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Technical Assignment 3

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Executive Summary

The purpose of this report is to analyze and gain a greater understanding of the lateral system for the Albany Medical Center Patient Care Pavilion. The structure of the Patient Pavilion is composite steel framing with 14 braced frames as well as 4 moment frames, sitting on a mat foundation or subgrade concrete shear walls. This report includes code checks of drift, story drift, and torsion. Also strength checks were performed for the lateral resisting members.

To perform the analysis for this report dead, live, and snow loads had to be verified in the structural drawings. Next, both wind and seismic loads were obtained per ASCE 7-05; for wind the Main Wind Force Resisting System procedure was used and for seismic the Equivalent Lateral force procedure was used. It was found that in the lower levels wind controlled and in the upper levels seismic loading controlled, overall seismic controls.

Next, a model of only the lateral system was built in ETABS to confirm the strength of the lateral system as well as analyze its serviceability. Only the lateral frames of the Patient Pavilion were modeled for this report. The lateral frames consisted of braced frames in both directions and some moment frames in the East-West direction. The braces in the braced frames were assigned moment releases in the 3-3 direction at each end, accounting only for axial load in the braces. The shear walls in the basement were modeled as a membrane accounting only for in plane loading. The floor slab in the Patient Pavilion is 6 1/2” lightweight concrete on metal deck, this floor system provides enough rigidity to be modeled as a rigid diaphragm.

To verify the accuracy of the model, relative stiffness’s of each frame. Hand calculations were performed to find the combined torsional and direct shear at a given story in each frame. The combined torsional and direct shear was then distributed to each frame using the calculated relative stiffness. Section cuts were made in the ETABS model to get the shear in ach frame and this shear was verified with the hand calculations.

Thirteen different load cases were considered to perform code checks on the lateral system. Chapter 6 in ASCE7-05 defines eleven different wind load cases, and there are two seismic load cases, one case in each direction, including accidental torsion. The seismic drifts obtained from the ETABS model were checked with the allowable story drifts per Chapter 12 of the ASCE7-05. The wind drifts were verified with the rule of thumb per the commentary in the ASCE 7-05.

Introduction

The Patient Pavilion is located in Albany, NY, at the intersection of New Scotland Avenue and Myrtle Avenue, on the eastern end of the existing Albany Medical Center Hospital (AMCH) campus. Constructed as an expansion to the AMCH, the Patient Pavilion utilizes pedestrian bridges to tie into an existing parking structure across New Scotland Avenue, as well as tying into an existing building on the AMCH campus as shown in *Figure 1.*

Figure 1 – Pedestrian Bridges

The Patient Pavilion will retain the architectural style, forms, and materials of downtown Albany and the AMCH campus, as specified in the City of Albany Zoning Ordinance. The façade primarily consists of brick and stone with punched windows and white stone accenting the upper levels. To add emphasis to the pedestrian walkway over New Scotland Avenue, metal paneling and glazed aluminum curtain-walls added an integrated modern look to the traditional façade.

The Patient Pavilion consists of two phases; Phase 1, contains the demolition of an existing building on the AMCH campus, and the construction of a six story medical center see *Figure 2* and Phase 2 is a future four story vertical expansion of the Patient Pavilion see *Figure 3*. The building height of Phase 1 is 75 feet above grade and the vertical expansion of Phase 2 will increase the building height to 145 feet above grade. Due to a small site and large square footage demands, the building cantilevers over the site on the side of New Scotland Avenue, demanding for the design of cantilevered plate girders to support a column load from stories 2-10.

Figure 2 – Phase 1 of Patient Pavilion; Initial Design

This patient care facility, contributes 229 patient beds, 20 operating rooms, and 1000 new permanent jobs to the AMCH campus. The 348,000 square foot expansion consists of six stories above grade with a four story vertical expansion in the future. Phase 1 construction on the Patient Pavilion began in September of 2010 and projects to finish in June of 2013.

Figure 3 – Phase 2 of the Patient Pavilion; Vertical Expansion

To better understand the terminology used for referring to designated levels, an architectural elevation is provided on the next page.

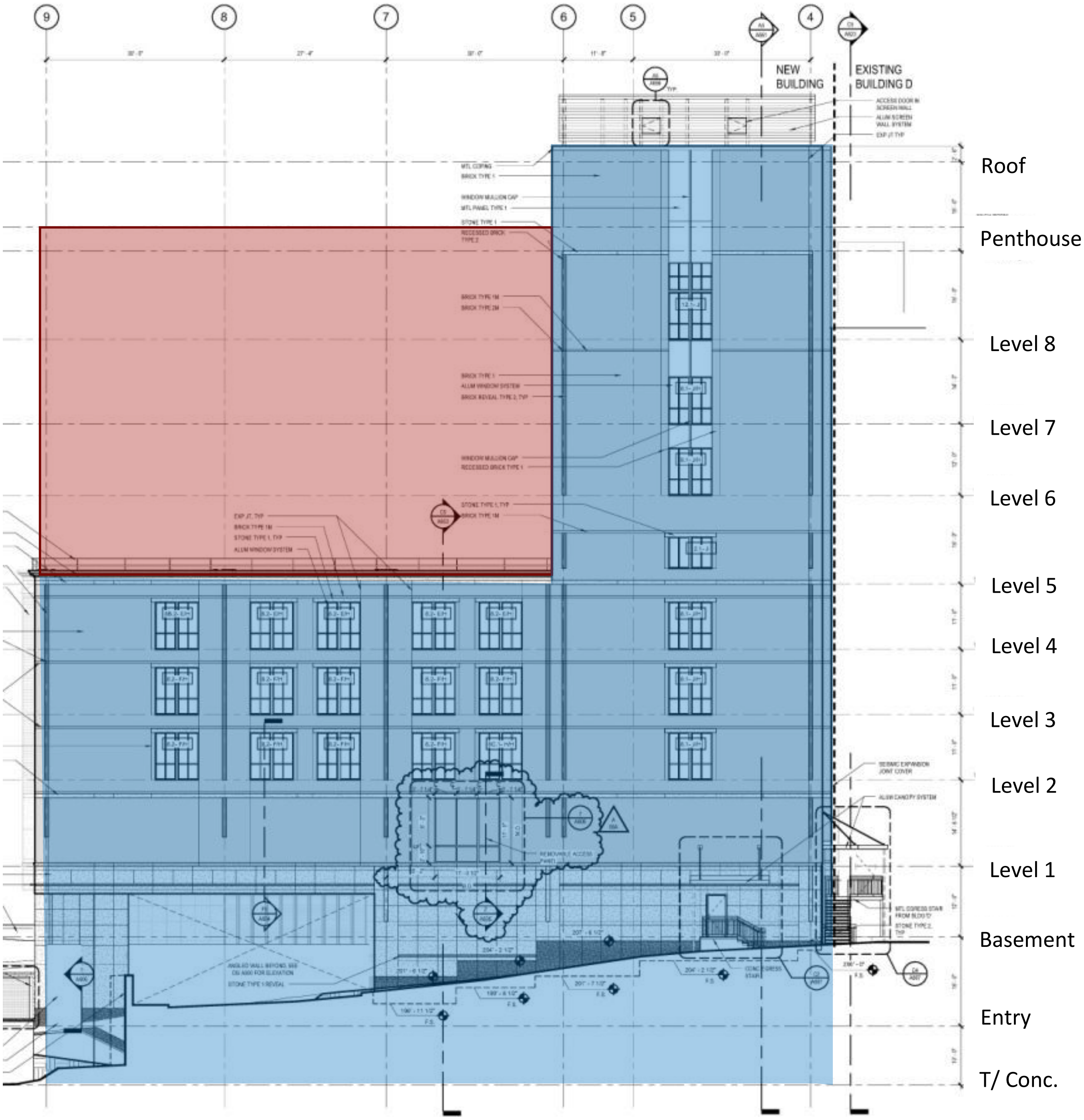


Figure 4 – South Elevation

Phase 1

Phase 2

Figure 5 – Site Plan

New Scotland Avenue

Myrtle Avenue

Structural Overview

The majority of the Patient Pavilion rests on a 36" thick mat foundation, and some piles located near existing buildings. The floor system utilizes composite beams, girders, and slabs to carry the loads derived from ASCE07-02. The lateral forces are collected on the brick non-bearing façade, transfers into the slab and is distributed to the foundation/grade by the integration of braced and moment frames. On the southern end of the site, 62" deep plate girders are utilized to cantilever nine stories over the edge of the site. Multi-story trusses are utilized to carry multiple levels with a large clear span, these are located over the emergency access ramp and at the pedestrian bridge that ties into an existing AMCH building see *Figure 6.*

Figure 6 - Span over Emergency Access Ramp and Street Labels

Foundation

Vernon Hoffman PE Soil and Foundation Engineering supplied the geotechnical report for the Patient Pavilion site. Procedures used were site boring, vane shear testing, pressure testing, and cone testing. Soil testing concluded that foundations must be designed to a net bearing pressure of 3000psf. Design ground water level was reported to be between 4' and 10' throughout the site. After a full analysis of the site, the geotechnical report recommended the building to sit on a mat foundation resting on a controlled fill.

Because of the relatively low allowable soil bearing pressure, the majority of the Patient Pavilion sits on a 36" mat foundation resting on a 4" mud slab with a 12" compacted aggregate base. Alternatively, 20'-0" deep piles are utilized in order to prevent unwanted settlement of the existing buildings. Piles are utilized in place of shallow foundations because piles will control settlements and provide uplift resistance more effectively than shallow foundations.

Foundation walls are utilized along existing building C and along New Scotland Avenue to lessen the demand on the excavation shoring; these walls also serve the purpose of shear walls in the lateral system. Backfilling behind these walls was needed to provide construction access for equipment and materials to build the pile caps and grade beams.

Floor System

The Patient Pavilion utilizes 3”x20ga galvanized composite steel deck with 3 1/2” lightweight topping, reinforced with #4’s at 16” O.C. for shrinkage and temperature, this floor system is typical throughout the levels, unless otherwise noted. On level 2, the floor slab is thickened with a 3” lightweight concrete topping in order to reduce floor vibrations in the operating rooms. The entry level utilizes an 8” lightweight concrete slab on 3 1/2”x16ga composite metal deck because of longer deck spans and larger live loads. In areas where radiation is prevalent, the slabs above and below that level are stiffened with a steel plate anchored to the slab with angles. These plates are located on levels 2 and 3 and their function is to provide a shield from the radiation for adjacent areas, refer to *Figure 7* for radiation slab details.

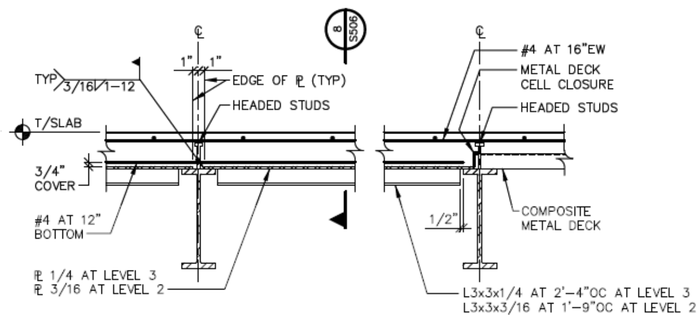


Figure 7 – Slab Detail; Radiation Shielding Plate

Typical beam spacing throughout is 10’-0” O.C., creating a 10’-0” deck span requirement, all beams are composite beams, typically W12’s. However, on the Basement Level and Level 2, typical beams range from W16’s to W18’s. Reasons for deeper beams are that the live load requirements on the Entry Level through Level 2 are greater than the other floors. However, the Basement Level and Level 2 utilize deeper beams than the Entry Level and Level 1 due to greater floor-to-floor heights.

Typical beams span 27’-4”, these beams sit on girders that typically span 30’-0”. Girder sizes range from W14’s to W18’s; however, on the Basement Level and Level 2 girder sizes fluctuate from W18’s to W24’s, refer to *Figure 8* for a typical bay on Level 3.

A demand for specialty framing is needed in certain areas in this project; on the southern end of the site, a column is cantilevered 18’ over the edge of the site resting on a 62” plate girder. The pedestrian bridge on the tying into the existing AMCH building spans 83’ over another existing AMCH building. A two-story truss was designed on bottom two levels of this pedestrian bridge, consisting of W10x77’s and W10x100’s.

Figure 8 – Typical Floor Plan

Lateral System

The lateral system for the Patient Pavilion predominantly consists of braced frames, with some moment frames. Within the structure, there are 14 braced frames and 5 moment frames, because of the locations of the braced frames, Chevron bracing is utilized to allow openings for doorways and corridors. See *Figure 8* for a typical braced frame. Figure *7* shows the locations of the braced and moment frames, the location of some braced frames fluctuate from level to level. For instance, braced frame 13 is braced between the Basement Level through Level 2 and above Level 2 is a moment frame.

The braced frames along the western side of the site sit on retaining walls in the basement, which also act as concrete shear walls. A strong connection is required to transfer the shear load as well as to resist upift, for these connections a 30”x30”x3½” baseplate with a 2” diameter anchor bold anchored 42” into the wall is specified. Diagonal bracing on the lower levels range from W10’s to W12’s and HSS8x6’s to HSS8x8’s on the upper levels. Heavier bracing on the lower levels provides a greater resistance to shear, which increases as the force moves down the frame. Columns used in these lateral resisting frames range from W14x43 to W 14x233.

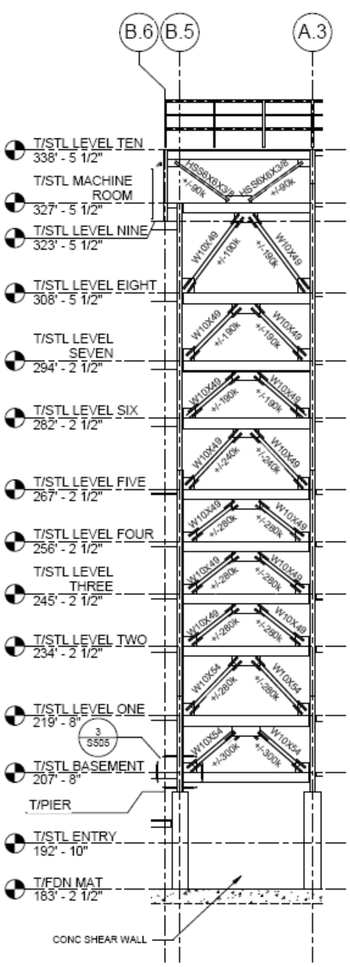


Figure 10 – Typical Braced Frame

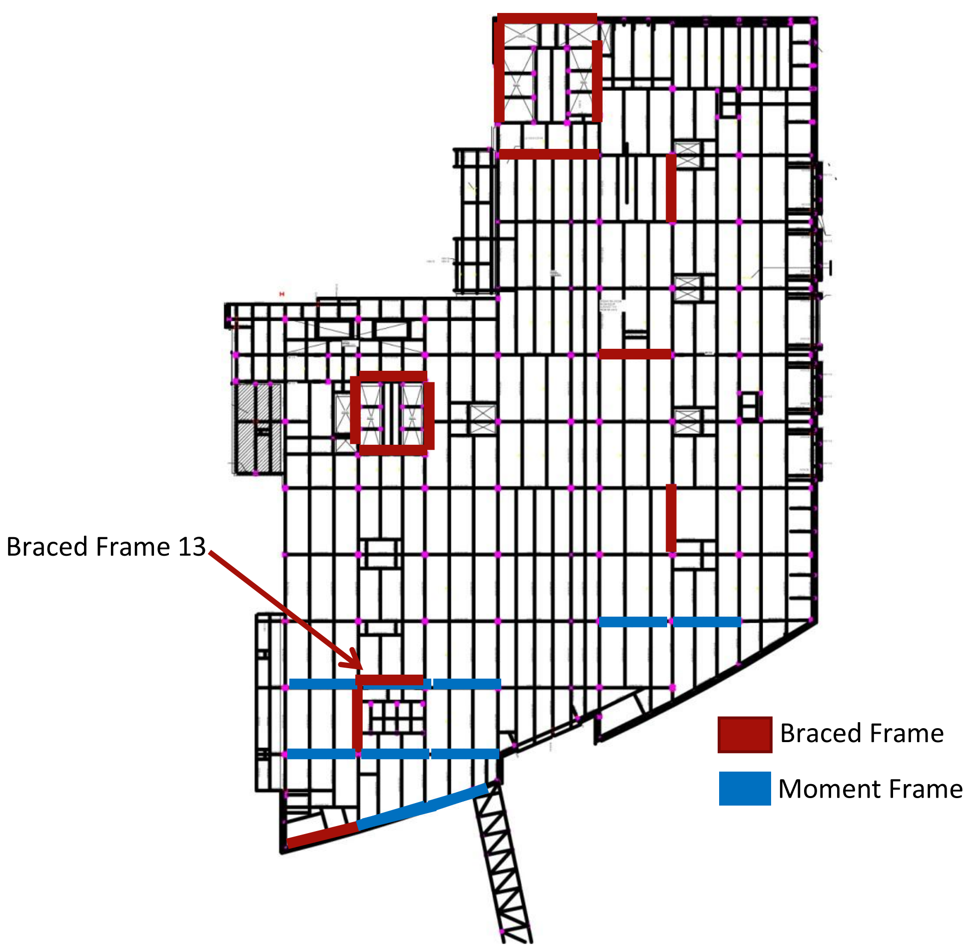


Figure 9 – Typical Layout of Lateral System

Design Codes and Standards

Ryan-Biggs Associates abided by these standards and codes when developing the design of the Patient Pavilion:

- AISC 13th Edition Manual
- AISC Specification 360-05
- 2007 Building Code of New York State (BCNYS)
- Minimum Design Loads for Buildings and Other Structures (ASCE7-02)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

Standards and codes utilized for this report:

- AISC 14th Edition Manual
- AISC Specification 360-10
- 2006 International Building Code (IBC 2006)
- Minimum Design Loads for Buildings and Other Structures (ASCE7-05)
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD)

Materials

The structural materials designated by the AISC 13th Ed. were used in the design of the Patient Pavilion by Ryan-Biggs; see *Table 1* for the capacities of the large variety of structural elements. The materials were specified on the General Notes page, S001, on the Construction Documents provided via Gilbane Building Company. All steel materials below are according to ASTM standards.

Table 1 – Material Properties

Material Properties		
Material		**Strength**
Rolled Steel	**Grade**	**f_y = ksi**
W Shapes	A 992	50
C, S, M, MC, and HP Shapes	A 36	36
Plates, bars, and angles	A 36	36
HSS pipe	A53 type E or S Grade B	35
Reinforcing Steel	A 615	60
Concrete	**Weight (lb/ft^3)**	**f'_c = psi**
Footings/mat foundation	145	3,000
Interior S.O.G or Slab on Deck	145	3,500
Foundation Walls, Shear walls, Piers, Pile caps, and Grade beams	145	4,000
Exterior S.O.G.	145	4,500
Masonry	**Grade**	**f'_m = psi**
Concrete Block	C 90	2,800
Mortar	C 270 Type S	n/a
Unit Masonry	n/a	2,000
Grout	C 476	2,500
Brick	C 216 type FBS Grade SW	3,000
Welding Electrodes	E70 XX	70 ksi

Loads

In the following tables, dead and live loads that were used to analyze and design the Patient Pavilion are listed as well as the loads used for this thesis. Live loads interpreted by the designer were derived from ASCE7-02, live loads used in this thesis were derived from ASCE 7-05; dead loads were assumed or calculated and verified with specified dead loads on the structural general notes.

Dead Loads

The dead loads listed on the general notes of the structural drawings are listed below in *Table 2.* Upon further analysis shown in *Table 3* and *Table 4*, the assumptions of these loads were verified to be accurate and conservative in some cases. The MEP is larger than typical because in a hospital the MEP weight is to be assumed larger than typical.

Table 2 – Superimposed Dead Loads

Dead Loads (As Shown on General Notes S100)	
Description	**Weight (psf)**
Roof Without Conc. Slab	30
Roof With Conc. Slab	95
Roof Garden	325
Floor	95
Level 9 Mechanical Penthouse	125

Table 3 – Roof without Conc. Slab Verification

Roof Without Conc. Slab Verification (ASCE7-05 and Vulcraft)	
Description	**Weight (psf)**
MEP	12
3”x16ga decking	5
Rigid Insulation (tapered starting at 8”)	.75psf per in thickness=(.75x8x.5)= 12
Total	**29**

Table 4 – Roof with Conc. Slab and Floor Verification

Roof With Conc. Slab and Floor Verification (ASCE7-05 and Vulcraft)	
Description	**Weight (psf)**
MEP	12
3”x20ga Composite Decking	48
Steel Framing	13
Finishes and Partitions	15
Fireproofing	2
Miscellaneous	5
Total	**95**

Live Loads

See *Table 5* for the controlling live load description per each level with the exception of elevator lobbies and stairs. The live loads given on the structural general notes were obtained using ASCE7-02, they were rechecked according to ASCE7-05 and were deemed accurate, see *Table 6.*

Table 5 – Live Loads

Live Loads (As Shown on General Notes S100)	
Description	**Weight (psf)**
Entry	100
Basement	100
Level 1	100
Level 2	100
Level 3	80
Level 4	80
Level 5	80
Level 6	80
Level 7	80
Level 8	80
Level 9 (Mechanical Penthouse)	125
Elevator Lobbies and Stairs	100

Table 6 – Verifying Live Loads per ASCE7-05

Level 1 – Level 2; Verification (ASCE7-05)	
Occupancy	**Weight (psf)**
Assembly Areas – Lobby	100
Hospitals – OR Rooms	60 + Partitions
Hospitals – Patient Rooms	40 + Partitions
Hospitals – Corridors above 1st Floor	80

Snow Load

The snow load for the Patient Pavilion was determined per ASCE7-05 section 7.3. Following the procedure described in this section, the flat roof snow load was calculated to be 37 psf, approximately 40psf, which was listed on the structural general notes. Hand calculations can be found in Appendix A.

Upon finding the density of the snow, and back figuring the density to find the height, it was determined the flat roof snow load height was 2 feet; this eliminates drift along the parapets, which are 2 feet high. Snowdrifts were calculated against the stair towers (See *Figure 9)* where windward drift loads control because of a larger l_u. Due to the windward forces control, the height of the snow load was reduced by using 3/4 of h_d, however after interpretation of the code the full h_d was used to calculate the drift width W. The height and weight of the drift is shown below in *Figure 9*, the location of each drift calculated is shown in *Figure 10.*

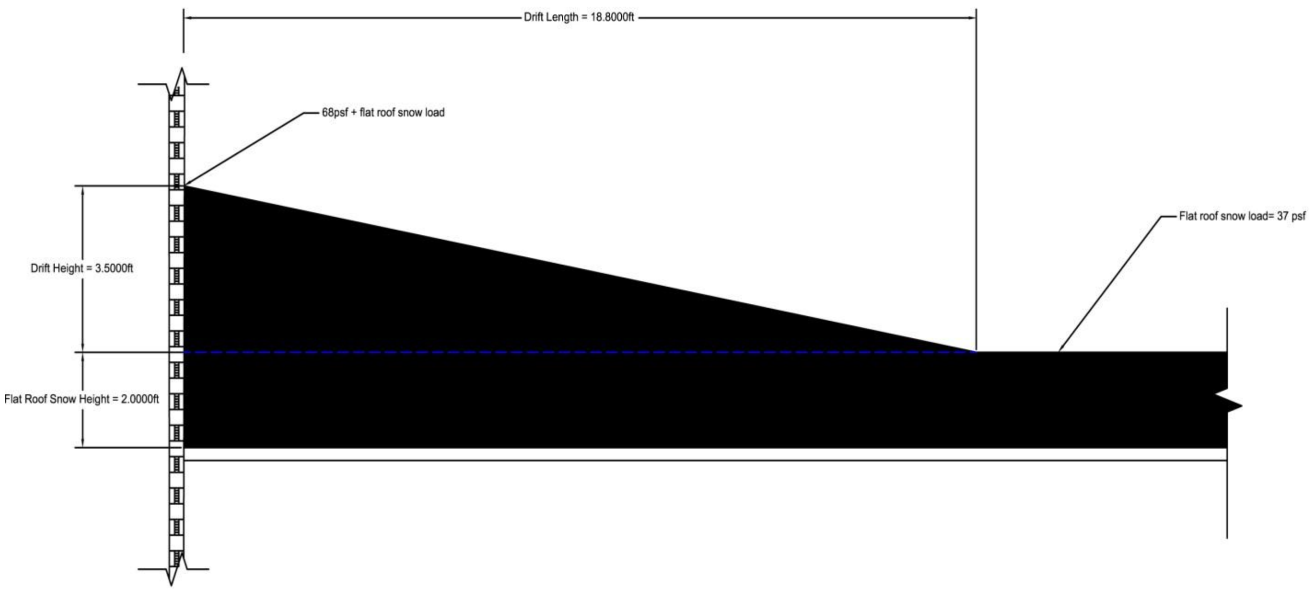


Figure 11 – Snow Drift

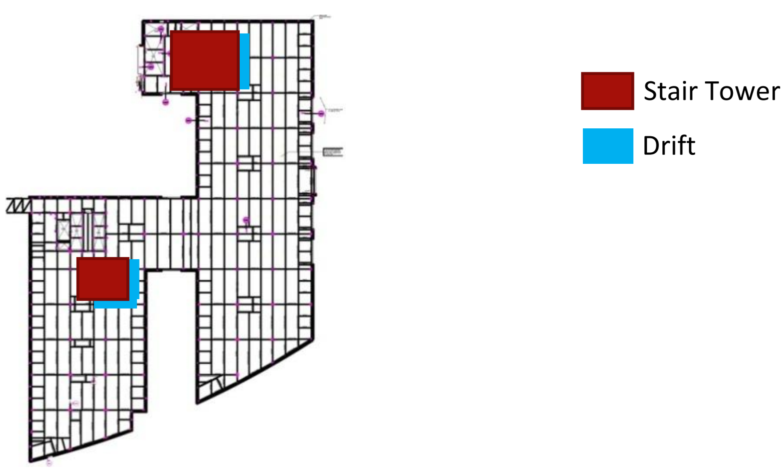


Figure 12 Drift and Stair Tower Locations

Wind Loads

Wind loads were calculated by Method 2, Main Wind Force Resisting System (MWFRS), provided in ASCE7-05 Chapter 6 to determine wind pressures in both the North-South direction and East-West direction. Initial assumptions had to be made for this procedure; the building footprint had to be projected into a rectangle, which is a valid assumption because the lateral systems run in two orthogonal directions (*See Figure 11*). Also the structure had to be assumed as a flexible structure and later verified through calculations which can be found in Appendix B.

A flexible building is defined in the ASCE7-05 as building with a frequency of 1Hz or less, equations to calculate the natural frequency are provided in the commentary in the ASCE7-05. Calculating the lower bound frequency (Eq C6-17) and the Average Value frequency (Eq C6-18), the natural frequency was less than 1Hz, the assumption of a flexible building was verified.

The calculations required for this analysis are redundant and time consuming; to simplifying the redundant process, a Microsoft Excel spreadsheet was created. The spreadsheet calculates windward and leeward forces, as well as story shear and overturning moment, in the North-South direction and East-West direction. The final forces in the North-South direction and East-West direction are shown in the following tables, as well as a schematic depiction showing the wind pressures and wind forces along the building height.

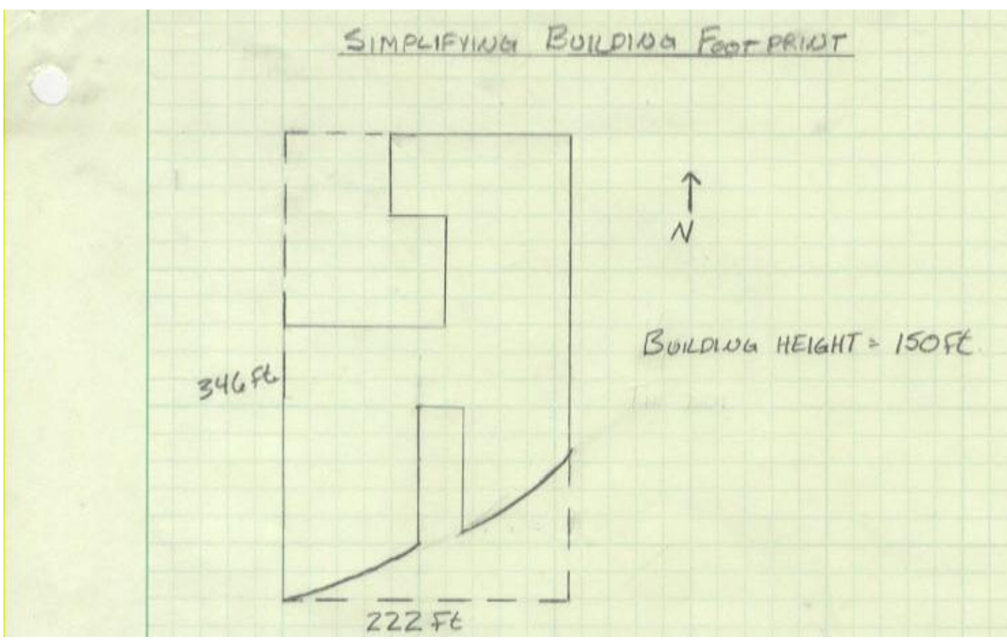


Figure 13 – Simplified Building Footprint

Table 7 – Wind Pressures; North-South Direction

Wind Pressure					
	Windward (psf)	Leeward (psf)	Internal Pressures (+/-)	Net Pressure	
				$(+GC_{pi})$	$(-GC_{pi})$
Entry Level	7.77	-7.27	4.01	3.75	11.78
Basement	7.77	-7.27	4.01	3.75	11.78
Level 1	9.21	-7.27	4.01	5.20	13.22
Level 2	10.46	-7.27	4.01	6.45	14.48
Level 3	11.17	-7.27	4.01	7.16	15.18
Level 4	11.77	-7.27	4.01	7.76	15.78
Level 5	12.37	-7.27	4.01	8.36	16.38
Level 6	13.08	-7.27	4.01	9.07	17.09
Level 7	13.49	-7.27	4.01	9.47	17.50
Level 8	14.03	-7.27	4.01	10.02	18.05
Level 9	14.58	-7.27	4.01	10.56	18.59

Table 8 – Roof Uplift; North-South Direction

Roof	Uplift (psf)	Internal Pressures (+/-)	$(+GC_{pi})$	$(-GC_{pi})$
0 to 75 ft	-16.86	4.01	-20.87	-12.85
75 to 150 ft	-16.86	4.01	-20.87	-12.85
150 to 300 ft	-9.37	4.01	-13.38	-5.35
>300 ft	-5.62	4.01	-9.63	-1.61

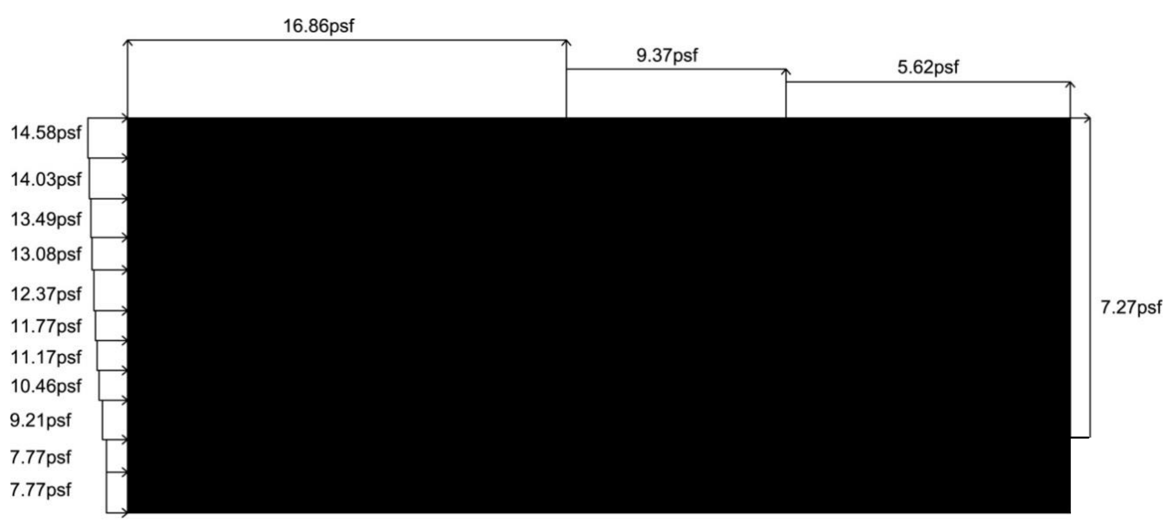


Figure 14 – Wind Pressures; North-South Direction

Table 9 – Wind Forces; North-South Direction

Wind Forces							
	Trib Heights		Elevation	Wall Width (Perp. To N-S)	Trib. Area	Story Force (kips)	Story Shear (kips)
	Below	Above					
Entry Level	0	7.5	0	222	1665	25.03	616.67
Basement	7.5	6	15	222	2997	45.06	591.64
Level 1	6	7.25	27	222	2941.5	48.47	546.58
Level 2	7.25	5.5	41.5	222	2830.5	50.19	498.11
Level 3	5.5	5.5	52.5	222	2442	45.03	447.93
Level 4	5.5	5.5	63.5	222	2442	46.49	402.90
Level 5	5.5	7.5	74.5	222	2886	56.68	356.40
Level 6	7.5	6	89.5	222	2997	60.98	299.73
Level 7	6	7.125	101.5	222	2913.75	60.48	238.75
Level 8	7.125	7.5	115.75	222	3246.75	69.16	178.27
Level 9	7.5	7.5	130.75	222	3330	72.74	109.12
Level 10	7.5	0	145.75	222	1665	36.37	37.22
						Total Base Shear=	**616.67**

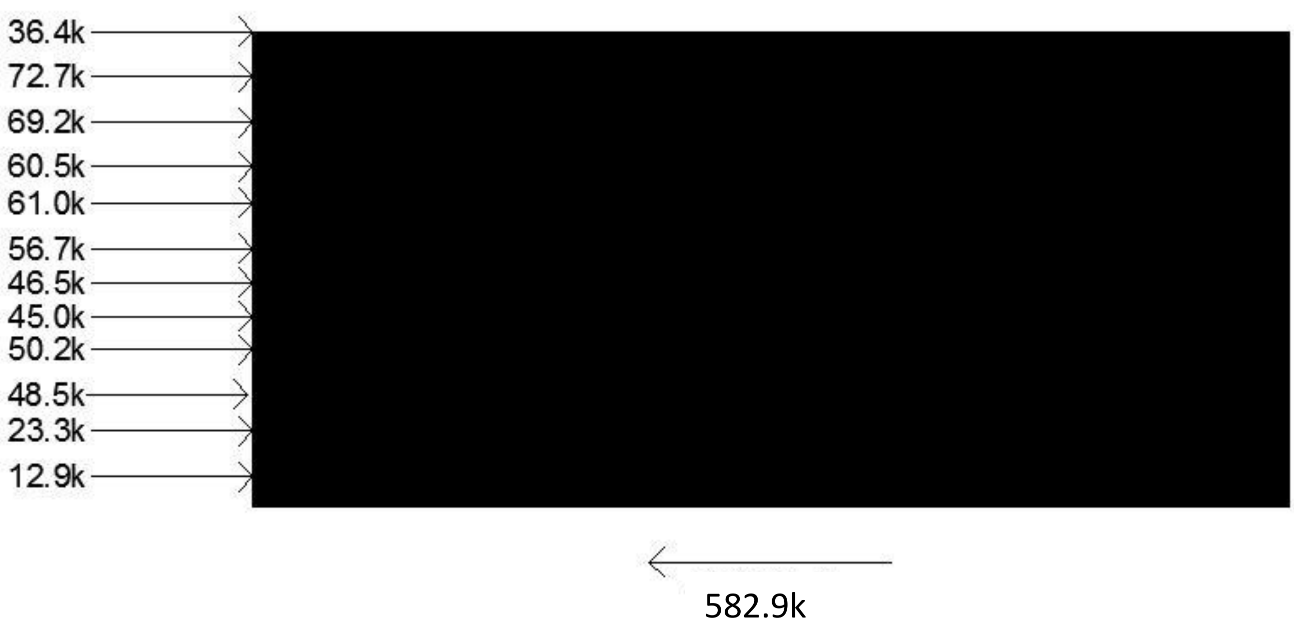


Figure 15 – North-South Wind Forces

Table 10 –Wind Pressures; East-West Direction

Wind Pressure					
	Windward (psf)	Leeward (psf)	Internal Pressures (+/-)	Net Pressure	
				($+GC_{pi}$)	($-GC_{pi}$)
Entry Level	7.56	-9.11	4.01	3.54	11.57
Basement	7.56	-9.11	4.01	3.54	11.57
Level 1	8.96	-9.11	4.01	4.95	12.97
Level 2	10.18	-9.11	4.01	6.17	14.19
Level 3	10.87	-9.11	4.01	6.86	14.88
Level 4	11.45	-9.11	4.01	7.44	15.47
Level 5	12.04	-9.11	4.01	8.02	16.05
Level 6	12.73	-9.11	4.01	8.71	16.74
Level 7	13.12	-9.11	4.01	9.11	17.14
Level 8	13.65	-9.11	4.01	9.64	17.67
Level 9	14.18	-9.11	4.01	10.17	18.20

Table 11 – Roof Uplift; East West Direction

Roof	Uplift (psf)	Internal Pressure (+/-)	($+GC_{pi}$)	($-GC_{pi}$)
0 to 75 ft	-19.48	4.01	-23.49	-15.47
75 to 150ft	-15.55	4.01	-19.56	-11.53
150 to end	-10.68	4.01	-14.69	-6.66

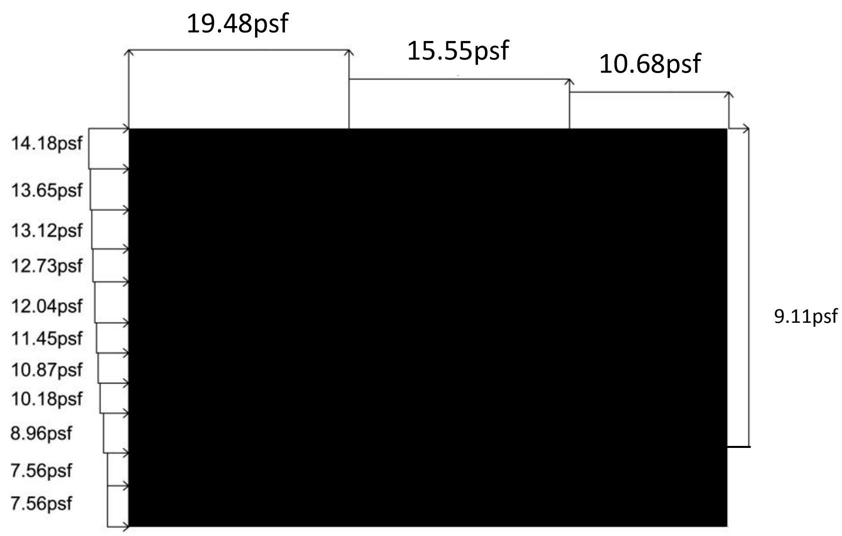


Figure 16 – Wind Pressures; East-West Direction

Table 12 – Wind Forces; East-West Direction

Wind Forces							
	Trib Heights		Elevation (ft)	Wall Width (ft)	Trib. Area (sf)	Story Force (k)	Story Shear (k)
	Below	Above					
Entry	0	7.5	0	346	2595	19.6	971
Basement	7.5	6	15	346	4671	35.3	952.33
Level 1	6	7.25	27	346	4584.5	82.86	917.03
Level 2	7.25	5.5	41.5	346	4411.5	85.12	834.17
Level 3	5.5	5.5	52.5	346	3806	76.06	749.05
Level 4	5.5	5.5	63.5	346	3806	78.28	672.99
Level 5	5.5	7.5	74.5	346	4498	95.13	594.72
Level 6	7.5	6	89.5	346	4671	102.01	499.58
Level 7	6	7.125	101.5	346	4541.25	100.99	397.57
Level 8	7.125	7.5	115.75	346	5060.25	115.21	296.58
Level 9	7.5	7.5	130.75	346	5190	120.92	181.37
Level 10	7.5	0	145.75	346	2595	60.46	60.46
						Total Base Shear=	**972**

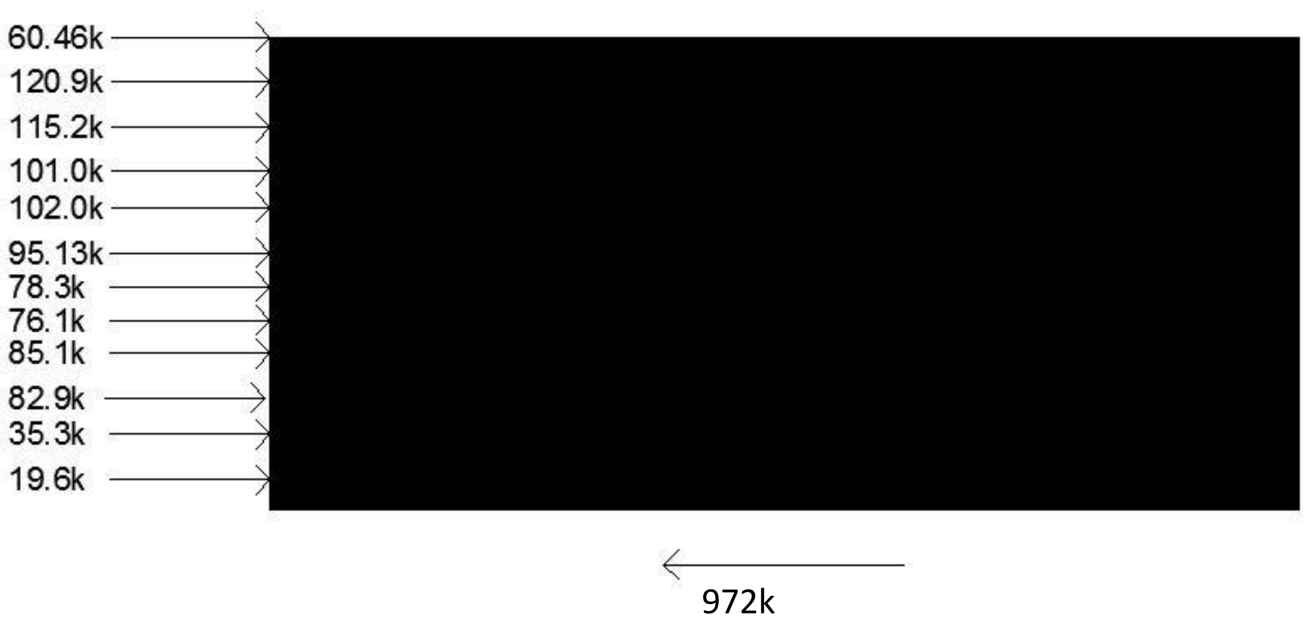


Figure 17 – Wind Forces; East-West Direction

Eleven serviceability load combinations are used to check total and story drifts for wind, these wind load cases are defined in Fig. 6-9, Chapter 6 in ASCE7-05. By inspection and knowledge of the center of rigidity and center of mass of a structure, several of these load combinations can be disregarded. However the load cases that are disregarded vary from project to project, they depend on the moment induced in the structure, which causes additive and subtractive forces in the lateral frames. Below in *Table 13* are the eleven load cases specified in the ASCE7-05, Chapter 6.

Table 13 – Wind Load Cases

Case 1	**$PW_x + PL_x$**
	$PW_y + PL_y$
Case 2	$0.75P_{Wx} + 0.75P_{Lx} + M_T$
	$0.75P_{Wx} + 0.75P_{Lx} - M_T$
	$0.75P_{Wy} + 0.75P_{Ly} + M_T$
	$0.75P_{Wy} + 0.75P_{Ly} - M_T$
Case 3	$0.75(P_{Wx} + P_{Lx})+0.75(P_{Wy} + P_{Ly})$
Case 4	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T(+e_x,+e_y)$
	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T(-e_x,-e_y)$
	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T(+e_x,-e_y)$
	$0.563P_{WX} + 0.563P_{LX} + 0.563P_{WY} + 0.563P_{LY} + M_T(-e_x,+e_y)$

Seismic Loads

The seismic design of the Patient Pavilion follows the Equivalent Lateral Force Procedure (ASCE7-05) described in Chapter 12. Seismic Ground Motion Values were obtained per ASCE7-05, Chapter 11.4, the initial parameter necessary for the Equivalent Lateral Force Procedure were calculated, and parameters S_s and S_1 were found using this online reference (*http://earthquake.usgs.gox/research/hazmaps/design/*) provided in graduate course AE597A. After reviewing the geotechnical report, it was determined that the average shear wave velocity, $\overline{v_s}$, was 716 feet per second, from table 20.3-1 a $\overline{v_s}$ of 716 feet per second classifies the soil as class D, stiff soil.

Following the Equivalent Lateral Force Procedure, the building weight must be determined in order to find the seismic response coefficient, C_s. This was performed by counting the beams and columns and multiplying the length by their unit weights. The tributary height of the columns was found by taking half of the height to next level up plus half the height from the lower level. Using the Vulcraft Metal Decking catalog a floor load of 48psf was determined for 3 1/2"x20ga composite decking with lightweight concrete. Superimposed dead loads were determined by subtracting the floor dead load of 45psf from the given floor dead load on the structural general notes. The weight of the exterior façade was determined by assuming dead load of 48psf for exterior stud walls with brick veneers via table C3-1 (ASCE7-05). To apply this load to each level the self-weight was multiplied by the perimeter and the tributary height of each level. Summarized in *Table 14* below are the weights of each element contributing to the seismic calculation.

Table 14 – Building Weight

	Framing	Floor	Columns	Façade	Dead	20% snow	Total Weight (k)
Entry	375.9115885	2138.454	211.5	789.6	2093.903		5609
Basement	375.9115885	2138.454	211.5	789.6	2093.903		5609
Level 1	581.5651741	2559.648	213.7	838.2394	2506.322		6699
Level 2	570.97604	2565.843	165.32	1198.337	2483.01		6983
Level 3	534.66928	2092.368	136.4	1108.8	2048.777		5921
Level 4	396.15239	2114.496	135.6	1064.448	2070.444		5781
Level 5	396.15239	2113.872	157	1257.984	2069.833		5995
Level 6	396.15239	2113.872	154.64	1306.368	2069.833		6041
Level 7	396.15239	2113.872	148.7	1270.08	2069.833		5999
Level 8	396.15239	2113.872	166.1	1415.232	2069.833		6161
Level 9	396.15239	2113.872	88.84	1451.52	2069.833	352.312	6473
Level 10	25.62584	88.992	2.9	180	87.138	14.832	399
						Total Weight=	**67671**

After obtaining the weights of each level, the seismic coefficient was determined using equation 12.8-3 (ASCE) because the value calculated from equation, 12.8-2 was larger than the allowable upper limit defined in equation 12.8-3. An excel spreadsheet (provided in AE597A) was utilized to determine the shear distribution and overturning moment for each level, refer to *Table 15* below for the Excel spreadsheet. Provided below is a schematic description showing the story forces, base shear, and overturning moment. Hand calculations can be found in Appendix C.

Table 15 – Seismic Distribution

	i	h_i (ft)	h (ft)	W (kips)	$w*h^k$	C_{VX}	f_i (k)	V_i (k)	Bx (ft)	5%Bx	Ax	M_z (k-ft)
E-W Direction	10	15.0	155	399	3985254	0.018	43	43	339	17	1.0	737
	9	15.0	140	6473	53721073	0.244	586	630	339	17	1.0	9941
	8	14.3	125	6161	41605387	0.189	454	1084	339	17	1.0	7699
	7	12.0	111	6000	32512839	0.148	355	1439	339	17	1.0	6016
	6	15.0	99	6040	26570786	0.121	290	1729	339	17	1.0	4917
	5	11.0	84	5995	19551044	0.089	213	1942	339	17	1.0	3618
	4	11.0	73	5781	14600735	0.066	159	2101	339	17	1.0	2702
	3	11.0	62	5921	11108047	0.051	121	2223	339	17	1.0	2056
	2	14.5	51	6983	9182133	0.042	100	2323	339	17	1.0	1699
	1	12.0	37	6700	4785959	0.022	52	2375	339	17	1.0	886
	Basement	15.0	25	5609	1941543	0.009	21	2396	339	17	1.0	359
	Entry	9.6	10	5609	349616	0.002	4	2400	339	17	1.0	65
			Σ	67671	219914416		2400					40695
N-S Direction	i	h_i (ft)	h (ft)	W (kips)	$w*h^k$	C_{VX}	f_i (k)	V_i (k)	Bx (ft)	5%Bx	Ax	M_z (k-ft)
	10	15.0	155	399	3985254	0.018	43	43	216	11	1.0	470
	9	15.0	140	6473	53721073	0.244	586	630	216	11	1.0	6332
	8	14.3	125	6161	41605387	0.189	454	1084	216	11	1.0	4904
	7	12.0	111	6000	32512839	0.148	355	1439	216	11	1.0	3832
	6	15.0	99	6040	26570786	0.121	290	1729	216	11	1.0	3132
	5	11.0	84	5995	19551044	0.089	213	1942	216	11	1.0	2304
	4	11.0	73	5781	14600735	0.066	159	2101	216	11	1.0	1721
	3	11.0	62	5921	11108047	0.051	121	2223	216	11	1.0	1309
	2	14.5	51	6983	9182133	0.042	100	2323	216	11	1.0	1082
	1	12.0	37	6700	4785959	0.022	52	2375	216	11	1.0	564
	Basement	15.0	25	5609	1941543	0.009	21	2396	216	11	1.0	229
	Entry	9.6	10	5609	349616	0.002	4	2400	216	11	1.0	41
			Σ	67671	219914416		2400					25920

Serviceability load combinations for seismic are shown in *Table 15* and are to be used to calculate total drift and story drift. The M_a which is defined in ASCE7-05 12.8.4.2, is the accidental moment due to an eccentricity of 5% the width of the floor plan. For example, seismic loading in the X-Direction, the eccentricity of the accidental moment will be 5% of the Y-Direction.

Table 16 – Seismic Servicability Load Cases

X-Direction	$E_{Qx+}M_a$
Y-Direction	$E_{Qy+}M_a$

Computer Model

A computer model was created using ETABS to simplify the analysis of the multiple load cases on the Patient Pavilion. Material properties were created first and all masses were turned off on the materials to not double count the mass applied to the rigid diaphragm. In ETABS, mass is input in mass/area, therefore the asses were determined by dividing the total weight of the floor by the area of the floor and 32.2 ft-sec^2 and 12^3. For the concrete shear walls a property modifier of f_{22}=0.5 was applied to account for cracking.

The lateral frames were modeled and not the gravity frames because only the lateral response to the seismic and wind load combinations were analyzed for this report. The lateral frames consisted of braced frames in both directions and some moment frames in the East-West direction. The braces in the braced frames were assigned moment releases in the 3-3 direction at each end, accounting only for axial load in the braces. The shear walls in the basement were modeled as a membrane accounting only for in plane loading. For example, a 28" thick concrete wall was assigned a membrane thickness of 28" as well as a bending thickness of 28". The floor slab in the Patient Pavilion is 6 1/2" lightweight concrete on metal deck, this floor system provides enough rigidity to be modeled as a rigid diaphragm.

An assumption was made to model the base of the columns as fixed due to strong connections detailed in the structural drawings. The subgrade shear walls were modeled due to a sloping site. On the Southern side the ground floor is the basement level, but on the Northern side of the site the lower two stories are exposed.

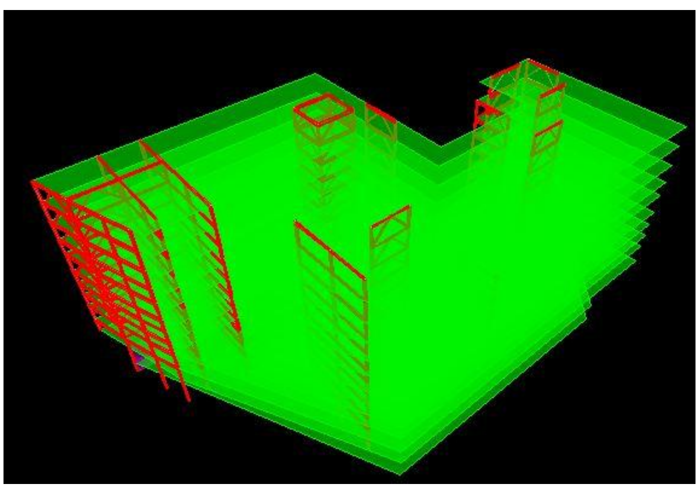


Figure 18 - ETABS Model

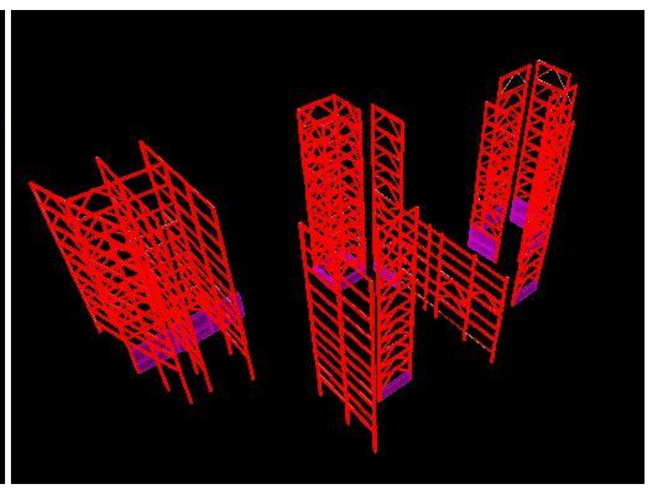


Figure 19 - ETABS Model; Diaphrams Hidden

Relative Stiffness

Relative stiffness is defined as the stiffness of a specific member in relation to the total stiffness of all the members on a given level. The relative stiffness was calculated by applying a 1000 kip load in each direction at the COR in the ETABS model, then for each story taking section cuts of every frame in a given direction. The load was applied at the COR with an eccentricity of zero in order to not account for torsion. After taking section cuts of all the frames in one direction, the total force shall equal 1000 kips or be relatively close. Once all the forces were obtained from the section cuts, relative stiffness was obtained by dividing the force in a given frame by the summation of the forces in all the frames in that story, see *Table 17 and 18* for relative stiffness's of the Patient Pavilion's frames per story.

Table 17 – Relative Stiffness in East-West Direction

E-W Direction	BF-8	BF-9	BF-10	BF-11	BF-12	MF-1	BF-13	MF-2	MF-3	MF-4	BF-14
Entry	-	-	-	-	-	-	-	-	-	-	-
Basement	-	-	-	-	-	-	-	-	-	-	-
Level 1	13%	17%	22%	5%	9%	2%	29%	3%	0.1%	-0.1%	0.2%
Level 2	12%	13%	27%	8%	7%	1%	31%	2%	0.4%	0.5%	0.4%
Level 3	13%	4%	42%	4%	10%	2%	3%	3%	2%	2%	16%
Level 4	15%	17%	13%	6%	15%	2%	3%	4%	2%	3%	21%
Level 5	14%	16%	13%	7%	14%	2%	4%	4%	3%	3%	20%
Level 6	12%	15%	15%	15%	14%	1%	2%	3%	2%	2%	19%
Level 7	14%	17%	16%	11%	7%	2%	3%	3%	3%	3%	21%
Level 8	12%	16%	14%	13%	13%	2%	2%	3%	3%	2%	19%
Level 9	14%	20%	15%	7%	13%	2%	3%	3%	1%	2%	21%
Level 10	11%	0%	22%	21%	17%	2%	3%	4%	1%	2%	17%

Table 18 – Relative Stiffness in North-South Direction

N-S Direction	BF-1	BF-2	BF-3	BF-4	BF-5	BF-6	BF-7	MF-3	MF-4	BF-14
Entry	-	-	-	-	-	-	-	-	-	-
Basement	-	-	-	-	-	-	-	-	-	-
Level 1	10%	10%	18%	19%	8%	7%	30%	0%	0%	0%
Level 2	10%	10%	13%	17%	6%	5%	38%	0%	0%	0%
Level 3	10%	10%	13%	17%	10%	10%	22%	0%	0%	7%
Level 4	10%	10%	12%	16%	11%	11%	22%	0%	0%	8%
Level 5	11%	11%	11%	15%	11%	11%	22%	0%	0%	9%
Level 6	11%	11%	11%	15%	12%	12%	21%	0%	0%	8%
Level 7	11%	11%	11%	14%	12%	13%	21%	0%	0%	9%
Level 8	11%	11%	11%	14%	12%	13%	20%	0%	0%	8%
Level 9	11%	11%	12%	14%	12%	13%	19%	0%	0%	8%
Level 10	14%	14%	7%	5%	14%	14%	22%	0%	0%	9%

A weak link was found in the East-West framing direction on Level 3, circled above in *Table 17*. At this level the geometry of BF-9 changes dramatically and this reduces it's relative stiffness. To compensate for the loss in stiffness in BF-9 the other lateral frames must take more load. In the table above if is shown that BF-10 acquires the additional load that was lost from BF-9.

An issue arose when trying to find relative stiffness for each floor. When taking section cuts were made through the subgrade walls the total force in all the frames in a given direction were much larger than the applied 1000kip load. It was thought that torsion may be an issue, but that was ruled out because the load was applied at the COR. It was found that the issue was with shear reversal, which occurs when there is a dramatic change in stiffness in the lateral system and the shear is amplified in the opposite direction. This could have been prevented by modeling the diaphragm as semi-rigid which would allow for some deformation of the diaphragm, see *Fig. 20* below for a braced frame experiencing shear reversal.

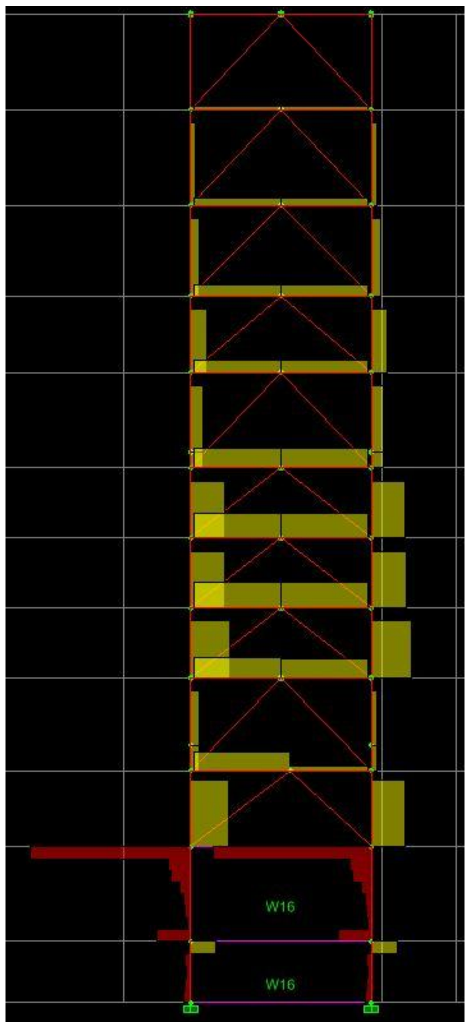


Figure 20 – Shear Reversal in Braced Frame

Shear

Shear consists of two components, direct and torsional shear. When a wind force or seismic force is applied to a building it is collected on the exterior façade and distributed from the façade to the diaphragm. The force is transferred through the diaphragm into the lateral frames, which distribute the load into the foundations then into the soil.

Direct Shear

Direct shear is calculated for each frame by multiplying the total shear force for a specific story by the relative stiffness of a lateral frame in consideration within that story. The principal of load follows stiffness is very applicable with direct shear.

Torsional Shear

In addition to direct shear, torsional shear must be considered to obtain total shear in a given story. Torsional shear is the shear induced by a moment in a story under consideration; the moment is produced by the offset of the COM and COR and by accidental torsion. The Patient Pavilion will experience torsion from the lateral loads because the COM and COR of the building is not in the same locations. The COM and COR was obtained from the ETABS model and hand calculations, in addition to the torsion induced by the direct shear, the accidental moment from seismic loading induces torsional shear into the frames.

To calculate torsional shear, the relative stiffness values were used in place of the actual rigidity of each frame. In addition, an assumption had to be made to include the lateral frames, BF-14, MF-4, and MF-3 on the Southern end of the building. These frames are not in either the X or Y-Axis, therefore their relative stiffness's are rotated off these axes. Geometry was used to rotate the local axis to align with the global axis, see *Fig. 21* below.

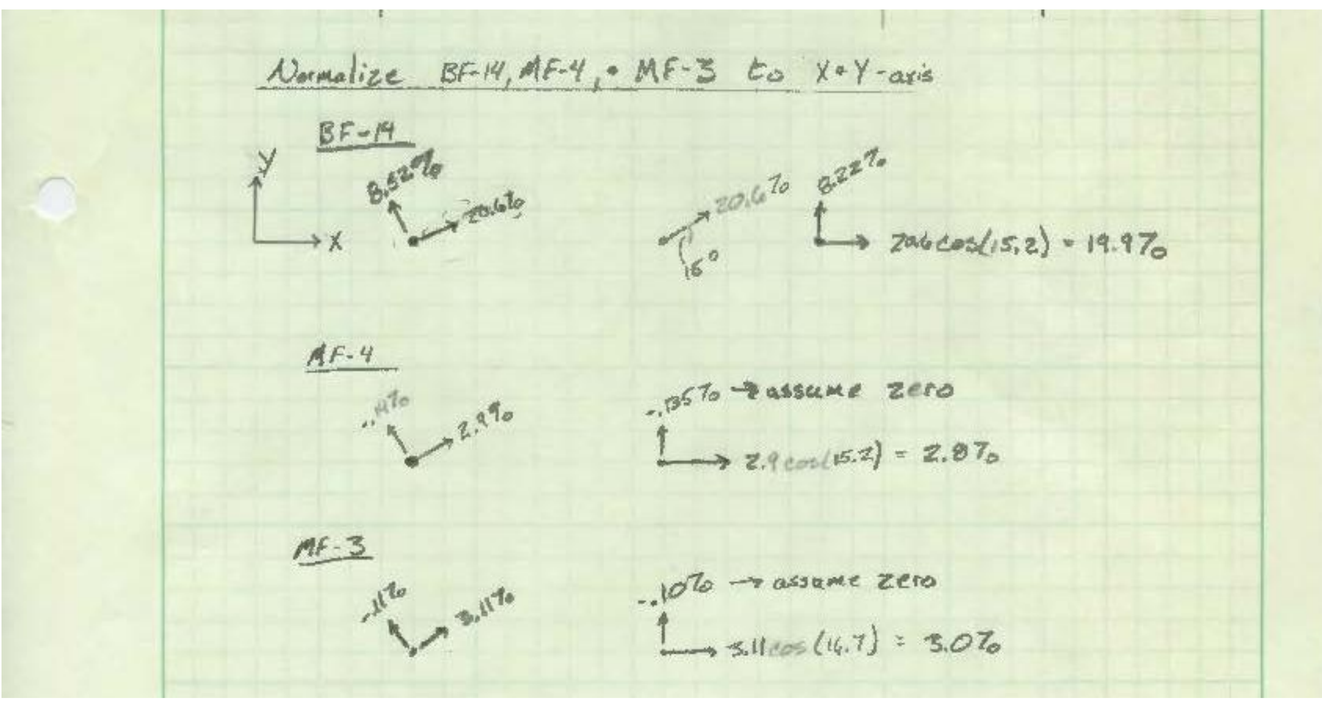


Figure 21 – Normalizing Relative Stiffness

Below, *Table17* shows the results of calculations of the total shear in each lateral frame at Level 7 in the Patient Pavilion. To verify the ETABS model, hand calculations were performed for a single force applied at Level 7 and section cuts were made through BF-10 and BF-11 , see *Figure 22*. See more detailed hand calculations in Appendix D.

Table 19 – Total Shear in Lateral Frames at Level 7

	Frame	V_{Direct} (k)	$V_{torsion}$ (k)	V_{total} (k)
X-Direction	BF-14	71.3	14.1	85.4
	MF-4	10.03	1.7	11.73
	MF-3	10.8	1.7	12.5
	BF-13	11.4	1.6	13
	MF-2	12.2	1.4	13.6
	MF-1	8.6	0.8	9.4
	BF-12	25.9	0.6	26.5
	BF-11	38.4	-0.22	38.18
	BF-10	55.6	-0.97	54.63
	BF-9	60.3	-6.3	54
	BF-8	50.6	-8.1	42.5

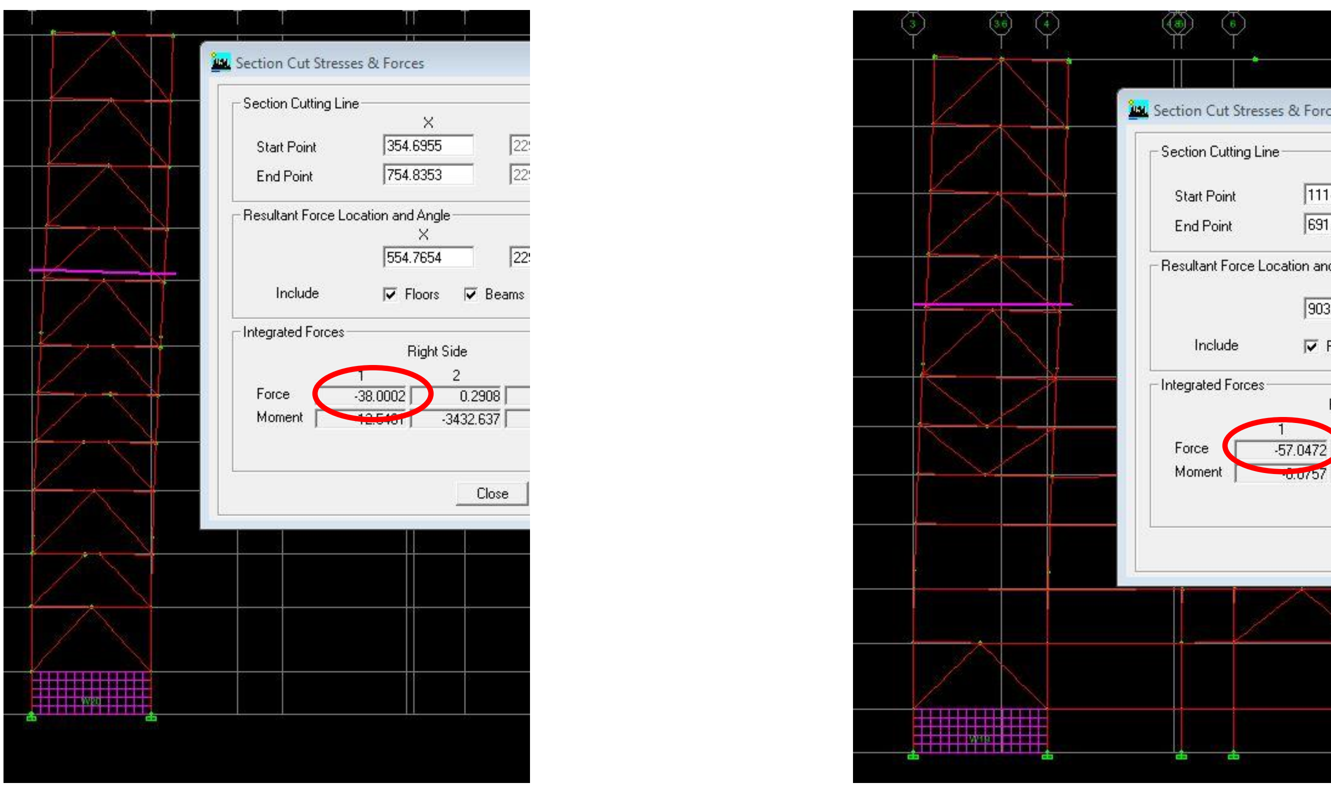


Figure 22 – ETABS Section Cut; BF-11(left) and BF-10(right)

Torsion

Torsion occurs when there is an eccentricity between the building's COM and COR, this eccentricity is the moment arm producing the torsion within the structure. For seismic, torsional irregularities were considered , these are controlled by ASCE7-05 Chapter 12, Table 12.3-1. Maximum and minimum drift values were obtained in the ETABS model and it was found that in the East-West direction there are no torsional irregularities, however in the North-South Direction there is Category 1a torsional irregularity. This means that the maximum drift for a given story is more than 1.2 times the average story drift at that level. *Tables 25 and 26* below show the torsional irregularity calculations for each direction.

Table 20 – Torsional Irregularity Checks East-West Direction

E-W Direction		Δ_{max}	Δ_{min}	$\Delta_{max}/\Delta_{ave}$	No Torsional Irregularity
	Entry	0	0	0	
	Basement	0	0	0	
	Level 1	0.18	0.23	0.88	
	Level 2	0.5	0.54	0.96	
	Level 3	0.84	0.83	1.01	
	Level 4	1.24	1.18	1.02	
	Level 5	1.7	1.53	1.05	
	Level 6	2.3	2.05	1.06	
	Level 7	2.8	2.5	1.06	
	Level 8	3.4	2.9	1.08	
	Level 9	4.03	3.3	1.10	
	Level 10	4.6	3.6	1.12	

Table 21 – Torsional Irregularity Checks North-South Direction

N-S Direction		Δ_{max}	Δ_{min}	$\Delta_{max}/\Delta_{ave}$	Torsional Irregularity
	Entry	0	0	0	
	Basement	0	0	0	
	Level 1	Level 2	0.12	1.29	
	Level 2	Level 3	0.3	1.30	
	Level 3	Level 4	0.51	1.26	
	Level 4	Level 5	0.74	1.24	
	Level 5	Level 6	1	1.21	
	Level 6	Level 7	1.4	1.19	
	Level 7	Level 8	1.7	1.18	
	Level 8	Level 9	2.02	1.18	
	Level 9	Level 10	2.4	1.17	
	Level 10	0	2.71	1.17	

Load Combinations

According to ASCE7-05 Chapter 2 there are 7 possible load combinations for strength they are as follows:

1. 1.4(D + F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(Lr or S or R)
3. 1.2D + 1.6(Lr or S or R) + (L or 0.8W)
4. **1.2D + 1.6W + L + 0.5(Lr or S or R)**
5. **1.2D + 1.0E + L + 0.2S**
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

The controlling load case for wind: **1.2D + 1.6W + L + 0.5(Lr or S or R)**
The controlling load case for seismic: **1.2D + 1.0E + L + 0.2S**

In Table below, the controlling load case for each floor, in each direction are shown. Wind forces were factored by 1.6 per Load Combination 4 and the seismic loads were factored by 1.0 as required per Load Combination 5. The wind load increases less exponentially with building height, however seismic loading increases greatly with building height, therefore wind mostly controls the lower stories and seismic mostly controls the upper stories.

Table 22 – Controlling Load Strength Case Per Level

		Seismic	Wind			Seismic	Wind
E-W Direction	Entry	4	31	N-S Direction	Entry	4	21
	Basement	21	56		Basement	21	37
	Level 1	52	133		Level 1	52	78
	Level 2	100	136		Level 2	100	80
	Level 3	121	122		Level 3	121	72
	Level 4	159	125		Level 4	159	74
	Level 5	213	152		Level 5	213	91
	Level 6	290	163		Level 6	290	98
	Level 7	355	162		Level 7	355	97
	Level 8	454	184		Level 8	454	111
	Level 9	586	193		Level 9	586	116
	Level 10	43	97		Level 10	43	58

Drift

Serviceability considerations take account for drift and displacement to reduce non-structural component damage when a building is experiencing maximum loading. Serviceability also includes the comfort of the building's inhabitants; large deflections or torsions on the upper level of a high rise can be very disturbing and uncomfortable. For each loading, seismic and wind, different provisions must be accounted for when considering drift. The total drifts and story drifts found in ETABS are summarized in *Table 21 and Table 22* below.

To verify the ETABS model, the deflected shape of each frame was considered. Braced frames and moment frames act differently when a load is applied, for braced frames the system deflects in a parabolic shape however a moment frame deflects linearly. When looking at the deflected frames it could also be seen if a member was not connected to the correct node because these parabolic and linear shapes would not be present. In *Fig. 23* below, the deflected shapes of two frames from the ETABS model are shown; a distinct parabolic shape is seen in the braced frame as well as a linear shape in the moment frame.

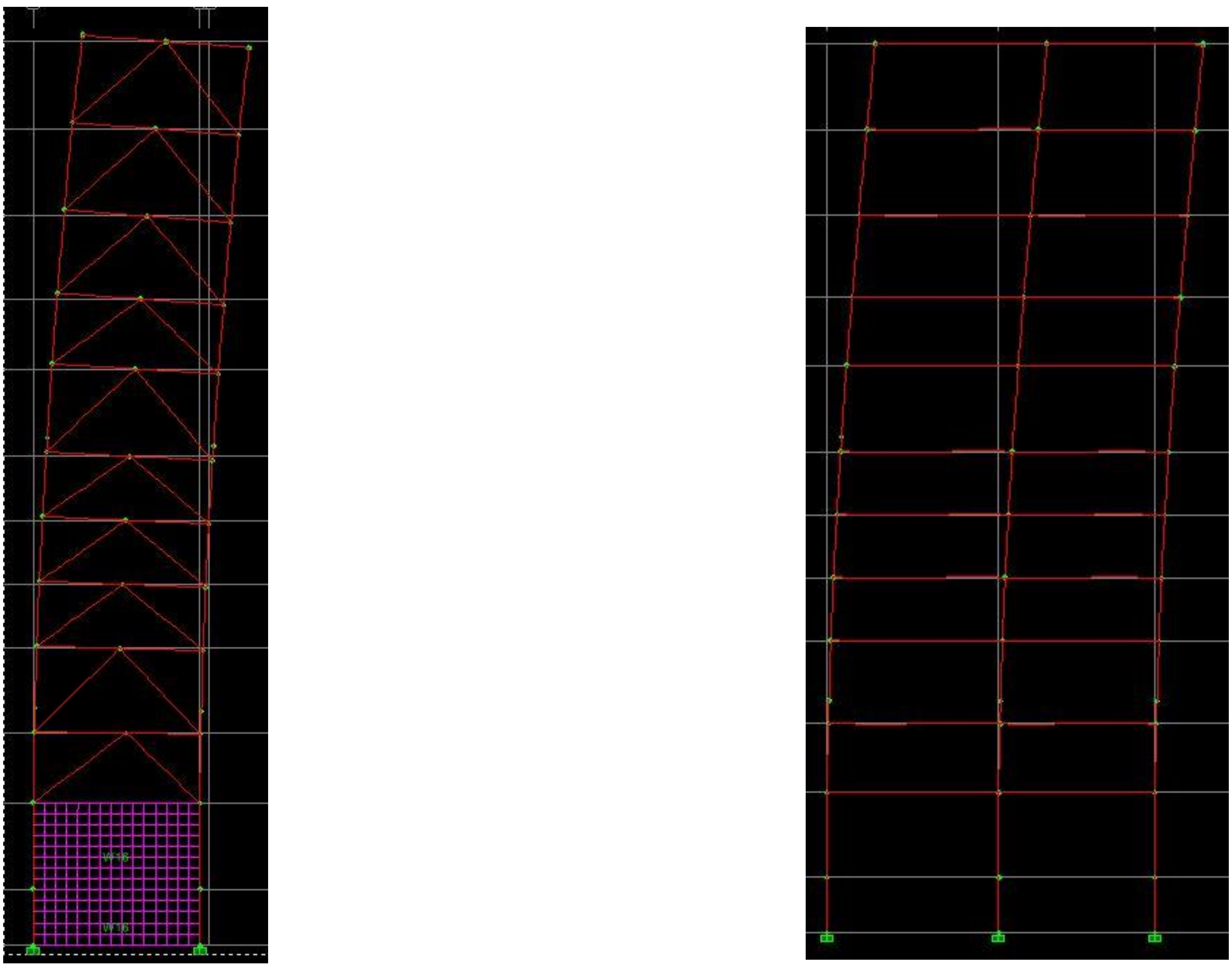


Figure 23 – Deflection; Braced Frame(left) and Moment Frame(right)

For seismic loading, the allowable story drift per ASCE7-05 Table 12.12-1 is $\Delta_{max} \leq 0.01h_{sx}$ and these were compared to the story drifts found in ETABS. The seismic drifts must be factored by C_d/I in order to get the building's actual response to the drift, and these factored values were used to calculate the story drift. All seismic drifts found in ETABS were deemed acceptable with the ASCE provisions.

Table 23 – Seismic Drift in East-West Direction

E-W Direction		P(kips)	Moment(k-ft)	δ_{xe}	δ_{ye}	$\delta_{xe}C_d/I$	$\delta_{ye}C_d/I$	Δ_{xe}	Δ_{ye}	Δ_{max}=0.010h_{sx}
	Entry Level	4	65	0	0	0	0	0	0	1.155
	Basement	21	359	0	0	0	0	0		1.8
	Level 1	52	1125	0.18	0.014	0.30	0.02	0.300	0.023	1.44
	Level 2	100	1954	0.5	0.004	0.83	0.01	0.533	-0.017	1.44
	Level 3	121	2158	0.84	0.05	1.40	0.08	0.567	0.077	1.32
	Level 4	159	2702	1.24	0.1	2.07	0.17	0.667	0.083	1.32
	Level 5	213	3618	1.7	0.17	2.83	0.28	0.767	0.117	1.32
	Level 6	290	4917	2.3	0.25	3.83	0.42	1.000	0.133	1.8
	Level 7	355	6016	2.8	0.34	4.67	0.57	0.833	0.150	1.44
	Level 8	454	7699	3.4	0.46	5.67	0.77	1.000	0.200	1.71
	Level 9	586	9941	4.03	0.6	6.72	1.00	1.050	0.233	1.8
	Level 10	43	737	4.6	0.76	7.67	1.27	0.950	0.267	1.8
	V_{base} =	2398								

Table 24 – Seismic Drift in North-South Direction

N-S Direction		P(kips)	Moment(k-ft)	δ_{xe}	δ_{ye}	$\delta_{xe}C_d/I$	$\delta_{ye}C_d/I$	Δ_{xe}	Δ_{ye}	Δ_{max}=0.010h_{sx}
	Entry Level	4	41	0	0	0	0	0	0	1.155
	Basement	21	229	0	0	0	0	0		1.8
	Level 1	52	621	0.04	0.22	0.07	0.37	0.067	0.367	1.44
	Level 2	100	1212	0.13	0.56	0.22	0.93	0.150	0.567	1.44
	Level 3	121	1388	0.19	0.87	0.32	1.45	0.100	0.517	1.32
	Level 4	159	1721	0.26	1.2	0.43	2.00	0.117	0.550	1.32
	Level 5	213	2304	0.32	1.54	0.53	2.57	0.100	0.567	1.32
	Level 6	290	3132	0.42	2.05	0.70	3.42	0.167	0.850	1.8
	Level 7	355	3832	0.49	2.45	0.82	4.08	0.117	0.667	1.44
	Level 8	454	4904	0.58	2.92	0.97	4.87	0.150	0.783	1.71
	Level 9	586	6332	0.67	3.4	1.12	5.67	0.150	0.800	1.8
	Level 10	43	470	0.75	3.8	1.25	6.33	0.133	0.667	1.8
	V_{base} =	2398								

Wind drifts are not controlled in the code, however in the commentary in the ASCE7-05 CC.1.2, for serviceability it is a rule of thumb to control the maximum story and total drift to L/400. The story drifts and the comparison to the rule of thumb are shown below in *Table 23*, the wind drifts and story drifts both met the rule of thumb.

Table 25 – Wind Drift

			$P_W + P_L$ (k)	M (ft-k)	δ_X	δ_Y	Δ_X	Δ_Y	Δ_a=L/400	$\Delta > \Delta_a$
Case 1	$PW_x + PL_x$	Entry Level	19.61	0	0	0	0	0	0.28875	Yes
		Basement	35.29	0	0	0	0	0	0.44499	Yes
		Level 1	82.86	0	0.106	0.016	0.106	0.016	0.36	Yes
		Level 2	85.12	0	0.248	0.036	0.142	0.02	0.436251	Yes
		Level 3	76.06	0	0.368	0.052	0.12	0.016	0.33	Yes
		Level 4	78.28	0	0.503	0.071	0.135	0.019	0.33	Yes
		Level 5	95.13	0	0.638	0.091	0.135	0.02	0.33	Yes
		Level 6	102.01	0	0.832	0.121	0.194	0.03	0.45	Yes
		Level 7	100.99	0	0.979	0.141	0.147	0.02	0.36	Yes
		Level 8	115.21	0	1.14	0.163	0.161	0.022	0.4275	Yes
		Level 9	120.92	0	1.3	0.182	0.16	0.019	0.45	Yes
		Level 10	60.46	0	1.43	0.193	0.13	0.011	0.45	Yes
	$PW_y + PL_y$		$P_W + P_L$ (k)	M (ft-k)	δ_X	δ_Y	Δ_X	Δ_Y	Δ_a=L/400	$\Delta > \Delta_a$
		Entry Level	12.93	0	0	0	0	0	0.28875	Yes
		Basement	23.27	0	0	0	0	0	0.44499	Yes
		Level 1	48.47	0	0.015	0.0523	0.015	0.015	0.36	Yes
		Level 2	50.19	0	0.041	0.131	0.026	0.0787	0.436251	Yes
		Level 3	45.03	0	0.061	0.197	0.02	0.066	0.33	Yes
		Level 4	46.49	0	0.081	0.266	0.02	0.069	0.33	Yes
		Level 5	56.68	0	0.103	0.336	0.022	0.07	0.33	Yes
		Level 6	60.98	0	0.132	0.437	0.029	0.101	0.45	Yes
		Level 7	60.48	0	0.156	0.517	0.024	0.08	0.36	Yes
		Level 8	69.16	0	0.185	0.611	0.029	0.094	0.4275	Yes
		Level 9	72.74	0	0.213	0.705	0.028	0.094	0.45	Yes
		Level 10	36.37	0	0.24	0.793	0.027	0.088	0.45	Yes

Overturning and Foundation Considerations

Overturning moments on a building are due to lateral loads on the building and the distance from the base of the building to the height of the load is the moment arm for that level. The moment is resisted in the foundation to prevent the building from overturning. In the Patient Pavilion's mat foundation, additional reinforcement is used as well as a strong connection from the column to the foundation in order to resist the uplift caused by overturning. When an overturning moment occurs, one end of the building will be forced into the ground demanding for sufficient soil bearing, where the other side of the building will be trying to pry away from the earth or out of it's foundation. *Table 24* below shows the overturning for the given loads at each level as well as the total over turning moment. The seismic load was factored using Load Combination 5 using 1.0E and the wind loads were totaled up then factored using Load Combination 4, 1.6W.

Table 26 – Overturning Moments

Overturning Moments							
		Seismic		Wind (East-West)		Wind (North-South)	
	Height (ft)	Lateral Force (k)	Moment (ft-k)	Lateral Force (k)	Moment (ft-k)	Lateral Force (k)	Moment (ft-k)
Level 10	155.417	43	6759	36.37	5653	19.61	3047
Level 9	140.417	586	82323	72.74	10215	35.29	4956
Level 8	125.417	454	56946	69.16	8673	82.86	10393
Level 7	111.167	355	39445	60.48	6723	85.12	9462
Level 6	99.167	290	28756	60.98	6047	76.06	7542
Level 5	84.167	213	17958	56.68	4770	78.28	6588
Level 4	73.167	159	11659	46.49	3402	95.13	6961
Level 3	62.167	121	7536	45.03	2799	102.01	6342
Level 2	51.167	100	5127	50.19	2568	100.99	5167
Level 1	36.625	52	1913	48.47	1775	115.21	4220
Basement	24.625	21	522	23.27	573	120.92	2978
Entry	9.625	4	37	12.93	124	60.46	582
		Total Overturning Moment=	**258981**	M_u= $1.6M_u$=	53323 **85317**	M_u= $1.6M_u$=	68237 **109180**

Spot Checks

Spot checks were performed for a diagonal brace and a column on Level 7, see *Fig. 25* below for location of members. For the brace, the member forces were found by cutting a section thru the brace and obtaining the shear values from the table. The axial force in the brace was determined from the shear force to properly analyze the brace. Checks done for the brace include tension yielding and rupture, as well as compression because the building will translate back and forth, therefore when one brace is in tension the other is in compression. The brace was deemed adequate for the derived loads in the ETABS model.

A column was analyzed in Level 7 also, the moments in the column were obtained by using the pier labeling tool in ETABS. Hand calculations were done to obtain the axial force in the column, considering dead live and snow loads, Load Combination 5 was used for this spot check. Live load reducing was not allowed for this calculation because the influence area was less than 400 sq ft. Considering both flexural and axial loads for this spot check, Table 6-1 in the 14 Ed. of the AISC was used to simplify the calculations. The column was deemed adequate for the derived loads.

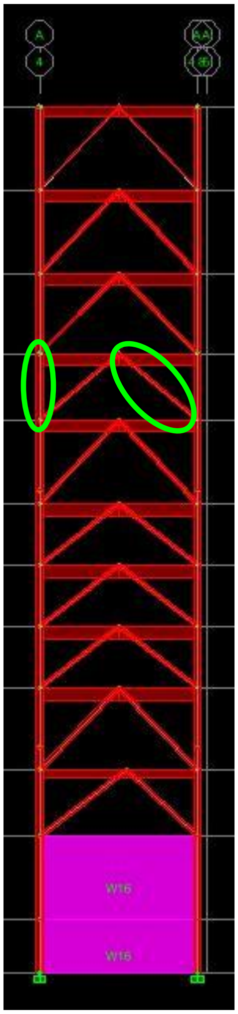


Figure 24 – Location of Members Analyzed

Conclusion

After a thorough analysis, the lateral system of the Patient Pavilion was found to be sufficient to carry the lateral loads determined per ASCE7-05. Conclusions were based up the use of computer modeling in ETABS, as well as hand calculations to find lateral loads and to verify the computer model. Both wind and seismic loads were obtained per ASCE 7-05; for wind the Main Wind Force Resisting System procedure was used and for seismic the Equivalent Lateral force procedure was used. It was found that in the lower levels wind controlled and in the upper levels seismic loading controlled, overall seismic controls.

A model of only the lateral system was built in ETABS to confirm the strength of the lateral system as well as analyze its serviceability. Appropriate assumptions had to be made in order to properly model the lateral system of the Patient Pavilion. Only the lateral frames of the Patient Pavilion were modeled for this report. The lateral frames consisted of braced frames in both directions and some moment frames in the East-West direction. The braces in the braced frames were assigned moment releases in the 3-3 direction at each end, accounting only for axial load in the braces. The shear walls in the basement were modeled as a membrane accounting only for in plane loading. The floor slab in the Patient Pavilion is 6 1/2” lightweight concrete on metal deck, this floor system provides enough rigidity to be modeled as a rigid diaphragm.

To verify the accuracy of the model, relative stiffness’s of each frame. Hand calculations were performed to find the combined torsional and direct shear at a given story in each frame. The combined torsional and direct shear was then distributed to each frame using the calculated relative stiffness. Section cuts were made in the ETABS model to get the shear in ach frame and this shear was verified with the hand calculations. The hand calculations verified that the forces obtained in the frames in the computer model were within 10% of the hand calculations.

Drift values were obtained from the ETABS model which were used to verify the serviceability of the Patient Pavilion. For strength, it was found that different loads can control throughout the building due to factoring wind by 1.6. It was found that the drifts derived from the ETABS model were acceptable per ASCE7-05 and the strength of the members were adequate for the forces within the lateral systems.

Appendix A: Snow Load Calculations

Thomas Kleinosky | Snow Loads | Pg 1 of 3

Flat Roof Snow Load; p_f

$p_f = 0.7 C_e C_t I p_g$

But Not Less Than

$p_f = 20(I)$

(table 7-2) $C_e = 1.0$ → Site Class B → (Page 288 ASCE 7-05)
Partially Exposed

(table 7-3) $C_t = 1.1$ →

(table 7-4) $I = 1.2$ → Category IV
↳ Hospitals + Other Health Care Facilities (Table 1-1)

$p_g = 40$ psf → per (Figure 7-1)

$p_f = 0.7(1.0)(1.1)(1.2)(40) = 36.96\text{ psf} \approx \boxed{37\text{ psf}}$

$37 \geq 20(1.2) = 24 \therefore$ OK ✓

THOMAS KLEINOSKY | SNOW LOADS | Pg 2 of 3

DRIFT CALCULATIONS

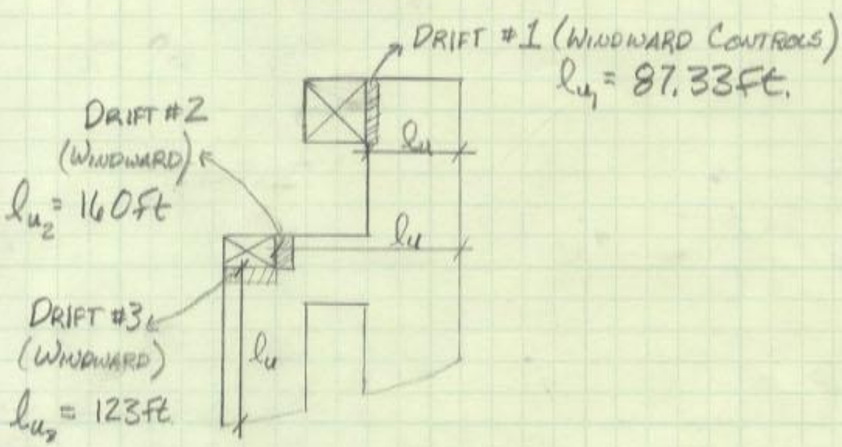
Drift does not apply at parapit because the height of the parapit is 2Ft. but the height of the Flat roof snow is approximately 2Ft.

$$\gamma = .13 p_g + 14 \leq 30 \text{pcf}$$

$$\gamma = .13(40) + 14 = 19.2 \text{pcf}$$

$$h = \frac{37 \text{psF}}{19.2} = 1.93 \text{ Feet}$$ ∴ drift can only be .07Ft high.

DRIFT AT STAIR TOWERS



WINDWARD

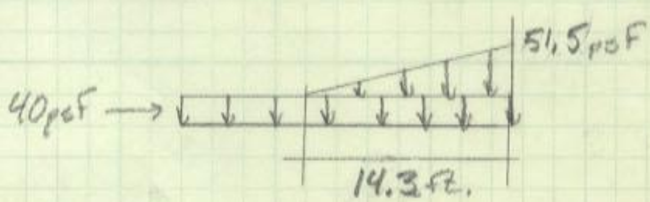
DRIFT # 1

$$h_d = \left[0.43 \sqrt[3]{l_u} \sqrt[4]{p_g + 10}\right] - 1.5 = \left[\left[0.43 \sqrt[3]{87.33} \sqrt[4]{40+10}\right] - 1.5\right] \times \frac{3}{4}$$

$$h_{d_1} = 2.68 \text{ Ft.}$$

$$\omega_1 = 2.68\text{Ft} \times 19.2 \text{ lb/ft}^3 = 51.5 \text{ psF}$$

$$\text{DRIFT WIDTH} = 4 h_d = 4\left(\frac{2.68}{.75}\right) = 14.3 \text{ Ft.}$$



THOMAS KLEINOSKY | SNOW LOADS | Pg 3 of 3

DRIFT # 2

$$h_d = \left[.43\sqrt[3]{\ell_u}\sqrt[4]{p_g+10}\right] - 1.5 = \left[.43\sqrt[3]{160}\sqrt[4]{40+10}\right] - 1.5 =$$

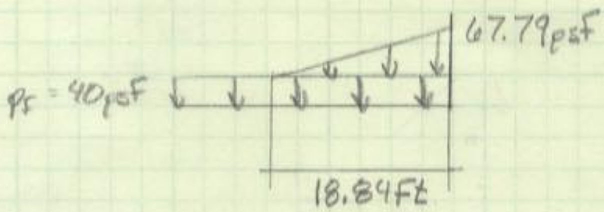
$$h_d = 4.71\text{ft}$$

because of windward, use $3/4\, h_d$

$$3/4\, h_d = 3.53\text{Ft}$$

$$\omega = 3.53\text{ft} \times 19.2 = 67.79\text{psf}$$

DRIFT WIDTH $= 4h_d = 4.71(4) = 18.84\text{Ft.}$



DRIFT # 3

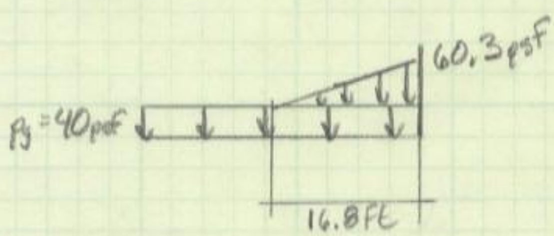
$$h_d = \left[0.43\sqrt[3]{\ell_u}\sqrt[4]{p_g+10}\right] - 1.5 = \left[.43\sqrt[3]{123}\sqrt[4]{50}\right] - 1.5$$

$$h_d = 4.2\text{Ft}$$

WINDWARD → $3/4\, h_d = 3.14\text{Ft.}$

$$\omega = 3.14\text{Ft} \times 19.2\text{pcF} = 60.3\text{psf}$$

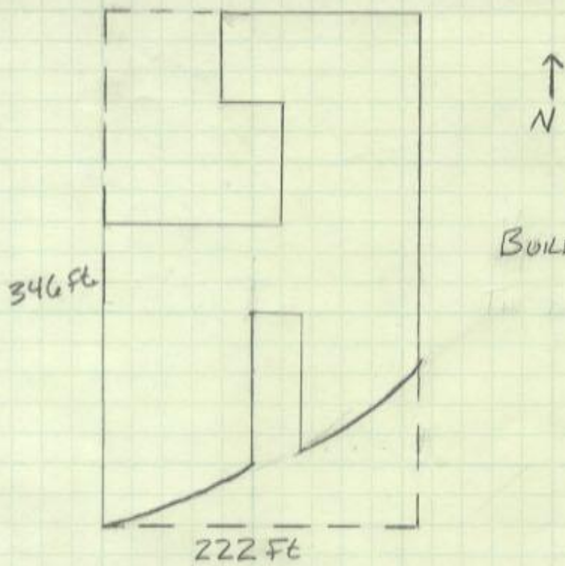
DRIFT WIDTH $= 4h_d = 4(4.2) = 16.8\text{Ft.}$



Appendix B: Wind Calculations

THOMAS KLEINOSKY | WIND CALCS | Page 1 of 6

SIMPLIFYING BUILDING FOOT PRINT



BUILDING HEIGHT = 150 ft

N-S DIRECTION

L = 346 ft
B = 222 ft

E-W DIRECTION

L = 222 ft.
B = 346 ft.

THOMAS KLEINOSKY | WIND CALCS. | Page 2 of 6

DESIGN CRITERIA PER ASCE 7-05

(Fig 6-1) BASIC WIND SPEED (V) = 90 MPH

(TABLE 6-4) WIND DIRECTIONALITY Factor (K_d) = 0.85

(TABLE 6-1) IMPORTANCE FACTOR (I) = 1.15 → OCCUPANCY CATEGORY IV
↳ V = 85-100 MPH

(PAGE 288) EXPOSURE TYPE = B

TOPOGRAPHIC FACTOR (K_{zt}) = 1.0 → Does not meet all requirements in 6.5.7.1 ∴ use 1.0

G_f CALCULATIONS

ASSUME A FLEXIBLE STRUCTURE

$$G_F = 0.925\left(\frac{1+1.7I_{\bar{z}}\sqrt{g_Q^2Q^2+g_R^2R^2}}{1+1.7g_v I_{\bar{z}}}\right)$$

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}}$$

n_1 = building natural Frequency → REFER TO COMMENTARY TO FIND n_1

$n_1 = \frac{100}{H} = \frac{100}{150} = 0.667$ (average value ; C6-17)

$n_1 = \frac{75}{H} = \frac{75}{150} = 0.50$ (Lower bound value ; C6-18)

USE $n_1 = 0.667$ ∴ Assumption of a flexible building is OK.

$$g_r = \sqrt{2\ln(3600(.667))} + \frac{0.577}{\sqrt{2\ln(3600(.667))}} = 4.092$$

Thomas Kleinosky | Wind Calcs | Page 3 of 6

Resonant Response Factor

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + .47R_L)}$$

hourly wind speed $\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right)$

(table 6-2) $\bar{b} = 0.45 \rightarrow$ Exposure B

(table 6-2) $\bar{\alpha} = 1/4.0 \rightarrow$ Exp B

$\bar{z} = 150ft(0.6) = 90ft \geq 30ft \therefore OK$

$\rightarrow \bar{V}_{\bar{z}} = .45\left(\frac{90}{33}\right)^{1/4.0}(90)\left(\frac{88}{60}\right) = \underline{76.33 \text{ mph}}$

$\rightarrow L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}$

(table 6-2) $\ell = 320 ft$

(table 6-2) $\bar{\epsilon} = 1/3.0$

$L_{\bar{z}} = 320\left(\frac{90}{33}\right)^{1/3.0} = \underline{447.09 ft}$

$\rightarrow N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.667(447.09)}{76.33} = \underline{3.9}$

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

$c = 0.3$

$I_{\bar{z}} = .3\left(\frac{10}{90}\right)^{1/6} = 0.253$

$\rightarrow \beta = 0.01 \rightarrow$ Per Chapter 6 commentary on structural damping

$\rightarrow R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = 0.059$

THOMAS KLEINOSKY | WIND CALCS | Page 4 of 6

NORTH - SOUTH

$n_H = 4.6 n_1 (h/\bar{V}_z)$

$= 4.6(2/3)(150/76.33) = 6.02$

$R_H = \frac{1}{n_H} - \frac{1}{2n_H^2}(1 - e^{-2n_H})$

$= \frac{1}{6.02} - \frac{1}{2(6.02^2)}(1 - e^{-2(6.02)})$

$= 0.152$

$n_B = 4.6 n_1 B/\bar{V}_z = 4.6(2/3)\left(\frac{222}{76.33}\right)$

$= 8.92$

$R_B = 1/8.92 - \frac{1}{2(8.92^2)}(1 - e^{-2(8.92)})$

$= 0.106$

$n_L = 15.4 n_1 (L/\bar{V}_z) = 15.4(2/3)\left(\frac{346}{76.33}\right)$

$= 46.54$

$R_L = \frac{1}{46.5} - \frac{1}{2(46.5^2)}(1 - e^{-2(46.5)})$

$= 0.021$

$R = \sqrt{\frac{1}{0.01}(.059)(.152)(.106)(.53 + .47(.021))}$

$= \underline{0.227}$

$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{.63}}}$

$= \sqrt{\frac{1}{1 + 0.63\left(\frac{222+150}{447.09}\right)^{.63}}}$

$= 0.8$

$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$

$= .925\left(\frac{1 + 1.7(.253)\sqrt{3.4^2(.8^2) + 4.1^2(.227^2)}}{1 + 1.7(3.4)(.253)}\right)$

$= 0.840$

EAST - WEST

$n_H = 6.02$

$R_H = 0.152$

$n_B = 4.61(2/3)\left(\frac{346}{76.33}\right)$

$= 13.93$

$R_B = \frac{1}{13.93} - \frac{1}{2(13.93^2)}(1 - e^{-2(13.93)})$

$= 0.069$

$n_L = 15.4(2/3)\left(\frac{222}{76.33}\right) = 29.86$

$R_L = \frac{1}{29.86} - \frac{1}{2(29.86^2)}(1 - e^{-2(29.86)})$

$= 0.033$

$R = \sqrt{\frac{1}{0.01}(.059)(.152)(.069)(.53 + .47(.033))}$

$= \underline{0.184}$

$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{346+150}{447.09}\right)^{.63}}}$

$= 0.77$

$G_f = .925\left(\frac{1 + 1.7(.253)\sqrt{3.4^2(.77^2) + 4.1^2(.18)^2}}{1 + 1.7(3.4)(.253)}\right)$

$= 0.815$

Thomas Kleinosky | Wind Calcs | Page 5 of 6

Building Is Enclosed ∴ $GC_{pi} = \pm 0.18$

External Pressure Coefficients (Fig 6-6)

Walls

Windward → $C_p = 0.8$

Leeward → E-W Direction $L/B = 222/346 = 0.64$

$C_p = -0.5$

N-S Direction $L/B = \frac{346}{222} = 1.56$

Interpolate Table

1	-.5
1.56	x
2	-0.3

$$x = \frac{1.56-1}{2-1}(-0.3-(-.5)) + (-.5) = -0.388$$

Side Wall → $C_p = -0.7$

Roof $\theta = 0°$

N-S Direction

$h/L = \frac{150}{346} = 0.433$

$0 \rightarrow h/2 \Rightarrow 0 \rightarrow 75ft$

$C_p = -0.9, -.18$

$75ft \rightarrow 150ft$

$C_p = -0.9, -.18$

$150 \rightarrow 300ft$

$C_p = -0.5, -.18$

$>300ft$

$C_p = -0.3, -.18$

E-W Direction

$h/L = \frac{150}{222} = 0.676$

$0 \rightarrow 75ft$

$C_p = \frac{.676-.5}{.5}(-1.3-(-.9)) + -.9 = -1.04, -0.18$

$75 - 150ft$

$C_p = \frac{.676-.5}{.5}(-.7-(-.9)) + -.9$

$= -0.83$

150 to end

$\frac{.676-.5}{0.5}(-0.7-(-.5)) + -0.5$

$C_p = -0.57$

THOMAS KLEINOSKY | WIND CALCS | Page 6 of 6

VELOCITY PRESSURE

$$q = 0.00256 K_z K_{zt} K_d V^2 I$$

DESIGN WIND FORCES

MAIN WIND FORCE → FLEXIBLE BUILDINGS

$$p = q G_f C_p - q_i (GC_{pi})$$

WINDWARD WALLS

$$p = q_z G_f C_p - q_h (GC_{pi})$$

LEEWARD WALLS, SIDE WALLS, & ROOFS

$$p = q_h (G_f C_p - GC_{pi})$$

PARAPIT

$$p = q_p (GC_p - GC_{pi})$$

Appendix C: Seismic Calculations

THOMAS KLEINOSKY | SEISMIC CALCS | Page 1 of 1

ASCE 7-05 → Ch 11.4.1 - Seismic Ground Motion Values

Referenced: http://earthquake.usgs.gov/research/hazmaps/design/ to find S_S and S_{D1}

Based on Geotechnical Report this is site class D

$S_S = 0.229$ $S_{D1} = 0.069$

$S_{MS} = F_a S_S$ $S_{M1} = F_v S_1$

(Table 11.4-1) $F_a = 1.6$ (Table 11.4-2) $F_v = 2.4$

$S_{MS} = 1.6(0.229) = 0.3664$ $S_{M1} = 2.4(0.069) = 0.1656$

(11.4-3) $S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.3664) = 0.245$

(11.4-4) $S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.1656) = 0.1104$

$I_e = 1.5$ for Seismic Occupancy IV

ASCE 7-05 → Ch 12.8 - Equivalent Lateral Force

$V = C_s W$

$W = 62200$ kips

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \leq \frac{S_{D1}}{T_a\left(\frac{R}{I}\right)}$ R = 3 for Ordinary composite Steel + Concrete Braced Frames

$= \frac{.245}{\left(\frac{3}{1.5}\right)} = 0.125$

(C6-17) $n_1 = \frac{100}{H} = \frac{100}{145} = 0.687$

$T = \frac{1}{n} = \frac{1}{0.687} = 1.45$

$C_s = \frac{0.1104}{1.45\left(\frac{3}{1.5}\right)} = 0.38 < 0.125 \therefore$ use 0.38

$\boxed{V = 0.38(62{,}200) = 2370^k}$

$2370^k \approx 2400^k$

↳ indicated on structural drawings

Appendix D: Torsional Shear Calculations

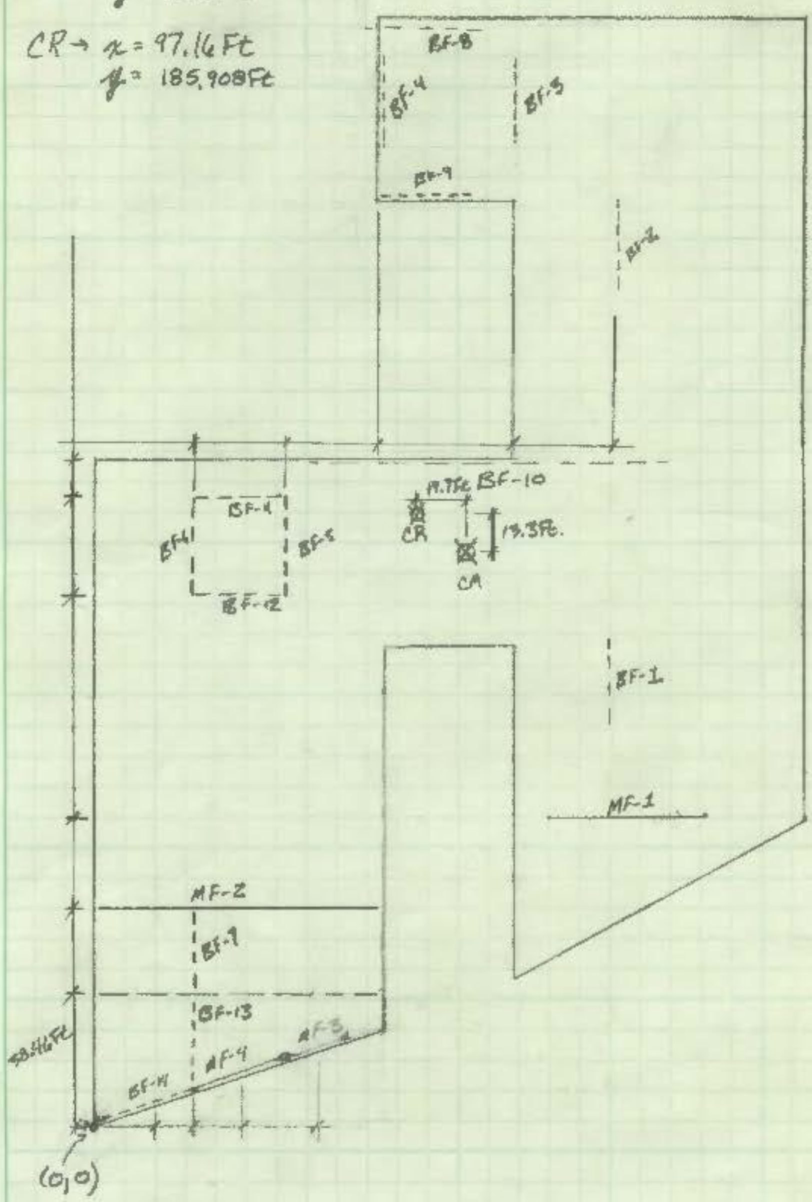
Thomas Kleinosky | Torsional Shear | 1 of 6

Level 7

From ETABS Model

CM → $x = 117.059$ Ft
$y = 172.444$ Ft

CR → $x = 97.16$ Ft
$y = 185.908$ Ft



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Distance to Frame (d_i)

X-direction Frames

Frame	dist. off Y-axis	Rel. Rigidity
BF-14	4.07	19.9
MF-4	12.46	2.8
MF-3	20.16	3.0
BF-13	38.46	3.17
MF-2	65.79	3.41
MF-1	93.13	2.4
BF-12	161.71	7.23
BF-11	191.54	10.71
BF-10	202.46	15.5
BF-9	284.46	16.82
BF-8	339.13	14.11

Y-direction Frames

Frame	dist. off x-axis	Rel. Rigidity
BF-14	15	8.22%
MF-4	43.67	-0
MF-3	67.15	-0
BF-7	30	20.52
BF-6	30	12.53
BF-5	57.33	12.25
BF-4	87.33	13.98
BF-3	129	10.59
BF-2	159	10.92
BF-1	159	10.92

Normalize BF-14, MF-4, & MF-3 to X & Y-axis

BF-14

Y, X

8.52%, 20.6%

20.6%, 15°

8.22%

$20.6\cos(15.2) = 19.9\%$

MF-4

-.14%, 2.9%

-.135% → assume zero

$2.9\cos(15.2) = 2.8\%$

MF-3

-.11%, 3.11%

-.10% → assume zero

$3.11\cos(16.7) = 3.0\%$

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d_i

X-direction

	d_i
BF-14	181.8
MF-4	173.4
MF-3	165.8
BF-13	147.5
MF-2	120.1
MF-1	92.8
BF-12	24.2
BF-11	5.63
BF-10	16.6
BF-9	98.6
BF-8	153.2

Y-direction

BF-14	82.2
MF-4	53.5
MF-3	30
BF-7	67.2
BF-6	67.2
BF-5	39.8
BF-4	9.8
BF-3	31.8
BF-2	61.8
BF-1	61.8

Torsional Rigidity

$$J = \Sigma R_i d_i^2$$

$$J = 19.9(181.8^2) + 2.8(173.4)^2 + 3(165.8^2) + 3.17(147.5^2) + 3.41(120.1^2) + 2.4(92.8^2) + 7.23(24.2)^2 + 10.71(5.63^2) + 15.5(16.6^2) + 16.8(98.6)^2 + 14.11(153.2^2) + 8.22(82.2^2) + 0 + 0 + 20.52(67.2^2) + 12.53(67.2^2) + 12.25(39.8^2) + 13.98(9.8^2) + 10.57(31.8^2) + 10.92(61.8^2) + 10.92(61.8^2)$$

$$J = 1786196.311\left(\frac{k}{in}\right)ft^2$$

Direct Shear

$$\Sigma R_x = 19.9 + 2.8 + 3 + 3.17 + 3.41 + 2.4 + 7.23 + 10.71 + 15.5 + 16.82 + 14.11 = 99.05$$

$V_{BF-14} = \frac{19.9}{99.05} \times 355 = 71.3^k$

$V_{MF-4} = 10.03^k$

$V_{MF-3} = 10.8^k$

$V_{BF-13} = 11.4^k$

$V_{MF-2} = 12.2^k$

$V_{MF-1} = 8.6^k$

$V_{BF-12} = 25.9^k$

$V_{BF-11} = 38.9^k$

$V_{BF-10} = 55.6^k$

$V_{BF-9} = 60.3^k$

$V_{BF-8} = 50.6^k$

$V_{tot} = 355.13^k$ (error due to rounding)

Torsional Shear

$$V = \frac{(M + V \cdot e)(R_i d_i)}{J} \qquad e = 13.3\text{ ft}$$

BF-14

$$V = \frac{[6016 + 71.3(13.3)](19.9 \times 181.8)}{1786196.311} = 14.1^K$$

MF-4

$$V = \frac{[6016 + 10.03(13.3)](2.8 \times 173.4)}{1786196.311} = 1.7^K$$

MF-3

$$V = \frac{[6016 + 10.8(13.3)](3 \times 165.8)}{1786196.311} = 1.7^K$$

BF-13

$$V = \frac{[6016 + 11.4(13.3)](3.17 \times 147.5)}{1786196.311} = 1.6^K$$

MF-2

$$V = \frac{[6016 + 12.2(13.3)](3.41 \times 120.1)}{1786196.311} = 1.4^K$$

MF-1

$$V = \frac{[6016 + 8.6(13.3)](2.4 \times 92.8)}{1786196.311} = 0.8^K$$

BF-12

$$V = \frac{[6016 + 25.9(13.3)](7.23 \times 24.2)}{1786196.311} = 0.6^K$$

BF-11

$$V = \frac{[6016 + 38.4(13.3)](10.71 \times 5.63)}{1786196.311} = 0.22^K$$

BF-10

$$V = \frac{[6016 + 55.6(13.3)](16.6 \times 16.5)}{1786196.311} = 0.97^K$$

BF-9

$$V = \frac{[6016 + 60.3(13.3)](16.82 \times 98.6)}{1786196.311} = 6.3^K$$

BF-8

$$V = \frac{[6016 + 50.6(13.3)](14.11 \times 153.2)}{1786196.311} = 8.1^K$$

Total Shear

$V_{BF-14} = 71.3 + 14.1 = 85.4^K$

$V_{MF-4} = 10.03 + 1.7 = 11.7^K$

$V_{MF-3} = 10.8 + 1.7 = 12.5^K$

$V_{BF-13} = 11.4 + 1.6 = 13.0^K$

$V_{MF-2} = 12.2 + 1.4 = 13.6^K$

$V_{MF-1} = 8.6 + 0.8 = 9.4^K$

$V_{BF-12} = 25.9 + 0.6 = 26.5^K$

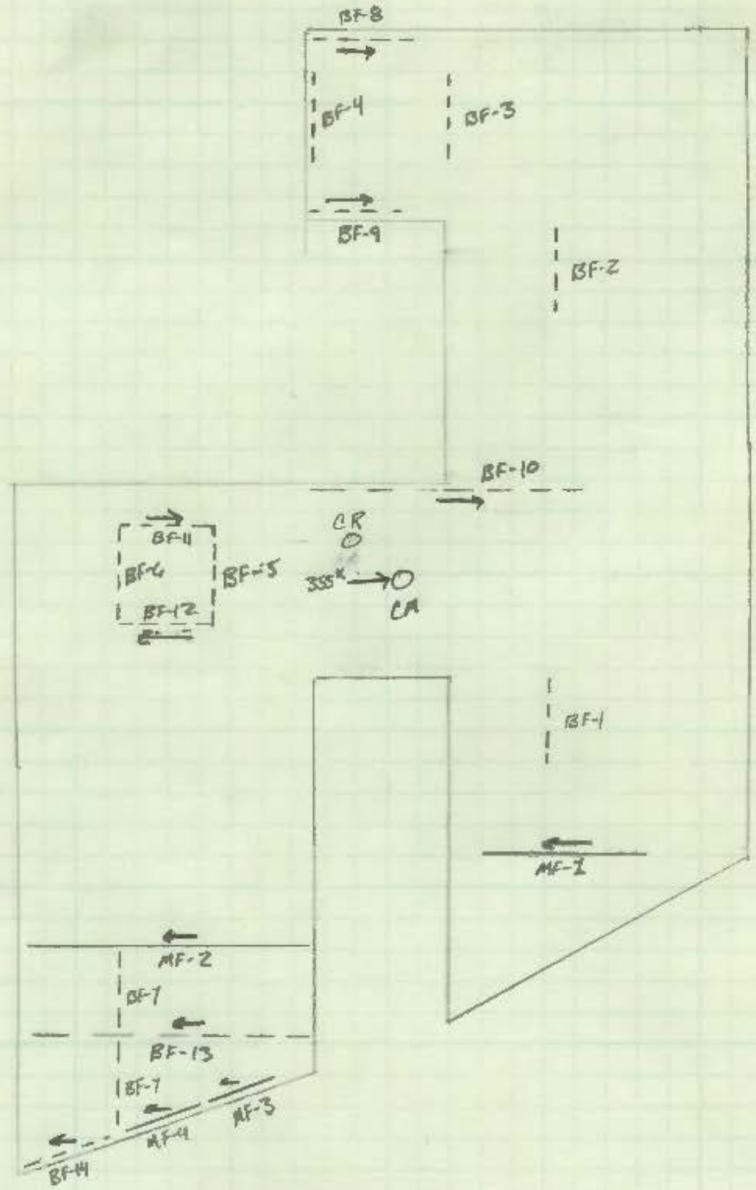
$V_{BF-11} = 38.4 - 0.22 = 38.18^K$

$V_{BF-10} = 55.6 - 0.97 = 54.4$

$V_{BF-9} = 60.3 - 6.3 = 54^K$

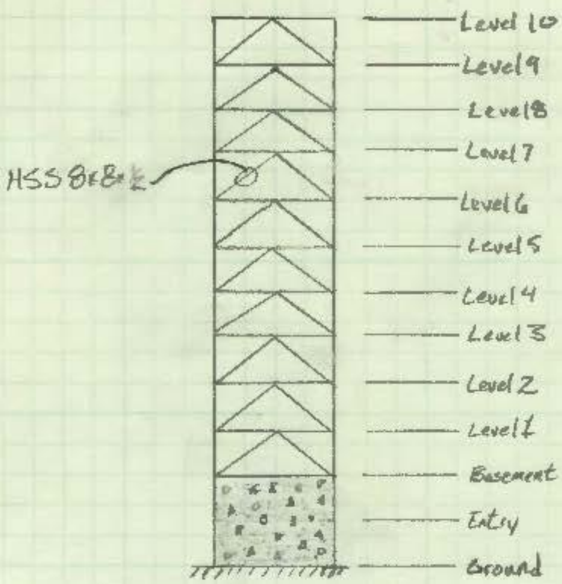
$V_{BF-8} = 50.6 - 8.1^K = 42.5^K$

THOMAS KLEINOSKY | Torsional Shear | 6 of 6



Appendix E: Spot Checks

Thomas Kleinosky | Spot Checks | 1 of 3

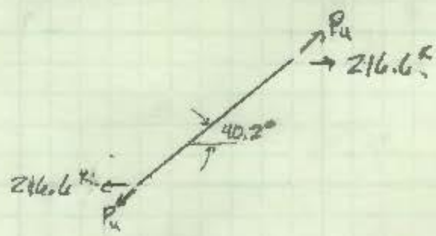


HSS 8x8x½

Area = 13.5 in^2

L = 18.6 ft

Load Case 7 0.9D+1.0E **Governs**



Find Axial Load

$$\cos(40.2) = \frac{216.6}{T}$$

$$\boxed{P_u = 284^k}$$

Tension

Yield → $\phi P_n = 0.9(13.5)(36) = 437.4^k > 284 \therefore$ OK

Rupture → $\phi P_n = 0.75(13.5)(58) = 587^k > 284 \therefore$ OK

Compression

$I = 125 \, in^4$

$$P_{cr} = \frac{\pi^2(29000)(125)}{(18.6 \times 12)^2} = 718^k$$

$$\phi P_{cr} = 0.9(718) = 647^k > 284^k \therefore \text{OK}$$

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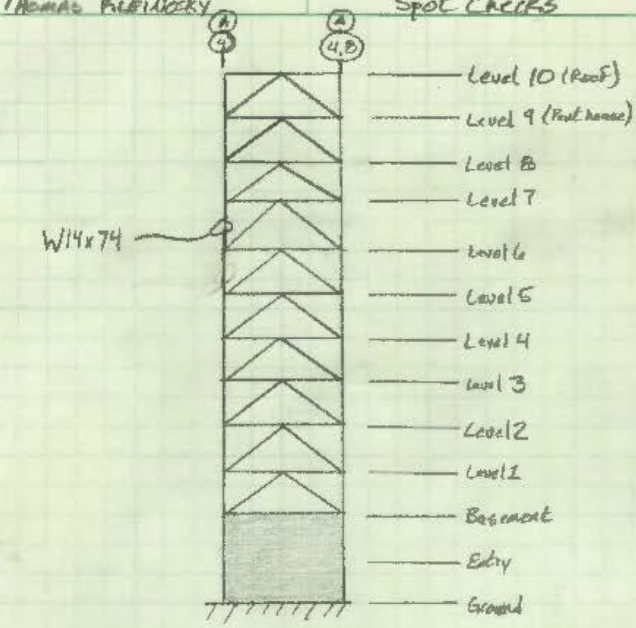


Loads

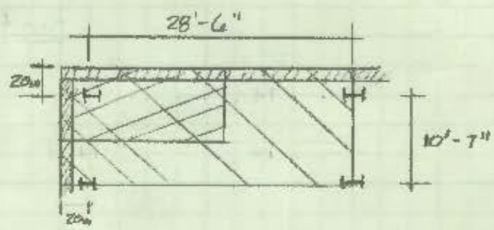
Dead:

Roof = 30 psf
Floors = 95 psf
Penthouse = 125
Wall = 48 psf

Live:

Snow = 37 psf
Floor = 100 psf
Penthouse = 125 psf

Load Case 5 $1.2D + 1.0E + 1.0L + 0.2S$ Governs



$$\text{Trib Area} = \left[\left(\frac{28.5}{2}\right)+1.67\right] \times \left[\frac{10.58}{2}+1.67\right]$$
$$= 111\ ft^2$$

$$\text{Influence Area} = (28.5+1.67) \times (10.58+1.67)$$
$$= 370\ ft^2 < 400\ ft^2$$

No LLred!

Level 10

$P_D = 30 \times 111 = 3.33^k$ $\quad P_S = 37 \times 111 = 4.11^k$

Level 9

$P_D = 125 \times 111 = 13.9^k$ $\quad P_L = 13.9^k$

Level 8

$P_D = 95 \times 111 = 10.55^k$ $\quad P_L = 100 \times 111 = 11.1^k$

Level 7

$P_D = 10.55^k$ $\quad P_L = 11.1^k$

$$P_u = 1.2(3.33 + 13.9 + 2(10.55)) + (13.9 + 11.1 + 11.1) + 0.2(4.11)$$
$$P_u = 82.9^k$$

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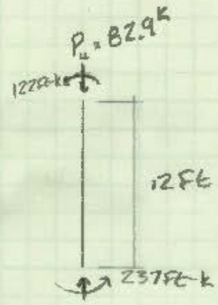

Table 6-1 in AISC Manual

W14x74 @ 12ft

$p = 1.3 \times 10^{-3}$

$b_x = 1.99 \times 10^{-3}$

$pP_r = 1.3\times10^{-3}(82.9) = 0.11 < 0.2$

$\frac{1}{2}\, pP_r + \frac{9}{8}(b_x M_{rx}) \le 1.0$

$\frac{1}{2}(0.11) + \frac{9}{8}(1.99\times10^{-3} \times 237) = 0.59 \le 1.0 \therefore \underline{OK}$