2011

Technical Assignment 1

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Patient Care Pavilion; Albany, NY

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Table of Contents

Executive Summary

The purpose of the first technical report is to analyze, understand, and report the existing structural conditions for the Patient Care Pavilion in Albany, NY. The Patient Pavilion is an expansion of the Albany Medical Center Hospital (AMCH) campus, completion scheduled for June of 2013. The Patient Pavilion consists of two phases, Phase 1 is to construct a new six-story medical center, and Phase 2 is a four story vertical expansion of the Patient Pavilion. The structural analysis and design of the hospital was for a ten-story building, preventing the task of reinforcing lower existing members for the vertical expansion.

Dead and live loads were established through analysis of the structure, its building components, both architectural and structural, and the occupancy use of each level. loads were found per ASCE7-05. Dead and live loads were obtained per the ASCE7-05 and then verified with the specified dead and live loads on the structural drawings. Assumptions of gravity loads were deemed accurate, with little discrepancy, live loads were verified to be accurate without discrepancy.

Gravity, wind, and seismic loads were calculated to provide a preliminary basis to verify the existing typical members and to find lateral forces needed in future tech reports. Gravity spot checks were performed on five different gravity components: floor deck, composite beam, composite girder, interior column, and exterior column. All gravity components were found to be adequate per specified dead and live loads.

The seismic base shear calculated was within 5% of the base shear indicated on the structural drawings. Comparing the resultant base shears for wind and seismic, the seismic base shear is over two times larger than the wind base shear, therefore seismic will control the lateral design of the Patient Pavilion. The large base shear is likely due to having a soil rating of D and a seismic occupancy of IV.

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 (See *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 (See *Figure 2*) contains the demolition of an existing building on the AMCH campus, and the construction of a six story medical center, and Phase 2 (See *Figure 3*) is a future four story vertical expansion of the Patient Pavilion. 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.

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

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

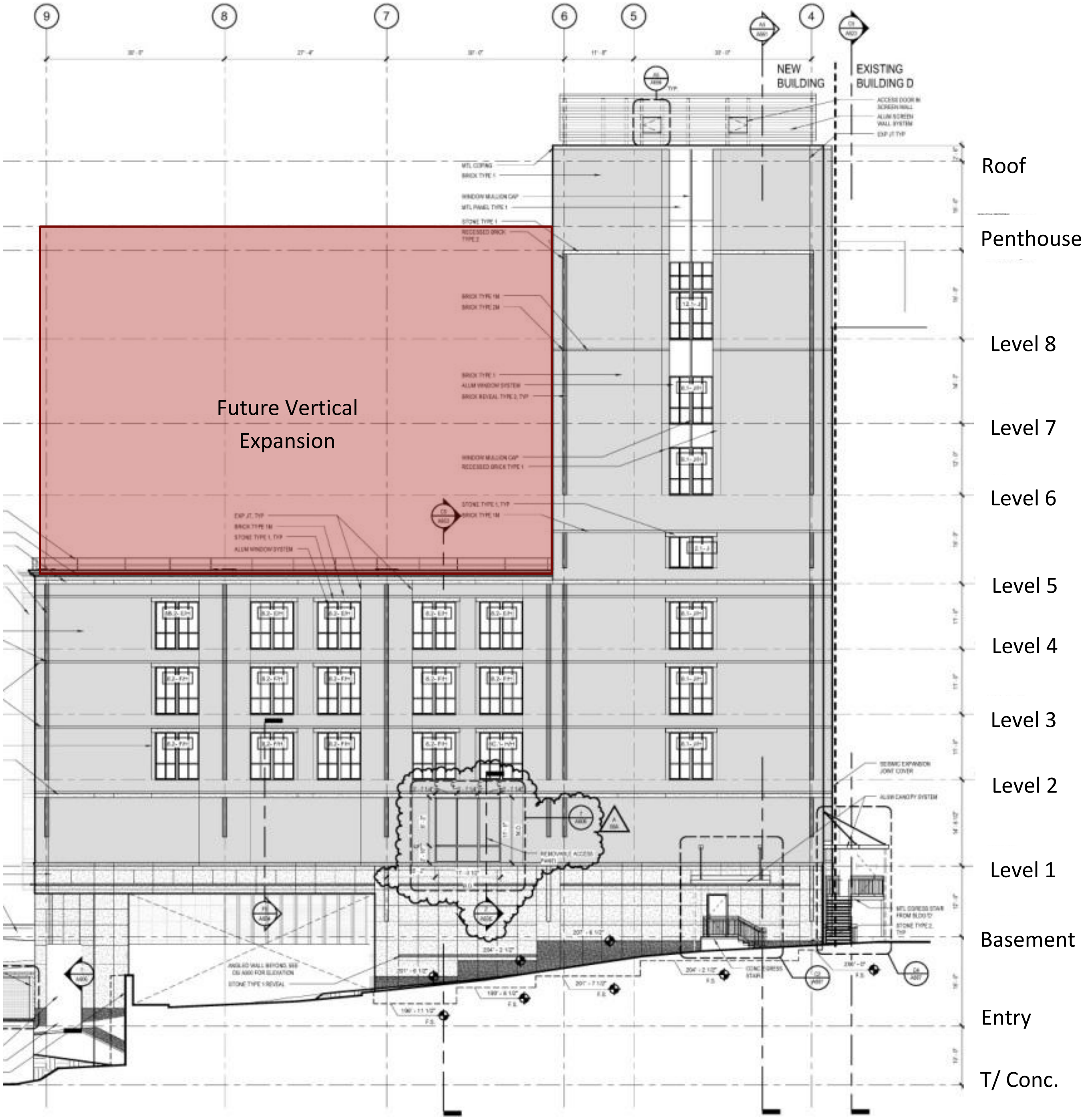


Figure 4 – South Elevation

Structural Overview

The majority of the Patient Pavilion rests on 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 in to 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 (*Figure 5)*.

Figure 5 - Span over Emergency Acess Ramp

Foundation

Vernon Hoffman PE Soil and Foundation Engineering performed 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.

Abiding to the analysis, the majority of the Patient Pavilion sits on a 36" mat foundation resting on a 4" mud slab with a 12" compacted aggregate base. However, 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 steel deck with 3 1/2” topping, reinforced with #4’s at 16” O.C., 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 from 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 6* for slab details.

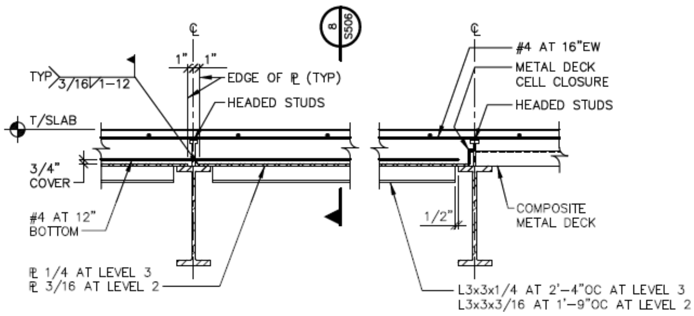


Figure 6 – Slab Detail; Radiation Shielding Plate

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 (*See Table 5)* are greater than the other floors. However, the Basement Level and Level Two utilize deeper beams than the Entry Level and Level 1 due to greater floor-to-floor height on the basement level and Level 2 so there is no framing depth restriction.

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. A typical girder span is 30’-0”, combined with the beam span produces a typical bay size of 27’-4” by 30’-0”.

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 tieing into the existing AMCH building spans 83’ over another existing AMCH building. The bottom two levels of this bridge, a two-story truss was designed by Ryan-Biggs, consisting of W10x77’s and W10x100’s.

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 from Level 2 up 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 from the column into the concrete shear wall, 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.

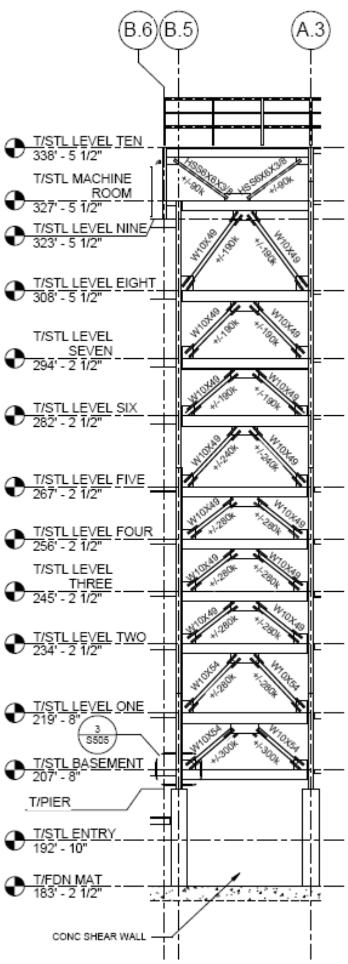


Figure 8 – Typical Braced Frame

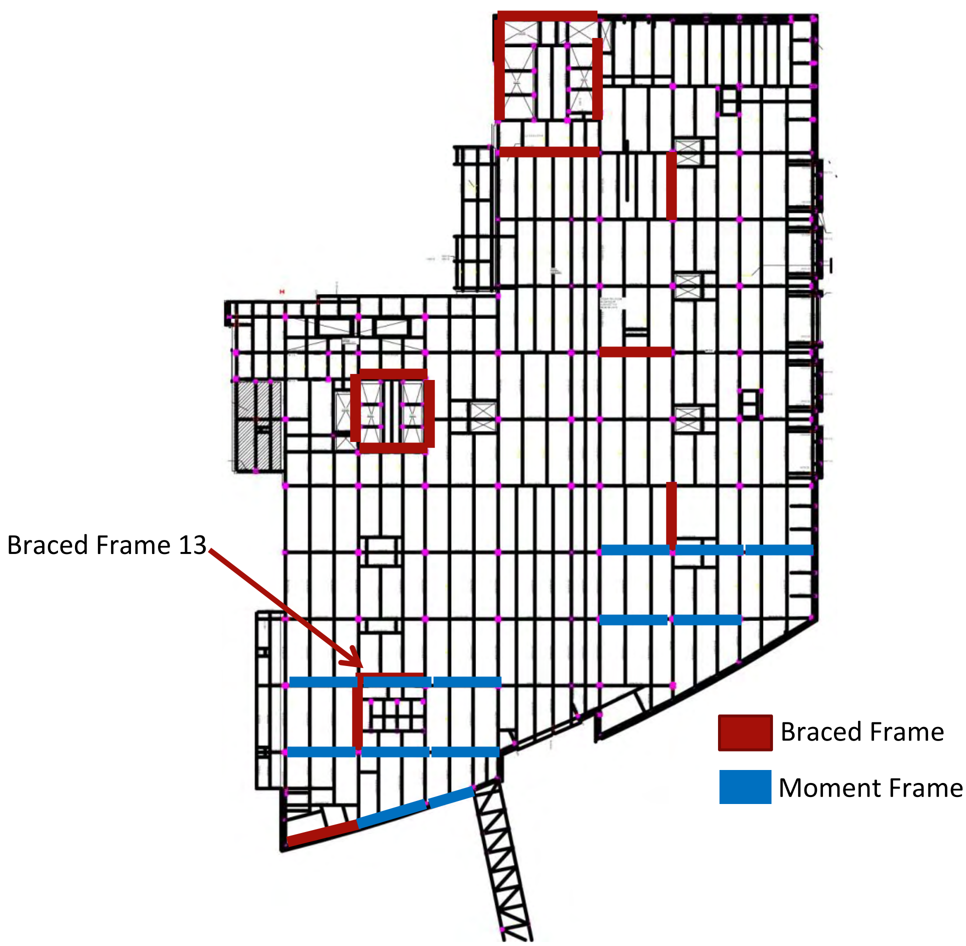


Figure 7 – Typical Layout of Lateral System

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)

These are the standards and codes I utilized:

- 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*), listed below are the various capacities for 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	52
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		3,000
Interior S.O.G or Slab on Deck		3,500
Foundation Walls, Shear walls, Piers, Pile caps, and Grade beams		4,000
Exterior S.O.G.		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	
Welding Electrodes	E70 XX	70 ksi

Gravity 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 normal.

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

Table 5 describes the controlling live load on each level with the exception of elevator lobbies and stairs. *Table 6 and Table 7* are verifying the live loads from the initial design per ASCE7-02, and the code used for this thesis, ASCE7-05.

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
Hospitals – Patient Rooms	40
Hospitals – Corridors above 1st Floor	80

Table 7 – Verifying Live Loads per ASCE7-05

Level 3 – Level 8; Verification (ASCE7-05)	
Occupancy	**Weight (psf)**
Hospitals – OR Rooms	60
Hospitals – Patient Rooms	40
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 cand 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.*

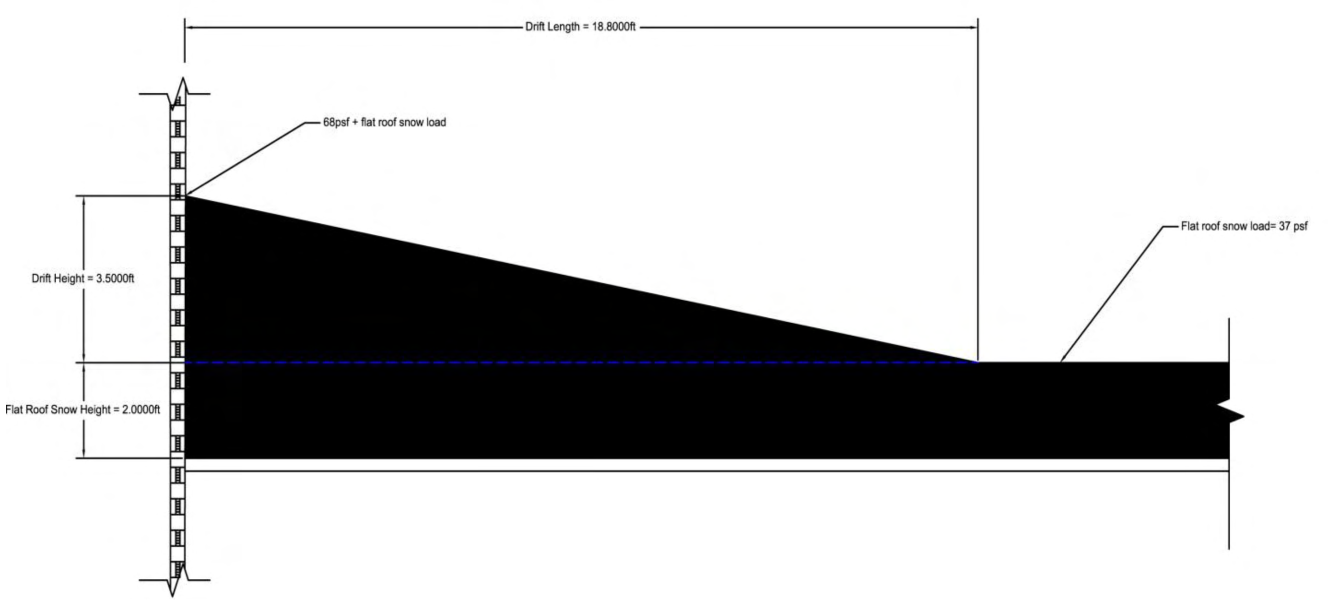


Figure 9 – Snow Drift

Figure 10 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 simplified 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.

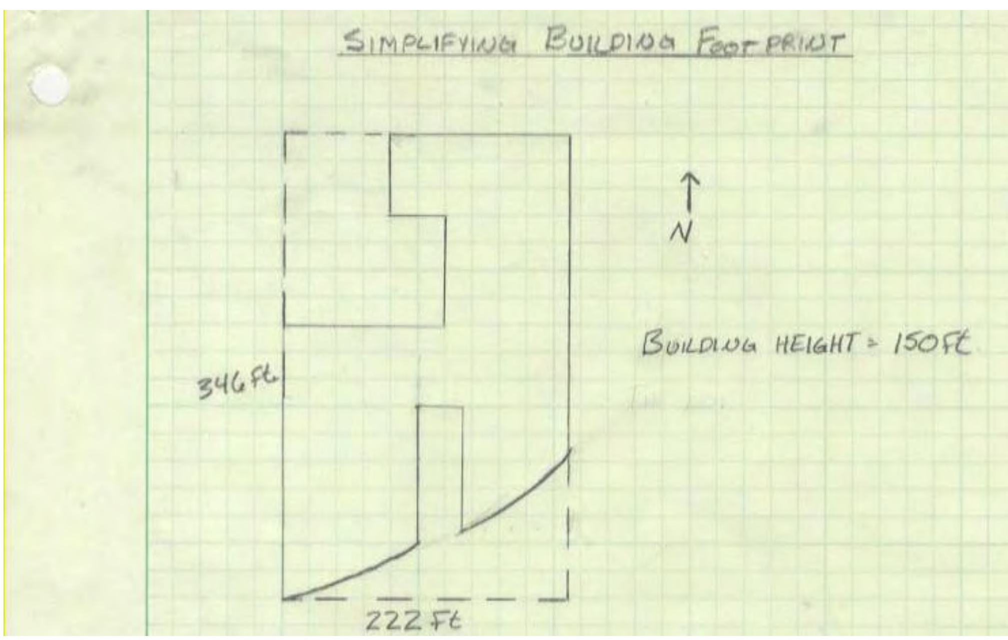


Figure 11 – Simplified Building Footprint

Table 8 – 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 9 – 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

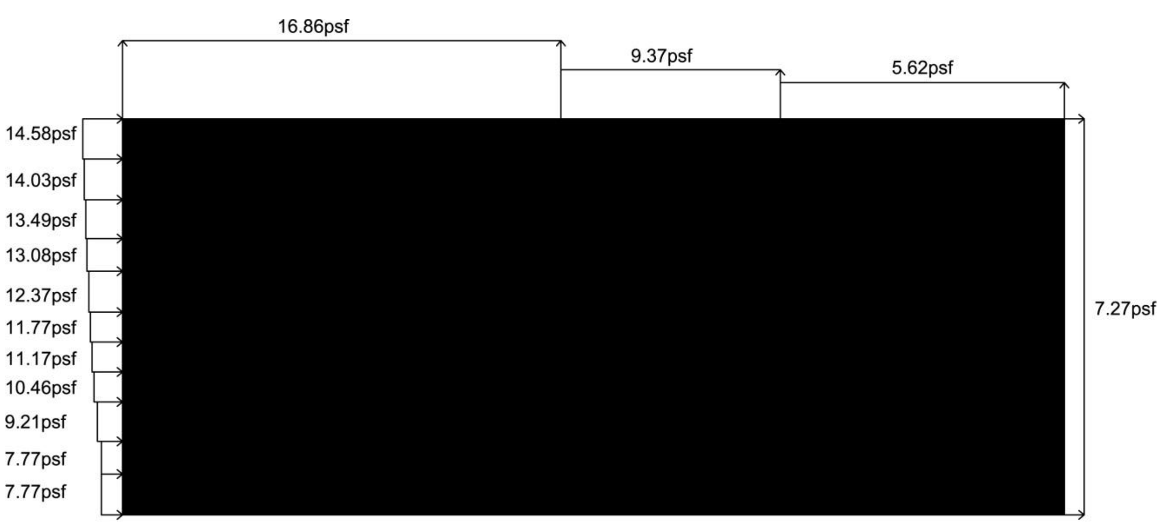


Figure 12 – Wind Pressures; North-South Direction

Table 10 – Wind Forces; North-South Direction

Wind Forces								
	Trib Heights		Elevation	Wall Width (Perp. To N-S)	Trib. Area	Story Force (kips)	Story Shear (kips)	Overturning Moment (k-ft)
	Below	Above						
Entry Level	0	7.5	0	222	1665	25.03	616.67	0.00
Basement	7.5	6	15	222	2997	45.06	591.64	675.83
Level 1	6	7.25	27	222	2941.5	48.47	546.58	1308.66
Level 2	7.25	5.5	41.5	222	2830.5	50.19	498.11	2082.78
Level 3	5.5	5.5	52.5	222	2442	45.03	447.93	2364.02
Level 4	5.5	5.5	63.5	222	2442	46.49	402.90	2952.29
Level 5	5.5	7.5	74.5	222	2886	56.68	356.40	4222.36
Level 6	7.5	6	89.5	222	2997	60.98	299.73	5457.62
Level 7	6	7.125	101.5	222	2913.75	60.48	238.75	6138.31
Level 8	7.125	7.5	115.75	222	3246.75	69.16	178.27	8004.90
Level 9	7.5	7.5	130.75	222	3330	72.74	109.12	9511.37
Level 10	7.5	0	145.75	222	1665	36.37	37.22	5301.27
						Total Base Shear=	**616.67**	
						Total Overturning Moment=		**48019.40**

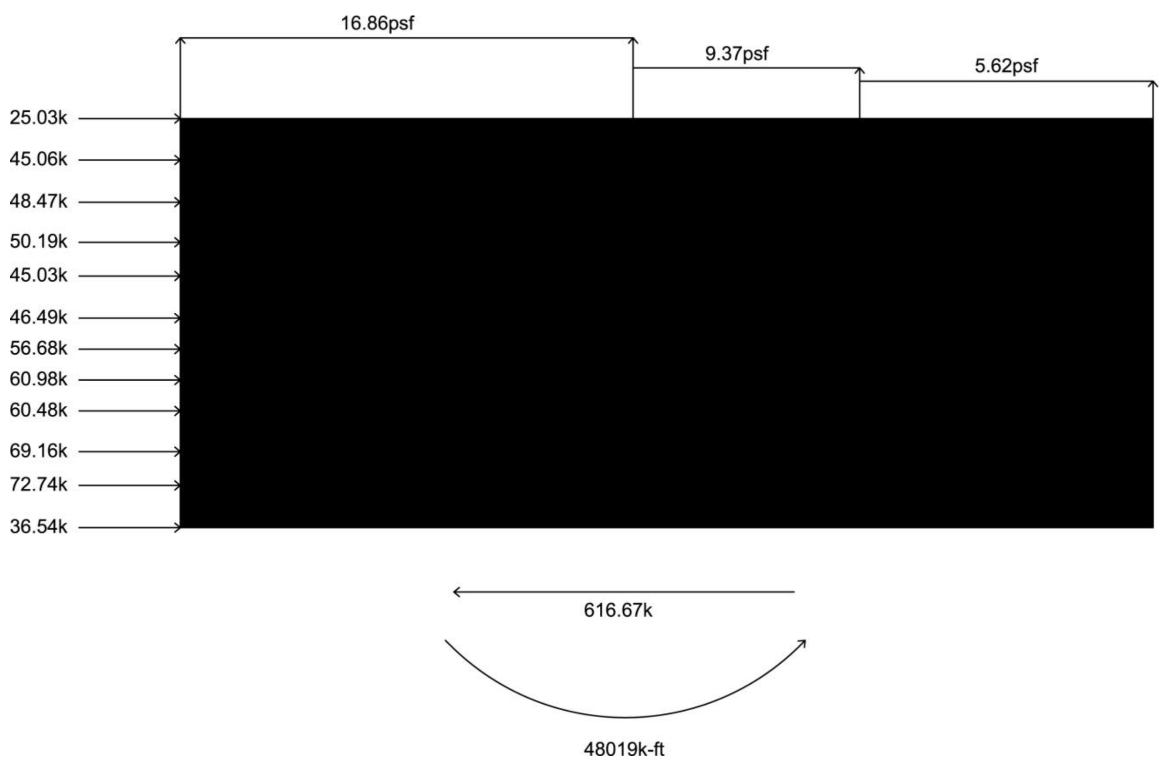


Figure 13 – North-South Wind Forces

Table 11 –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 12 – 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

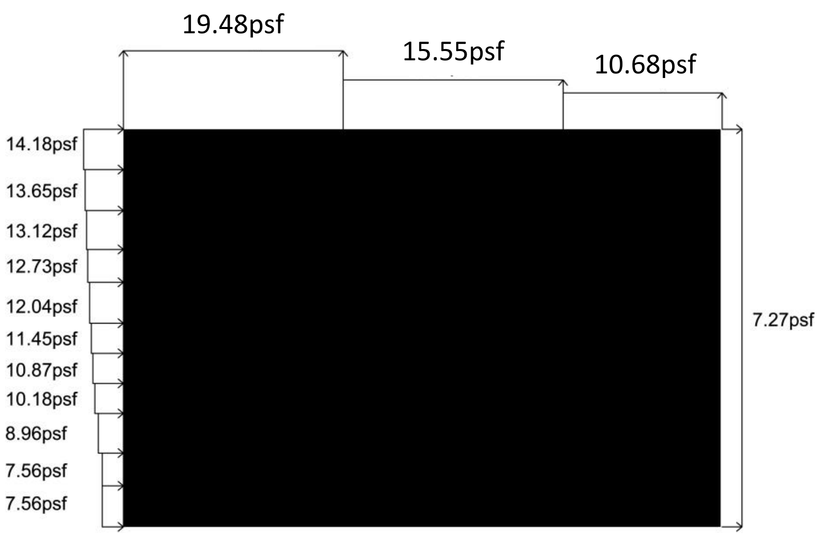


Figure 14 – Wind Pressures; East-West Direction

Table 13 – Wind Forces; East-West Direction

Wind Forces								
	Trib Heights		Elevation (ft)	Wall Width (ft)	Trib. Area (sf)	Story Force (k)	Story Shear (k)	Overturning Moment (k-ft)
	Below	Above						
Entry Level	0	7.5	0	346	2595	43.26	1038.15	0.00
Basement	7.5	6	15	346	4671	77.86	994.90	1167.96
Level 1	6	7.25	27	346	4584.5	82.86	917.03	2237.33
Level 2	7.25	5.5	41.5	346	4411.5	85.12	834.17	3532.37
Level 3	5.5	5.5	52.5	346	3806	76.06	749.05	3993.05
Level 4	5.5	5.5	63.5	346	3806	78.28	672.99	4970.66
Level 5	5.5	7.5	74.5	346	4498	95.13	594.72	7087.49
Level 6	7.5	6	89.5	346	4671	102.01	499.58	9130.15
Level 7	6	7.125	101.5	346	4541.25	100.99	397.57	10249.99
Level 8	7.125	7.5	115.75	346	5060.25	115.21	296.58	13335.50
Level 9	7.5	7.5	130.75	346	5190	120.92	181.37	15809.72
Level 10	7.5	0	145.75	346	2595	60.46	60.46	8811.72
						Total Base Shear=	**1038.15**	
						Total Overturning Moment=		**80325.95**

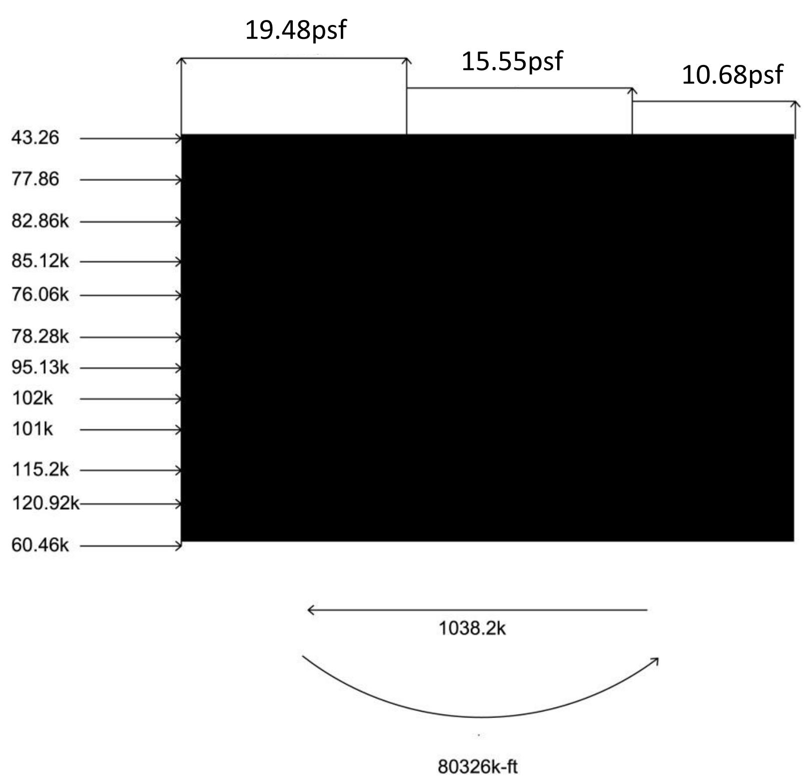


Figure 15 – Wind Forces; East-West Direction

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 (kips)
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=	**62062**

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. Avoiding redundancy, 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 Force; Story Distribution

T=	**1.408**	**s**							
k=	1.454								
V_b=	2400	kips							
i	h_i	h	w	$w*h^k$	C_{VX}	f_i	V_i	M	
	ft	ft	kips			kips	kips	kip-ft	
12	15	145.7917	399	558522	0.017	40	40	5795	
11	15	130.7917	6473	7737720	0.229	551	590	72021	
10	14.25	115.7917	6161	6169330	0.183	439	1029	50837	
9	12	101.5417	6000	4963775	0.147	353	1383	35869	
8	15	89.54167	6040	4161803	0.123	296	1679	26520	
7	11	74.54167	5995	3164153	0.094	225	1904	16785	
6	11	63.54167	5781	2419080	0.072	172	2076	10939	
5	11	52.54167	5921	1879350	0.056	134	2210	7027	
4	14.54167	41.54167	6983	1575135	0.047	112	2322	4657	
3	12	27	6700	807751	0.024	57	2380	1552	
2	15	15	5609	287688	0.009	20	2400	307	
		Σ	62062	33724307	V_s=	**2400**		Overturning Moment=	**232310**

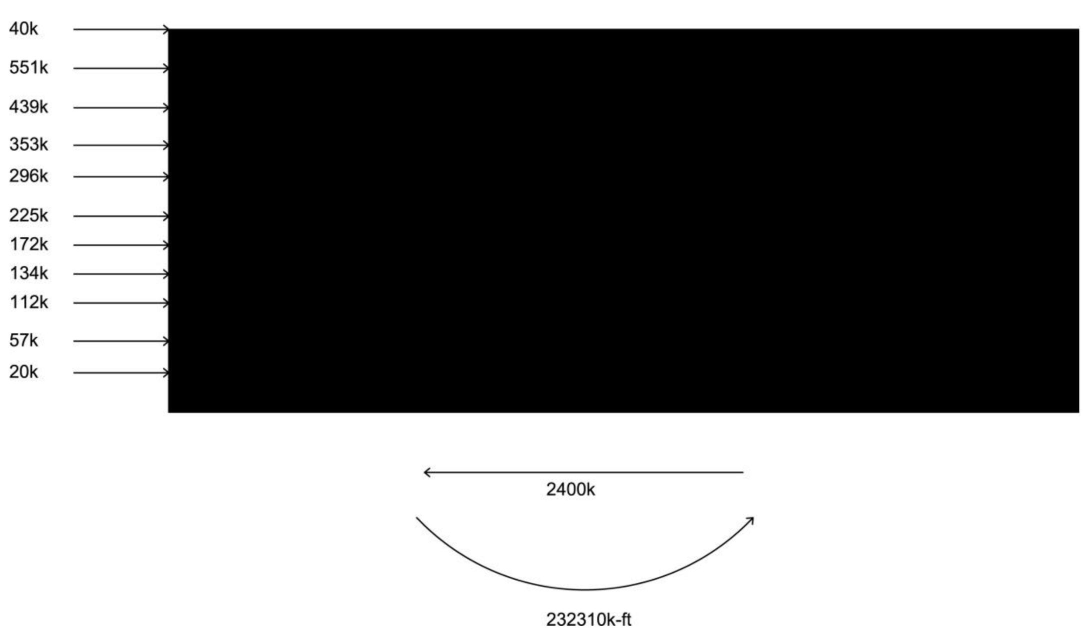


Figure 16 – Seismic Forces

Gravity Load Checks

Spot checks were performed on a typical bay located on Level 3, columns K-1, K-2, J-1, and J-2 make up the corners for the bay. Spot checks were performed to utilize knowledge learned in past courses to verify the structural system of the building. Complete hand calculations are located in Appendix D.

Decking

Typical floor construction for the Patient Pavilion utilizes a 3"x20ga composite steel deck with 3 1/2" lightweight concrete topping. Using Vulcraft Steel Decking(2008) catalog, the specified deck type is acceptable according to the allowable strengths and spans for a 3VLI20. The allowable superimposed live load is 149psf, roughly double the live load on Level 3; a possible reason for this is because the live load at the penthouse is 125psf increasing the demand for strength in the deck. Verifying the fire rating with the 2001 Fire Resistance Directory, a 3 1/2" lightweight concrete topping allows for a 2-hour rating.

Beam & Girder

Gravity spot checks were performed on a beam and girder in a typical bay. Strength and deflection checks were performed for both pre-concrete curing and post concrete curing. The members were adequate for the specified loads in flexure, shear, and deemed adequate for serviceability requirements.

The nominal flexural strength of the beam was within 10% of the ultimate moment per specified live and dead loads. This is because for Levels 3 to 8 the live load does not fluctuate, nor do the bay sizes so a less conservative approach is acceptable. The strength of the girder was approximately 45% larger than the ultimate moment of the floor loads; this could be because the plastic neutral axis on the composite girder was found to be in the web of the girder, so a stiffer heavier member is needed to preven web crippling. A shallower beam must be utilized to accommodate the low floor-to-floor heights on the typical stories. To resist the ultimate flexural capacities, a heavier, non-slender section needs to be used.

Column

Two column checks were made, one interior column J-4, and one exterior column B-9. Lateral forces were excluded from the calculations, and due to having a lateral system of braced framed, transfer of moments from adjacent bays into the column were not necessary.

Column J-4 is a W14x193 and was analyzed on the Entry level of the Patient Pavilion, above it are eight floors, a penthouse, and a roof. Live load reduction was performed where applicable, using the influence area instead of K_{LL} values given in the ASCE7-05. Loads were calculated were calculated at each level and a final check was performed on the entry level. Total dead and live loads were summed up resulting in an ultimate axial load of 1650 kips. The nominal strength of a W14x193 at an unbraced length of 15 feet is 2210 kips, which well satisfies the calculated ultimate axial load. Dead and live load discrepancies could not contribute to the large difference between the ultimate and nominal axial load; the live and dead loads assumed were accurate to the assumed dead and live loads. The possibility of a partial moment transfer at the column due to a continuous slab could induce more load in the column and therefore create a larger axial force.

Column B-9 is a W14x99, its base is located on level 2, and it is an exterior column. The same procedure was followed to calculate this ultimate axial load; however, the weight of the exterior façade was included in the dead load on the column. The ultimate axial strength culminated to 962 kips, less than 1190 kips, the nominal axial strength of a W14x99 with an unbraced length of 11 feet.

Figure 17 – Typical Column Layout

Conclusion

Technical Report 1 analyzed and summarized the existing structural conditions of the Patient Pavilion. Examining the Patient Pavilion a greater understanding of the structure, its particular elements, and the building as a whole was obtained. Heavy evaluation of the foundation, lateral system, floor/framing systems, and columns was performed to describe the full structural system.

ASCE7-05 code analysis was utilized to obtain superimposed dead and live loads for the Patient Pavilion, which were checked against the loads provided on the structural drawings. Code analysis was also utilized to obtain snow loads and snowdrifts, as well as wind and seismic loads.

The seismic base shear calculated was within 5% of the base shear indicated on the structural drawings. Comparing the resultant base shears for wind and seismic, the seismic base shear is over two times larger than the wind base shear, therefore seismic will control the lateral design of the Patient Pavilion. The large base shear is likely due to having a site class of D and a seismic occupancy of IV. In Technical Report 3, the lateral system will be analyzed to verify the strengths of the existing bracing systems.

Gravity checks were performed on five members on a typical floor to show a representation of the entire building. Spot checks verified the typical members were adequate for the required loads and their deflections met the live load deflection, construction load deflection, and wet concrete deflection criteria. Further knowledge and comprehension of composite beams was acquired due challenges of heavy, shallow composite members.

Appendix A: Snow Load and Drift Hand 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)

$p_g = 40\text{psf}$ → per (figure 7-1)

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

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

Thomas Kleinosky | Snow Loads | Pg 2 of 3

Drift Calculations

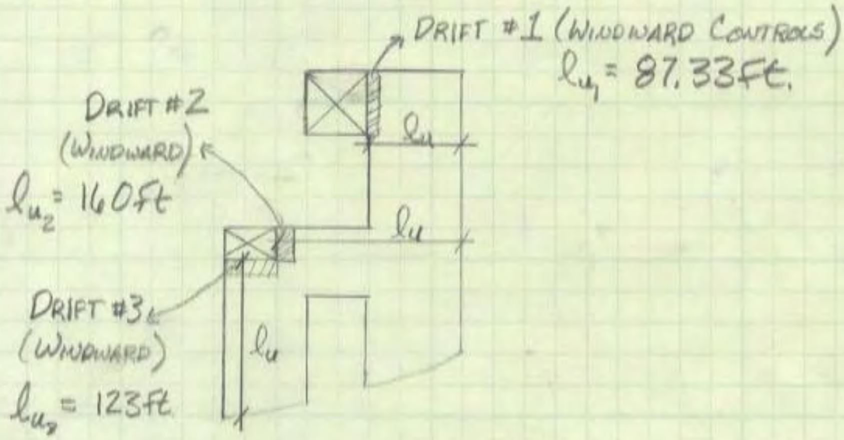
Drift does not apply at parapit because the height of the parapit is 2ft. but the height of the flat roof snow is approximately 2ft.

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

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

$$h = \frac{37 psf}{19.2} = 1.93 \text{ feet} \therefore \text{ drift can only be } .07 \text{ ft high.}$$

Drift At Stair Towers



WINDWARD

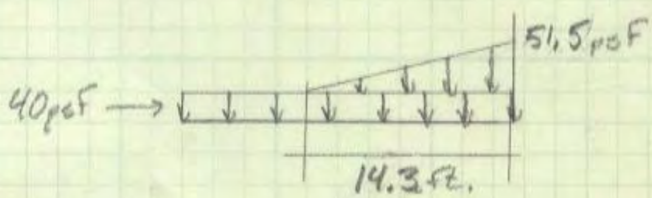
DRIFT #1

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

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

$$\omega_1 = 2.68 ft \times 19.2 \, {}^{lb}/_{ft^3} = 51.5 \, psf$$

$$\text{DRIFT WIDTH} = 4 h_d = 4\left({}^{2.68}/_{.75}\right) = 14.3 \text{ ft.}$$



THOMAS KLEINOSKY | SNOW LOADS | Pg 3 of 3

DRIFT # 2

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

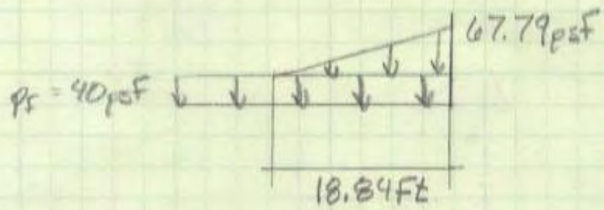
$$h_d = 4.71\,ft$$

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

$$3/4\, h_d = 3.53\,ft$$

$$\omega = 3.53\,ft \times 19.2 = 67.79\,psf$$

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



DRIFT # 3

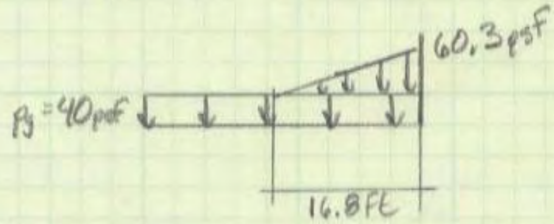
$$h_d = \left[0.43\sqrt[3]{\ell_u}\sqrt[4]{p_g+10}\right] - 1.5 = \left[.43\sqrt[3]{123}\sqrt[4]{50}\right] - 1.5$$

$$h_d = 4.2\,ft$$

WINDWARD → $3/4\, h_d = 3.14\,ft.$

$$\omega = 3.14\,ft \times 19.2\,pcf = 60.3\,psf$$

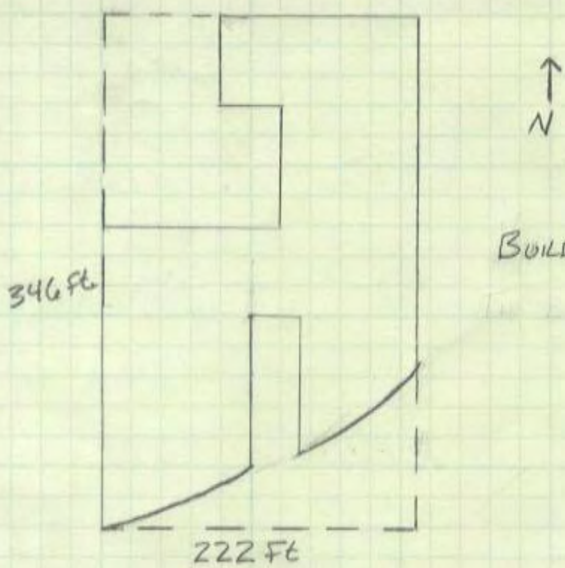
$$\text{DRIFT WIDTH} = 4h_d = 4(4.2) = 16.8\,ft.$$



Appendix B: Wind Hand Calculations

THOMAS KLEINOSKY | WIND CALCS | Page 1 of 6

SIMPLIFYING BUILDING FOOTPRINT



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

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

$$g_Q = g_v = 3.4$$

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

$c = 0.3$

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

$\rightarrow \beta = 0.01 \rightarrow$ 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

$n_H = 4.6n_1 (h/V_{\bar{z}})$

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

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

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

$= 0.152$

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

$= 8.92$

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

$= 0.106$

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

$= 46.54$

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

$= 0.021$

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

$= \underline{0.227}$

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

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

$= 0.8$

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

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

$= 0.840$

EAST - WEST

$n_H = 6.02$

$R_H = 0.152$

$n_B = 4.6(2/3)\left(\frac{346}{76.33}\right)$

$= 13.93$

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

$= 0.069$

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

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

$= 0.033$

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

$= \underline{0.184}$

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

$= 0.77$

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

$= 0.815$

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

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

$0 \to h/2 \to 0 \to 75$ft
$C_p = -0.9, -.18$

75ft → 150ft
$C_p = -0.9, -.18$

150 → 300ft
$C_p = -0.5, -.18$

>300ft
$C_p = -0.3, -.18$

E-W Direction

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

0 → 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 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

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

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

$C_p = -0.9, -.18$

$75ft \rightarrow 150ft$

$C_p = -0.9, -.18$

$150 \rightarrow 300ft$

$C_p = -0.5, -.18$

$>300ft$

$C_p = -0.3, -.18$

E-W Direction

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

$0 \rightarrow 75ft$

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

$75 \rightarrow 150ft$

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

$= -0.83$

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 B.1

Gust Factor, Velocity pressure (q_z) and (q_h) Calculations per Excel Spread Sheets

Gust Factor (G_f)	
$g_q=g_v$	3.4
n_1	0.666666667
$g_R=$	4.091678828
b	0.45
α	0.25
z	90
V	90
V_z	76.33409962
L_z	447.0878162
l	320
ϵ	0.333333333
c	0.3
N_1	3.904657887
I_z	0.253804797
β	0.01
R_n	0.05930516

Velocity Pressure (q_z)	
Entry Level	11.5535808
Basement	11.5535808
Level 1	13.70214144
Level 2	15.56692992
Level 3	16.6209408
Level 4	17.51279616
Level 5	18.40465152
Level 6	19.4586624
Level 7	20.0667456
Level 8	20.8775232
Level 9	21.6883008
Level 10	22.296384

Velocity Pressure (q_h)	
$q_h=$	22.296384

Appendix C: Seismic Hand 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_{01}

Based on Geotechnical Report this is site Class D

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

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

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

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

(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

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} \leq \frac{S_{D1}}{T_4\left(\frac{R}{I}\right)}$ 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

Addendix D: Gravity Load Check Hand Calculations

Floor Construction

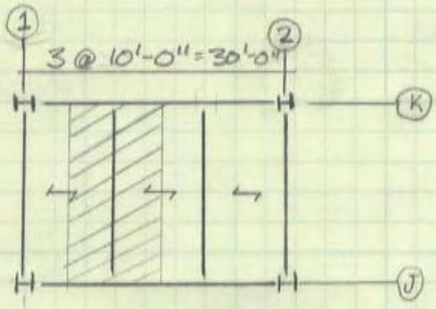
3" x 20GA Composite Deck
3½" LW Concrete

$t = 3 + 3\frac{1}{2} = 6\frac{1}{2}"$

$f'_c = 3500\,psi$

Floor Loads

DL = 95 psf
LL = 80 psf



Typical Bay

Decking (Per Vulcraft 2008)

Unshored Length

3VLI20 w/ 3½" topping - 3 span condition
13'-3"
13'-3" > 10'-0" ∴ OK ✓

Super Imposed live loads

allowable = 149 psf @ 10'-0" Clear span
149 psf > 80 psf ∴ OK ✓

Fire Resistance

From the 2001 Fire Resistance Directory
3½" LW Conc. Allows a 2hr Fire rating

Thomas Kleinosky | Gravity Checks | Page 2 of 10

Beam Checks:

W 12×30 [14] <c + 3/4">

Tributary width = 10'-0"
Span = 27'-4"

W12×30 Properties

$F_y = 50\text{ksi}$
$I_x = 238\text{in}^3$
$A = 8.79\text{in}^2$

Loads

LL = 80psf → non-reducable
SDL = 47psf
DL = 48psf → Slab + Deck Self wt.

$w_u = 1.2D + 1.6L$

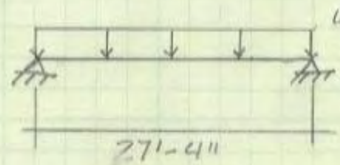
Dead

$(47+48) \times 10/1000 = 0.95\text{klf}$

Live

$\left(\frac{80\text{psf}}{1000}\right) \times 10\text{ft} = 0.8\text{klf}$

$w_u = 1.2(.95) + 1.6(0.8) = 2.42\text{ klf}$ + Beam Self wt.



$M_u = \frac{2.42(27.33^2)}{8} = 225\text{k-ft}$

$V_u = \frac{2.42(27.33)}{2} = 33^k$ → drawings call out 35^k ∴ OK ✓

$b_{eff} = \min \begin{cases} 2 \times \frac{\text{Span}}{8} = \frac{2(27.33)}{8} \times 12 = 82\text{in.} \rightarrow \text{Controls} \\ 2 \times \frac{1}{2} \text{ dist to adjacent beam} = 2 \times \frac{10}{2} \times 12 = 120\text{in.} \end{cases}$

Thomas Kleinosky | Gravity Checks | Page 3 of 10

For 3/4" ϕ studs + 3500 psi conc.

$Q_n = 17.2^k$ (table 3-21)

assume $a \approx 1$ in

$Y_2 = 6.5 - 1\text{in}/2 = 6$ in

$\phi M_n = 244$ k-ft > 225 k-ft ∴ ok ✓

$\Sigma Q_n = 110^k$

number of studs $= \frac{\Sigma Q_n}{Q_n} = \frac{110^k}{17.2} = 6.4$ stud/side

$6.4 \times 2 = 12.8$ studs → use 14 studs

$\phi V_n = 95.9^k$ (table 3-6)

$95.9^k > 33^k$ ∴ ok ✓

Check unshored length:

W12x30 → $\phi M_p = 162$ k-ft

Construction Loads = 20 psf

$w_u = 1.2D + 1.6L$

$w_u = 1.2(48\text{psf} \times 10\text{ft}) + 1.2(30\text{plf}) + 1.6(20\text{psf} \times 10\text{ft})$
$= 0.932$ klf

$M_u = \frac{0.932(27.33^2)}{8} = 87$ k-ft

$\phi M_p > M_u$ ∴ ok ✓

Thomas Kleinosky | Gravity Checks | Page 4 of 10

Check wet concrete deflection:

$w_{wc} = (10 \times 48) + 30 = 0.51\,klf$

$\Delta_{wc} = \frac{5wl^4}{384EI} = \frac{5(.51)(27.33)^4(1728)}{384(29000)(238)} = 0.93\,in$

$\Delta_{wc_{max}} = \frac{l}{240} = \frac{27.33(12)}{240} = 1.4\,in.$

$0.93\,in < 1.4\,in \therefore OK$ ✓

Check Live Load Deflection (Δ_{LL}):

$w_{LL} = 80\,psf \times 10\,ft = 0.8\,klf$

(table 3-20) $I_{LB} = 498\,in^4$

$\Delta_{LL} = \frac{5(0.8)(27.33^4)(1728)}{384(29000)(498)} = 0.695\,in.$

$\Delta_{LL_{max}} = \frac{l}{360} = \frac{27.33 \times 12}{360} = 0.911\,in.$

$\Delta_{LL} < \Delta_{LL_{max}} \therefore OK$ ✓

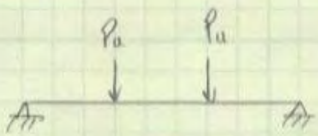
$\boxed{\text{Use a W12×30 [14]}}$

THOMAS KLEINOSKY | Gravity Checks | Page 5 of 10

Composite Girder: W16 × 89 [36]

Properties:

$F_y = 50$ ksi
$I_x = 1300\ in^3$
$A = 26.2\ in^2$
$t_f = 0.89\ in$
$b_f = 10.4\ in$



$P_u = P_{u_{DEAD}} + P_{u_{Live}}$

$L_{red} = 0.25 + \frac{15}{\sqrt{2(819.9)}} = 0.62$

Dead:
$P_{u_D} = (95psf \times 10 \times 27.33) = 26^k$

Live:
$P_{u_L} = (80psf \times 10ft \times 27.33ft) \times 0.62 = 13.6^k$

$P_u = 1.2(26) + 1.6(13.6) = 53^k$

$V_u = 53^k$ $\quad M_u = 53^k \times 10ft = 530$ k-ft

Find b_{eff}

$b_{eff} = \min \begin{cases} 2 \times \frac{Span}{8} = 2 \times \frac{30}{8} \times 12 = 90\ in. \\ \text{Clear Span} = \cancel{27.33 \times 12} = \end{cases}$

Find ΣQ_n

For Girder, $Q_n = \min \begin{cases} R_g R_p A_{sc} F_u = 1(.75)(\frac{.75}{2})^2 \pi (65) = 21.5^k \\ .5 A_{sc} \sqrt{f'_c E_c} = .5(\frac{.75}{2})^2 \times \pi \sqrt{3.5(2160)} \end{cases}$

$= 19.2^k \rightarrow$ Use

$\Sigma Q_n = 19.2^k \times \frac{36 \text{ studs}}{2} = 346^k$

Is it Partially composite?

$V'_c = 0.85(3.5)(90)(6.5) = 1740^k$

$V'_s = 26.2(50) = 1310^k$

$\Sigma Q_n = 346 \le \begin{cases} 1740^k \\ 1310^k \end{cases}$ Yes, it's Partially Composite

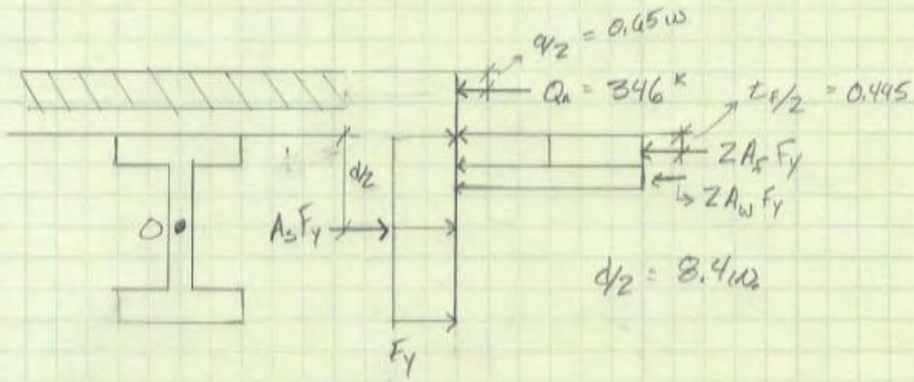
Thomas Kleinosky | Gravity Checks | Page 6 of 10

Find a

$$a = \frac{346}{.85(3.5)(90)} = 1.3\text{ in}$$

Find PNA

$$PNA = \frac{1310 - 346}{2(50)(10.4)} = 0.93\text{ in} > 0.89\text{ in} \therefore \text{PNA in web}$$



$$\text{Force in Web} = A_sF_y - \Sigma Q_n - 2A_fF_y$$

$$= 1310 - 346 - 2(10.4 \times .875)(50) = 54^k$$

$$2A_wF_y = 2(t_f)y(F_y) = 2(0.5)y(50) = 54^k$$

$$y = 1.08\text{ in}$$

① $$\Sigma M_{n_o} = \left[8.4 - 0.875 - \left(\frac{1.08}{2}\right)\right] \times (2 \times 54) + \left[8.4 - \left(\frac{.875}{2}\right)\right] \times (2 \times 0.875 \times 10.4 \times 50)$$

$$+ \left[8.4 + (6.5 - 0.65)\right] \times 346 = 12931\text{ k-in}$$

$$\phi M_n = 0.9\left(\frac{12931}{12}\right) = 969\text{ k-ft}$$

$$969\text{ k-ft} > 530\text{ k-ft} \therefore \underline{OK}$$

Page 7 of 10

CHECK UNSHORED LENGTH:

$W16\times89 \rightarrow \phi M_p = 657\,ft\text{-}k$

$w_u = 1.2(48psf \times 27.33) + 1.2(89plf) + 1.6(20psf \times 27.33)$

$= 2.6\,klf$

$M_u = \frac{2.6(30^2)}{8} = 292.5\,ft\text{-}5 \therefore ok$ ✓

Check wet concrete deflection:

$w_{wc} = (48 \times 27.33) + 89 = 1.4\,klf$

$\Delta_{wc} = \frac{5(1.4)(30)^4(1728)}{384(29000)(1300)} = 0.382\,in$

$\Delta w_{c\,max} = \frac{l}{240} = \frac{30(12)}{240} = 1.5\,in \therefore OK$

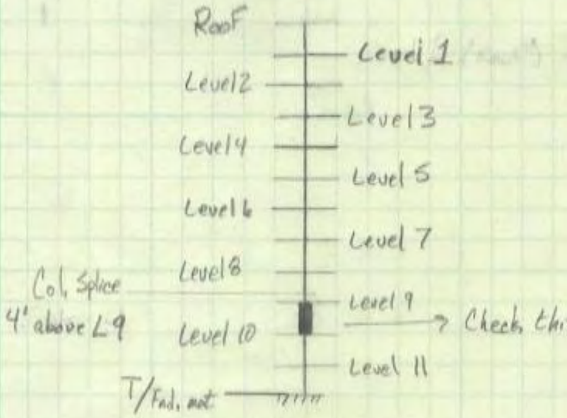
Check Δ_{LL}:

$\Delta_{LL} = \frac{5(0.8)(30^4)(1728)}{384(29000)(1300)} = 0.386\,in.$

$\Delta_{LL\,max} = \frac{30(12)}{360} = 1\,in \therefore \underline{OK}$

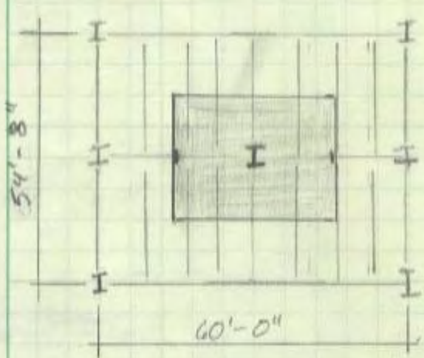
Column Check

Interior Column → J-4



W14x120 is from Splice to roof.

→ Check this Column, it's on Basement Level, W14x193



Tributary Area $= \left(\frac{60'-0"}{2}\right)\left(\frac{54'-8"}{2}\right) = 820\,ft^2$

Influence Area $= 60' \times 54'-8" = 3280\,ft^2$

ϕP_n for a W14x193 @ 15 ft

$\phi P_n = 2210^k$

Loads

Dead

Floors = 95psf
Roof = 30psf
Level 1 = 125psf

Live

Level 11 - 8 = 100psf
Level 7 - 2 = 80psf
Level 1 = 125psf
Roof = 40psf

THOMAS KLEWOSKY | Gravity Checks | Page 1 of 10

Column Loads

Roof:

$P_D = 30\,psf \times 820\,ft^2 = 24.6^k$

$P_L = 40\,psf \times 820\,ft^2 = 32.8^k$

Penthouse:

$P_D = P_L = 125\,psf \times 820\,ft^2 = 102.5^k$

Level 2:

$P_D = 95\,psf \times 820\,ft^2 = 77.9^k$

$LL_{red} = .25 + \frac{15}{\sqrt{3280}} = 0.51$

$P_L = .51(80 \times 820) = 33.4^k$

Level 3

$P_D = 95\,psf \times 820 = 77.9^k$

$LL_{red} = .25 + \frac{15}{\sqrt{2(3280)}} = 0.44$

$P_L = .44(80 \times 820) = 28.9^k$

Level 4

$P_D = 95 \times 820 = 77.9^k$

$LL_{red} = .25 + \frac{15}{\sqrt{3(3280)}} = 0.25 \rightarrow 0.4$

$P_L = 0.4(80 \times 820) = 26.2^k$

Level 5

$P_D = 77.9^k$

$P_L = 0.4(80 \times 820) = 26.2^k$

Level 6 & 7

$P_D = 77.9^k$

$P_L = 26.2^k$

Level 8 & 9

$P_D = 77.9^k$

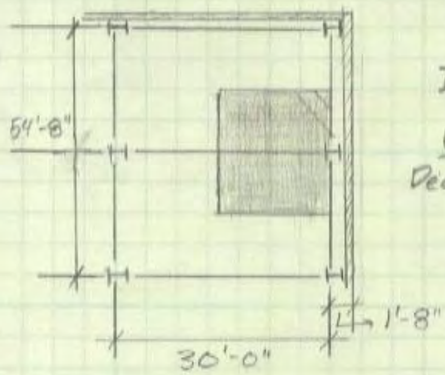
$P_L = (100\,psf \times 820) = 82^k$

Total $P_D = 750.3^k$ $P_L = 466.4$

$\boxed{P_u = 1.2(750) + 1.6(466) = 1650^k < 2210^k \therefore OK \checkmark}$

Thomas Kleinosky | Gravity Checks | Page 10 of 10

Exterior Column – B-9



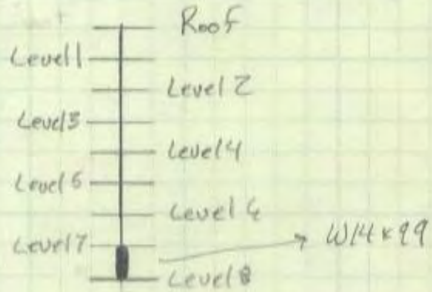
Tribatary Area = 27'-4" × 16'-8" = 456 sq ft.

Influence Area = 54'-8" × 31'-8" = 1731 sq ft

Loads

Dead:
- Floor = 95psf
- Roof = 30psf
- Level 1 = 125psf
- Facade = 48psf

Live:
- Level 11-8 = 100psf
- Level 7-2 = 80psf
- Level 1 = 125psf
- Roof = 40psf



Dead Loads

$P_{Roof} = 30 \times 456 = 13.7^k$

$P_{Floors} = 95 \times 456 = 43.3^k$

$P_{wall} = 89.25ft \times (54'\text{-}8") \times 48psf = 234.2^k$ total

$P_{Level 1} = 125psf \times 456 = 57^k$

Live Loads

Level	LL_{red}	P_L
Roof	N/A	$40 \times 456 = 18.2^k$
Level 1	N/A	$125 \times 456 = 57^k$
Level 2	$.25 + 15/\sqrt{1731} = 0.61$	$0.61(80 \times 456) = 22.2^k$
Level 3	$.25 + 15/\sqrt{(2)1731} = 0.5$	$0.5(80 \times 456) = 18.2^k$
Level 4	$.25 + 15/\sqrt{(3)(1731)} = 0.46$	$0.46(80 \times 456) = 16.8^k$
Level 5	$.25 + 15/\sqrt{(4)(1731)} = 0.43$	$0.43(80 \times 456) = 15.7^k$
Level 6	$.25 + 15/\sqrt{(5)(1731)} = 0.41$	$0.41(80 \times 456) = 15.0^k$
Level 7	0.40	$0.4(80 \times 456) = 14.6^k$
	Total	177.7^k

$P_u = 1.2(13.7) + 1.2(43.3 \times 6) + 1.2(57) + 1.2(234.2) + 1.6(177.7^k)$

$P_u = 962^k$

W14×99 @ 11ft → $\phi P_n = 1190^k > 962^k$ ∴ OK ✓