum of R	Sum of R
	(ft.)	Shear (kips)	(kips)	L (ft.) =	67.9167	37.75	26.8333	28.75	26.8333	28.75	N-S	E-W
				t (in.) =	15	15	16	16	16	16		
Roof	260.5	22.061	22.061		0.0632329	0.011235	0.0043373	0.0053285	0.0043373	0.0053285	0.0719075	0.0218919
25	249.6667	44.002	66.063		0.0715189	0.0127443	0.0049233	0.0060478	0.0049233	0.0060478	0.0813655	0.0248398
24	239	43.908	109.971		0.0811391	0.0145057	0.0056079	0.006888	0.0056079	0.006888	0.0923549	0.0282816
23	228.3333	46.649	156.62		0.0925446	0.0166059	0.0064254	0.007891	0.0064254	0.007891	0.1053954	0.0323879
22	216.3333	43.562	200.182		0.1080485	0.0194807	0.0075462	0.0092659	0.0075462	0.0092659	0.1231409	0.0380126
21	207.1667	37.733	237.915		0.1222738	0.0221381	0.008584	0.0105387	0.008584	0.0105387	0.1394418	0.0432154
20	198	36.624	274.539		0.1390719	0.0252991	0.0098208	0.0120549	0.0098208	0.0120549	0.1587134	0.0494089
19	188.8333	36.228	310.767		0.1590417	0.0290884	0.0113063	0.0138756	0.0113063	0.0138756	0.1816542	0.0568397
18	179.6667	35.911	346.678		0.1829546	0.0336692	0.0131061	0.0160809	0.0131061	0.0160809	0.2091668	0.0658309
17	170.5	35.664	382.342		0.2118145	0.039258	0.0153079	0.0187774	0.0153079	0.0187774	0.2424303	0.0768129
16	161.3333	35.382	417.724		0.2469406	0.0461459	0.0180298	0.0221095	0.0180298	0.0221095	0.2830003	0.0903649
15	152.1667	34.912	452.636		0.2900866	0.05473	0.0214344	0.0262748	0.0214344	0.0262748	0.3329554	0.1072797
14	143	34.767	487.403		0.3436153	0.0655614	0.0257486	0.0315497	0.0257486	0.0315497	0.3951125	0.1286608
13	133.8333	34.159	521.562		0.4107474	0.0794179	0.031296	0.0383269	0.031296	0.0383269	0.4733395	0.1560718
12	124.6667	34.159	555.721		0.4959379	0.0974197	0.0385476	0.0471776	0.0385476	0.0471776	0.5730331	0.1917749
11	115.5	33.218	588.939		0.6054468	0.1212174	0.0482061	0.0589523	0.0482061	0.0589523	0.7018591	0.239122
10	106.3333	33.218	622.157		0.748205	0.153302	0.0613497	0.0749522	0.0613497	0.0749522	0.8709044	0.3032063
9	97.1667	32.439	654.596		0.9371829	0.1975401	0.0796845	0.0972305	0.0796845	0.0972305	1.0965519	0.392001
8	88	31.861	686.457		1.1915829	0.2601272	0.1060128	0.129146	0.1060128	0.129146	1.4036086	0.5184191
7	78.8333	34.461	720.918		1.54041	0.3513445	0.1451329	0.1764209	0.1451329	0.1764209	1.8306758	0.7041863
6	67.8333	36.734	757.652		2.1485051	0.5244987	0.221595	0.2683822	0.221595	0.2683822	2.5916951	1.0612631
5	56.8333	34.672	792.324		3.0900275	0.8257145	0.3606891	0.4344262	0.3606891	0.4344262	3.8114057	1.694567
4	46.5	38.993	831.317		4.4940288	1.3427214	0.6150361	0.734745	0.6150361	0.734745	5.7241011	2.8122114
3	36.1667	42.213	873.53		6.8133057	2.3467978	1.1562712	1.3632278	1.1562712	1.3632278	9.1258482	5.0732533
2	18.6667	49.901	923.431		16.527293	7.6255364	4.6598752	5.258547	4.6598752	5.258547	25.847043	18.14263

Shear Wall Lateral Load Distribution - Concentric Shear							
Wall 2	Story Shear	Wall 4	Story Shear	Wall 6	Story Shear		Moment
Proportion	Shear	Proportion	Shear	Proportion	Shear		(ft-k)
0.5132029	11.32177	0.2433985	5.3696151	0.2433985	5.3696151		238.99343
0.5130601	33.894292	0.2434699	16.084354	0.2434699	16.084354		469.35613
0.5129005	56.404183	0.2435497	26.783408	0.2435497	26.783408		468.35346
0.5127185	80.301969	0.2436408	38.159016	0.2436408	38.159016		559.788
0.5124814	102.58955	0.2437593	48.796223	0.2437593	48.796223		399.31543
0.5122726	121.87733	0.2438637	58.018833	0.2438637	58.018833		345.88709
0.5120349	140.57356	0.2439825	66.982722	0.2439825	66.982722		335.72122
0.5117629	159.03901	0.2441186	75.863996	0.2441186	75.863996		332.08758
0.5114495	177.30829	0.2442752	84.684853	0.2442752	84.684853		329.18536
0.5110862	195.40971	0.2444569	93.466147	0.2444569	93.466147		326.92119
0.5106617	213.31566	0.2446691	102.20417	0.2446691	102.20417		324.33264
0.5101619	230.91762	0.2449191	110.85919	0.2449191	110.85919		320.02783
0.5095679	248.36491	0.2452161	119.51905	0.2452161	119.51905		318.69866
0.508855	265.39941	0.2455725	128.08129	0.2455725	128.08129		313.12189
0.5079899	282.30063	0.2460051	136.71018	0.2460051	136.71018		313.12531
0.5069269	298.54905	0.2465365	145.19497	0.2465365	145.19497		304.49944
0.5056028	314.56433	0.2471986	153.79633	0.2471986	153.79633		304.49612
0.5039274	329.86886	0.2480363	162.36357	0.2480363	162.36357		297.35858
0.50177	344.44353	0.249115	171.00674	0.249115	171.00674		292.06023
0.4989368	359.69255	0.2505316	180.61272	0.2505316	180.61272		379.071
0.4942211	374.44763	0.2528894	191.60218	0.2528894	191.60218		404.074
0.4872717	386.07705	0.2563642	203.12347	0.2563642	203.12347		358.27618
0.4774611	396.9215	0.2612695	217.19775	0.2612695	217.19775		402.92637
0.4625824	404.07962	0.2687088	234.72519	0.2687088	234.72519		738.7275
0.4203104	388.12766	0.2898448	267.65167	0.2898448	267.65167		931.487
						SUM =	9807.8916

Shear Walls - Torsional Shear (8th Floor)					
Wall #	R	x (ft.)	Rx^2	Rx/ΣRx^2	Torsional
					Shear (k)
1	1.192	14.854	263.00445	0.0043579	14.364147
2	0.26	65.327	1109.5804	0.0041805	13.779271
3	0.106	97.896	1015.8644	0.0025541	8.4184277
4	0.129	79.34	812.03379	0.0025191	8.3031346
5	0.106	69.146	506.80395	0.001804	5.9461122
6	0.129	52.507	355.65107	0.0016671	5.4949923
		SUM =	4062.9381		

SHEAR WALL CHECK:



Elevator/Stairway Core Walls
$t = 16''$

Other Shear Walls
$t = 15''$

→ N

* Use the E-W shear forces from the wind analysis.

Rigidity:

$$r = \frac{1}{\Delta}$$

$$R = Et\left(4\left(\frac{h}{L}\right)^3 + 3\left(\frac{h}{L}\right)\right)^{-1}$$

* E is constant btw/ all the walls so it can be divided out
* t is not constant for all the walls

Center of Rigidity:

* use R values from 8th floor ← see excel spreadsheet

Wall #	R
1	1.192
2	0.260
3	0.106
4	0.129
5	0.106
6	0.129

$$X_{CR} = \frac{0.26(180.5) + 0.129(35.75) + 0.129(62.583)}{0.26 + 0.129 + 0.129}$$

$$= 115.09'$$

$$Y_{CR} = \frac{1.192(64.5) + 0.106(177.25) + 0.106(148.5)}{1.192 + 0.106 + 0.106}$$

$$= 79.354'$$

SHEAR WALLS

Torsional Moment at Base due to eccentricity:

$T = 897.305\left(115.09 - \frac{222.833}{2}\right) = 3296.1$ ft-k

cum. shear @ 8^{th} floor = 897.305^k

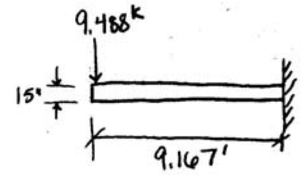
Wall #	Concentric Shear (k)	Eccentric Shear (k)
1	0	14.364
2	344.4	13.779
3	0	8.418
4	171.0	8.303
5	0	5.946
6	171.0	5.495

· lateral check of Wall 2:

total $V = 344.4 + 13.779 = 358.18^k$

distributed across wall: $\omega = \frac{358.18^k}{37.75'} = 9.488$ klf

° treating the wall as a cantilevered beam and taking a unit foot strip of wall:



$M = 9.488(9.167) = 87$ ft-k

Req'd $M_n = \frac{87}{0.9} = 96.7$ ft-k

Req'd $A_s = \frac{M_n}{f_y(0.9d)} = \frac{96.7}{60(0.9)(7.5)} = 0.239\ in^2$

· assume d is at the middle of the wall $\therefore d = 7.5"$

try #5, $A_s = 0.31\ in^2$

$a = \frac{A_s f_y}{0.85 f'_c b} = \frac{0.31(60)}{0.85(7)(12)} = 0.261$

$\phi M_n = 0.9 A_s f_y \left(d - \frac{a}{2}\right) = 0.9(0.31)(60)\left(7.5 - \frac{0.261}{2}\right)$

$= 123.4$ ft-k > 87 ft-k

$\therefore$ use #5@12" reinforcement for 8^{th} floor shear wall #2