Technical Report I

Roberts Pavilion
Camden, NJ

Andrew Voorhees | Structural

Existing Conditions
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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 ordinary steel concentrically braced frames.

Purpose and Scope

The purpose of this report is to provide an analysis of the Roberts Pavilion and demonstrate an understanding of the structural systems. The scope of which will include an analysis of different structural elements, applicable codes, building materials, gravity loads, and wind and seismic analyses.

One of the main functions of this report is to provide a thorough description of the structural system of the building. This will include a description of the foundation; the slab on grade, bearing walls, piles, and piers. It will also discuss the typical floor framing, including decking and typical wide flange members such as beams and columns. Typical framing members were checked and verified that the calculated member was the in the same range as the designed member.

Lateral loads were also verified. Simple wind and seismic loads were calculated in accordance with ASCE 7-05. The analyses proved to result in values different from those specified on the drawings. The seismic base shear calculated was about 12% higher than the design shear. This was due to the fact that the pavilion was designed under ASCE 7-02. Seismic parameters changed between editions of the code. (A discussion on this can be found in the seismic section of this report) Being a summary of the existing conditions, wind and seismic loads were not the focus of this report, and thus calculations were simplified. More detailed lateral calculations will be performed in Technical Report III.

Hand calculations are included in the Appendices of this report for reference. Additional drawings and floor plans are included there as well for clarification.

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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.

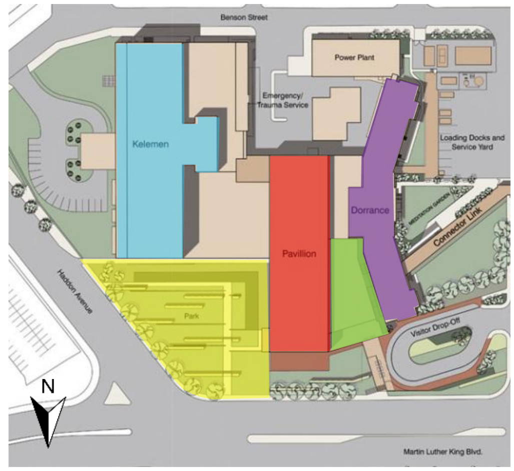


Figure 1 : Site plan (Courtesy of EwingCole)

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 puple 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.

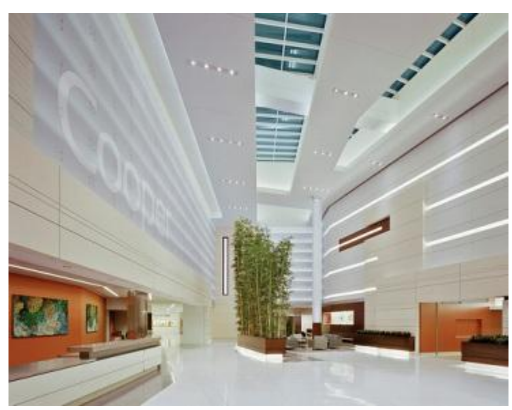


Figure 3 : Lobby (Courtesy of Eduard Hueber/Arch Photo, Inc.)

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.

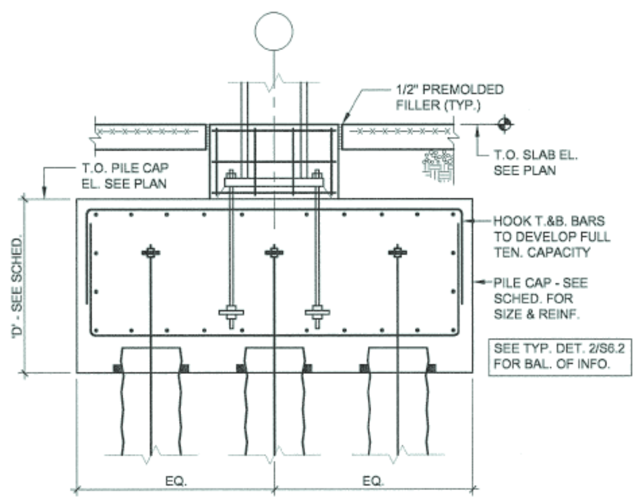


Figure 4 : Typical pile cap without pedestal

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”.

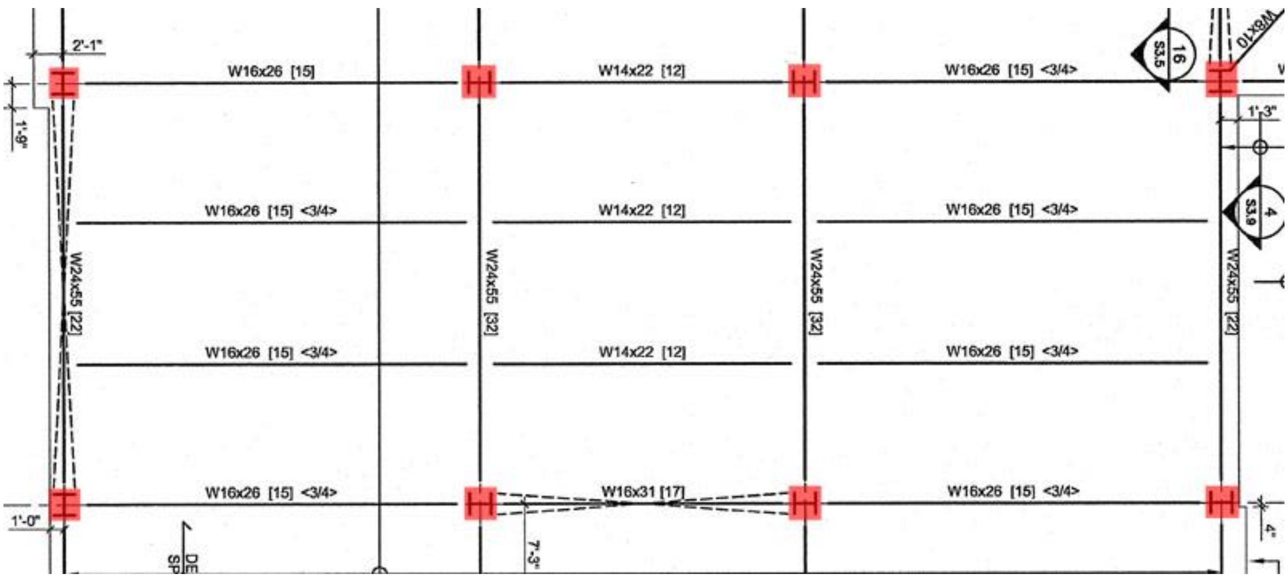


Figure 5 : Typical bay (See Appendix A for full framing plan)

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

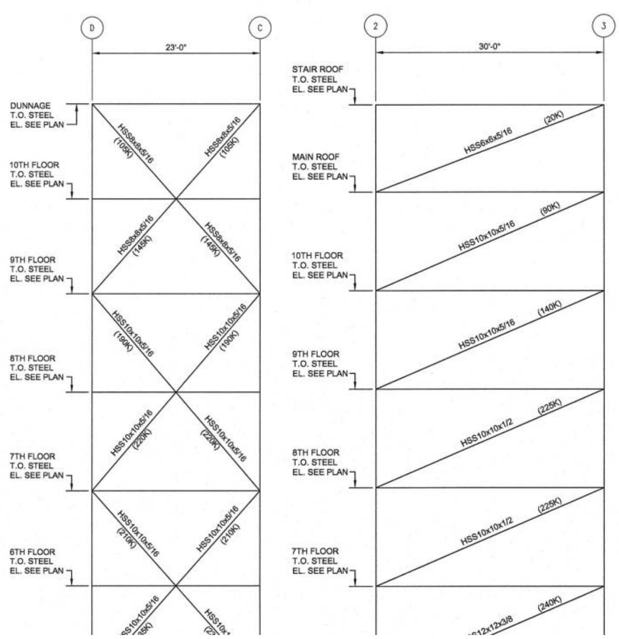


Figure 7 : Two typical braced frames (OSCBF)

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 ratings

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

Gravity Spot Checks

Three gravity load spot checks were completed for this report. The first being the composite decking system, the second: a typical beam, and the third: a typical interior column. Live and dead loads used in the calculations are shown on the previous page. **Additionally, detailed calculations are shown in Appendix C.**

Floor Decking

The first spot check was performed on a typical span of floor decking (See Figure 10). The specifications require a minimum of 2”, 18 gauge decking. Acceptable manufacturers included Vulcraft Division of Nucor Corp. and Wheeling Corrugating Co. Division of Wheeling-Pittsburgh Steel Corp. The calculations in this report were done using the Vulcraft catalog.

To determine the thickness of the concrete topping, the fire rating chart in the Vulcraft catalog was consulted. For a 2” composite deck (2VLI) requiring a two hour rating and using sprayed fireproofing, the largest topping thickness required is 3¼” LW or 4½” NW concrete. Since the drawings specified lightweight concrete, we will consider the 3¼” LW topping for the calculations. Next, the decking was picked based on loading tables in the Vulcraft catalog. **See detailed calculations in Appendix C.**

It was found that 18 gauge 2VLI decking with a 3¼” lightweight topping could be used. This decking also meets the requirements for 3-span unshored construction and thus is an economical choice. It also corresponds with the decking specified in the drawings.

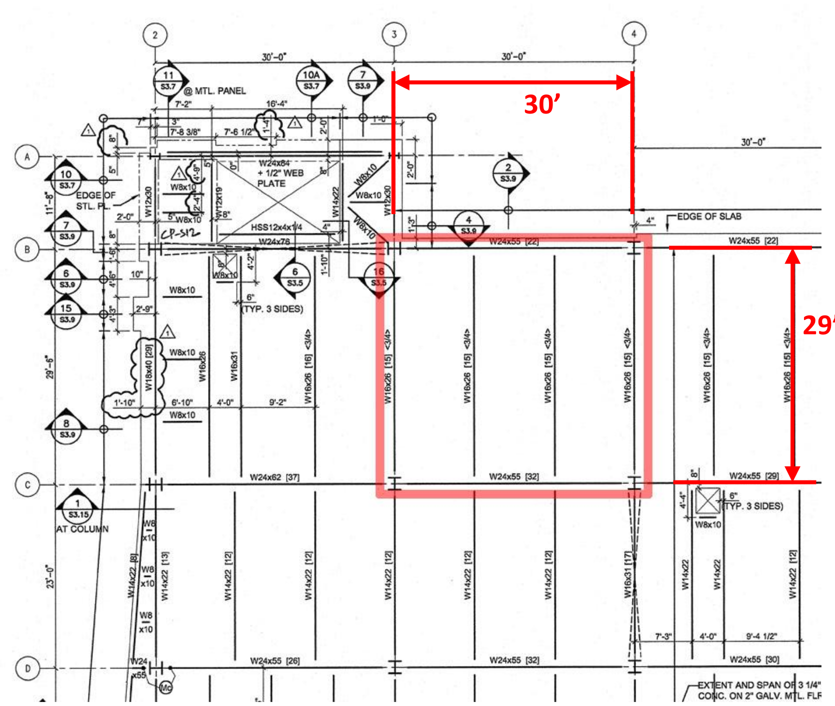


Figure 8 : Decking considered in 5th floor typical bay

Typical Composite Beam

The second spot check was done on a composite beam in a typical bay on the 5th floor. Beam spacing is typical at 10' on center. These calculations considered a beam in the 29'-6" span, shown in Figure 9. This member was specified in the drawings as a W16x26. Loading follows the typical dead and live loads for a patient floor. Additionally, live load was able to be reduced.

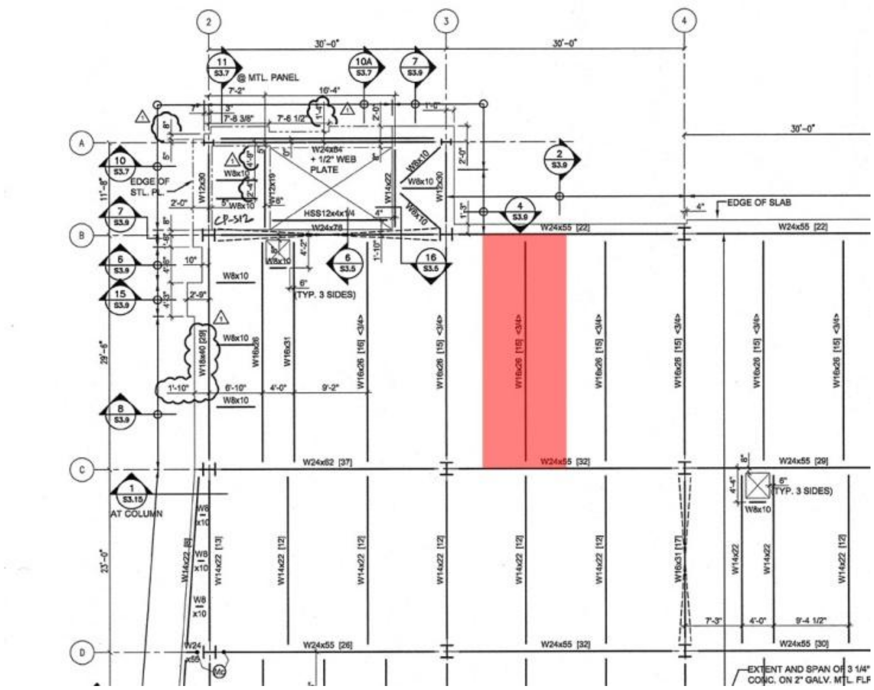


Figure 9 : Typical composite beam in 5th floor bay

Calculations used the weak stud position value. As a designer you cannot guarantee that the studs will be placed correctly, and it is more conservative to use the weak position value. Unshored strength, wet concrete deflection, and live load deflection were all calculated. **See Appendix C for detailed calculations.**

Several different beam options were considered (See Figure 12). The W16x26 proves to be the most economical with the least weight in steel. Therefore the W16x26 with 12 shear studs was picked. This is similar to the drawings which specify a W16x26 with 15 shear studs. The difference in number of shear studs is most likely a safety precaution. Additional shear studs are often added as a precaution in case some are damaged or installed incorrectly.

Composite Beams		
Member	# of Studs	Weight (lbs)
W14x26	16	927
W16x26	12	887
W16x31	14	1055

Figure 5 : Composite beam choices

Interior Column

The interior column C3 that was chosen for a spot check is shown in Figure 14. The column supports loads from floors 5-8 directly and supports the above column and upper floors as well. Typical loading was considered on each floor. Additionally because the column is supporting roof loads, the snow load and mechanical equipment load were also taken into account. Tributary areas from the main roof and the upper roof are shown in Figure 15 and 16. Once the loads were totaled, they were factored into the axial load on the column. Internal moment was also considered by using pattern loading. Live load was placed in one adjacent 30' bay. Floors average in height of 13' and thus the column is braced at every floor in both directions. Assuming a K factor of 1 is conservative and gives an effective length of 13'. **See detailed calculations in Appendix C.**

Using the combined axial and flexure equation and a trial size column depth of 14", a column size of W14x109 was selected from Table 4-1 in the steel manual. The column size given in the column schedule is a W14x99; the next size down from a W14x109. This difference is probably from an overestimation of the axial loading on the column. All columns in the building have a web of 14", therefore it is important to main this depth. Due to the fact that the given and the calculated column depths are the same, the weight difference is acceptable considering that the W14x99 is only slightly more economical.

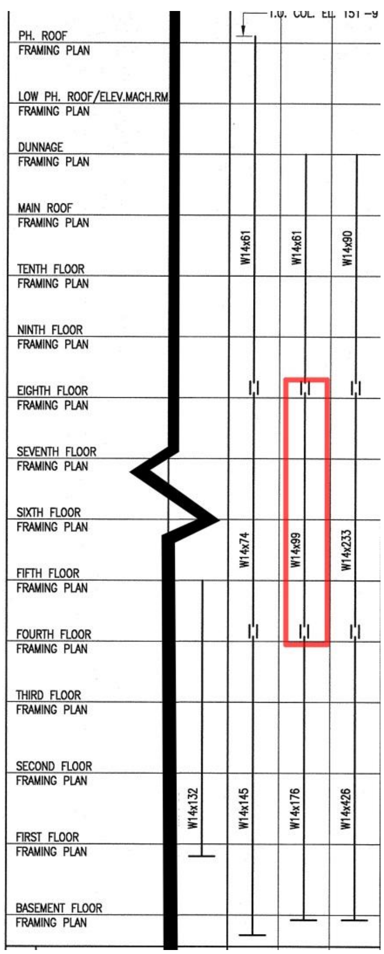


Figure 6 : Column schedule denoting C3

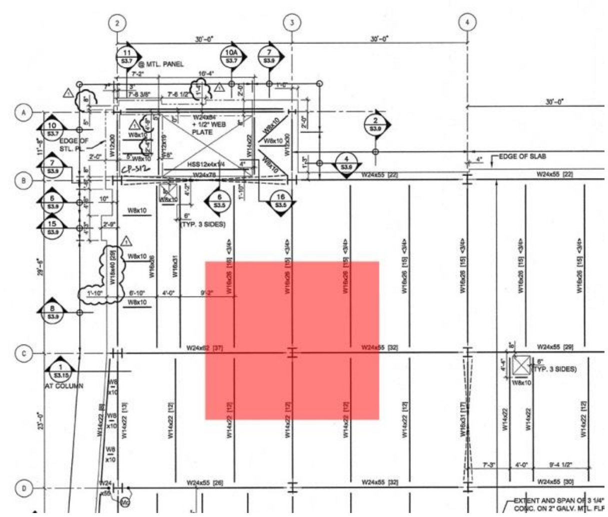


Figure 7 : C3 Typical floor tributary area

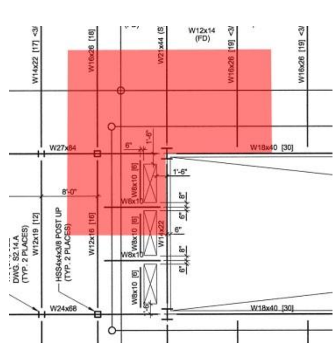


Figure 9 : C3 main roof tributary area

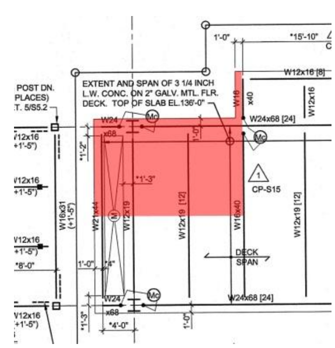


Figure 8 : C3 upper roof tributary area

LATERAL LOADS

Seismic Loads

Seismic loads were calculated based on ASCE 7-05 provisions. A major difference in the design of the building and the analysis is that the building was designed under ASCE 7-02. This difference was very evident in the response modification coefficient of the building, as well as ground acceleration factors. Shown below are different factors that are relevant to the seismic analysis calculations in this report.

Values for S_S, S_1, S_{DS}, and S_{D1} were determined via the USGS geo-hazards website. The values were then checked for accuracy by using the contour maps in ASCE 7-05 chapter 22.

Seismic Design Values		
Factor/Parameter	Design	Analysis
R	5	3.25
C_d	4.5	3.25
Ω	2	2
I	1.5	1.5
Use Group	III	III
Design Category	C	C
Site Class	D	D
S_s	0.321	0.267
S_1	0.08	0.059
S_{DS}	0.3296	0.282
S_{D1}	0.128	0.095
Base Shear, V	1300	1462

After calculating the approximate fundamental period of the building, Cs was able to be determined. Then floor weights were totaled using an excel spreadsheet. Finally the base shear was able to be calculated **(See Appendix D for detailed calculations)**. Forces were then distributed to each story level to find story forces and story shears. For simplicity, both roof level's masses were lumped together at the main roof level (h=133'). **For force distribution and story shears, see excel table on next page**.

The base shear determined in this report's analysis was 1462 k while the base shear the building was designed for was 1300 k. This is approximately a 12% difference and was caused by the changes in code. Changes in the ground motion response maps affecting S_{D1} directly affected the value of C_S and by association, the base shear.

A computer model was not created for this stage of analysis. However, analyzing the building in a computer model would give different values of the fundamental frequency of the building. Since the code allows the use of the approximate period, the building's response to seismic activity is considered the same in all directions. In reality the building would react differently to North-South ground motion as opposed to East-West ground motion. These effects will be dealt with at a later stage, specifically Technical Report III.

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	3237	0	0.00	0.00	1461.68	0.00
2nd	14	2563	52133	0.02	24.72	1461.68	346.14
3rd	28	2652	118994	0.04	56.43	1436.96	1580.12
4th	42	2725	194242	0.06	92.12	1380.52	3869.02
5th	55	2168	210239	0.07	99.71	1288.40	5483.84
6th	68	2106	260116	0.08	123.36	1188.70	8388.50
7th	81	2100	316751	0.10	150.22	1065.34	12167.77
8th	94	2100	375412	0.12	178.04	915.12	16735.69
9th	107	2100	435235	0.14	206.41	737.08	22085.93
10th	120	2100	496098	0.16	235.27	530.67	28233.00
Roof	133	2344	622862	0.20	295.39	295.39	39287.23
Sum		**26195**	**3082083**	**1.00**	**1461.68**		**138177.23**

*Table shows seismic force distribution per story height as well as overturning moment per level

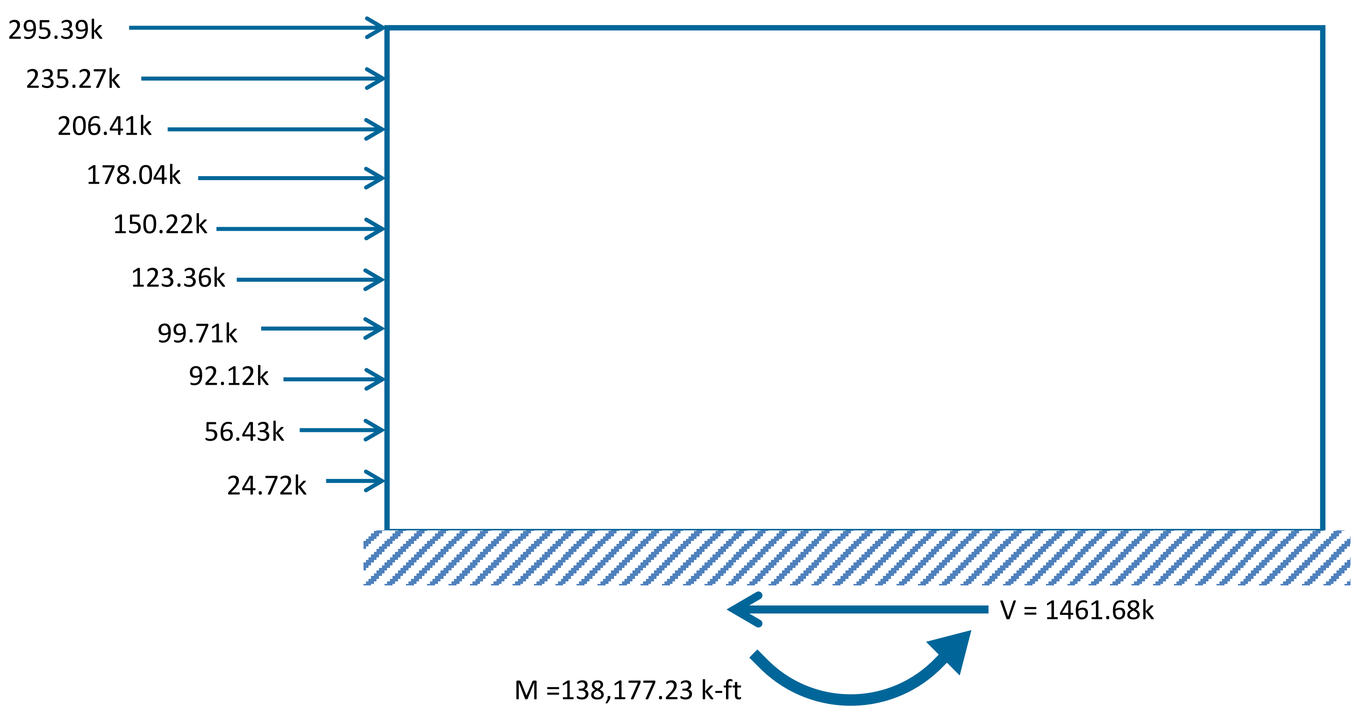


Figure 10 : Seismic story forces in N-S and E-W

Wind Loads

Wind loads on the Main Wind-Force Resisting System (MWFRS) were calculated in accordance with ASCE 7-05. The code provisions call for the fundamental frequency to be calculated in order to determine if the building is flexible or not. From there, the gust factor can be determined. In order to determine the fundamental frequency, the code provides the approximation 75/H. This is more conservative than using the approximate frequency determined from $1/T_a$.

Calculations determined that the building was flexible; therefore the gust factor was determined by the procedure outlined in the code for a flexible building. **Detailed calculations can be seen in Appendix E**. Diagrams depicting the wind pressures on the building are shown on the next two pages. Also shown are the pressures for the roof. The values calculated were checked with those on the drawings and found to match. Although, a computer model will be constructed at a later date in order to perform more detailed calculations.

Since the pavilion is not a perfect rectangular box on the first 4 floors, it was approximated as a rectangle with the dimensions 86'x285' which are the dimensions of the upper floors (See Figures 20 & 21, Appendix A). Figure 18 shows the wind pressures in the North-South direction and Figure 19 shows the East-West direction.

It should be noted that for the wind analysis, the height of the building was taken as 152' which is the dimension to the top of the parapet. This is different from the seismic analysis which took the lumped roof mass at a height of 133'.

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

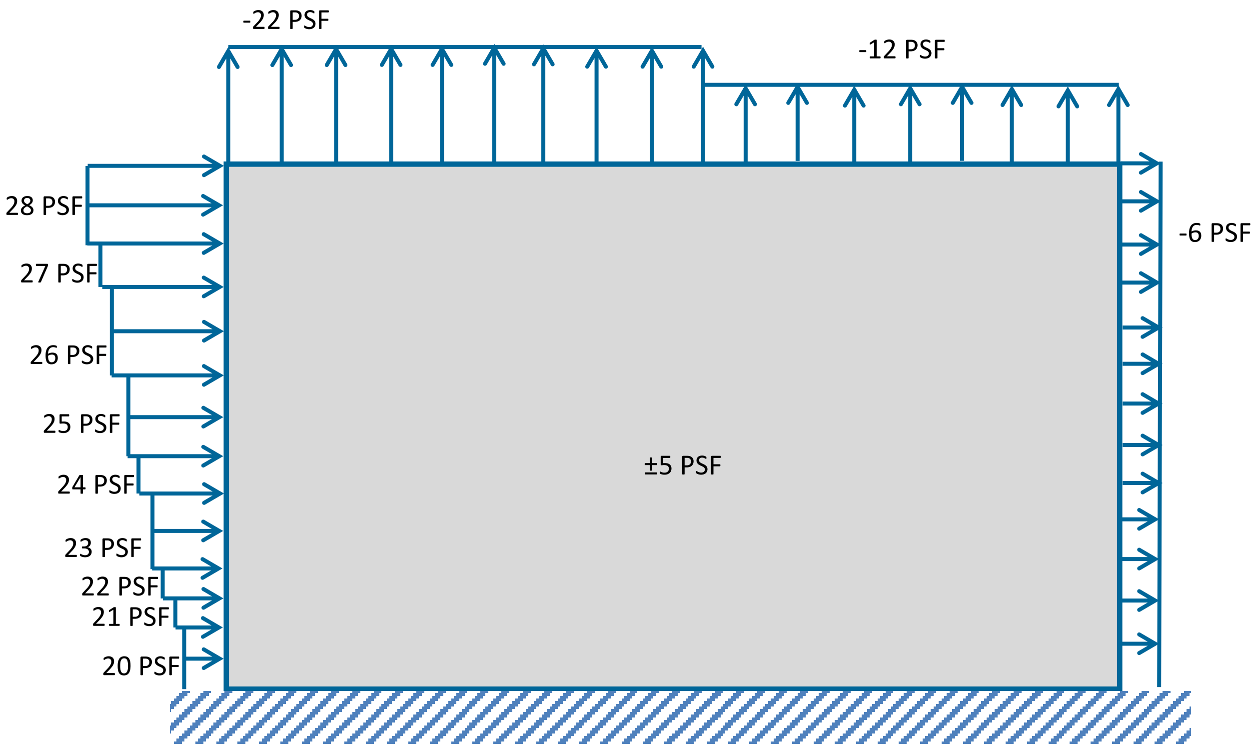


Figure 18: North-South Wind Pressures

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

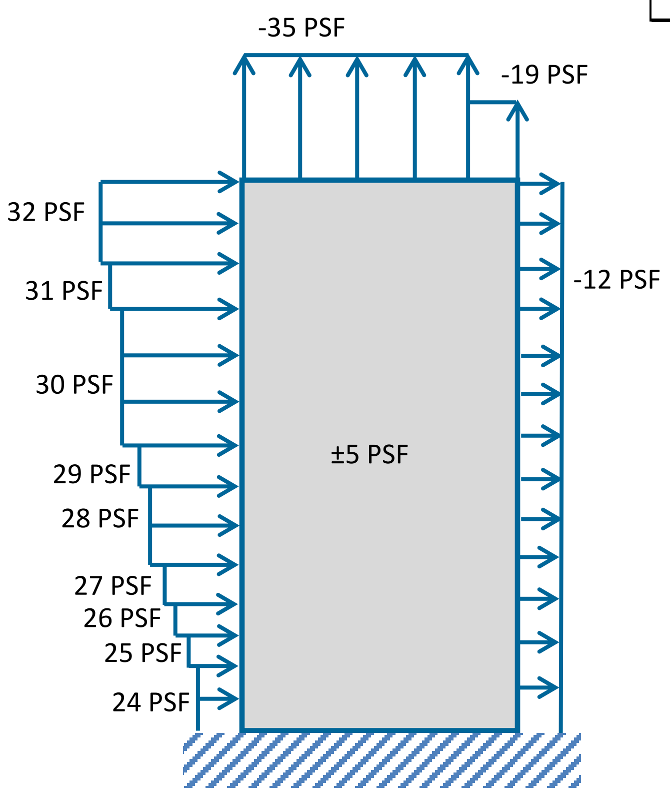


Figure 19: East-West Wind Pressures

CONCLUSION

This report was meant to analyze the existing conditions of the Roberts Pavilion. This consisted of the foundations, floor systems, framing system, lateral systems, as well as codes and loadings. These were summarized in detail and the systems were then analyzed under the ASCE 7-05 code. Loadings were determined based on code values as well as values given in the drawings. From the code and the drawings the lateral and framing systems were able to be analyzed and verified.

A typical bay was spot-checked to determine if the steel decking was adequate, which it was. Then a composite beam was designed for the same bay based on the loading assumed. It was then checked against the beam that was actually used in the framing and they were the same size. They only differed in number of shear studs used, which was most likely an overdesign safety precaution. Finally a typical interior column was designed and then checked against the column used in framing. The column that the calculations required was a W14x109 which is the next size up from the column that was used in framing: a W14x99. This discrepancy was most likely based on the calculations overestimating the axial load or on the difference in steel manual editions.

Lateral systems were also evaluated. The seismic forces were determined and the base shear was approximately 12% off from what was given in the drawings. This difference was caused by code changes and is not an issue of design error. Wind forces were also calculated, and the pressures were equal to those given on the drawings. The fundamental frequency that was calculated based on the code is an approximated value for both directions. In reality the frequency would be different in each direction. The design team determined the wind forces by a computer model. However, since this report does not utilize a computer model for the analysis, the values calculated were slightly lower.

Appendix A: Typical Plans

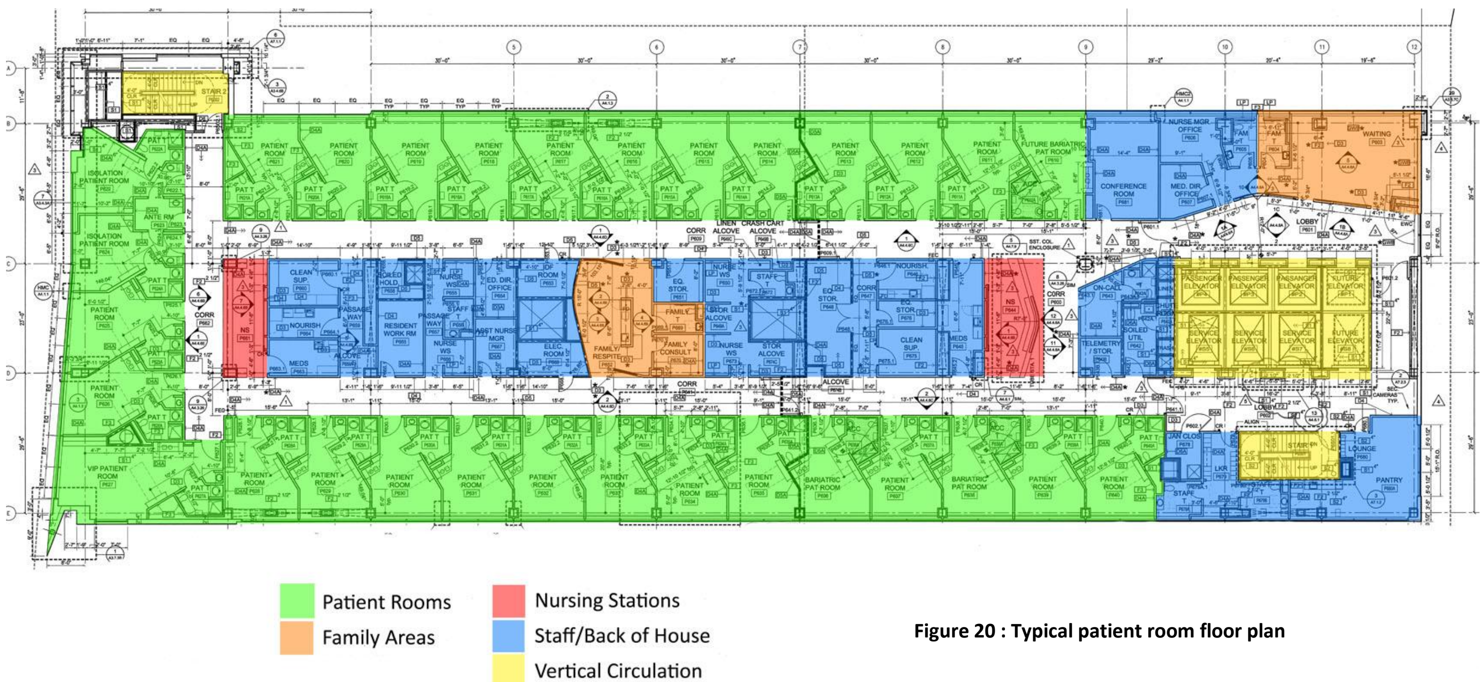


Figure 20 : Typical patient room floor plan

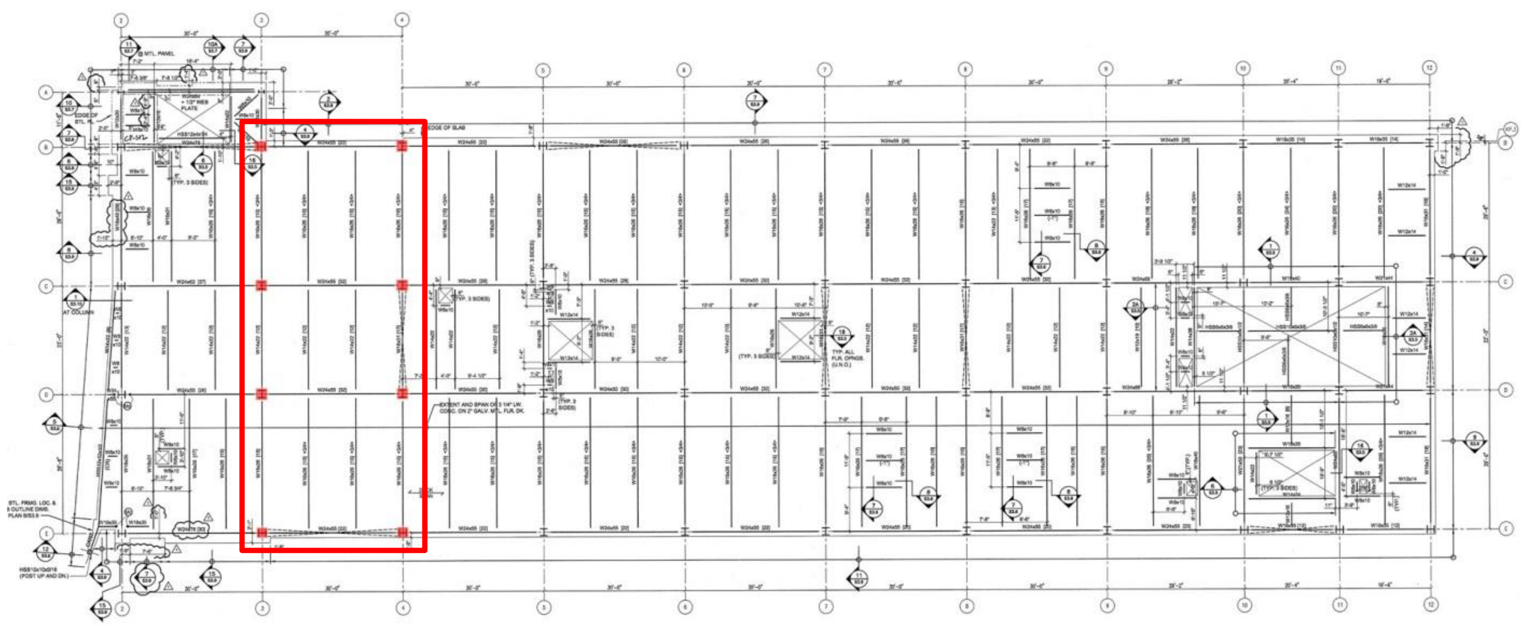


Figure 21 : Typical floor framing plan (typ bay shown)

Figure 22 : Lobby floor plan

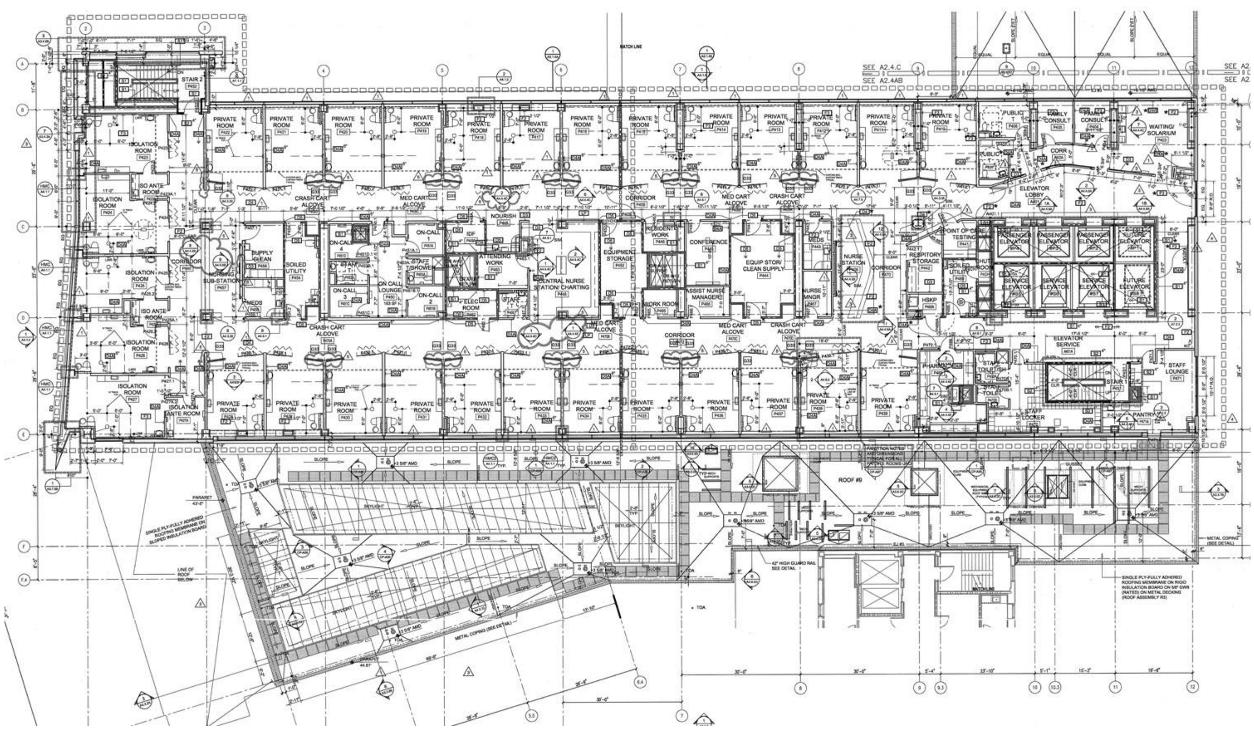


Figure 23 : Lobby roof and 4th floor architectural plan

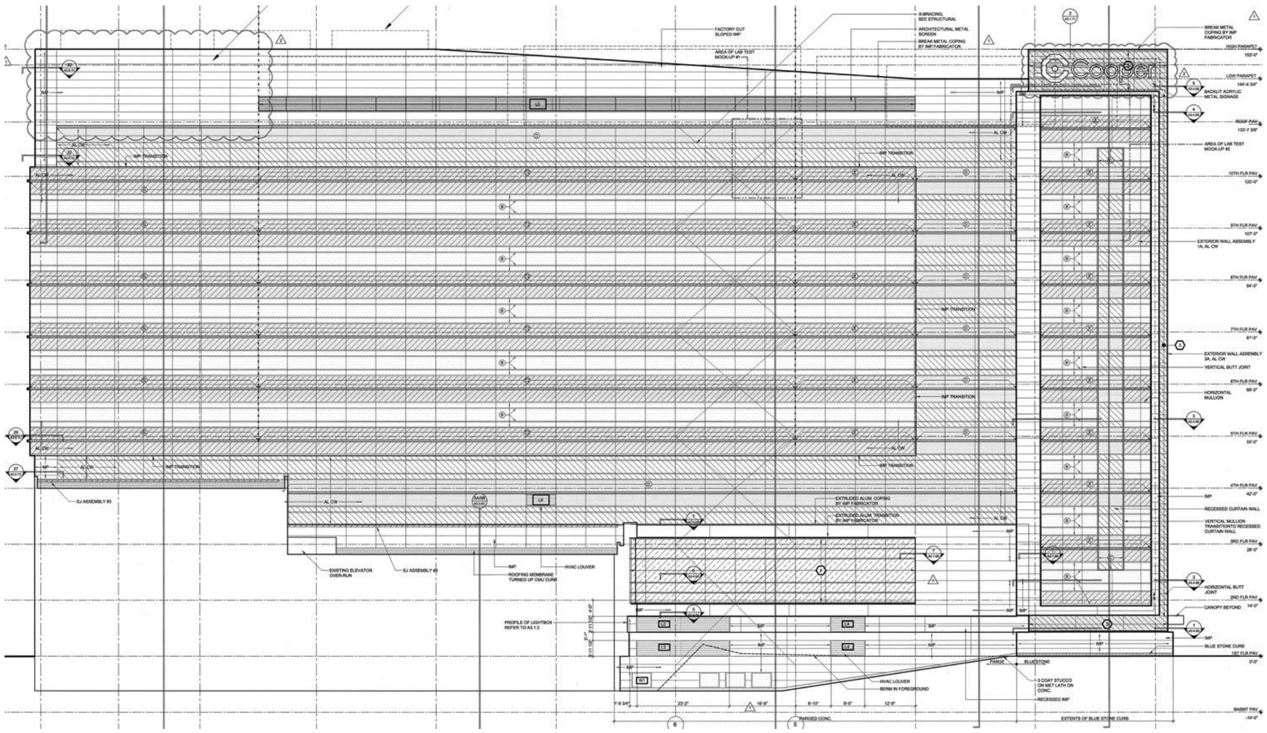


Figure 24 : East elevation

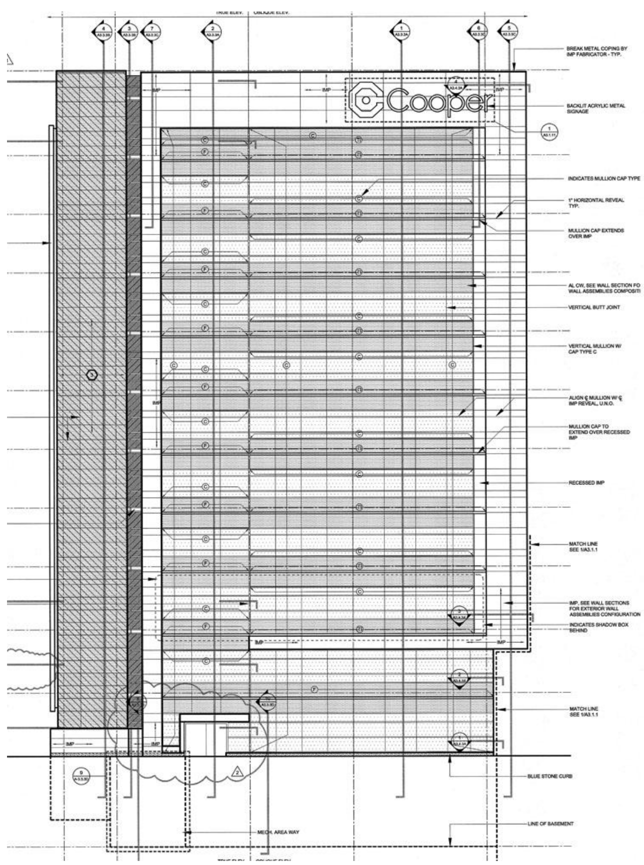


Figure 25 : North elevation

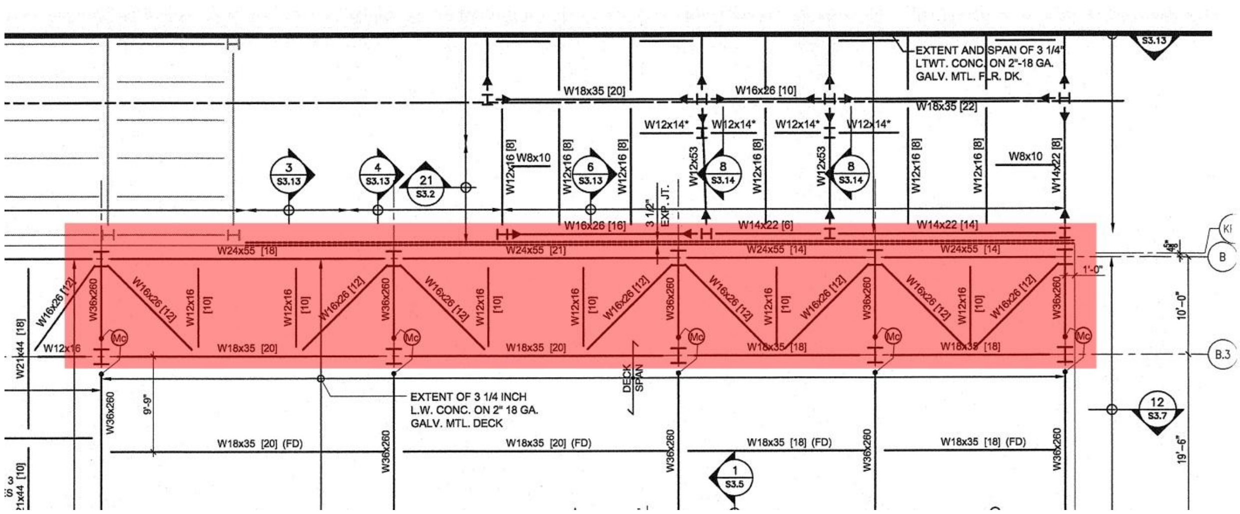


Figure 26 : Atypical bay framed with W36x260

Appendix B: Snow Load Calculations

SNOW LOAD | Tech I Report | Andrew Voorhees | 1

Roof Snow Load

$P_f = 0.7\, C_e C_t I p_g$

$C_e = 1.0$ partially exposed (Table 7-2)

$C_t = 1.0$ Table 7-3

$I = 1.2$ Category IV Table 7-4

$P_g = 25\text{psf}$ from Figure 7-1

$P_g = 25\text{psf} > 20\text{psf}$

$\therefore$

$P_f = 20(I)$

$= 20(1.2) = 24\text{ psf}$

compare to value given in drawings $P_f = 24\text{ psf}$ ✓ OK

Snow Drift

$\gamma = 0.13 P_g + 14 \leq 30\text{ pcf}$

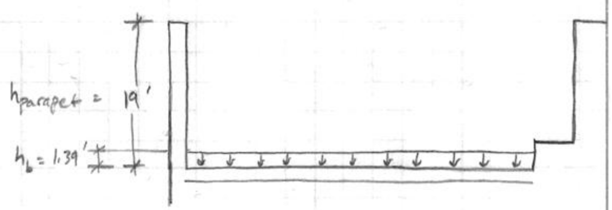
$\gamma = 0.13(25) + 14 = 17.25\text{ pcf} \leq 30\text{ pcf}$ ✓ OK

$h_b = \frac{P_s}{\gamma} \longrightarrow P_s = C_s P_f$

$C_s = 1.0$ Figure 7-2

$P_s = 1.0(24) = 24$

$h_b = \frac{24}{17.25} = 1.39$

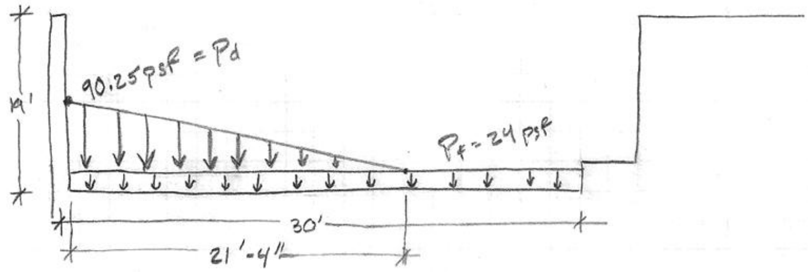


$h_c = h_{parapet} - h_b$

$= 19' - 1.39' = 17.61'$

$\frac{h_c}{h_b} = \frac{17.61}{1.39} = 12.67 > 0.2$ $\therefore$ Must consider Drift

SNow Load | Tech I Report | Andrew Voorhees | 2



Leeward drift:

$l_u = 285$ ft

$P_g = 25$ psf

$h_d = 5.38'$ from figure 7-9 ← Controls

$h_d = 0.43 \sqrt[3]{285} \sqrt[4]{25+10} - 1.5$
$= 5.38$

Windward Drifts

$l_u = 240$ ft

$P_g = 25$ psf

$h_d = 3/4 (5) = 3.75'$

$h_d = 5$ controls

$h_d = 5 < h_c = 17.61 \longrightarrow w = 4h_d = 4(5.38) = 21.52 < 8h_c = 140.88$ ✓ OK

$P_d = \gamma h_d = 17.25 (5.38') = 92.8$ psf

Compare to value on drawings $P_{d\,max} = 94$ psf ✓ OK

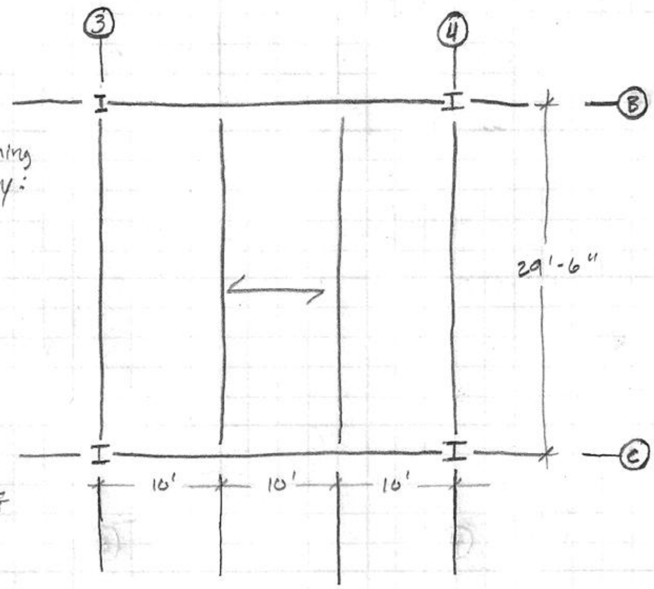
Appendix C: Gravity Loads Spot Check Calculations

SPOT CHECKS: Deck | Tech 1 Report | Andrew Voorhees | 1

Floor Decking

Location: 5th Floor
East-side exterior
Typ. Framing Bay:

- 2 hr Fire rating req'd
- composite action



- Live Load
  - patient room: 40 psf
  - corridors above First Floor: 80 psf
  - Moveable Partitions: 20 psf
- Dead Load
  - Framing: 10 psf
  - SDL: 10 psf
  - MEP: 5 psf

$L_{max} = \left| \begin{array}{l} 40+20 \\ 80+20 \end{array} \right. = 100$ psf

$D = 10 + 10 + 5 = 25$ psf

$1.2D + 1.6L = 1.2(25) + 1.6(100) = 190$ psf

* Assuming sprayed Fiber Fireproofing @ 2hrs. + 3 1/4" LW conc.

⟹ composite 2VLI deck

Clear span = 10'

2VLI 18 table value = 205 > 190 psf ✓ ok

∴ Use 2VLI 18 decking with 3 1/4" LTWT concrete

Drawings spec 2" deck, 18 gauge, 3 1/4" LTWT concrete ✓

slab + deck weight = 42 psf

3 span unshored clear span = 12'-7" > 10' ✓ ok
1 span unshored clear span = 10'-6" > 10' ✓ ok

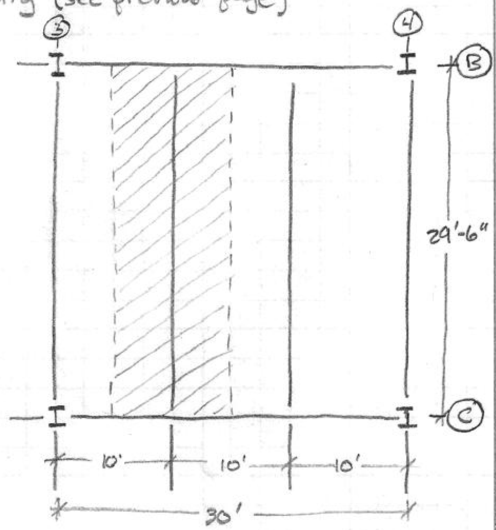
SPOT CHECKS: Beam | Tech I Report | Andrew Voorhees | 2

BEAM

Using the same bay as the floor decking (see previous page)

Live Load : 80 psf
Partitions : 20 psf

Framing : 10 psf
SDL : 10 psf
MEP : 5 psf
Slab + Deck : 42 psf
Self weight : 10 psf



Live Load Reduction

$K_{LL} A_T = 2(295) = 590 \text{ ft}^2 > 400$

$\therefore$ Live Load reduction applies

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

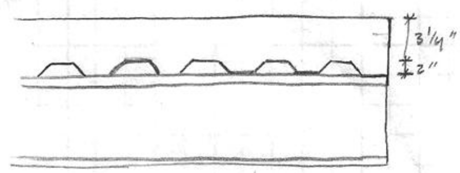
$$= 100\left(0.25 + \frac{15}{\sqrt{590}}\right) = 86.75 \text{ psf}$$

$A_T = 29.5'(10') = 295 \text{ ft}^2$

$w_u = 1.2(42+5+10+10) + 1.6(86.75) = 219.2 \text{ psf} \times 10' = 2192 \text{ lb/ft}$

$$M_u = \frac{w_u l^2}{8} = \frac{(2.192 \text{ k/ft})(29.5)^2}{8} = 238.45 \text{ k-ft}$$

Assume $a = 1.5" \rightarrow Y = 4.5"$
Assume 1 weak stud per rib $f'c = 3\text{ ksi}$ LtWt



Try W14x26
$\phi M_n = 240$ k-ft $\Sigma Q_n = 135$

$\frac{135}{17.2} = 7.85 \times 2 \Rightarrow 16$ studs

Economical check

W14x26: $26 \times 29.5 + 16(10) = 927$

Try W16x26 $\Rightarrow$ Most Economical
$\phi M_n = 241$ k-ft $\Sigma Q_n = 96$ $\frac{96}{17.2} = 5.58 \Rightarrow 12$ studs

W16x26: $26 \times 29.5 + 12(10) = 887$

W16x31: $31 \times 29.5 + 14(10) = 1059.5$

Try W16x31
$\phi M_n = 292$ $\Sigma Q_n = 114$ $\frac{114}{17.2} = 6.63 \Rightarrow 14$ studs

Spot Checks: Beam | Tech 1 Report | Andrew Voorhees | 3

Check a: $b_{eff} = 2 \times \min \left| \begin{array}{l} 5' \times 12'' = 60'' \\ \frac{29.5 \times 12''}{8} = 44.25 \rightarrow \times 2 = 88.5 \end{array} \right.$

$a = \dfrac{103.2}{0.85(88.5)(3)} = 0.457'' < 1.5''$ ✓ OK

$Y = 4.5''$ ok

use $\Sigma Q_n = \dfrac{12}{2} \times 17.2 = 103.2$ (12 = studs)

Check unshored strength:

W16×26 $\phi M_p = 166$ k-ft

$w_u = 1.4(26) + 1.4(42 \times 10') = 624.4$ plf

$w_u = 1.2(26 + 42 \times 10') + 1.6(20 \times 10') = 771.2$ plf ← controls

(20 × 10' = constr. load)

$M_u = \dfrac{0.7712(29.5)^2}{8} = 83.9$ k-ft $< 166 = \phi M_p$ ✓ OK for unshored

Check wet concrete deflection:

$W_{wc} = 42 \times 10' + 26 = 446$ plf

$\Delta_{wc} = \dfrac{5}{384} \dfrac{W_{wc} \ell^4}{E I_x} = \dfrac{5(0.446)(29.5)^4(1728)}{384(29{,}000)(301 \text{ in}^4)} = 0.871''$

$\Delta_{wc} = \dfrac{\ell}{240} = \dfrac{29.5 \times 12''}{240} = 1.475''$

$0.871 < 1.475''$

✓ Deflection OK

Check LL Deflection:

$w_{LL} = 86.75$ psf $(10') = 0.8675$ klf

$I_{LB} = 535 \text{ in}^4$ @ $Y_2 = 4.5''$ (point 7, $\Sigma Q_n = 96$)

$\Delta_{LL} = \dfrac{5}{384} \dfrac{w \ell^4}{EI} = \dfrac{5(0.8675)(29.5)^4(1728)}{384(29{,}000)(535)} = 0.953''$

$\Delta_{LL\,max} = \dfrac{\ell}{360} = \dfrac{29.5 \times 12''}{360} = 0.983''$

$0.953'' < 0.983''$

✓ LL Defl. OK

Summary:

Use W16×26 w/ 12 studs along the length of the beam

compare w/ drawings: used W16×26 w/ 15 studs (see report for discussion)

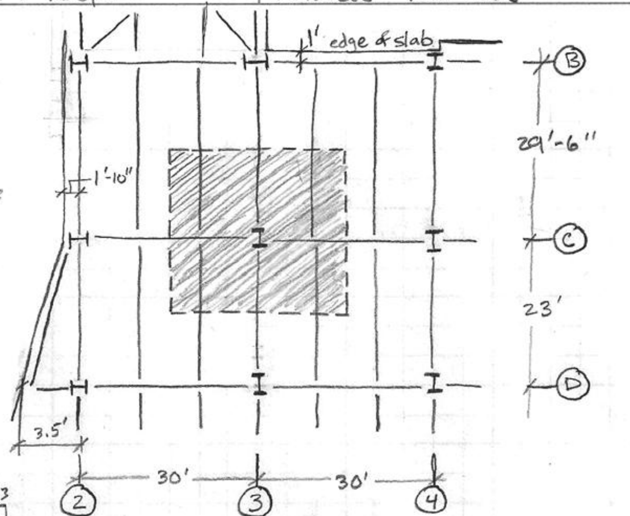
Spot Checks: Column | Tech I Report | Andrew Voorhees | 4

W14x99

Interior Column

Fifth Floor, Interior column

Loads
- 5th – 10th floor : LL = 80 psf
  partitions = 20 psf
- Snow Load : 24 psf
- Main Roof : R_L = 30
- Snow : Max drift = 94 psf
- Mech equip. : 10.38^k Dead
- SDL = 10 psf
- Framing = 10 psf
- MEP = 5 psf
- Deck/slab = 42 psf



$$A_T = \left(\frac{29.5}{2} + \frac{23}{2}\right)(30) = 787.5\ ft^2 > 400$$

∴ LL reduction is applicable

5th Floor – 10th Floor:

$K_{LL} A_T$ =
$(30' + 30')(23 + 29.5) + 1'(30')$
$+ 1.83'(29.5) + 1.83'(23')$
$+ \frac{1}{2}(3.5 - 1.83)(23)$

$= 3295.28$

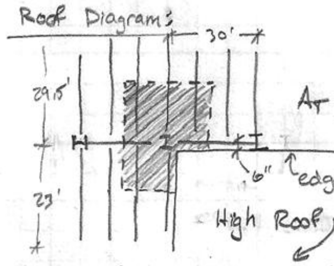
$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right)$$

$$= L_o\left(0.25 + \frac{15}{\sqrt{3295.28}}\right) = 0.51\,L_o$$

$0.51L_o > 0.40\,L_o$ ✓ ok

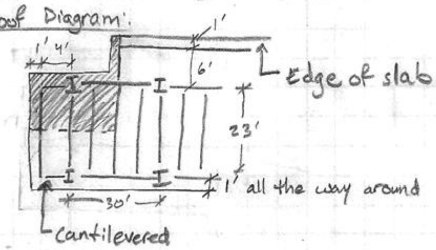
$L_o = 0.51(100) = 51$ psf

Roof Diagram:



$A_T = 628\ ft^2$

High Roof Diagram:

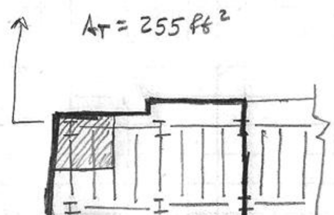


$A_T = 255\ ft^2$

<u>Level 5</u>

$P_L = 51\,psf(787.5\ ft^2) = 40.16^k$
$P_D = 787.5\ ft^2(42 + 10 + 10 + 5) = 52.76^k$

Values same for 6–10



outline shows 83,000 lb. AHU
∴ Col. sees 10.38^k Dead Load

spot checks : Column | Tech | Report | Andrew Voorhees | 5

Roof Level : Accounting for Drift

$P_S = 94(628) = 59.03^k$
$P_D = 628(42+10+10+5) = 42.08^k$
$P_{RL} = 30(628) = 18.84^k$

* Mechanical Equipment is located on High Roof Level

High Roof level : No Drift at this level

$P_S = 24(255) = 6.12^k$
$P_D = 255(42+10+10+5+10.38) = 19.73^k$
$P_{RL} = 255(30) = 7.65^k$

controls on main roof; controls on high level roof

$P_u = 1.2D + 1.6L + 0.5S + 0.5L_R$

$P_u = 6[1.6(40.16) + 1.2(52.76)] + 0.5[59.03 + 7.65] + 1.2[42.08 + 19.73] = 872.92^k$

6 floors

@ 13' braced length

W14×82 $\phi P_n = 802^k$ from Table (4-1) AISC

N.G.

W14×90 $\phi P_n = 1050^k$ from table (4-1) AISC

$1050 > 872.92^k$ ✓ ok

Compare to column schedule

W14×99 $\phi P_n = 1150$

Larger than W14×90, but this makes sense, this gravity calculation hasn't considered any lateral load, therefore selection would be smaller ✓ok.

Appendix D: Seismic Load Calculations

Seismic | Tech Report 1 | Andrew Voorhees | 1

Basic Seismic-Force-Resisting System:

- Ordinary steel concentrically braced frame

$R = 3\frac{1}{4}$
$C_d = 3\frac{1}{4}$
$\Omega = 2$
$I = 1.5$
Seismic Use Group = III
Seismic Design Category = C

Site Class = D

$S_s = 0.267\,g$
$S_1 = 0.059\,g$

$S_{MS} = 0.423\,g$
$S_{M1} = 0.142\,g$

$S_{DS} = 0.282\,g$
$S_{D1} = 0.095\,g$

USGS geohazards website - Referencing ASCE 7-05

Also checked with ASCE 7-05 seismic maps

√ ok

From Table 12.2-1

$R = 3\frac{1}{4}$
$\Omega = 2$
$C_d = 3\frac{1}{4}$

Approximate Fundamental Period:

$T_a = C_t h_n^x$ from table 12.8-2: $C_t = 0.02$, $x = 0.75$

$= 0.02(133)^{0.75} = 0.783$

From Figure 22-15 $T_L = 6$ sec

$T = 0.783 < T_L$

$$C_s = \frac{S_{DS}}{R/I} = \frac{0.282}{3.25/1.5} = 0.1301 < \frac{S_{D1}}{(R/I)T} = \frac{0.095}{(3.25/1.5)(0.783)} = 0.0558$$

$0.1301 \not< 0.0558$

$$\boxed{\therefore C_s = 0.0558}$$

> 0.01 √ ok

Seismic | Tech I Report | Andrew Voorhees | 2

Base shear

$V = C_s W$

$W = 26{,}195^k$ (calculated in spread sheet)

$V = 0.0558\,(26{,}195) = \boxed{1461.68^k}$

$C_{vx} = \dfrac{w_x h_x^k}{\Sigma w_i h_i^k}$ $\quad T = 0.783 > 0.5$, < 2.5

$\dfrac{0.783 - 0.5}{2.5 - 0.5} = \dfrac{k - 1}{2 - 1} \Rightarrow k = 1.1415$

$w_x h_x^k \Rightarrow$ see spreadsheet calculations

Story Force $F_x = C_{vx} V$

Story shear = Σ story forces above + including desired level

Overturning Moment = $F_x h_x$

* See spreadsheet for calculations

Base shear = 1461.88^k compared to value given in drawings = 1300^k

Difference based on code year ⇒ Bldg designed under ASCE 7-02
- Calculations done under ASCE 7-05

OSCBF $\begin{cases} R = 3\tfrac{1}{4} \\ \Omega = 2 \\ C_d = 3\tfrac{1}{4} \end{cases}$ $\qquad$ OSCBF $\begin{cases} R = 5 \\ \Omega = 2 \\ C_d = 4\tfrac{1}{2} \end{cases}$

$S_s = 0.267$, $S_1 = 0.059$ ⟵ compare values ⟶ $S_s = 0.3210$, $S_1 = 0.08$

$S_{DS} = 0.282$, $S_{D1} = 0.095$ $\qquad$ $S_{DS} = 0.3296$, $S_{D1} = 0.1280$

* See report for discussion

Appendix E: Wind Load Calculations

Wind Loads | Tech I Report | Andrew Voorhees | 1

$\boxed{\text{Wind Load Parameters}}$ • 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)

$\boxed{\text{Velocity Pressures}}$

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

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

$$@\ 152': \quad q_z = 0.00256(1.38)(1.0)(0.85)(90)^2(1.15) = 27.97 \text{ psf}$$

$\boxed{\text{Calculating } G}$

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

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

∴ Consider Building as Flexible (§6.5.8.2)

Wind Loads | Tech 1 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

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

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

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

$$\boxed{= 0.6064 \text{ N-S}}$$

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: $$G_f = 0.925 \left[\frac{1 + 1.7(0.1688)\sqrt{3.4^2(0.8616^2) + 4.0174^2(0.6064^2)}}{1 + 1.7(3.4)(0.1688)} \right]$$

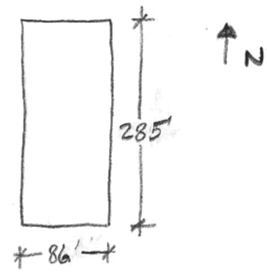
$$\boxed{G_f = 0.98 \text{ N-S}}$$

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

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

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