Technical Report III

Roberts Pavilion
Camden, NJ

Andrew Voorhees | Structural

Lateral Systems Analysis
November 12th, 2012
Faculty Advisor: Dr. Linda Hanagan

EXECUTIVE SUMMARY

The Roberts Pavilion is a patient care center located in Camden, NJ. It is part of the Cooper University Hospital and serves a large range of patient needs. Standing 10 stories above grade, it is a noticeable landmark when entering Camden. The pavilion was built between two existing hospital buildings and now serves to connect them. During construction, renovations updated the façades on the adjacent buildings to give a sense of uniformity to the complex. Aluminum and glass panels make up the main façade and give patients excellent views to the outside. Structurally, the building is framed in steel, with composite deck flooring. Lateral loads are resisted by four ordinary steel concentrically braced frames in each direction of the building.

Purpose and Scope

The purpose of this report is to provide an analysis of the Roberts Pavilion lateral system. This includes calculating wind and seismic loads on the building and determining the adequacy of the structure to resist them.

Story forces were determined using the procedures outlined in the code for wind and seismic loads. Comparing these forces, it was determined that seismic loads generally control in the North-South direction of the building, while wind loads control in the East-West direction. This is mostly because the building face normal to the East-West direction is much larger than the adjacent face, and thus provides much more contact area for wind pressures. The controlling base shear for wind was approximately 2,020 kips, and the controlling seismic base shear was approximately 1,644 kips.

A large portion of this report also focuses on a computer model that was generated with ETABS modeling software. The model was created for the purposes of observing the building's behavior in the applied loading. After verifying the model by hand calculations, each wind and seismic loading case was applied and the analysis was run. From the output, floor displacements and story shears were found. Displacements were compared to those allowable by code, and it was found that most of the cases passed. It was expected that there might be an issue with seismic displacement because of the larger forces being used compared to those used during design. This will be discussed fully in the report. Additionally, from the shears found through the model, members were able to be checked and verified for their designed forces.

TABLE OF CONTENTS

BUILDING INTRODUCTION

The Roberts Pavilion, as shown in red in Figure 1, is a recently constructed patient care center at the Cooper University Hospital in Camden, New Jersey. Completed in December 2008, the project cost about $220 million. The pavilion is approximately 320,000 GSF and occupies 10 stories above grade as well as one basement level. Additionally, during construction, the adjacent Kelemen and Dorrance Buildings, shown in Figure 1 in blue and purple respectively, underwent 51,000 GSF of renovations.

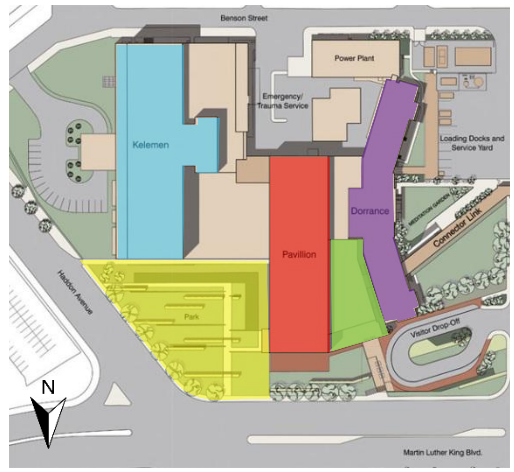


Figure 1 : Site plan (Courtesy of EwingCole)

Cooper has been a leading medical institution in southern New Jersey for many years. The Roberts Pavilion establishes Cooper's presence in Camden and upon entering the city, it is easily visible. Architecture and engineering systems were designed by EwingCole. They designed the façade, as shown in Figure 2, to be composed mostly of glass and aluminum panels. During renovations, façades of the adjacent buildings were updated to give the complex a sense of uniformity. The master plan also called for the demolition of the parking garage on the corner of Haddon Avenue and Martin Luther King Boulevard, as shown in yellow in Figure 1, and for the space to be turned into a park to improve the surrounding landscape.

Figure 2 : Roberts Pavilion (Courtesy of Halkin photography, LLC)

The lobby, shown in green in Figures 1 and 3, is a grand, open space with an abundance of natural light and warm colors. It also acts as a link between the new pavilion and the existing Dorrance Building which is shown in purple in Figure 1. Bamboo plantings and natural materials give the space a garden-like feel. Cooper wanted the pavilion to feel like a "healing garden" where patients experience a calm and peaceful atmosphere seemingly distant from the city outside. This idea is evident in the design from the lobby to the upper floors.

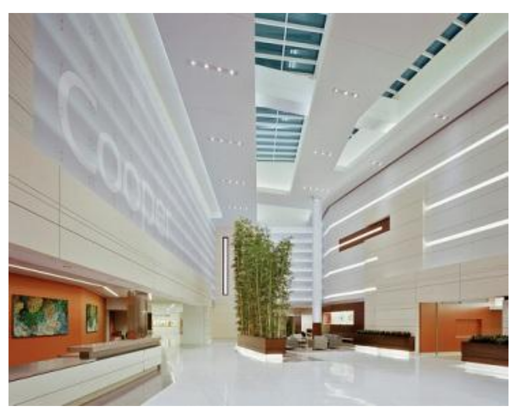


Figure 3 : Lobby (Courtesy of Eduard Hueber/Arch Photo, Inc.)

Each floor maintains a different function. The second floor houses clinical cardiology, while the third floor houses surgical suites, and the fourth and fifth floors hold the intensive care units. Typical patient rooms are located on floors six through ten.

STRUCTURAL OVERVIEW

Foundation

URS Corporation investigated the Roberts Pavilion site conditions by performing nine test borings. The top layer of soil in most of the drillings consisted of silty sand with some gravel and fragments of brick and concrete. This fill layer was classified as poorly to well-graded sand (SP-SW). Soil under the fill layer was classified as loose to dense silty sand with layers of clay becoming more firm with depth. 16" diameter reinforced piles were cast with a depth of -68' below the basement slab to reach firm soil. A minimum compressive strength of 4000 PSI concrete was used along with ASTM A615 Grade 60 reinforcement. Pile caps required concrete with minimum compressive strength of 5000 PSI and range in thickness from 3'-6" to 6'-0". The stratum layer under the footings was compacted to reach a bearing capacity of 4000 PSF.

The main basement will have an elevation of +8' above sea level (being about 5' above the water table), but elevator pits and mechanical space will be about +2' (1' below the water table). This means that the lower slab and walls will require waterproofing. Additionally these areas should be designed for hydrostatic uplift pressures. A permanent pump-operated subsurface drainage system was added to control the water level.

The main basement level is a 5" concrete slab, with a 16" slab poured in the north end under the mechanical room. Structural fill was placed for support under the foundations and used as backfill for the walls and footings. Soil pressures will need to be calculated when designing foundation walls.

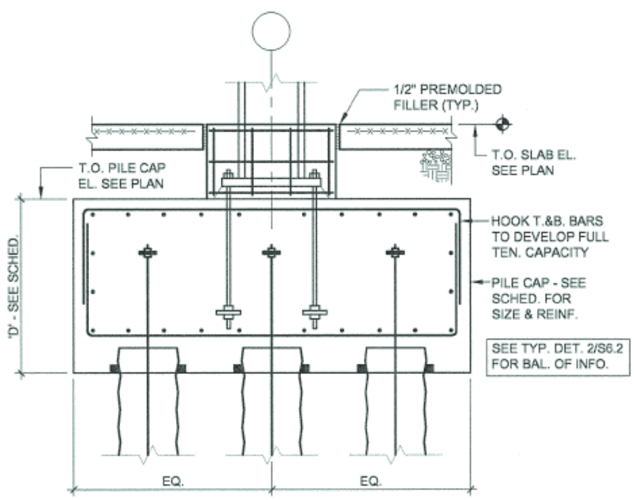


Figure 4 : Typical pile cap without pedestal

Floor System

Typical floor framing in the pavilion consists of a composite system. It incorporates a 2", 18-gauge steel deck with a 3¼" lightweight concrete topping reinforced with WWF (welded-wire-fabric). The Decking runs perpendicular to the beams and shear studs transfer the load to the beam to allow for composite behavior.

Framing System

All steel wide flange members in the building are A992 grade 50. Columns are typically spaced 30' on center in the North-South direction. In the East-West direction there are typically three bays; the interior span being 23', and the two exterior spans being 29'-6". Column spacing is shown in Figure 5 Column weights vary; with the heaviest being a W14x426. However, all columns have a 14" web.

Beams on floors 4 - 10 are typically wide flange members W16x26 and W14x22 spaced at 10' (See Figure 6). Floors 1 (ground) - 3 have larger beams, being that they are supporting heavier equipment. The 3rd floor holds the operating suites and part of the trauma unit thus it supports larger dead and live loads than most of the floors. It uses mostly W21x44 beams spaced at 7'-6".

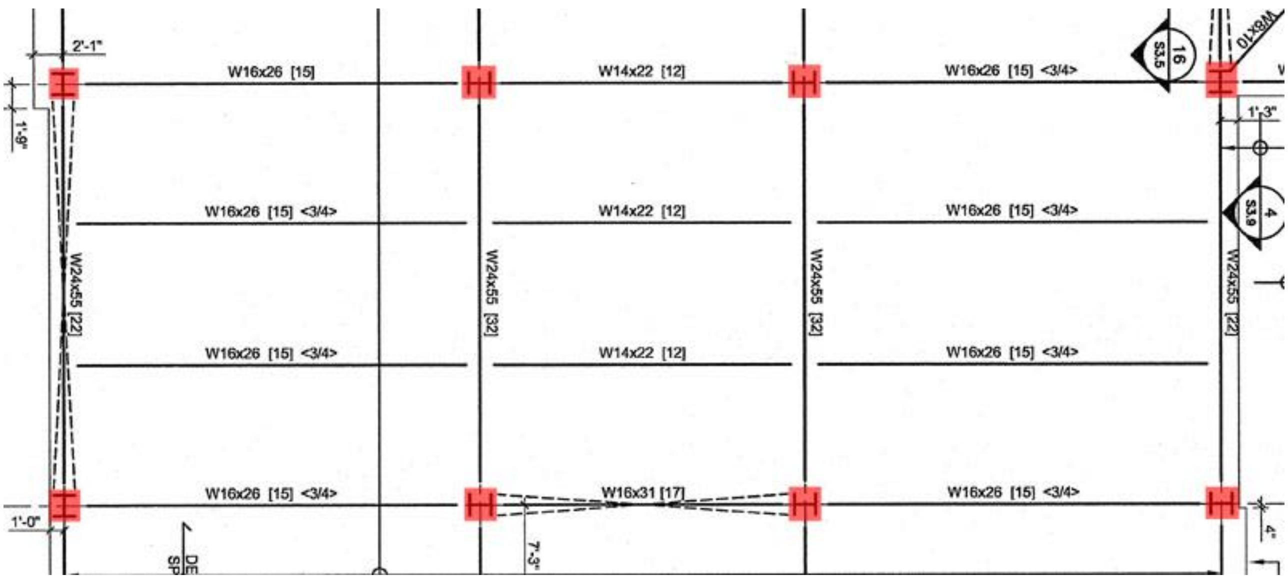


Figure 5 : Typical bay (See Appendix A for full framing plan)

Roof System

The roof of the pavilion supports mechanical equipment; specifically three cooling towers, an air cooled chiller, and three air handling units. It has two different levels, where the center level rises 3' above the main level to support the AHU's. Composite steel decking is also used on the roof, with the exception of the elevator core roof which is a poured slab. Wide flange members in the raised level are spaced at 6'-6" maximum to support the load from the mechanical units. In the south-west corner of the roof there is a small mechanical room with the roofing material being 1½", 20 gauge roof galvanized metal roof decking. All the mechanical systems on the roof are hidden by a 19' parapet.

Lateral System

The lateral resisting system in the pavilion consists of ordinary steel concentrically braced frames (OSCBF). There are four frames in each direction of the building as shown in Figure 6. Each frame extends through one full bay and through the full height of the building. Two typical frames are shown below in Figure 8. They consist of a variety of square HSS members with the most common being HSS10x10x1/2.

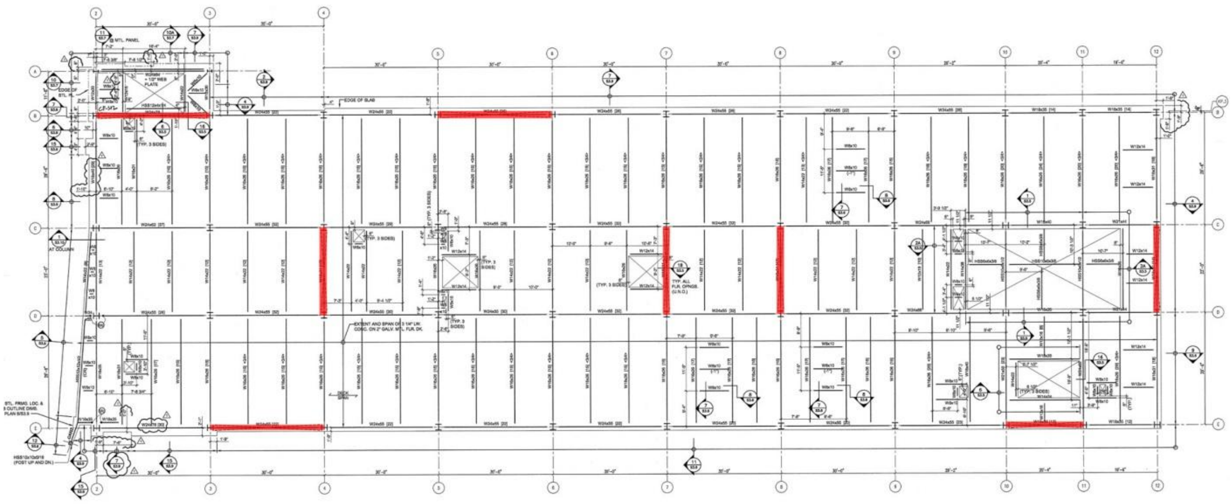


Figure 6 : Braced frame locations

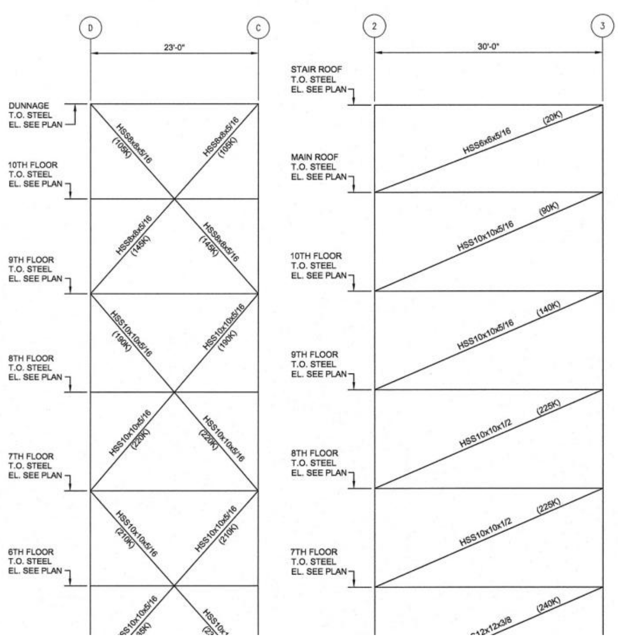


Figure 7 : Two typical braced frames (OSCBF)

Design Codes

Below is a list of the codes and standards applicable to the design of the Roberts Pavilion as used by the design team. Codes that were utilized in this report for analysis are listed separately.

Codes Used In Design:

- IBC 2000 (New Jersey Edition)
- ASCE 7-02 (Minimum Design Load for Buildings and Other Structures)
- ACI 318-02 (Building Code Requirements for Structural Concrete)
- PCI (Manual for Structural Design of Architectural Precast Concrete)
- AISC 12th Edition (Manual of Steel Construction)
- AWS D1.1 (Structural Welding Code for Steel
- ASTM (American Society for Testing and Materials)

Codes Used In Analysis:

- ASCE 7-05 (Minimum Design Load for Buildings and Other Structures)
- AISC 14th Edition (Manual of Steel Construction)

Materials

Below are listed the typical materials used in the construction of the Roberts Pavilion.

*Material strengths based on ASTM rating

Structural Steel	
Member Type	**Strength**
Wide Flange Member	A992 Grade 50
HSS Pipes	A500 Grade 46
Base Plates	A572 Grade 50
Lateral Moment Plates	A572 Grade 50
Splice Plates	A572 Grade 50
Angles	A36
Channels	A36
Anchor Bolts (1” and 2” Ø)	F1554 Grade 105
Bolts (¾” Ø)	A325 - X
Concrete Reinforcement	A615 Grade 60

Concrete	
Location	**Compressive Strength, f'_c (PSI)**
Slab on Grade	3000
Foundation Walls	4000
Piers	4000
Structural Slabs	4000
Beams	4000
Pedestals	4000
Equipment Pads	4000
Sidewalks	4000

Masonry	
Masonry	**Compressive Strength, f'_c (PSI)**
CMU	1500
Masonry Mortar	1500

Steel Deck		
Location	**Thickness (in)**	**Gauge**
Floor (composite)	2	18
Roof (composite)	2	18
Penthouse Roof	1.5	20

GRAVITY LOADS

Dead and Live Loads

Live load values were given on the structural drawings. These were similar to the values in ASCE 7-05 with the exception of several that aren't specified in the code. These values are denoted on the tables below with the value that was assumed. For spaces such as the operating rooms, that have a large difference between the code value and the value used for design, these calculations have used the value given in the drawings. This is because the live load may have been estimated larger because of specialized equipment, and it would be more conservative to use the larger value.

Dead loads are also shown below. An average value of 6.5 PSF for framing was calculated by summing the weight of framing on a given floor and dividing by the floor area. However, some floors are framed with larger members than the average floor (See Figure 26, Appendix A), thus 10 PSF was estimated as the maximum value. Although the value is larger than average, it provides a more conservative analysis.

Live Loads (PSF)		
Occupancy or Use	**As Designed**	**ASCE 7-05**
Lobby/Public Areas	100	100
1st Floor Corridor	100	100
Corridors above 1st Floor	80	80
Patient Rooms + Partitions	40+20	40+20
O.R.	100	60
O.R. Core	125	*60
Medical Equipment Rooms	100	*100
Stairways	100	100
Mechanical Rooms	150	*150
Conference Rooms	100	*100
Kitchen	125	*125
Roof	30	20

*Assumed Value

Dead Loads (PSF)	
System	**As Designed**
Framing	*10
Superimposed	*10
MEP	*5
Composite Floor	42

*Assumed Value

Snow Loads

Snow loads were calculated using ASCE 7-05. The ground snow load was given in the code as 25 PSF. Calculations in Appendix B show that the maximum design value for snow drift is approximately 93 PSF (94 PSF given in the drawings). Values used to calculate the flat roof snow load are shown to the right.

Flat Roof Snow Load	
Variable	**Value**
P_g (PSF)	25
C_e	1
C_t	1
I	1.2
P_f (PSF)	24

LATERAL LOADS

Wind Loads

To calculate wind loads on the Roberts Pavilion, a detailed analysis was conducted via the analytical procedure outlined in ASCE 7-05. For this procedure, the building shape was simplified to a rectangle with dimensions of 86' x 285'. Being 10 stories above grade, the building was assumed to be a flexible structure, meaning a gust factor of greater than 0.85. This was confirmed by calculations which determined the gust factor to be 0.89 in the East-West direction and 0.98 in the North-South direction. After obtaining the necessary variables, the wind pressures on each face of the building were determined. Net design pressures were cross referenced with values on the structural drawings and found to match. The pressures were then summed at each story level to find the forces in each direction. The story forces and overturning moments in each direction are shown in the table below. Base shear in the East-West direction was found to control at approximately 2,020 kips. The overturning moment in the East-West direction also controls at about 158,000 k-ft.

Wind Forces							
Level	Story Height, hx (ft)	N-S			E-W		
		Trib Area (SF)	Story Force, F_x (k)	Overturning Moment (k-ft)	Trib Area (SF)	Story Force, F_y (k)	Overturning Moment (k-ft)
Ground	0	602	18.91	0.00	1995	78.10	0.00
2	14	1204	38.56	539.85	3990	158.44	2218.17
3	28	1204	42.10	1178.90	3990	169.15	4736.22
4	42	1161	43.01	1806.22	3847.5	170.38	7156.05
5	55	1118	43.08	2369.19	3705	169.10	9300.57
6	68	1118	44.61	3033.27	3705	173.73	11813.58
7	81	1118	45.84	3713.24	3705	177.46	14374.63
8	94	1118	46.72	4391.41	3705	180.11	16930.25
9	107	1118	48.19	5156.68	3705	184.57	19749.18
10	120	1118	48.90	5868.48	3705	186.72	22406.45
Roof	133	2193	97.85	13013.51	7267.5	372.06	49484.07
Sum			**517.76**	**41,070.74**		**2019.83**	**158,169.17**

Wind pressures are distributed to the components and cladding in the applicable direction of interest. The forces are then transferred through the façade into the floor diaphragm, which transfers the forces into the braced frames acting in that direction. Wind pressures were calculated along the total face of the building to the top at 152'. However there is a parapet extending 19' from the roof level. Therefore, load on the parapet was transferred to the roof diaphragm at a height of 133' above the ground.

The controlling load case for wind forces as given in the code is shown below. Wind forces shown in the table above have been multiplied by 1.6.

$$1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$$

Shown below are the wind pressures acting in the North-South direction.

Wind Pressures: Walls North-South						
Bldg Height (ft)	K_z	q_z	Windward Pressure (PSF)	Leeward Pressure (PSF)	Interior Pressure (PSF)	Net Design Pressure (PSF)
0-15	0.85	17.23	13.51	-6.12	±4.89	19.63
15-20	0.9	18.24	14.30	-6.12	±4.89	20.43
20-25	0.94	19.05	14.94	-6.12	±4.89	21.06
25-30	0.98	19.86	15.57	-6.12	±4.89	21.70
30-40	1.04	21.08	16.53	-6.12	±4.89	22.65
40-50	1.09	22.09	17.32	-6.12	±4.89	23.45
50-60	1.13	22.90	17.96	-6.12	±4.89	24.08
60-70	1.17	23.72	18.59	-6.12	±4.89	24.72
70-80	1.21	24.53	19.23	-6.12	±4.89	25.35
80-90	1.24	25.13	19.71	-6.12	±4.89	25.83
90-100	1.26	25.54	20.02	-6.12	±4.89	26.15
100-120	1.31	26.55	20.82	-6.12	±4.89	26.94
120-140	1.36	27.57	21.61	-6.12	±4.89	27.74
140-152	1.38	27.97	21.93	-6.12	±4.89	28.05

Wind Pressures: Roof North-South		
Distance from edge (ft)	Suction (PSF)	Interior Pressure (PSF)
0-152	-21.86	±4.89
152-285	-12.14	±4.89

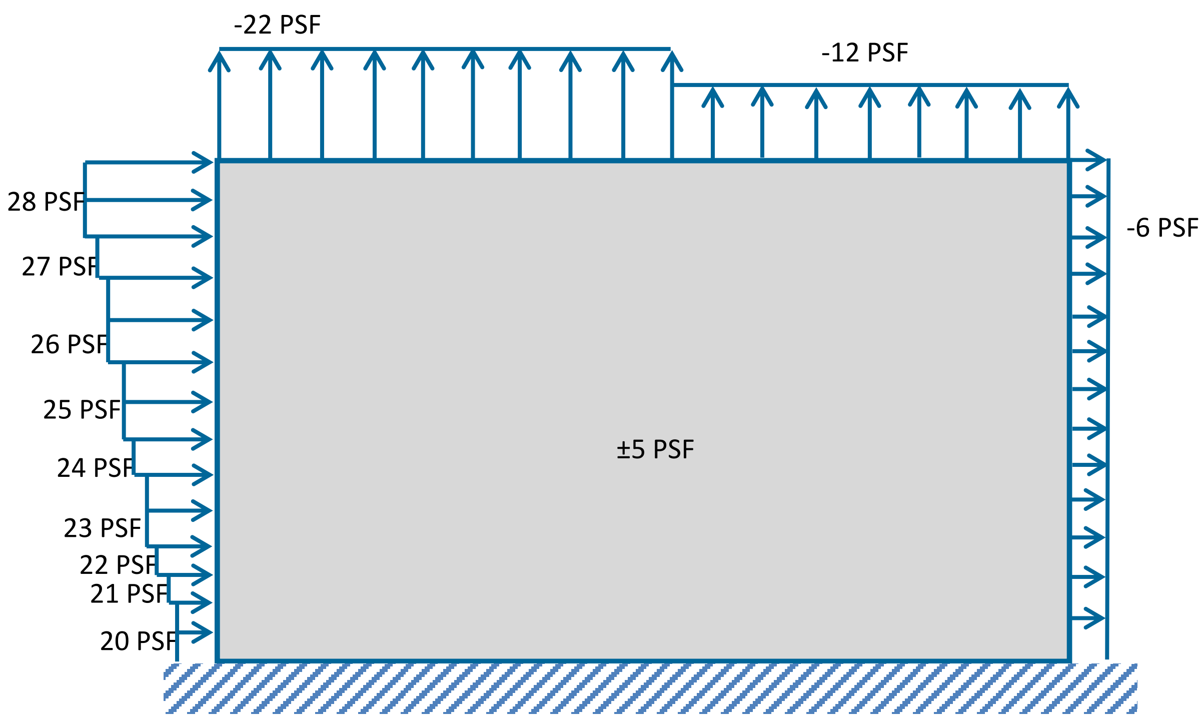


Figure 8: North-South Wind Pressures

Shown below are the wind pressures acting in the East-West direction.

Wind Pressures: Walls East-West						
Bldg Height (ft)	K_z	q_z	Windward Pressure (PSF)	Leeward Pressure (PSF)	Interior Pressure (PSF)	Net Design Pressure (PSF)
0-15	0.85	17.23	12.32	-12.14	±4.89	24.47
15-20	0.9	18.24	13.05	-12.14	±4.89	25.19
20-25	0.94	19.05	13.63	-12.14	±4.89	25.77
25-30	0.98	19.86	14.21	-12.14	±4.89	26.35
30-40	1.04	21.08	15.08	-12.14	±4.89	27.22
40-50	1.09	22.09	15.80	-12.14	±4.89	27.95
50-60	1.13	22.90	16.38	-12.14	±4.89	28.53
60-70	1.17	23.72	16.96	-12.14	±4.89	29.11
70-80	1.21	24.53	17.54	-12.14	±4.89	29.69
80-90	1.24	25.13	17.98	-12.14	±4.89	30.12
90-100	1.26	25.54	18.27	-12.14	±4.89	30.41
100-120	1.31	26.55	18.99	-12.14	±4.89	31.14
120-140	1.36	27.57	19.72	-12.14	±4.89	31.86
140-152	1.38	27.97	20.01	-12.14	±4.89	32.15

Wind Pressures: Roof East-West		
Distance from edge (ft)	Suction (PSF)	Interior Pressure (PSF)
0-76	-34.61	±4.89
76-86	-18.63	±4.89

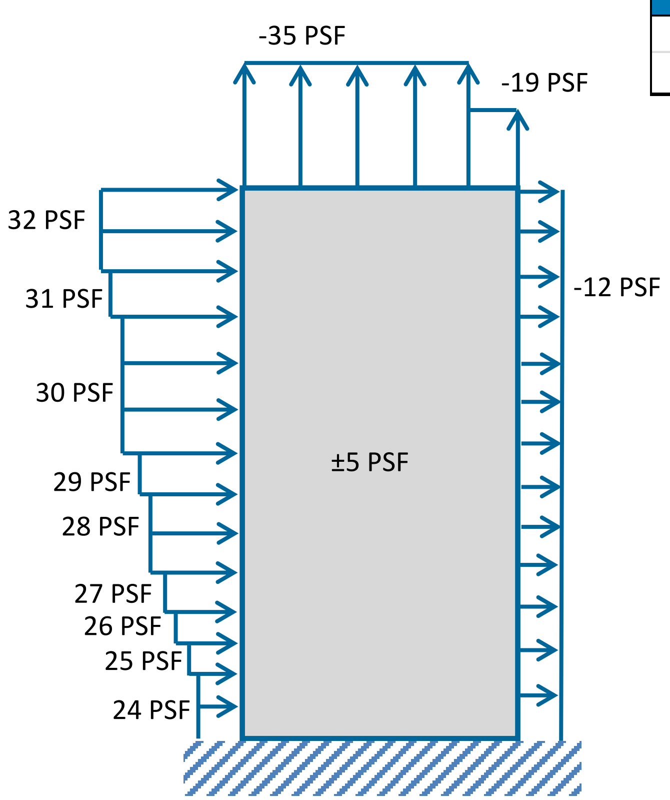


Figure 9: East-West Wind Pressures

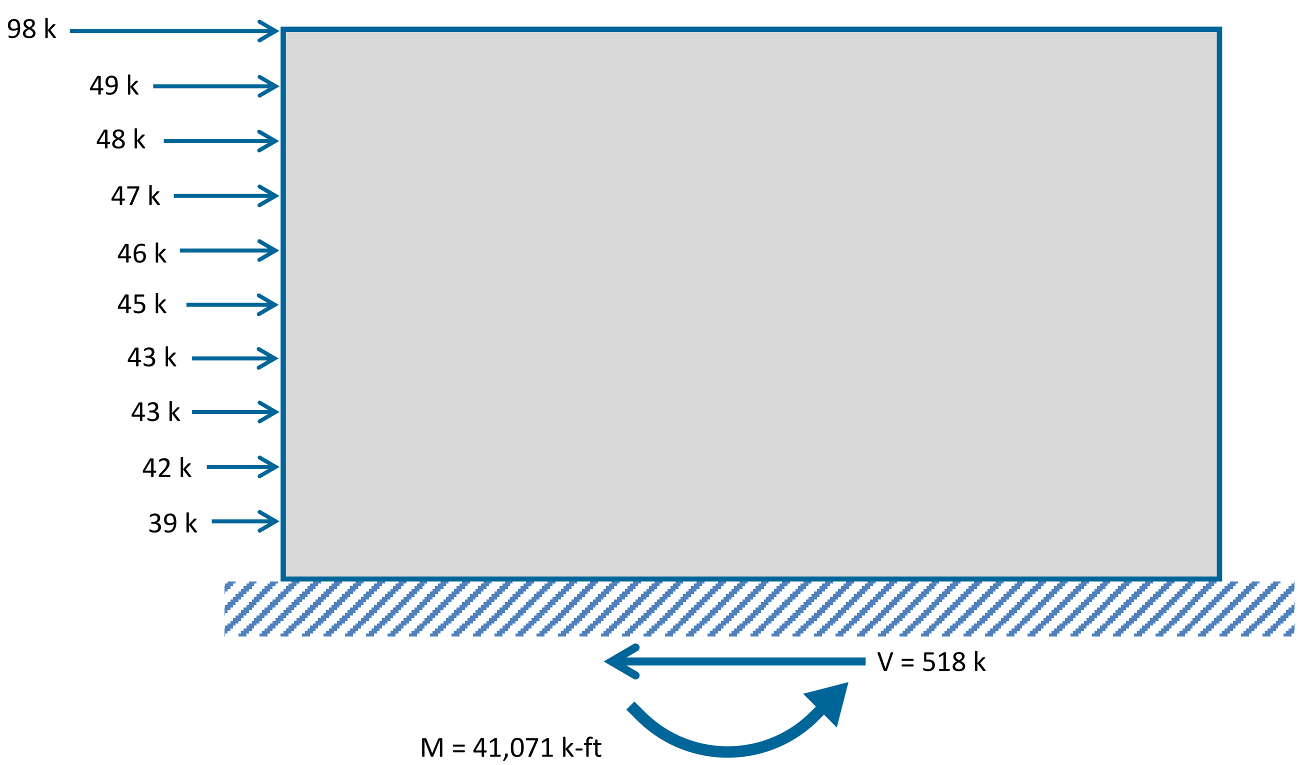


Figure 10: Wind Forces North-South

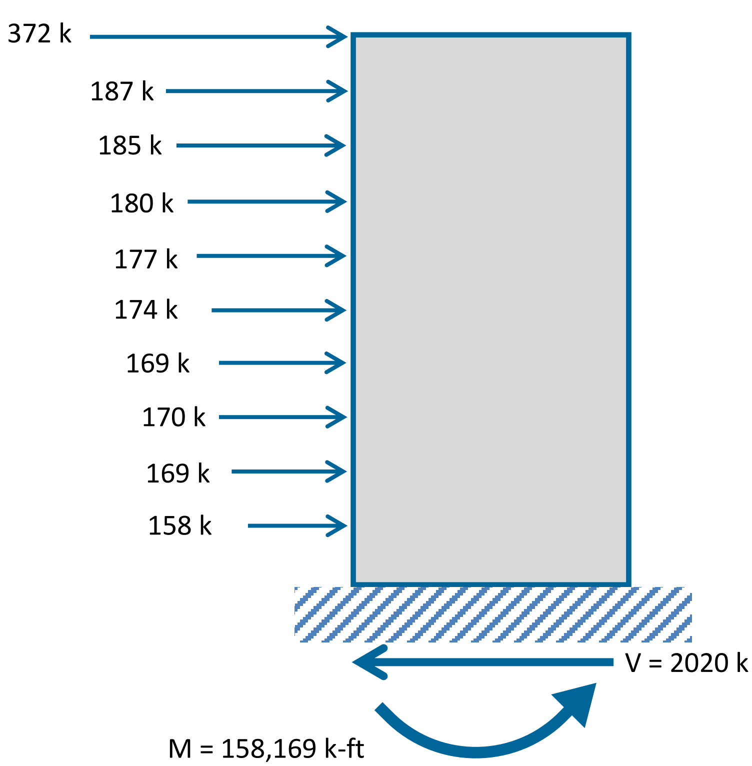


Figure 11: Wind Forces East-West

Seismic Loads

Seismic forces were calculated in compliance with ASCE 7-05 using the equivalent lateral force procedure. The building weight was totaled; resulting in a weight of approximately 27,136 kips. A detailed building weight summation was done in Microsoft Excel. A summary of the weight calculations is shown in the table below.

The seismic response coefficient found from the code was R=3. From this value and the approximate period, the base shear was able to be determined, and was found to be about 1644 kips. This value is the same in each direction because the code approximated period of the building is the same for both directions of the building. By code, the building's approximate period is 0.783 seconds, with an upper bound of 1.33 seconds. These values will be discussed later as part of the lateral system analysis.

The base shear calculated in this report is larger than the base shear recorded in the structural drawings: 1300 kips. This is due to the change in code from 2002 to 2005. Seismic design parameters for the region changed between the code issues. In 2002 the code called for a seismic response modification coefficient of R=5. However, the 2005 code calls for R=3. This change, along with changes in the maps for S_{DS} and S_{D1}, resulted in a larger base shear and larger seismic forces under the 2005 code. This does not mean that the building is not adequate under the current code. Further analysis in this report revealed that even under the larger loads, the building systems remain sufficient to resist the loads.

The controlling load combination for earthquake loads as given in the code is shown below. Forces shown in the table on the next page have been multiplied by 1.0.

$$1.2D + 1.0E + L + 0.2S$$

Building Weight (k)						
Level	**Floor (k)**	**Framing (k)**	**MEP (k)**	**SDL (k)**	**Walls (k)**	**Total (k)**
Ground	1654.72	354.19	196.99	1181.94	64.25	3452.08
2	1170.29	351.85	139.32	835.92	128.50	2625.88
3	1224.34	360.86	145.76	874.53	121.54	2727.02
4	1280.83	326.87	152.48	548.93	183.13	2492.24
5	963.10	240.60	114.66	687.93	222.25	2228.53
6	963.10	240.60	114.66	687.93	222.25	2228.53
7	963.10	240.60	114.66	687.93	222.25	2228.53
8	963.10	229.68	114.66	687.93	222.25	2217.61
9	963.10	218.76	114.66	687.93	222.25	2206.69
10	963.10	218.76	114.66	687.93	222.25	2206.69
Roof	975.33	292.39	496.43	230.86	526.93	2521.95
				Total Building Weight		**27,135.77**

Seismic forces applied at each level are shown in the table below.

Seismic Forces							
Level	**Story Height, h_x (ft)**	**Story Weight, w_x (k)**	**$w_x h_x^k$**	**C_{vx}**	**Story Force, F_x (k)**	**Story Shear (k)**	**Overturning Moment (k-ft)**
Ground	0	3452	0	0.00	0.00	1644.44	0.00
2nd	14	2626	53405	0.02	27.22	1644.44	381.09
3rd	28	2727	122355	0.04	62.37	1617.22	1746.23
4th	42	2492	177635	0.06	90.54	1554.85	3802.78
5th	55	2229	216094	0.07	110.15	1464.31	6057.98
6th	68	2229	275314	0.09	140.33	1354.17	9542.43
7th	81	2229	336167	0.10	171.35	1213.84	13879.12
8th	94	2218	396471	0.12	202.08	1042.49	18995.96
9th	107	2207	457386	0.14	233.13	840.40	24945.34
10th	120	2207	521347	0.16	265.74	607.27	31888.25
Roof	133	2522	670060	0.21	341.54	341.54	45424.22
Sum		**27136**	**3,226,233.28**	**1.00**	**1644.44**		**156,663.38**

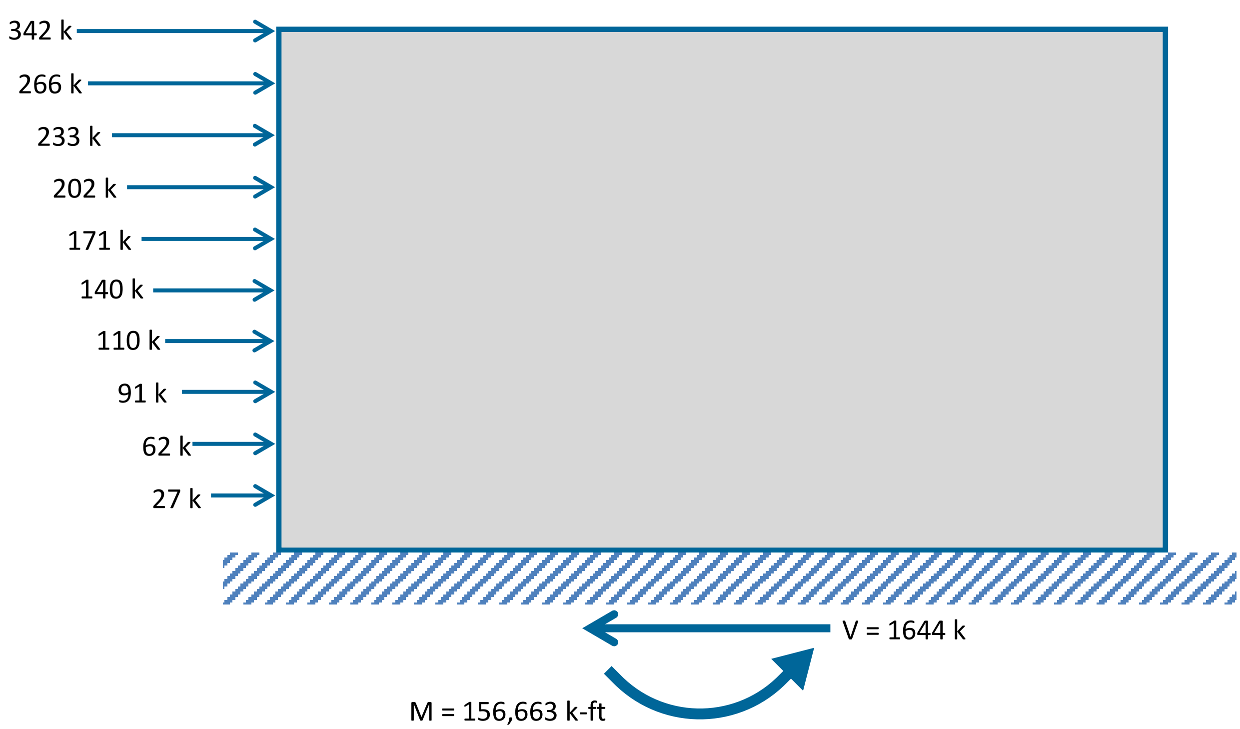


Figure 12: Seismic Forces North-South and East-West

Comparison

A comparison of wind and seismic loads is shown below. Wind loads control most of the lower floors in the East-West direction, as well as the roof level. This is because the wind loading on the parapet is transferred to the roof diaphragm and thus a larger force at the roof level is expected. Seismic loads control most of the upper floors in the North-South direction and also create a larger overturning moment. Seismic forces were expected to be larger in the upper levels because they are related to the height of each level.

Story Forces N-S (k)			
Level	Story Height (ft)	Wind	Seismic
Ground	0	18.91	0.00
2	14	38.56	27.22
3	28	42.10	62.37
4	42	43.01	90.54
5	55	43.08	110.15
6	68	44.61	140.33
7	81	45.84	171.35
8	94	46.72	202.08
9	107	48.19	233.13
10	120	48.90	265.74
Roof	133	97.85	341.54
Base Shear		**517.76**	**1644.44**

Overturning Moment N-S (k-ft)			
Level	Story Height (ft)	Wind	Seismic
Ground	0	0.00	0.00
2	14	539.85	381.09
3	28	1178.90	1746.23
4	42	1806.22	3802.78
5	55	2369.19	6057.98
6	68	3033.27	9542.43
7	81	3713.24	13879.12
8	94	4391.41	18995.96
9	107	5156.68	24945.34
10	120	5868.48	31888.25
Roof	133	13013.51	45424.22
Overturning Moment		**41,070.74**	**156,663.38**

Story Forces E-W (k)			
Level	Story Height (ft)	Wind	Seismic
Ground	0	78.10	0.00
2	14	158.44	27.22
3	28	169.15	62.37
4	42	170.38	90.54
5	55	169.10	110.15
6	68	173.73	140.33
7	81	177.46	171.35
8	94	180.11	202.08
9	107	184.57	233.13
10	120	186.72	265.74
Roof	133	372.06	341.54
Base Shear		**2019.83**	**1644.44**

Overturning Moment E-W (k-ft)			
Level	Story Height (ft)	Wind	Seismic
Ground	0	0.00	0.00
2	14	2218.17	381.09
3	28	4736.22	1746.23
4	42	7156.05	3802.78
5	55	9300.57	6057.98
6	68	11813.58	9542.43
7	81	14374.63	13879.12
8	94	16930.25	18995.96
9	107	19749.18	24945.34
10	120	22406.45	31888.25
Roof	133	49484.07	45424.22
Overturning Moment		**158,169.17**	**156,663.38**

LATERAL SYSTEM ANALYSIS

Computer Modeling

To analyze the lateral system of the Roberts Pavilion, a computer model, shown in Figure 13, was created in ETABS. Lateral frames were modeled as they appeared on the structural drawings. The frames consist of wide flange columns oriented in strong-axis bending with HSS members as the braces. Moment releases were used in braces and the beams. Also, after reviewing the structural drawings it was determined that columns should be fixed at the base. Walls and slabs in the basement level were not included because they were below the seismic base. Gravity columns and framing members were not considered in the model either because they are not meant to resist any lateral load. Although, a more detailed model may be considered as part of the proposal for the spring semester design work. Floors were modeled as rigid diaphragms. The actual floor system of the pavilion consists of composite deck and slab, therefore the rigid diaphragm is a good approximation of the floor's behavior. Self-weight of the model was neglected so that masses could be defined at each level according to the building weight that was previously calculated.

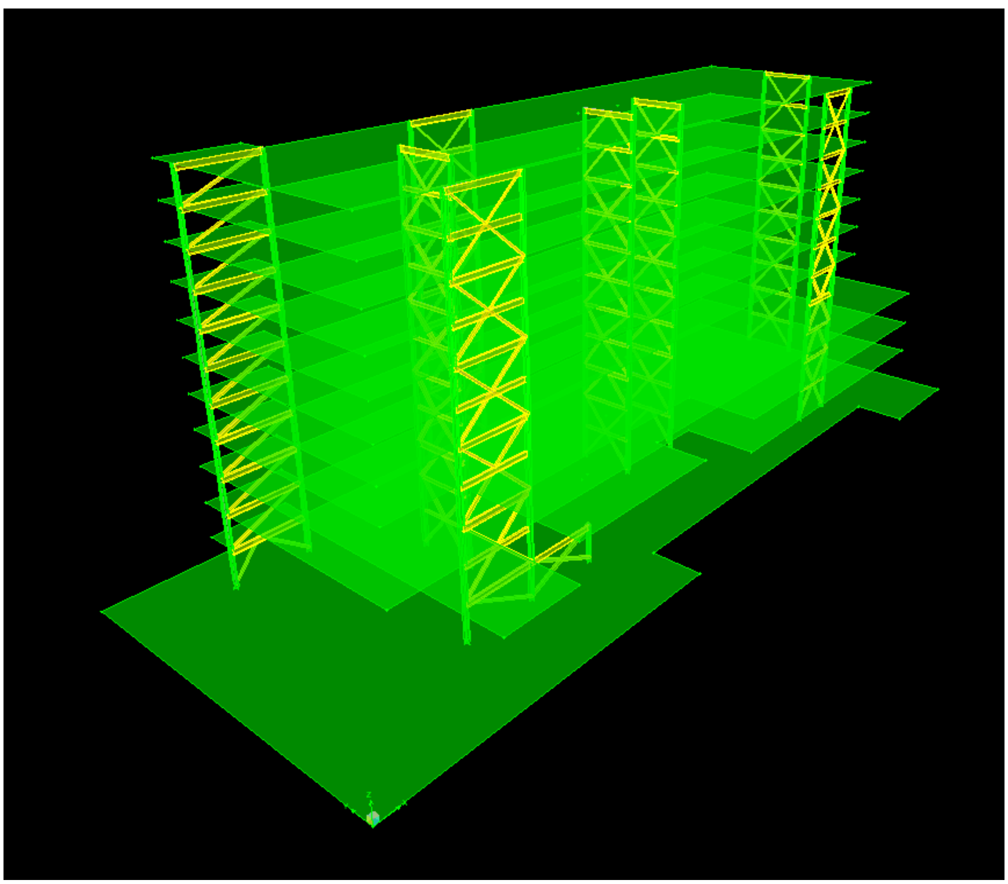


Figure 13: ETABS model

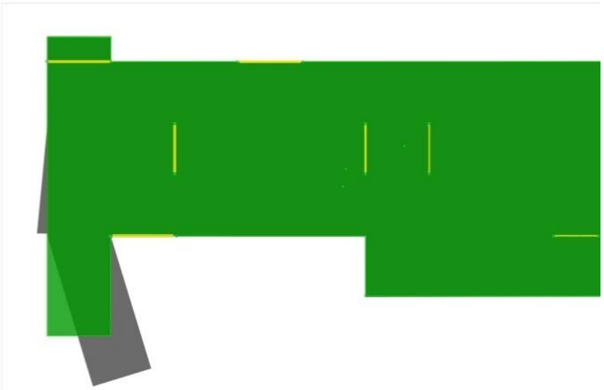


Figure 14: Floor 3 Diaphragm

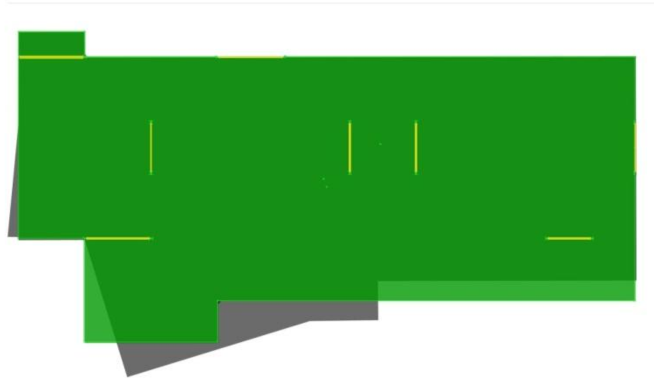


Figure 15: Floor 4 Diaphragm

For this analysis, the shape of the lobby was altered for simplicity. Shown in Figure 14 is the simplified shape of the third level. The gray rectangle that appears at an angle is the lobby roof that frames into floor 3, additionally, in Figure 15, the gray shape is the lobby roof at floor 4. In green is the shape that was used to approximate the diaphragm in the model. If a detailed model is created at a later point, the lobby will be formed more accurately.

An additional simplification was made at the roof level. In reality, mechanical equipment is elevated on a separate roof offset by approximately 3'. For this model, the roof was considered to be one level at a height of 133' from the ground. The masses of both levels were lumped at this level, and should have minimal effects on the model's response.

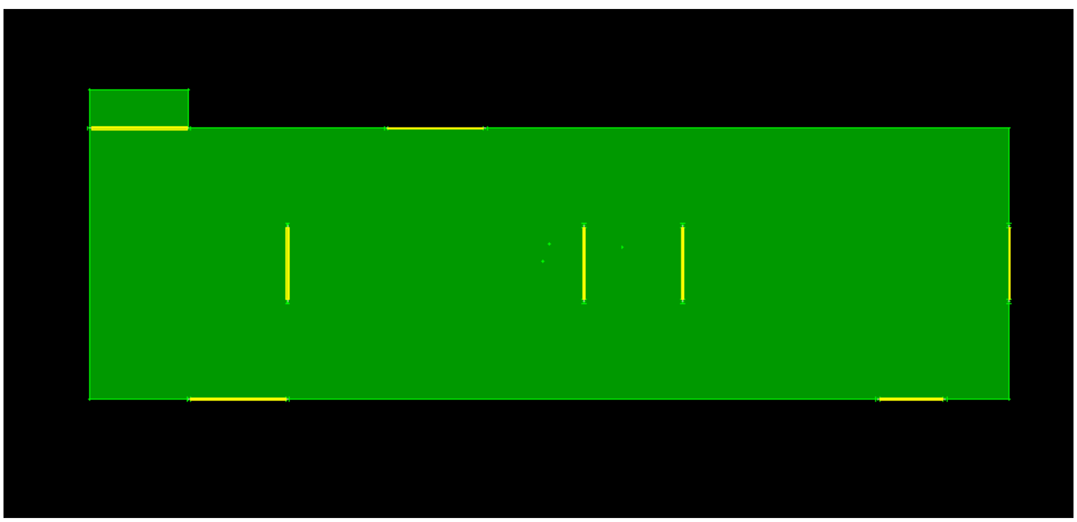


Figure 16: Typical Upper Floor Diaphragm Extents

Building Properties

The table below shows the center of mass and the center of rigidity coordinates of the pavilion as output from the model. A hand calculation confirming the coordinates of the roof level is shown in Appendix D. By inspection, coordinates for the other floors are were also verified. An inherent torsion in the building will need to be accounted for because of the offset between the center of mass and the center of rigidity.

Diaphragm Coordinates				
Story	XCM	YCM	XCR	YCR
Roof	137.48	41.72	161.66	45.95
10	137.45	41.73	161.50	45.90
9	137.44	41.73	161.29	45.62
8	137.38	41.75	161.00	45.02
7	137.32	41.75	160.89	44.21
6	137.31	41.74	161.21	43.50
5	137.30	41.74	161.72	43.41
4	138.21	26.79	163.79	42.77
3	140.72	31.58	168.36	42.25
2	147.16	34.36	163.77	41.59
Ground	157.73	69.37	0	0

Frame stiffnesses were determined by using a point load applied to each frame individually to find the displacement. These values, as well as relative stiffnesses to each other, are shown in the tables below. The average frame stiffness is around 40 k/in. From these values, as mentioned above, the center of rigidity was able to be calculated and confirmed with the program's output.

Frame Stiffnesses East-West (y-direction) (k/in)											
4 (D-C)			7 (D-C)			8 (D-C)			12 (D-C)		
Disp	K_{abs}	K_{rel}	Disp	K_{abs}	K_{rel}	Disp	K_{abs}	K_{rel}	Disp	K_{abs}	K_{rel}
23.01	**43.47**	1.00	25.81	**38.75**	0.89	26.07	**38.36**	0.88	28.26	**35.38**	0.81

Frame Stiffnesses North-South (x-direction) (k/in)											
B (2-3)			B (5-6)			E (3-4)			E (10-11)		
Disp	K_{abs}	K_{rel}	Disp	K_{abs}	K_{rel}	Disp	K_{abs}	K_{rel}	Disp	K_{abs}	K_{rel}
32.05	**31.21**	0.56	18.04	**55.44**	1.00	18.48	**54.12**	0.98	50.79	**19.69**	0.36

After the model had been verified, the period of the building was calculated by running the analysis. It was determined to be approximately 2.44 seconds. This is larger than the code limit of 1.33 seconds, found by C_uT_a, see Appendix C for calculations. Therefore, the lower period of 1.33 seconds should be used as it is more conservative.

Next, a 1000 kip load was applied at the roof level and shears in each frame were recorded to determine the percent shear that each frame takes. The table below shows the percent of the shear that each frame resists at individual levels. These percentages are necessary for member checks.

Story	Percent of Total Direct Shear (%)							
	Frames North-South (x-direction)				Frames East-West (y-direction)			
	B (2-3)	B (5-6)	E (3-4)	E (10-11)	4 (D-C)	7 (D-C)	8 (D-C)	12 (D-C)
1	9.99	54.44	33.73	1.83	15.17	9.56	28.15	47.12
2	18.31	32.92	31.07	17.70	30.34	29.88	16.74	23.04
3	14.32	37.93	35.97	11.79	33.89	24.07	20.36	21.67
4	21.75	35.21	33.91	9.13	33.53	25.27	22.68	18.52
5	16.77	31.27	39.84	12.12	31.85	24.13	22.10	21.91
6	26.06	32.82	32.11	9.01	31.48	25.74	23.45	19.32
7	24.16	35.79	29.44	10.61	28.97	23.77	25.40	21.86
8	33.64	27.44	26.27	12.65	24.51	26.15	27.58	21.76
9	22.27	33.12	31.97	12.64	24.65	25.84	26.96	22.55
10	25.60	26.61	33.72	14.08	23.39	28.18	26.70	21.73

Wind Analysis

A wind analysis of the Roberts Pavilion was run on the computer model. Based on ASCE 7-05, four different wind cases were considered, as shown in Figure 17.

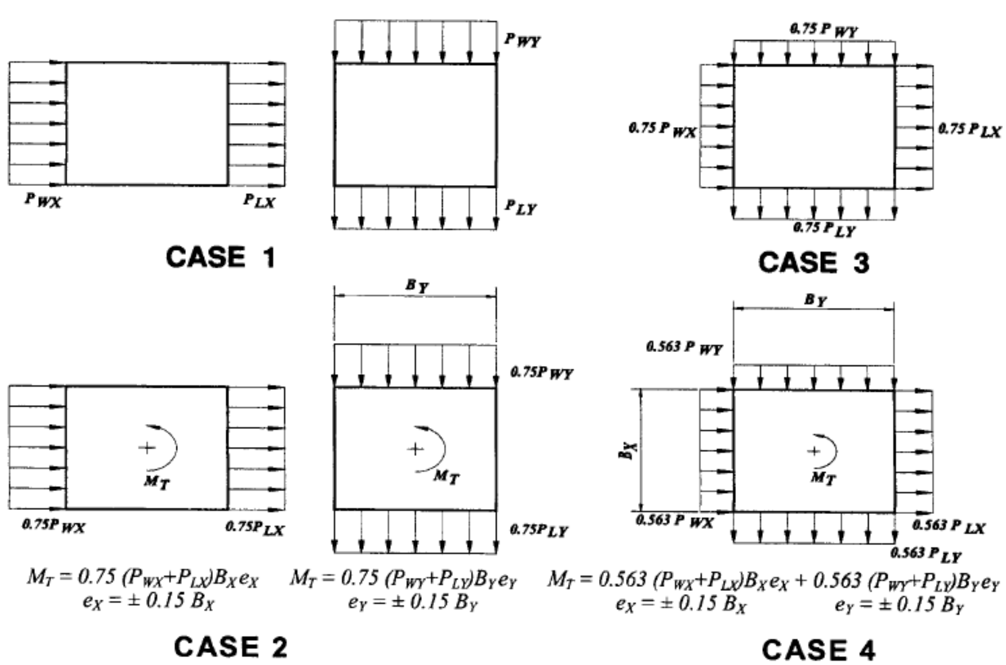


Figure 17: ASCE 7-05 Wind Load Cases

The wind load cases were applied to the model as shown below, resulting in 12 different cases:

$$P_X$$
$$P_Y$$
$$0.75\ P_X\ (-e_X)$$
$$0.75\ P_X\ (+e_X)$$
$$0.75\ P_Y\ (-e_Y)$$
$$0.75\ P_Y\ (+e_Y)$$
$$0.75\ P_X + 0.75\ P_Y$$
$$0.75\ P_X - 0.75\ P_Y$$
$$0.563\ P_X\ (-e_X) + 0.563\ P_Y\ (-e_Y)$$
$$0.563\ P_X\ (-e_X) + 0.563\ P_Y\ (+e_Y)$$
$$0.563\ P_X\ (+e_X) + 0.563\ P_Y\ (-e_Y)$$
$$0.563\ P_X\ (+e_X) + 0.563\ P_Y\ (+e_Y)$$

Rotations were applied using an eccentricity of 15% of the building length, offset from the center of pressure at that face. Each case was run in the model to find displacements and shears. ASCE 7-05 Chapter C Appendix C states that using wind loads factored by 1.6 is excessively conservative when considering serviceability. Therefore the code allows the use of the load combination:

$$D + 0.5L + 0.7W$$

Drifts for serviceability were recorded at points around the perimeter of the building under the load combination shown above. Using the drift limit of H/400, it was found that only case 1 P_y would not pass the serviceability drift limit. This limitation for drift is based on the movement of façade elements, in order to prevent cracking of wall elements etc. Thus, case 1 not passing the limit is not an issue of strength. The table below shows the drift calculations for case 1. Drift checks for the other cases are shown in Appendix F.

	Level	P_x				P_y				Δ_{allow}
		δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
Case 1	Roof	0.6434	-0.0013	0.0717	-0.0006	0.3462	3.8517	0.0405	0.4400	0.39
	10	0.5717	-0.0007	0.0746	-0.0007	0.3057	3.4117	0.0420	0.4564	0.39
	9	0.4971	0.0000	0.0756	-0.0015	0.2637	2.9553	0.0416	0.4594	0.39
	8	0.4215	0.0015	0.0709	-0.0012	0.2221	2.4959	0.0367	0.4246	0.39
	7	0.3506	0.0027	0.0725	0.0000	0.1854	2.0713	0.0347	0.4175	0.39
	6	0.2781	0.0027	0.0652	0.0032	0.1507	1.6538	0.0326	0.3782	0.39
	5	0.2129	-0.0005	0.0631	0.0012	0.1181	1.2756	0.0276	0.3571	0.39
	4	0.1498	-0.0017	0.0577	0.0011	0.0905	0.9185	0.0252	0.3202	0.42
	3	0.0921	-0.0028	0.0563	0.0002	0.0653	0.5983	0.0315	0.3208	0.42
	2	0.0358	-0.0030	0.0358	-0.0030	0.0338	0.2775	0.0338	0.2775	0.42
	Ground	0	0	0	0	0	0	0	0	0

Seismic Analysis

Seismic loads were evaluated in the computer model based on the story force in a single direction. Additionally, an accidental torsion was accounted for by offsetting the force from the center of mass by 5% in either direction. This resulted in six cases:

$$\mathbf{E_X}$$
$$\mathbf{E_Y}$$
$$\mathbf{E_X + E_X\,(-e_X)}$$
$$\mathbf{E_X + E_X\,(+e_X)}$$
$$\mathbf{E_Y + E_{YT}(-e_Y)}$$
$$\mathbf{E_Y + E_{YT}(+e_Y)}$$

The Roberts Pavilion was inspected for any vertical irregularities and none applied. However, it was noted that there was a possibility for horizontal irregularities such as torsional irregularity as shown in Figure 18.

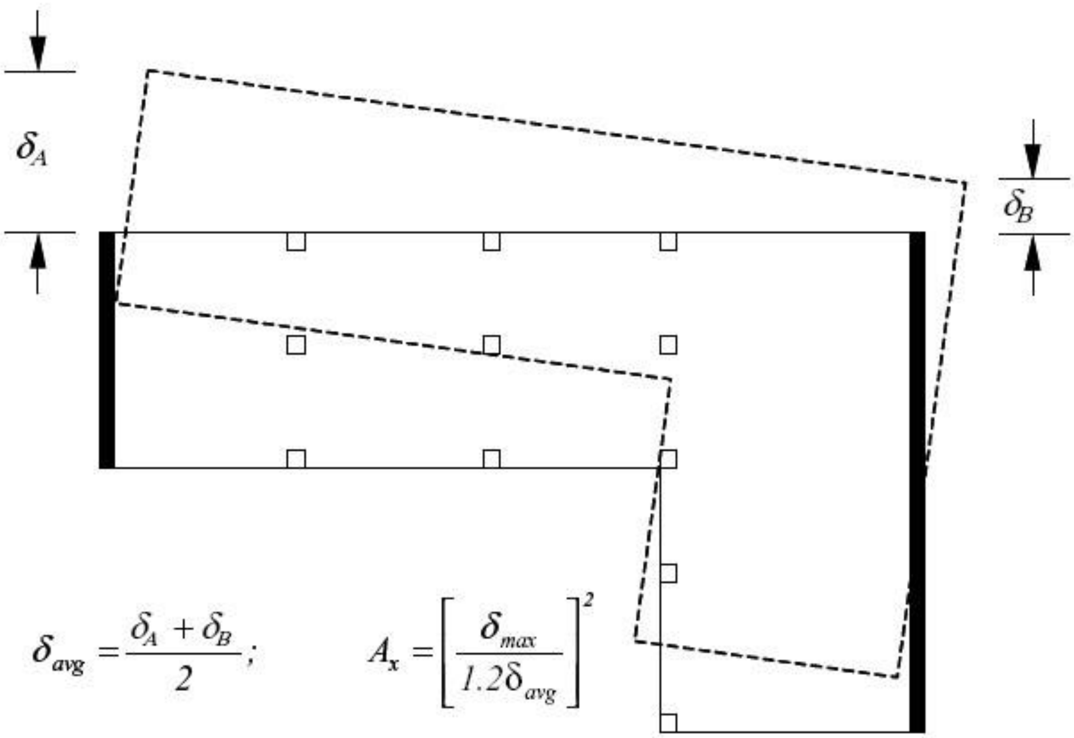


FIGURE 12.8-1 TORSIONAL AMPLIFICATION FACTOR, A_x

Figure 18: Torsional Amplification Factor

To check for irregularity, the model was run for each earthquake load case while considering a torsional amplification factor of A_X=1.0. Next, the deflections at each end of the building were determined for each case. It was determined that case **E_Y + E_{YT}(-e_Y),** shown below, had the largest difference in displacement from end to end. The maximum displacement divided by the average between the two ends was greater than 1.2, and even by 1.4. Therefore it was determined that the building exhibits extreme torsional irregularity. The values for the amplification factor A_X are shown below. Detailed calculations for all load cases are shown in Appendix G and H.

After the amplification factor was determined, it was multiplied by the moment created by the 5% offset. The load was then reapplied to the model and run again to give the final displacements. Detailed tables with displacements for each seismic case are shown in Appendix I.

Level	Y-Direction (-e)							Δ_{allow}
	δ_{xe1}	Δ_1	δ_{xe2}	Δ_2	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	A_x	
Roof	11.0807	1.27	2.7537	0.30	0.78	**1.62**	1.78	1.2
10	9.8109	1.35	2.4585	0.31	0.83	**1.62**	1.78	1.2
9	8.4598	1.37	2.1454	0.32	0.85	**1.63**	1.77	1.2
8	7.0863	1.27	1.8286	0.32	0.79	**1.60**	1.76	1.2
7	5.8168	1.24	1.5128	0.33	0.79	**1.58**	1.75	1.2
6	4.5726	1.11	1.1843	0.30	0.70	**1.57**	1.75	1.2
5	3.4658	1.03	0.8817	0.30	0.67	**1.55**	1.77	1.2
4	2.4314	0.89	0.5828	0.26	0.58	**1.55**	1.81	1.2
3	1.5377	0.85	0.32	0.21	0.53	**1.61**	1.90	1.2
2	0.6896	0.69	0.1143	0.11	0.40	**1.72**	2.04	1.2

Displacements were multiplied by C_d/I, then story drift was calculated. It was found that the displacements at the center of mass under earthquake loading caused relative story displacements that were just over those allowed by code. Allowable displacement values as prescribed in the code are shown on the next page in Figure 19. The displacements for the maximum case are recorded in the table on the next page, and can be seen in Appendix I. These displacements, with a maximum of 7.2" at the roof level, are expected to be large. Seismic forces determined in this report are larger than those that would have been determined in the design; this is due to the difference in the C_S value which caused a larger base shear. Therefore, it is expected that the displacements would be larger. These displacements, as calculations will later show, are not related to the strength of the structure. It will be shown later in this report, that even though the forces are larger, the frames are still adequate to support the loads.

Drift E-W (y - direction)										
	E_Y			$E_Y + E_{YT}$ ($+e_x$)			$E_Y - E_{YT}$ ($-e_x$)			
Level	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	Δ_{allow}
Roof	6.714	14.546	**1.642**	6.450	13.974	**1.576**	7.182	15.560	**1.760**	1.56
10	5.956	12.904	**1.747**	5.722	12.399	**1.675**	6.369	13.800	**1.875**	1.56
9	5.149	11.157	**1.773**	4.949	10.723	**1.701**	5.504	11.924	**1.902**	1.56
8	4.331	9.384	**1.667**	4.164	9.023	**1.602**	4.626	10.023	**1.781**	1.56
7	3.562	7.717	**1.658**	3.425	7.420	**1.597**	3.804	8.242	**1.766**	1.56
6	2.796	6.059	1.487	2.688	5.823	1.433	2.989	6.476	**1.582**	1.56
5	2.110	4.572	1.418	2.026	4.390	1.366	2.259	4.894	1.510	1.56
4	1.456	3.154	1.236	1.396	3.025	1.190	1.562	3.384	1.319	1.68
3	0.885	1.918	1.119	0.847	1.835	1.073	0.953	2.065	1.200	1.68
2	0.369	0.800	0.800	0.352	0.762	0.762	0.399	0.865	0.865	1.68
Ground	0	0	0	0	0	0	0	0	0	0

TABLE 12.12-1 ALLOWABLE STORY DRIFT, $\Delta_a{}^{a,b}$

Structure	Occupancy Category		
	I or II	III	IV
Structures, other than masonry shear wall structures, 4 stories or less with interior walls, partitions, ceilings and exterior wall systems that have been designed to accommodate the story drifts.	$0.025h_{sx}{}^{c}$	$0.020h_{sx}$	$0.015h_{sx}$
Masonry cantilever shear wall structures [d]	$0.010h_{sx}$	$0.010h_{sx}$	$0.010h_{sx}$
Other masonry shear wall structures	$0.007h_{sx}$	$0.007h_{sx}$	$0.007h_{sx}$
All other structures	$0.020h_{sx}$	$0.015h_{sx}$	$0.010h_{sx}$

Figure 19: Allowable drifts as prescribed by code

Foundation Impact

The foundation was checked for overturning moment and found that the resisting moment of the building was about 778,000 k-ft, which is much larger than the largest overturning moment of 158,000 k-ft. Therefore the foundation was found to be sufficient to resist the wind and seismic overturning moments, see detailed calculations in Appendix J. More in-depth calculations on the foundation would need to account for uplift, and the appropriate load combinations would need to be considered. Foundations may be analyzed more fully in the future.

Member Spot Checks

Spot checks on ground level members were conducted in two frames, one in each direction of the building. The columns were checked for their required axial load as well as the lateral load applied. Braces were checked for their required axial load. Detailed calculations can be seen in Appendix J. For strength checks it was determined that the frame in the North-South direction was controlled by seismic, and thus load combination 5 was checked. In the East-West direction it was determined that wind controlled and thus load combination 4 was checked. Frames that were check are shown below with the brace highlighted. Both columns and braces in each frame were verified as adequate.

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

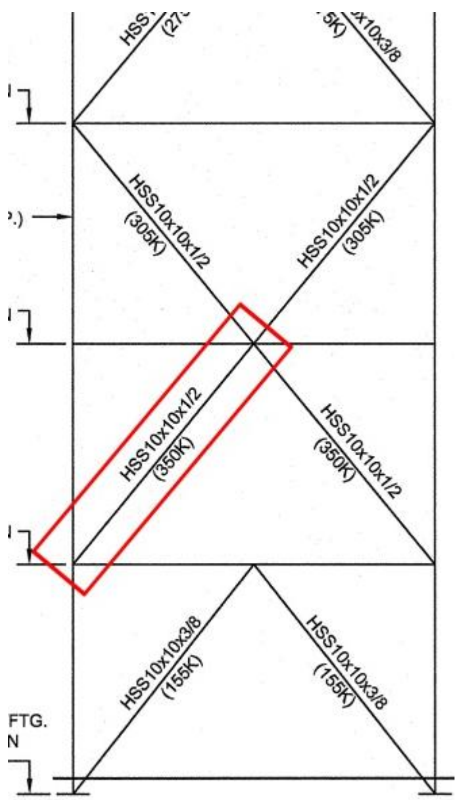


Figure 20: braced frame 8(D-C)

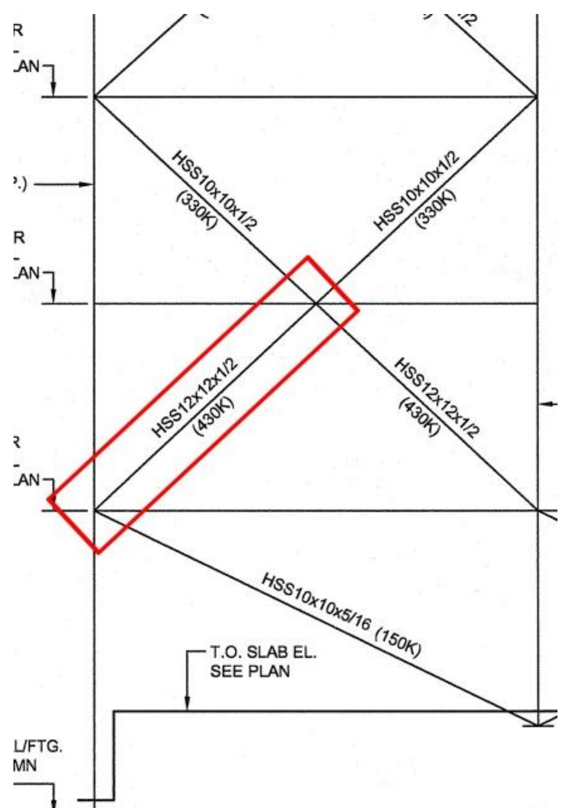


Figure 21: Braced Frame E(3-4)

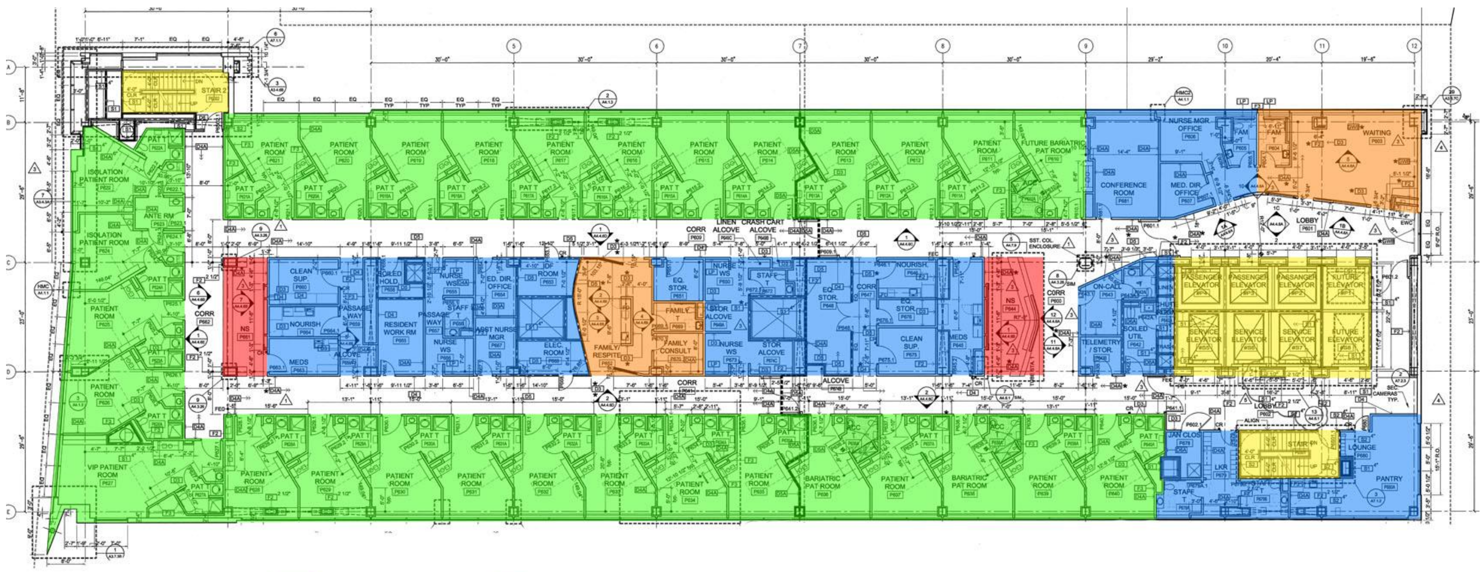
CONCLUSION

This report consisted of an analysis of the Roberts Pavilion lateral force resisting system. Wind and seismic loads were calculated for the building based on ASCE 7-05. The wind loads were verified with the values on the structural drawings. Seismic loads were determined to be larger than the values used during the design. This was due to differences in the code. However, calculations showed that even though the forces were larger, the structure was still adequate. It was determined that seismic forces controlled in the North-South direction with a base shear of approximately 1644 kips, while wind controlled in the East-West direction with a base shear of approximately 2020 kips. These forces were important for determining the design and verification of lateral resisting members.

An ETABS computer model was also created in order to observe building response to the forces. Centers of mass and rigidity were verified by hand calculations. The building was then checked for torsional irregularity, and subsequently, forces and moments were determined based on the amplification factor. After these adjustments, displacements were determined for all load cases. It was found that for the controlling cases of wind and seismic forces, the story drifts were just over the values determined by code. This is not an issue of strength but of serviceability, and may be evaluated further if a more detailed model is constructed.

The final step in this report was to determine the adequacy of lateral force resisting members. A frame in each direction was picked. Then the column and the brace on the ground floor were evaluated and determined to be adequate. This meant that even under the larger forces determined in this report, as compared to the design forces, the building was capable of withstanding the loads. A more comprehensive computer model may be constructed as part of the spring design project, in which lateral forces and their effects will be studied in further depth.

APPENDIX A: TYPICAL PLANS



Patient Rooms
Family Areas
Nursing Stations
Staff/Back of House
Vertical Circulation

Figure 22 : Typical patient room floor plan

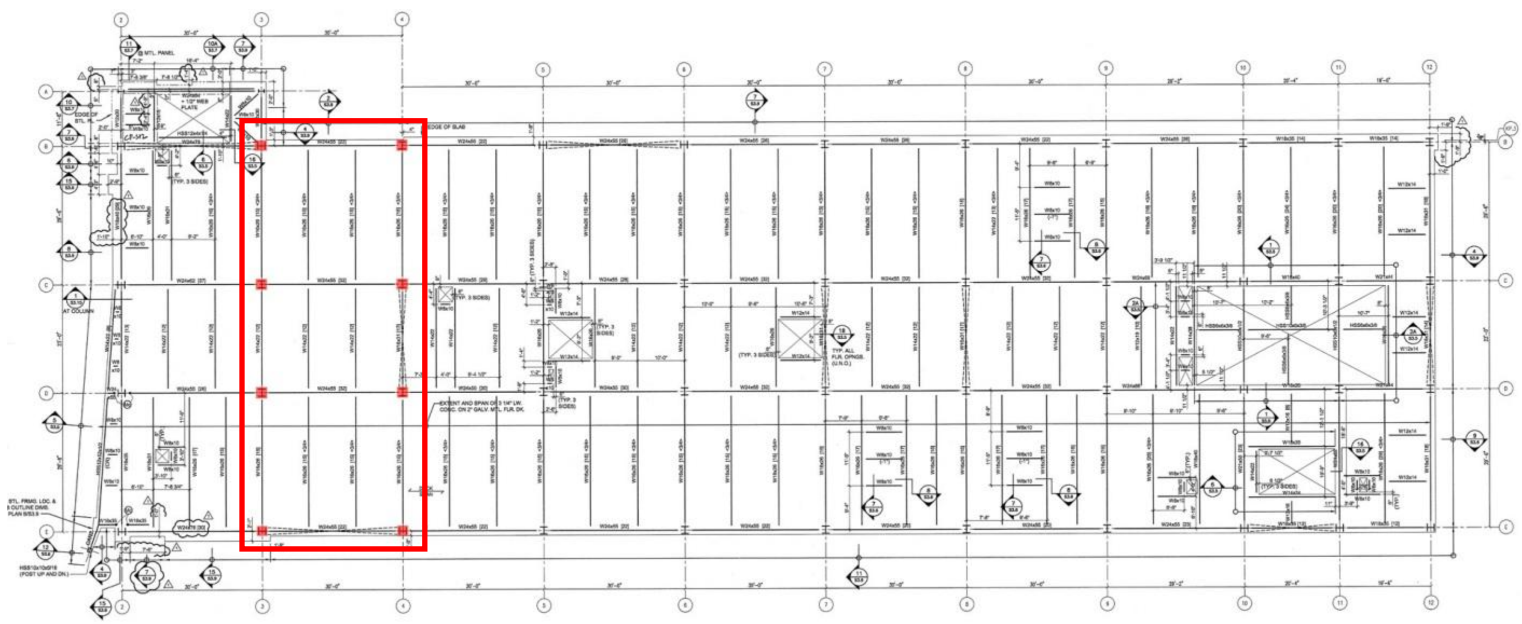


Figure 23 : Typical floor framing plan (typ bay shown)

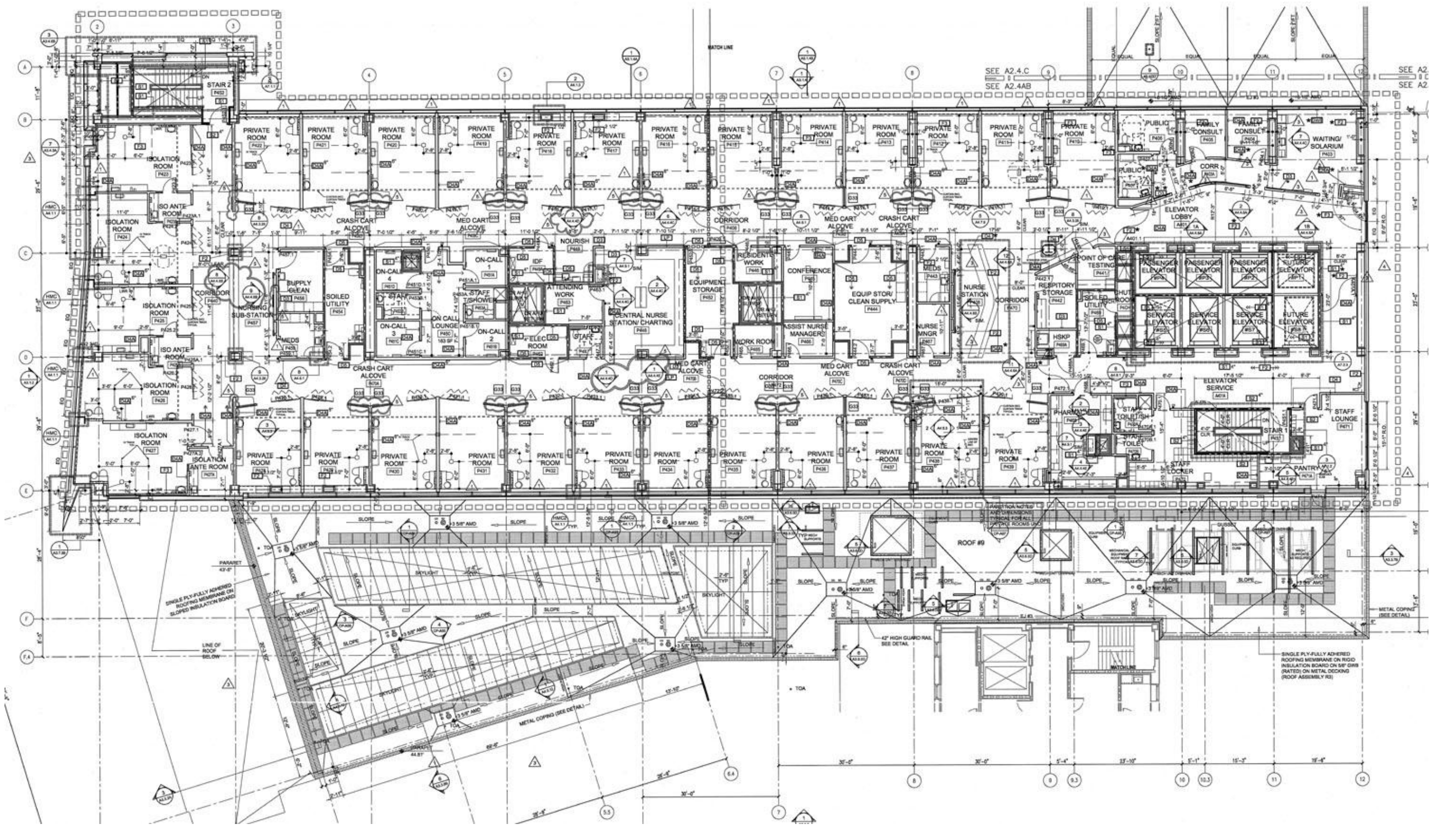


Figure 24: Ground Floor Architectural Plan

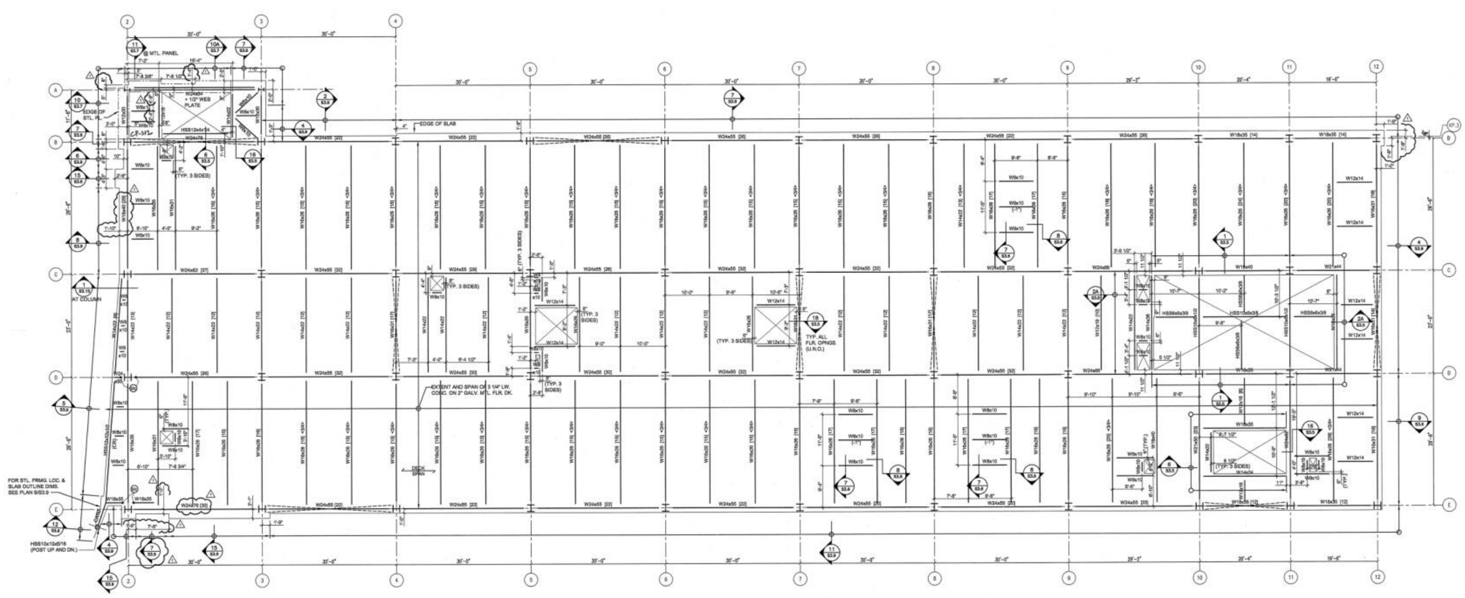


Figure 25: Typical Floor Framing Plan

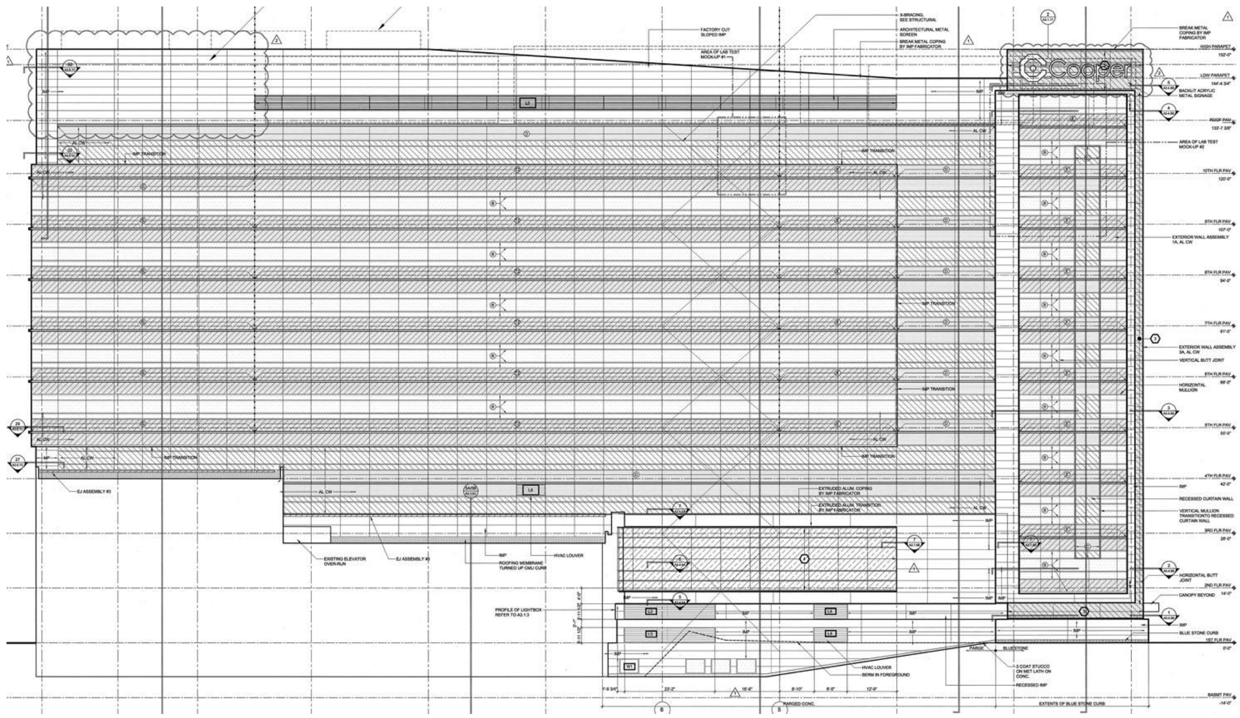


Figure 26: East Elevation

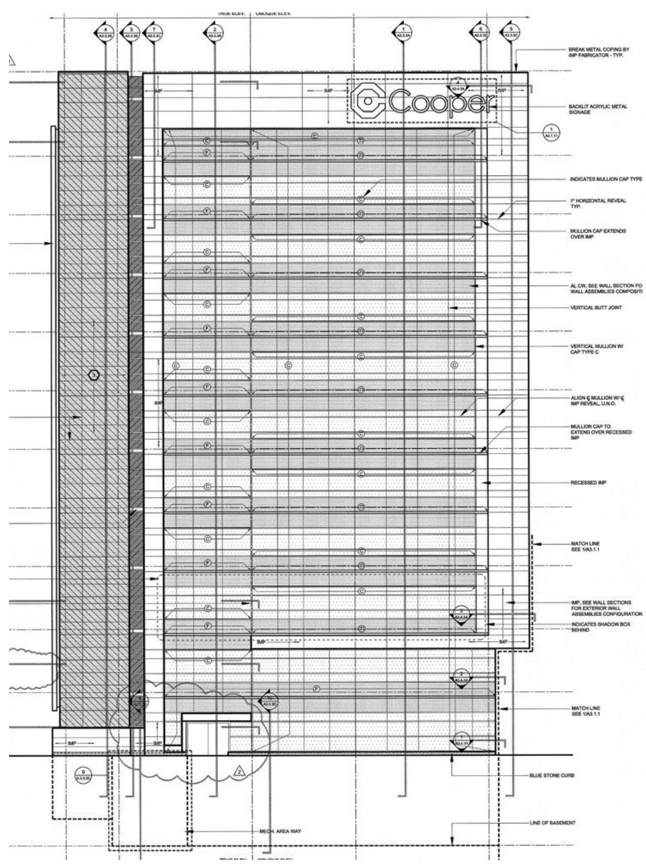


Figure 27: North Elevation

APPENDIX B: WIND DESIGN VALUES

Wind Loads | Tech I Report | Andrew Voorhees | 1

Wind Load Parameters • ASCE 7-05

$V = 90$ mph — Location: Camden, NJ (Fig 6-1)
$I = 1.15$ — Occupancy Category IV (Table 6-1) (Table 1-1)
Exposure C (§6.5.6.2)
$K_d = 0.85$ — Building MWRFS (Table 6-4)
$K_{zt} = 1.0$ — Homogeneous Topography

G = see below
$GC_{pi} = \pm 0.18$ — Enclosed Bldg (Fig. 6-5)

$K_z = 1.38$ @ 152' $\quad \dfrac{152-140}{160-140} = \dfrac{K_z - 1.36}{1.39 - 1.36}$ (Table 6-3)

Velocity Pressures

Building Height	K_z	q_z (psf)
0-15	0.85	17.23
20	0.90	18.24
25	0.94	19.05
30	0.98	19.86
40	1.04	21.08
50	1.09	22.09
60	1.13	22.90
70	1.17	23.72
80	1.21	24.53
90	1.24	25.13
100	1.26	25.54
120	1.31	26.55
140	1.36	27.57
152	1.38	27.97

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

@ 152': $q_z = 0.00256(1.38)(1.0)(0.85)(90)^2(1.15) = 27.97$ psf

Calculating G

Fundamental frequency $\rightarrow$ $n_1 = 100/H = 100/152 = 0.658$
$n_1 = 75/H = 75/152 = 0.493$

$n_1 = 0.493 < 1$ — * More conservative to use lower bound

∴ Consider Building as Flexible (§6.5.8.2)

Wind Loads | Tech Report | Andrew Voorhees | 2

$g_Q = g_v = 3.4$

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

$$= \sqrt{2\ln(3600 \times 0.493)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.493)}} = \boxed{4.0174}$$

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

$c = 0.2$ (Table 6-2)

$\bar{z} = 0.6h = 0.6(152) = 91.2$

$$= 0.2\left(\frac{33}{91.2}\right)^{1/6} = \boxed{0.1688}$$

Determine Q:

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

$\bar{\epsilon} = 1/5$ (Table 6-2)

$\ell = 500$ (Table 6-2)

$$= 500\left(\frac{91.2}{33}\right)^{1/5} = 612.73$$

Q: N-S

$B = 86'$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{86+152}{612.73}\right)^{0.63}}} = \boxed{0.8616}$$

Q: E-W

$B = 285'$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{285+152}{612.73}\right)^{0.63}}} = \boxed{0.8140}$$

Wind Loads | Tech 1 Report | Andrew Voorhees | 3

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

$\bar{\alpha} = 1/6.5$ (Table 6-2)
$\bar{b} = 0.65$ (Table 6-2)
$V = 90$ mph

$$= 0.65\left(\frac{91.2}{33}\right)^{1/6.5} 90\left(\frac{88}{60}\right)$$

$$= \boxed{100.32 \text{ ft/s}}$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.493 \times 612.73}{100.32} = \boxed{3.01}$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(3.01)}{[1 + 10.3(3.01)]^{5/3}} = \boxed{0.0697}$$

$$\eta_h = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.493)(152)}{100.32} = 3.436$$

$$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h}) = \frac{1}{3.436} - \frac{1}{2(3.436^2)}(1 - e^{-2(3.436)})$$

$$= \boxed{0.2487}$$

$$\eta_B = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}}$$

$B = 86'$ N-S $\Rightarrow \frac{4.6(0.493)(86')}{100.32} = 1.944$ N-S

$B = 285'$ E-W $\Rightarrow \frac{4.6(0.493)(285)}{100.32} = 6.443$ E-W

$$R_B = \frac{1}{\eta_B} - \frac{1}{2\eta_B^2}(1 - e^{-2\eta_B}) = \frac{1}{1.944} - \frac{1}{2(1.944^2)}(1 - e^{-2(1.944)}) = \boxed{0.3848 \text{ N-S}}$$

$$= \frac{1}{6.443} - \frac{1}{2(6.443^2)}(1 - e^{-2(6.443)}) = \boxed{0.1432 \text{ E-W}}$$

$$\eta_L = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}}$$

$L = 285'$ N-S $\Rightarrow \frac{15.4(0.493)(285)}{100.32} = 21.569$ N-S

$L = 86'$ E-W $\Rightarrow \frac{15.4(0.493)(86)}{100.32} = 6.508$ E-W

$$R_L = \frac{1}{\eta_L} - \frac{1}{2\eta_L^2}(1 - e^{-2\eta_L}) = \frac{1}{21.569} - \frac{1}{2(21.569^2)}(1 - e^{-2(21.569)}) = \boxed{0.0453 \text{ N-S}}$$

$$= \frac{1}{6.508} - \frac{1}{2(6.508^2)}(1 - e^{-2(6.508)}) = \boxed{0.1419 \text{ E-W}}$$

Wind Loads | Tech 1 Report | Andrew Voorhees | 4

Assume $\beta = 0.01$ (§ C6.5.8)

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$

N-S: $R = \sqrt{\frac{1}{0.01}(0.0697)(0.2487)(0.3848)(0.53 + 0.47 \times 0.0453)}$

$\boxed{= 0.6664 \text{ N-S}}$

E-W: $R = \sqrt{\frac{1}{0.01}(0.0697)(0.2487)(0.1432)(0.53 + 0.47 \times 0.1419)}$

$\boxed{= 0.3849 \text{ E-W}}$

Determine G_f:

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

N-S: $G_f = 0.925 \left[\frac{1 + 1.7(0.1688)\sqrt{3.4^2(0.8616^2) + 4.0174^2(0.6664^2)}}{1 + 1.7(3.4)(0.1688)} \right]$

$\boxed{G_f = 0.98 \text{ N-S}}$

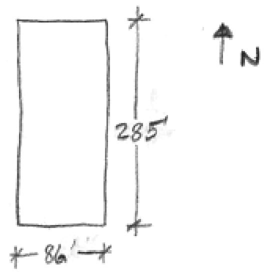
E-W: $G_f = 0.925 \left[\frac{1 + 1.7(0.1688)\sqrt{3.4^2(0.8140^2) + 4.0174^2(0.3849^2)}}{1 + 1.7(3.4)(0.1688)} \right]$

$\boxed{G_f = 0.89 \text{ E-W}}$

Wind Loads | Tech 1 Report | Andrew Voorhees | 5

Design Wind Pressures for MWRFS

Simplified Building Shape



C_p : Walls

N-S

Windward wall pressure coeff. = 0.8
Leeward wall pressure coeff. :

$L/B = 279/82 = 3.4$

$\frac{3.4-2}{4-2} = \frac{C_p - (-0.3)}{-0.2-(-0.3)} \Rightarrow C_p = -0.23$

E-W

Windward wall pressure coeff. = 0.8
Leeward wall pressure coeff. :

$L/B = 82/279 = 0.29$

$0 < 0.29 < 1 \Rightarrow C_p = -0.5$

C_p : Roof

$0 = \theta < 10°$

$h/L = 152/279 = 0.54 \Rightarrow$

$0 - h/2$	$C_p = -0.9$
$h/2 - h$	$C_p = -0.9$
$h - 2h$	$C_p = -0.5$
$> 2h$	$C_p = -0.3$

$152/82 = 1.85$

$\Downarrow$

$0 - h/2$	$C_p = -1.3$
$> h/2$	$C_p = -0.7$

see Excel tables for calculations

APPENDIX C: SEISMIC DESIGN VALUES

Seismic | Tech Report 3 | Andrew Voorhees | 1

Design Values

$R = 3$
$I = 1.5$
use group III
Design Category C
site Class D

Taken from USGS Website
$S_S = 0.267$
$S_1 = 0.059$
$S_{DS} = 0.282$
$S_{D1} = 0.095$
$T_L = 6$ sec.

Equivalent Lateral Force Method

$T_a = C_T h_N^x$

From table 12.8-2
$C_T = 0.02$
$x = 0.75$

$h_n = 133$ ft

$T_a = 0.02(133)^{0.75}$

$= 0.783$ sec ←

Fundamental Period:

$T = T_a C_u$

from table 12.8-1

$S_{DS} \le 0.1 \therefore C_u = 1.7$

$= 0.783(1.7)$

$= 1.33$ sec ← shall not exceed

Seismic Response Coefficient:

$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.282}{(3/1.5)} = 0.141$

*Code permits use of T_a when calculating C_s; $0.783 < 1.33$ more conservative

$T < T_L \therefore C_s \le \frac{S_{D1}}{T(R/I)} = \frac{0.095}{0.783(3/1.5)} = 0.0606 < 0.141$

↑ controls

$C_s \ge 0.044 S_{DS} I = 0.044(0.282)(1.5) = 0.0186 < 0.0357$ ✓ ok

$\boxed{\therefore C_s = 0.0606}$

Seismic | Tech Report 3 | Andrew Voorhees | 2

Base Shear

$V_b = C_s W$

$W = 27{,}136$ k   see excel Table for Calculations

$C_s = 0.0606$

$V_b = 0.0606(27{,}136) = 1644$ k   compared to 1300^k as designed

This difference in base shear is due to changes in the code

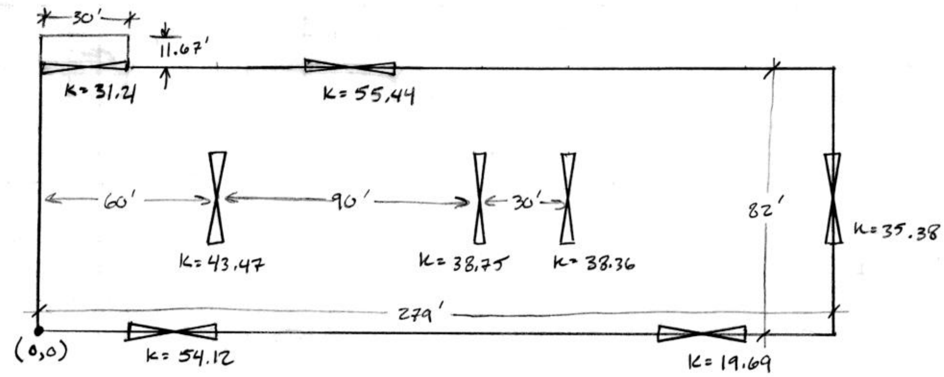
As designed ASCE 7-02	Per this Report ASCE 7-05
$R = 5$	$R = 3\frac{1}{4} \rightarrow$ use 3
$\Omega = 2$	$\Omega = 2$
$C_d = 4\frac{1}{2}$	$C_d = 3\frac{1}{4}$
$S_s = 0.3210$	$S_s = 0.267$
$S_1 = 0.08$	$S_1 = 0.059$
$S_{DS} = 0.3296$	$S_{DS} = 0.282$
$S_{D1} = 0.1280$	$S_{D1} = 0.095$

The change in design values S_{DS} and S_{D1} as well as the R value, impacted the C_s value and consequently the base shear.

APPENDIX D: CENTER OF MASS & RIGIDITY VERIFICATION

Center of Mass / Rigidity | Tech Report 3 | Andrew Voorhees | 1

Level 6 Typical upper floor



Center of Mass

$$\bar{x} = \frac{(30' \times 11.67')(15') + (279' \times 82')(139.5')}{(30' \times 11.67') + (279' \times 82')} = \boxed{137.62'}$$

Compare to ETABS Value $\bar{x} = 137.313'$ ✓ ok

$$\bar{y} = \frac{(30' \times 11.67')(87.835') + (279' \times 82')(41')}{(30' \times 11.67') + (279' \times 82')} = \boxed{41.71'}$$

Compare to ETABS Value $\bar{y} = 41.743'$ ✓ ok

Center of Rigidity

$$x_r = \frac{60'(43.47 \text{ k/in}) + 150'(38.75) + 180'(38.36) + 279'(35.38)}{43.47 + 38.75 + 38.36 + 35.38}$$

$$= \boxed{161.56'}$$

Compare to ETABS Value $x_r = 161.205'$ ✓ ok

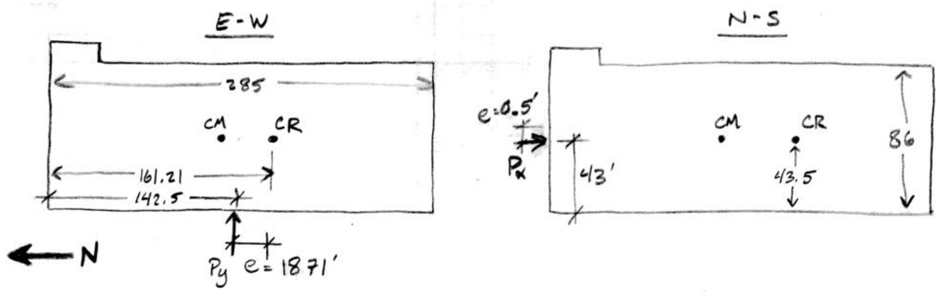
$$y_r = \frac{31.21(82') + 55.44(82')}{31.21 + 55.44 + 54.12 + 19.69} = \boxed{44.28'}$$

Compare to ETABS Value $y_r = 43.503'$ ✓ ok

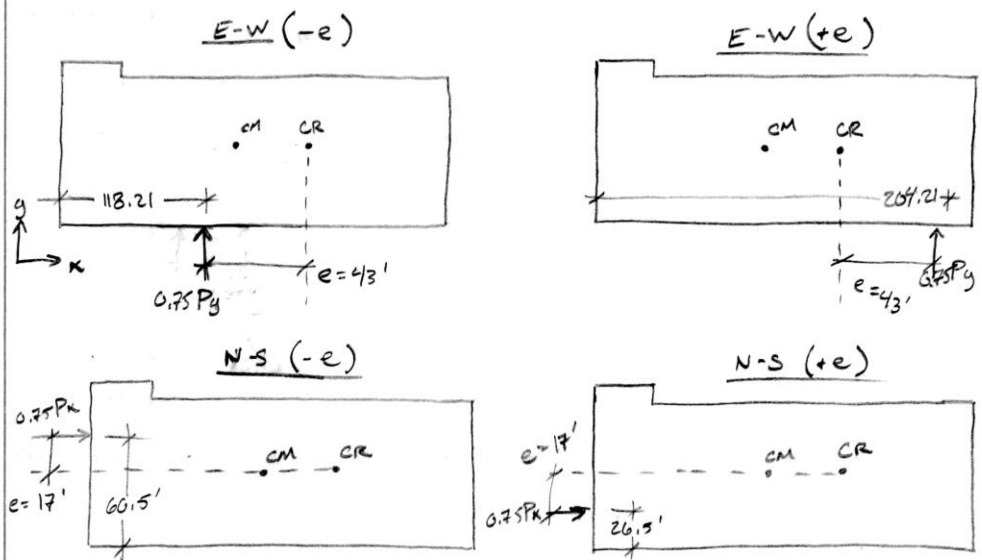
APPENDIX E: WIND LOAD CASES

Wind Load Cases | Tech Report 3 | Andrew Voorhees | 1

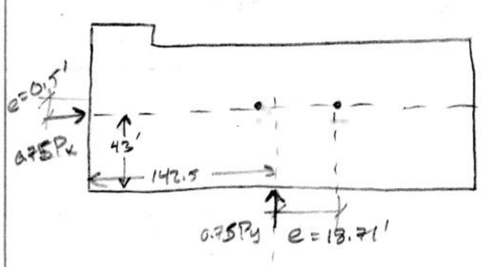
Case 1 : Roof Level



Case 2 : Roof Level

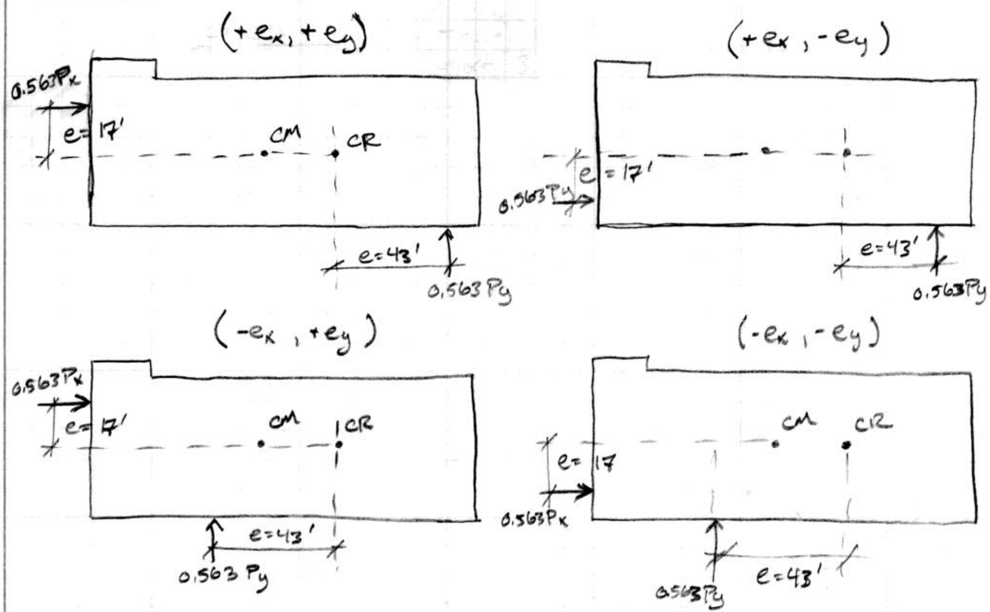


Case 3 : Roof Level



Wind Load Cases | Tech Report 3 | Andrew Voorhees | 2

Case 4: Roof Level



APPENDIX F: WIND LOAD DEFLECTIONS

Case 1

Level	P_x				P_y				Δ_{allow}
	δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
Roof	0.6434	-0.0013	0.0717	-0.0006	0.3462	3.8517	0.0405	0.4400	0.39
10	0.5717	-0.0007	0.0746	-0.0007	0.3057	3.4117	0.0420	0.4564	0.39
9	0.4971	0.0000	0.0756	-0.0015	0.2637	2.9553	0.0416	0.4594	0.39
8	0.4215	0.0015	0.0709	-0.0012	0.2221	2.4959	0.0367	0.4246	0.39
7	0.3506	0.0027	0.0725	0.0000	0.1854	2.0713	0.0347	0.4175	0.39
6	0.2781	0.0027	0.0652	0.0032	0.1507	1.6538	0.0326	0.3782	0.39
5	0.2129	-0.0005	0.0631	0.0012	0.1181	1.2756	0.0276	0.3571	0.39
4	0.1498	-0.0017	0.0577	0.0011	0.0905	0.9185	0.0252	0.3202	0.42
3	0.0921	-0.0028	0.0563	0.0002	0.0653	0.5983	0.0315	0.3208	0.42
2	0.0358	-0.0030	0.0358	-0.0030	0.0338	0.2775	0.0338	0.2775	0.42
Ground	0	0	0	0	0	0	0	0	0

Case 2

Level	$0.75\ P_x\ (-e_x)$				$0.75\ P_x\ (+e_x)$				Δ_{allow}
	δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
Roof	0.0412	0.1388	0.0047	0.0160	-0.0412	-0.1388	-0.0047	-0.0160	0.39
10	0.0365	0.1228	0.0048	0.0166	-0.0365	-0.1228	-0.0048	-0.0166	0.39
9	0.0317	0.1062	0.0047	0.0166	-0.0317	-0.1062	-0.0047	-0.0166	0.39
8	0.0270	0.0896	0.0045	0.0154	-0.0270	-0.0896	-0.0045	-0.0154	0.39
7	0.0225	0.0742	0.0045	0.0153	-0.0225	-0.0742	-0.0045	-0.0153	0.39
6	0.0180	0.0589	0.0044	0.0137	-0.0180	-0.0589	-0.0044	-0.0137	0.39
5	0.0136	0.0452	0.0041	0.0133	-0.0136	-0.0452	-0.0041	-0.0133	0.39
4	0.0095	0.0319	0.0038	0.0120	-0.0095	-0.0319	-0.0038	-0.0120	0.42
3	0.0057	0.0199	0.0035	0.0115	-0.0057	-0.0199	-0.0035	-0.0115	0.42
2	0.0022	0.0084	0.0022	0.0084	-0.0022	-0.0084	-0.0022	-0.0084	0.42
Ground	0	0	0	0	0	0	0	0	0

Level	$0.75\ P_y\ (-e_y)$				$0.75\ P_y\ (+e_y)$				Δ_{allow}
	δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
Roof	-0.5179	-1.7426	-0.0591	-0.2020	0.5179	1.7426	0.0591	0.2020	0.39
10	-0.4588	-1.5406	-0.0606	-0.2082	0.4588	1.5406	0.0606	0.2082	0.39
9	-0.3982	-1.3324	-0.0601	-0.2098	0.3982	1.3324	0.0601	0.2098	0.39
8	-0.3381	-1.1226	-0.0562	-0.1939	0.3381	1.1226	0.0562	0.1939	0.39
7	-0.2819	-0.9287	-0.0577	-0.1937	0.2819	0.9287	0.0577	0.1937	0.39
6	-0.2242	-0.7350	-0.0544	-0.1730	0.2242	0.7350	0.0544	0.1730	0.39
5	-0.1698	-0.5620	-0.0520	-0.1681	0.1698	0.5620	0.0520	0.1681	0.39
4	-0.1178	-0.3939	-0.0473	-0.1494	0.1178	0.3939	0.0473	0.1494	0.42
3	-0.0705	-0.2445	-0.0439	-0.1421	0.0705	0.2445	0.0439	0.1421	0.42
2	-0.0266	-0.1024	-0.0266	-0.1024	0.0266	0.1024	0.0266	0.1024	0.42
Ground	0	0	0	0	0	0	0	0	0

Case	Level	0.75 P_x + 0.75 P_y				0.75 P_x - 0.75 P_y				Δ_{allow}
		δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
Case 3	Roof	0.7423	2.8878	0.0843	0.3296	0.2229	-2.8897	0.0234	-0.3305	0.39
	10	0.6580	2.5582	0.0874	0.3418	0.1995	-2.5592	0.0244	-0.3428	0.39
	9	0.5706	2.2164	0.0879	0.3434	0.1751	-2.2164	0.0255	-0.3456	0.39
	8	0.4827	1.8730	0.0807	0.3175	0.1496	-1.8708	0.0257	-0.3193	0.39
	7	0.4020	1.5555	0.0804	0.3132	0.1239	-1.5515	0.0284	-0.3132	0.39
	6	0.3216	1.2423	0.0733	0.2859	0.0955	-1.2383	0.0244	-0.2812	0.39
	5	0.2483	0.9564	0.0680	0.2688	0.0711	-0.9571	0.0266	-0.2669	0.39
	4	0.1803	0.6876	0.0622	0.2410	0.0445	-0.6902	0.0244	-0.2394	0.42
	3	0.1181	0.4466	0.0659	0.2408	0.0201	-0.4508	0.0186	-0.2404	0.42
	2	0.0522	0.2058	0.0522	0.2058	0.0015	-0.2104	0.0015	-0.2104	0.42
	Ground	0	0	0	0	0	0	0	0	0

Case	Level	0.563 P_x (-e_x) + 0.563 P_y (-e_y)				0.563 P_x (-e_x) + 0.563 P_y (+e_y)				Δ_{allow}
		δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
Case 4	Roof	0.3578	1.2040	0.0408	0.1397	0.4197	1.4123	0.0478	0.1637	0.39
	10	0.3170	1.0643	0.0419	0.1439	0.3719	1.2486	0.0491	0.1687	0.39
	9	0.2751	0.9204	0.0415	0.1450	0.3228	1.0799	0.0488	0.1700	0.39
	8	0.2336	0.7754	0.0389	0.1340	0.2740	0.9099	0.0455	0.1571	0.39
	7	0.1947	0.6414	0.0399	0.1338	0.2285	0.7528	0.0467	0.1568	0.39
	6	0.1548	0.5076	0.0376	0.1196	0.1818	0.5960	0.0441	0.1402	0.39
	5	0.1172	0.3880	0.0359	0.1162	0.1377	0.4558	0.0421	0.1362	0.39
	4	0.0813	0.2718	0.0327	0.1032	0.0956	0.3196	0.0384	0.1211	0.42
	3	0.0486	0.1686	0.0303	0.0980	0.0572	0.1985	0.0356	0.1153	0.42
	2	0.0183	0.0706	0.0183	0.0706	0.0216	0.0832	0.0216	0.0832	0.42
	Ground	0	0	0	0	0	0	0	0	0
	Level	0.563 P_x (+e_x) + 0.563 P_y (-e_y)				0.563 P_x (+e_x) + 0.563 P_y (+e_y)				Δ_{allow}
		δ_x	δ_y	Δ_x	Δ_y	δ_x	δ_y	Δ_x	Δ_y	
	Roof	-0.4197	-1.4123	-0.0478	-0.1637	-0.3578	-1.2040	-0.0408	-0.1397	0.39
	10	-0.3719	-1.2486	-0.0491	-0.1687	-0.3170	-1.0643	-0.0419	-0.1439	0.39
	9	-0.3228	-1.0799	-0.0488	-0.1700	-0.2751	-0.9204	-0.0415	-0.1450	0.39
	8	-0.2740	-0.9099	-0.0455	-0.1571	-0.2336	-0.7754	-0.0389	-0.1340	0.39
	7	-0.2285	-0.7528	-0.0467	-0.1568	-0.1947	-0.6414	-0.0399	-0.1338	0.39
	6	-0.1818	-0.5960	-0.0441	-0.1402	-0.1548	-0.5076	-0.0376	-0.1196	0.39
	5	-0.1377	-0.4558	-0.0421	-0.1362	-0.1172	-0.3880	-0.0359	-0.1162	0.39
	4	-0.0956	-0.3196	-0.0384	-0.1211	-0.0813	-0.2718	-0.0327	-0.1032	0.42
	3	-0.0572	-0.1985	-0.0356	-0.1153	-0.0486	-0.1686	-0.0303	-0.0980	0.42
	2	-0.0216	-0.0832	-0.0216	-0.0832	-0.0183	-0.0706	-0.0183	-0.0706	0.42
	Ground	0	0	0	0	0	0	0	0	0

APPENDIX G: HORIZONTAL IRREGULARITIES

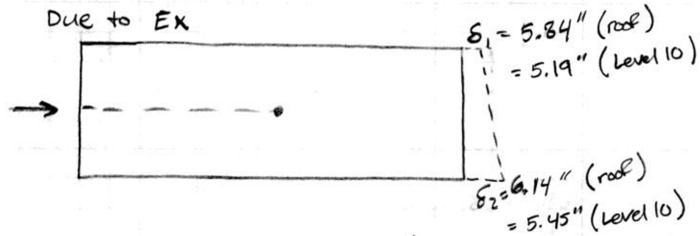
Check For Horizontal Irregularities (Table 12.3-1)

Torsional Irregularity Type 1a

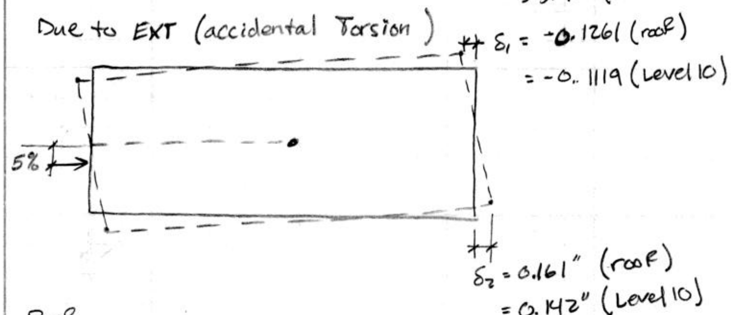
$$\Delta_{max} > 1.2\left(\frac{\Delta_1 + \Delta_2}{2}\right)$$

Example Calculation at Roof Level

Due to Ex



Due to EXT (accidental Torsion)



Roof

$\delta_{xe1} = 5.84'' - 0.1261'' = 5.71''$

$\delta_{xe2} = 6.14'' + 0.161'' = 6.30''$

Level 10

$\delta_{xe1} = 5.19'' - 0.1119'' = 5.08''$

$\delta_{xe2} = 5.45'' + 0.142'' = 5.59''$

Roof Drift : $\Delta_1 = \delta_{roof} - \delta_{10} = 5.71'' - 5.08'' = 0.63''$

$\Delta_2 = 6.30 - 5.59 = 0.71''$ ← max Δ @ roof Level

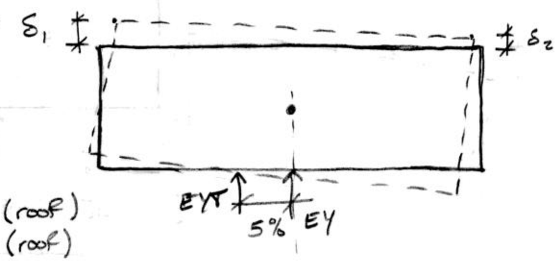
$$\Delta_{avg} = \frac{0.63'' + 0.71''}{2} = 0.67$$

$0.71 < 1.2(0.67) = 0.804$ ∴ No Torsional Irregularity

* See spreadsheet for calculations per each floor and direction

Horiz Irregularities | Tech Report 3 | Andrew Voorhees | 2

Due to EY + EYT (-e)



δ_1 = 11.0807 (roof)
δ_2 = 2.7537 (roof)

δ_1 = 9.8109 (level 10)
δ_2 = 2.4585 (Level 10)

Roof Drift: $\Delta_1 = \delta_{roof} - \delta_{10} = 11.0807 - 9.8109 = 1.27''$

$\Delta_2 = 2.7537 - 2.4585 = 0.30''$

max at roof level

$\Delta_{avg} = \dfrac{1.27 + 0.3}{2} = 0.78$

$1.27 > 1.2(0.78'') = 0.942''$

∴ Torsional Irregularity exists (extreme : > 1.4 Δ_{avg})

<u>Calculate A_x</u>

$A_x = \left[\dfrac{\delta_{max}}{1.2\,\delta_{avg}}\right]^2$ $\qquad \delta_{avg} = \dfrac{11.0807 + 2.7537}{2} = 6.92''$

$= \left[\dfrac{11.0807}{1.2\,(6.92'')}\right]^2 = 1.78$ at roof level

* See spreadsheet for calculations for each floor and direction

APPENDIX H: TORSIONAL AMPLIFICATION FACTORS

Level	X-Direction (-e)							X-Direction (+e)							Δ_{allow}
	δ_{xe1}	Δ_1	δ_{xe2}	Δ_2	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	A_x	δ_{xe1}	Δ_1	δ_{xe2}	Δ_2	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	A_x	
Roof	5.5493	0.61	6.1152	0.69	0.65	1.05	1.00	5.7977	0.64	5.8007	0.65	0.64	1.00	1.00	1.2
10	4.9356	0.66	5.4302	0.73	0.70	1.05	1.00	5.1559	0.69	5.1526	0.69	0.69	1.00	1.00	1.2
9	4.2774	0.68	4.6978	0.76	0.72	1.06	1.00	4.4681	0.71	4.4592	0.72	0.71	1.01	1.00	1.2
8	3.5999	0.64	3.9414	0.71	0.67	1.05	1.00	3.761	0.66	3.7428	0.67	0.67	1.01	1.00	1.2
7	2.9634	0.65	3.2326	0.71	0.68	1.05	1.00	3.0968	0.67	3.0712	0.68	0.68	1.00	1.00	1.2
6	2.3177	0.57	2.52	0.61	0.59	1.04	1.00	2.4226	0.59	2.3953	0.58	0.59	1.01	1.00	1.2
5	1.7505	0.54	1.9096	0.59	0.57	1.04	1.00	1.8286	0.57	1.8154	0.56	0.56	1.00	1.00	1.2
4	1.2092	0.48	1.3173	0.52	0.50	1.04	1.00	1.2623	0.50	1.2535	0.49	0.50	1.01	1.00	1.2
3	0.7295	0.45	0.7983	0.49	0.47	1.04	1.00	0.7604	0.47	0.7601	0.47	0.47	1.01	1.00	1.2
2	0.2751	0.28	0.3089	0.31	0.29	1.06	1.00	0.286	0.29	0.2935	0.29	0.29	1.01	1.00	1.2

Level	Y-Direction (+e)							Y-Direction (-e)							Δ_{allow}
	δ_{xe1}	Δ_1	δ_{xe2}	Δ_2	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	A_x	δ_{xe1}	Δ_1	δ_{xe2}	Δ_2	Δ_{avg}	$\Delta_{max}/\Delta_{avg}$	A_x	
Roof	7.4434	0.85	5.4269	0.60	0.73	1.17	1.00	11.0807	1.27	2.7537	0.30	0.78	1.62	1.78	1.2
10	6.5954	0.90	4.8236	0.64	0.77	1.17	1.00	9.8109	1.35	2.4585	0.31	0.83	1.62	1.78	1.2
9	5.6906	0.92	4.186	0.65	0.78	1.17	1.00	8.4598	1.37	2.1454	0.32	0.85	1.63	1.77	1.2
8	4.771	0.85	3.5393	0.63	0.74	1.15	1.00	7.0863	1.27	1.8286	0.32	0.79	1.60	1.76	1.2
7	3.9213	0.83	2.9129	0.64	0.74	1.12	1.00	5.8168	1.24	1.5128	0.33	0.79	1.58	1.75	1.2
6	3.0937	0.74	2.269	0.58	0.66	1.12	1.00	4.5726	1.11	1.1843	0.30	0.70	1.57	1.75	1.2
5	2.3534	0.68	1.689	0.57	0.63	1.09	1.00	3.4658	1.03	0.8817	0.30	0.67	1.55	1.77	1.2
4	1.6692	0.59	1.1178	0.50	0.54	1.09	1.00	2.4314	0.89	0.5828	0.26	0.58	1.55	1.81	1.2
3	1.0755	0.57	0.6225	0.40	0.49	1.17	1.00	1.5377	0.85	0.32	0.21	0.53	1.61	1.90	1.2
2	0.5015	0.50	0.2177	0.22	0.36	1.39	1.35	0.6896	0.69	0.1143	0.11	0.40	1.72	2.04	1.2

APPENDIX I: SEISMIC DEFLECTIONS

Drift N-S (x - direction)										
	E_X			$E_X + E_{XT}$ $(-e_y)$			$E_X - E_{XT}$ $(-e_y)$			
Level	δ_{xe}	$C_d\delta_{xe}/I$	Δ_x	δ_{xe}	$C_d\delta_{xe}/I$	Δ_x	δ_{xe}	$C_d\delta_{xe}/I$	Δ_x	Δ_{allow}
Roof	5.813	12.595	1.401	5.827	12.626	1.406	5.799	12.565	1.397	1.56
10	5.166	11.194	1.501	5.179	11.220	1.505	5.154	11.168	1.496	1.56
9	4.474	9.693	1.547	4.484	9.715	1.552	4.464	9.671	1.542	1.56
8	3.760	8.146	1.451	3.768	8.163	1.456	3.752	8.130	1.447	1.56
7	3.090	6.695	1.466	3.096	6.707	1.470	3.084	6.682	1.463	1.56
6	2.413	5.228	1.273	2.417	5.237	1.275	2.409	5.220	1.272	1.56
5	1.825	3.955	1.205	1.829	3.962	1.184	1.822	3.948	1.226	1.56
4	1.269	2.750	1.090	1.282	2.778	1.105	1.256	2.722	1.075	1.68
3	0.766	1.660	1.026	0.772	1.672	1.034	0.760	1.647	1.018	1.68
2	0.293	0.634	0.634	0.295	0.639	0.639	0.290	0.629	0.629	1.68
Ground	0	0	0	0	0	0	0	0	0	0

Drift E-W (y - direction)										
	E_Y			$E_Y + E_{YT}$ $(+e_x)$			$E_Y - E_{YT}$ $(-e_x)$			
Level	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	δ_{ye}	$C_d\delta_{ye}/I$	Δ_y	Δ_{allow}
Roof	6.714	14.546	**1.642**	6.450	13.974	**1.576**	7.182	15.560	**1.760**	1.56
10	5.956	12.904	**1.747**	5.722	12.399	**1.675**	6.369	13.800	**1.875**	1.56
9	5.149	11.157	**1.773**	4.949	10.723	**1.701**	5.504	11.924	**1.902**	1.56
8	4.331	9.384	**1.667**	4.164	9.023	**1.602**	4.626	10.023	**1.781**	1.56
7	3.562	7.717	**1.658**	3.425	7.420	**1.597**	3.804	8.242	**1.766**	1.56
6	2.796	6.059	1.487	2.688	5.823	1.433	2.989	6.476	**1.582**	1.56
5	2.110	4.572	1.418	2.026	4.390	1.366	2.259	4.894	1.510	1.56
4	1.456	3.154	1.236	1.396	3.025	1.190	1.562	3.384	1.319	1.68
3	0.885	1.918	1.119	0.847	1.835	1.073	0.953	2.065	1.200	1.68
2	0.369	0.800	0.800	0.352	0.762	0.762	0.399	0.865	0.865	1.68
Ground	0	0	0	0	0	0	0	0	0	0

APPENDIX J: OVERTURNING MOMENT CHECK

Overturning Moment | Tech Report 3 | Andrew Voorhees

$M_{resist} = W(\frac{1}{2}B)$ F.S.

B = least width of building

= 86'

$M_{resist} = 27{,}135.77^{k}\,(\frac{1}{2}\cdot 86')(\frac{2}{3}) = 777{,}892.07$ kft

Wind Load Overturning Moment

N-S = 153,169.17 kft

777,892.07 >> 153,169.17 kft

∴ Building weight resists overturning

Foundation <u>OK</u>

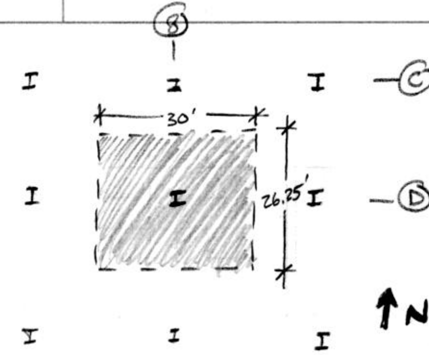
APPENDIX K: LATERAL MEMBER SPOT CHECKS

Member Spot Check | Tech Report 3 | Andrew Voorhees | 1

Frame 8-(D-C) : E-W direction

Column D8 Ground floor

Trib Area = 787.5 ft^2



Load Combos :

$1.2D + 1.6W + L + 0.5(L_r, S, R)$

$1.2D + 1.0E + L + 0.2S$

Roof

S = 94 psf

L_r = 150 psf (mech. equip load)

D = 110 psf

East-West direction controlled by wind

$\therefore\ 1.2D + 1.6W + L + 0.5(L_r, S, R)$

floor 10 : $P_u = [1.2(110) + 0.5(150)]\ 787.5 = 163.01^k$

floor 9 : $P_u = [1.2(96.2\text{ psf}) + 80]\ 787.5 = 153.91^k$

floor 8 : $P_u = [1.2(96.2) + 80]\ 787.5 = 153.91^k$

floor 7 : $P_u = [1.2(96.7) + 80]\ 787.5 = 154.38^k$

floor 6 : $P_u = [1.2(97.2) + 80]\ 787.5 = 154.85^k$

floor 5 : $P_u = [1.2(97.2) + 80]\ 787.5 = 154.85^k$

floor 4 : $P_u = [1.2(97.2) + 80]\ 787.5 = 154.85^k$

floor 3 : $P_u = [1.2(81.7) + 100]\ 787.5 = 155.9^k$

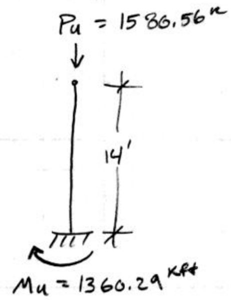
floor 2 : $P_u = [1.2(93.5) + 100]\ 787.5 = 167.1^k$

Ground floor : $P_u = [1.2(94.2) + 100]\ 787.5 = 167.8^k$

Total $P_u = 1580.56^k$

Total $M_u = 1360.29^{k\text{ft}}$ (see spreadsheet for calculation)

Member Spot Check | Tech Report 3 | Andrew Voorhees | 2



fixed - pinned :
$K = 0.7$

$KL = 0.7(14) = 9.8'$

Combined Axial + Flexure : Table 6-1

W 14 × 342

use $kL = 10'$ → conservative

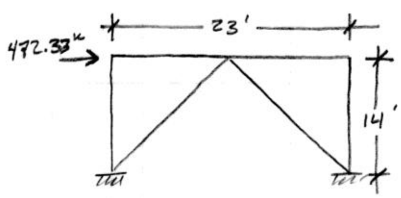
$p \times 10^3 = 0.236$ $\quad b_x \times 10^3 = 0.353$

$$pP_r = \frac{0.236\,(1580.56)}{1000} = 0.373 \geq 0.2$$

$$\rightarrow 0.373 + \frac{0.353\,(1360.29)}{1000} = 0.853 < 1.0$$

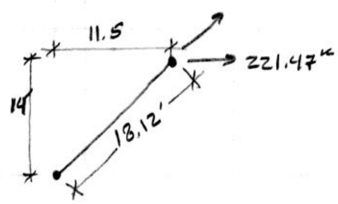
✓ ok - Column is adequate

Brace 8 (D-C)



Columns take 14.7^k each (x-dir)

∴ Each Brace takes 221.47^k



Axial force :

$$\frac{221.47^k}{11.5'} = \frac{x}{18.12}$$

Force $= 348.96^k$

Drawings detail brace for 350^k

HSS 10 × 10 × ½

Tension :
$\phi P_n = 561^k$

Compression
$\phi P_n = 577^k$

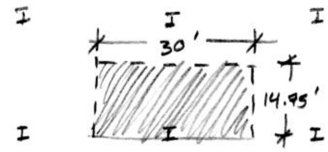
✓ ok the Brace is adequate

Member Spot Check	Tech Report 3	Andrew Voorhees	3

Frame E (3-4) : N-S Direction

Column E4 Ground Floor

Trib Area = 442.5 ft²



Load combos:

$1.2D + 1.6W + L + 0.5(L_r, S, R)$

$1.2D + 1.0E + L + 0.2S$

Roof

S = 94 psf

L_r = 150 psf (mech equip load)

D = 110 psf

N-S direction controlled by Earthquake

$\therefore 1.2D + 1.0E + L + 0.2S$

floor 10: $P_u = [1.2(110) + 0.2(94)]\,442.5 = 66.73^k$

floor 9: $P_u = [1.2(96.2) + 80]\,442.5 = 86.48^k$

floor 8: $P_u = [1.2(96.2) + 80]\,442.5 = 86.48^k$

floor 7: $P_u = [1.2(96.7) + 80]\,442.5 = 86.75^k$

floor 6: $P_u = [1.2(97.2) + 80]\,442.5 = 87.01^k$

floor 5: $P_u = [1.2(97.2) + 80]\,442.5 = 87.01^k$

floor 4: $P_u = [1.2(97.2) + 80]\,442.5 + 0.2(94)\,442.5 = 95.33$

floor 3: $P_u = [1.2(81.7) + 100]\,442.5 = 87.63^k$

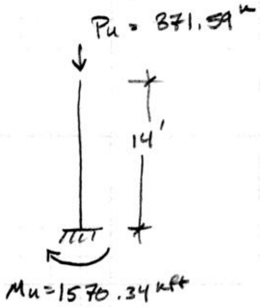
floor 2: $P_u = [1.2(93.5) + 100]\,442.5 = 93.90^k$

Ground Floor: $P_u = [1.2(94.2) + 100]\,442.5 = 94.27^k$

Total P_u = 871.59^k

Total M_u = 1570.34 kft (see spreadsheet for calculation)

Member Spot Checks | Tech Report 3 | Andrew Voorhees | 4



fixed-pinned
$k = 0.7$

$kl = 9.8'$

combined axial + flexure : Table 6-1

W 14 x 370

use $kl = 10'$ → conservative

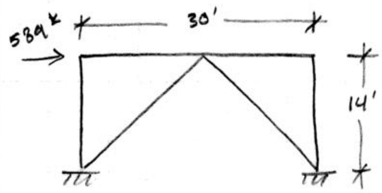
$p \times 10^3 = 0.219$ $\quad b_x \times 10^3 = 0.322$

$$pP_r = \frac{0.219\,(871.59)}{1000} = 0.191 < 0.2$$

$$\rightarrow \frac{1}{2}(0.191) + \frac{9}{8} \times \frac{(1570.34)}{1000}(0.322) = 0.66 < 1.0$$

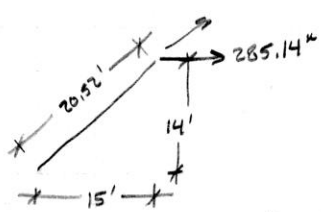
√ ok the column is adequate

Brace E (3-4)



Columns take 9.36^k each (x-dir)

∴ each brace takes 285.14^k



Axial Force :

$$\frac{285.14^k}{15'} = \frac{x}{20.52}$$

Force $= 390.07^k$

Drawings detail brace for 430^k

HSS 12 x 12 x 1/2

Tension : $\phi P_n = 683^k$

Compression : $\phi P_n = 750^k$

√ ok The brace is adequate