Technical Report 1

The University Medical Center of Princeton

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Structural Option
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Executive Summary

Technical Report 1 was written to gain an in-depth understanding of the University Medical Center of Princeton's (UMCP) structural design. This is first shown through a written description of the foundation, floor system, framing system, lateral system, and the roof system throughout the building. The report also includes figures that will help clarify the designed systems more thoroughly. A brief listing of what codes, means, and design loads were used during the design process is discussed. Finally, calculations of gravity loads that define the sizes of the members used in the structure, and lateral load forces will be analyzed, including spot checks.

Turner construction provided existing drawings and specifications which aided in the analysis of the UMCP building. These drawings and specifications provided many of the figures in this report. Appendix 1 has sections and floor plans to provide a better understanding of the hospitals layout. All of the calculations were designed with ASCE7-10 procedures and values.

The lateral design calculations concluded that the wind forces control over the seismic forces in both the East/West and North/South direction. To accommodate for these forces, steel moment frames were placed on the East/West outside walls while braced frames were placed on the outside walls of the North/South walls as well as the core elevator shaft. A few general assumptions were made depending on how to get certain variables for the wind and seismic design.

Part of this tech report was to determine how the designers came up with the building's framing system. A typical bay, 29.5'x26', was analyzed with an assumed live load of 80psf, superimposed dead load of 20psf, and an MEP load of 15psf. Through the calculations, the products for the slab and beam came out to be the same, but with different girder and column sizes. The difference in sizes was relatively small and within a 20% difference. In the end, the spot check concluded that the framing is structurally sound.

One structural aspect of the UMCP building that was not taken into account was the curtain wall system. In addition, soil pressures were not measured in this report, but are known to have an effect on the design of the foundation walls. These systems will be further analyzed in later technical reports on their structural integrity.

Contents

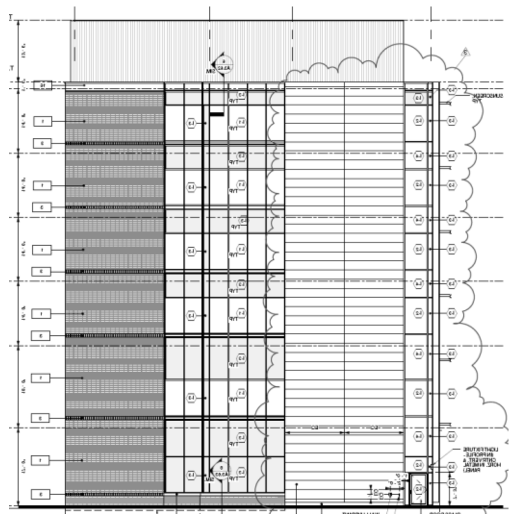
Building Introduction

Princeton University Medical Center was in a big need of change. The rapid growth of people plus the outdated building design and equipment were the main reasons to upgrade their old medical center.

FIGURE 1: UMCP SITE LOCATION SHOWN IN BLUE
SATELLITE PHOTO COURTESY OF GOOGLE MAPS

The University Medical Center at Princeton (UMCP) will also be joining the Pebble Project. Pebble Project is a research effort between The Center for Health Design and selected healthcare providers to measure the layout and design of a hospital and how it can increase quality care and economic performance. The design of this building is not just for looks, but to help operate a hospital in a healthy and efficient manner.

This six story tall building has a long and curving body that encases the parking lot to draw people into the building. Lighting is not going to be an issue during the day as the glass curtain wall is used on the south face of the building. Furthermore, it will provide a view to the outside for all the patients and workers in the building. The curtain wall is framed with aluminum reliefs and metal panels. The West and East elevations have a CMU ground face with a brick façade on the top floors, and there are very few windows since these walls are framed with steel bracing. The mechanical equipment is encased in 13.5' parapets. Floors two through six almost mimic each other in framing and room layout. The entrance of the building has a wide atrium open to the second floor with interior wood shading panels. The overall design of the building is simple, sleek, and efficient.



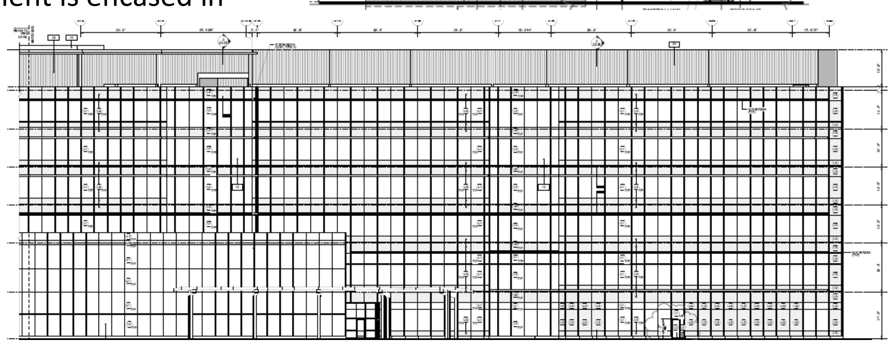
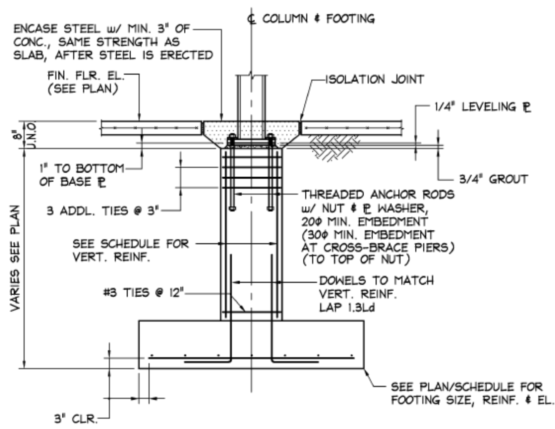


FIGURE 2: EAST AND SOUTH BUILDING ELEVATIONS
DRAWINGS COURTESY OF TURNER CONSTRUCTION

Structural Overview

The foundation plan for the University Medical Center is built on 4" to 5" Slab-On-Grade basement floor with interior concrete piers stabilizing wide flange columns, and an exterior 2' thick foundation wall partially incasing mini tension piles. The design of the superstructure is primarily steel framing. The framed floors consist of a 3 span 3 ¼" lightweight concrete composite decking system with composite steel framing. Roof decking is type B 1 ½" galvanized metal deck, and 6 ½" normal weight concrete composite metal deck for the roof Penthouse area. There is also a massive curtain wall spanning the South end of the curving building, but this will not be analyzed in this technical report.

Foundations



TYPICAL COLUMN FOOTING WITH PIER

FIGURE 3: TYPICAL COLUMN FOOTING WITH PIER
DRAWING COURTESY OF TURNER CONSTRUCTION

According to drawing S3.01 all the subgrade footings were poured under the supervision of a registered Soils Engineer. The capacity of the soils, shown in the boring test specifications, came out to be 4,000psf and 8,000psf for the compacted/native soils (medium-dense/stiff) and decomposed bedrock respectively. The spread footings erect wide flange columns, varying from W10x54 to W14x311, to anchor the superstructure (Refer to Figure 3 for more detail). The spacing for the foundation columns is not consistent throughout the basement, which that is the reason for the varying column sizes. Figure 3 shows a typical spread footing supporting a steel column. Outlying the basement is a 2' thick foundation wall with mini tension piles that relives up to 150kips of tension from the concrete bearing wall.

Concrete Strengths:

- 3,000psi- Spread Footings, Wall Footings, Foundation Wall, & Retaining Walls
- Minimum of 3,000psi- Piers-match wall strength
- 3,500psi- Slab-On-Grade

Rebar Design:

- ASTM A615- Deformed Bars Grade 60
- ASTM A185- Welded Wire Fabric

Floor & Framing Systems

A typical beam spanning in the North/South direction, consists of a 26' span then a 15' span, and finally back to a 26' span. The East/West girders span 29 ½' typically. Floors two through six do not change in design other than the column thickness, all of the floors use a 3 span 3 ¼" lightweight concrete composite decking. This creates a one-way composite flooring system connected to composite beams. Even though the first floor has an additional atrium, the decking is still consistent to the floors above. Figure 4 shows the wide flange beams used in each span.

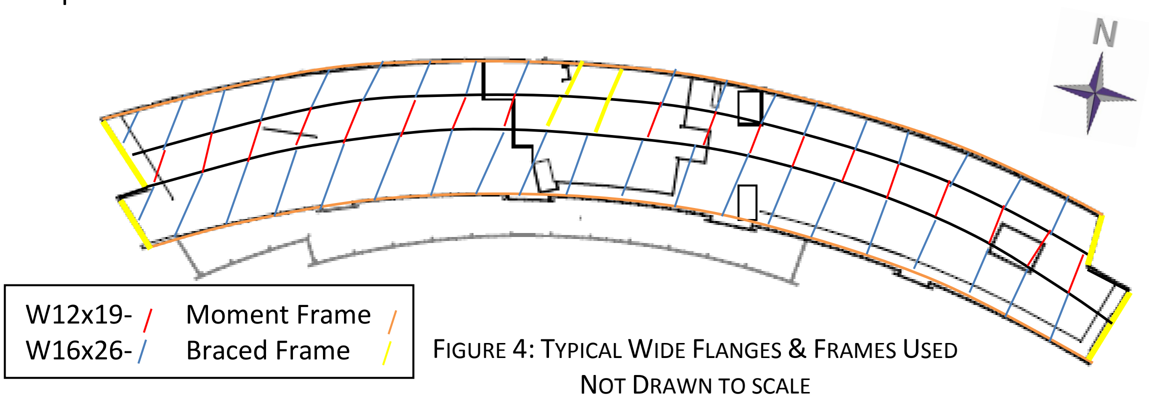


Figure 4: Typical Wide Flanges & Frames Used
Not Drawn to scale

The infill beams are usually at a spacing of 9.8' and they range from W16x26 for the 26' spans or W12x19 for the 15' spans. The girders typically span 29.5' and vary from W24x55 on the exterior girders to W21x44 on the interior girders. These composite beams use ¾" bolts to help anchor the decking. The typical bays then come out to be either 29.5'x26' or 29.5'x15'. There are also two transfer beams on the on column lines N2 and S3 to account for columns that do not line up on the first to second floor.

Steel Design:

- ASTM A992- Wide Flanges
- ASTM A500- Rectangular/Square Hollow Structural Sections Grade B, Fy=46ksi
- ASTM A500 or ASTM A53- Steel Pipe Type E or S Grade B
- ASTM F1554- Anchor Rods Grade 55

Lateral Systems

The UMCP lateral systems design was comprised of typical steel moment frames in the East/West direction and steel concentrically braced frames in the North and South direction. Those framing systems only occurred on the perimeter of the building. Around the elevator shaft is another place where the design is concentrically braced. The lateral forces will travel into the composite deck, and then through the wide flange beams or HSS braces into the columns to the piers to then dissipate into the ground.

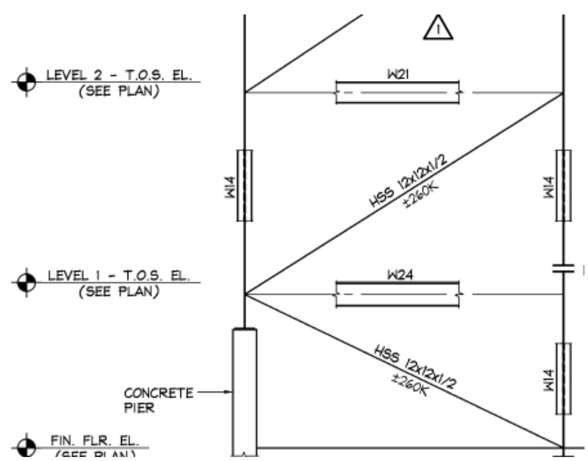


Figure 5: Typical Braced Frame
Courtesy of Turner Construction

Codes/Means Used

This building fit into an Occupancy Category III. Any Hospital/Medical Center needs to be designed with an Occupancy Category III as a safety factor.

Original design codes used on this building were:

- 2006 International Building Code (IBC) with New Jersey Uniform Construction Code
- 2006 International Mechanical Code (IMC)
- 2005 National Electric Code (NEC) with local amendments
- 2006 International Energy Conservation Code with other local amendments
- 2006 International Fuel Gas Code with local amendments
- New Jersey Department of Health and Senior Services - "Licensing Standards for Hospitals, N.J.A.C 8.43G" and the 2006 Edition - "Guidelines for Design and Construction of Hospital and Health Care Facilities."

Design codes used for Thesis Calculations:

- ASCE 7-10 Minimum Design Loads for Buildings and other Structures
- American Institute of Steel Construction, 14th Edition AISC Steel Construction Manual
- 2008 Vulcraft Steel Roof & Floor Deck Manual

Gravity Loads

The UMCP structure was designed by O'Donnel & Naccarato, Inc. using the 2006 International Building Code with New Jersey Amendments. For the thesis calculations performed, ASCE7-10 was used to determine the snow, dead, and live loads. Every calculation was performed by using the LRFD method, and in later tech reports these checks will be analyzed on a computer modeling system.

Snow Loads

All the snow load calculations were taken from chapter 7 of ASCE7-10. The only places that needed to be designed for drift were the 13.5' parapets, and the two story tall atrium extension from the South face of the building. Since the parapets are so tall, only one direction was taken into account for the atrium drift because no snow will blow over top of a 13.5' parapet. The drift calculations for the parapet were only taken for the longer direction, East/West, since the snow load would be greater. The flat roof snow load, P_f, came out to be 19.5psf.

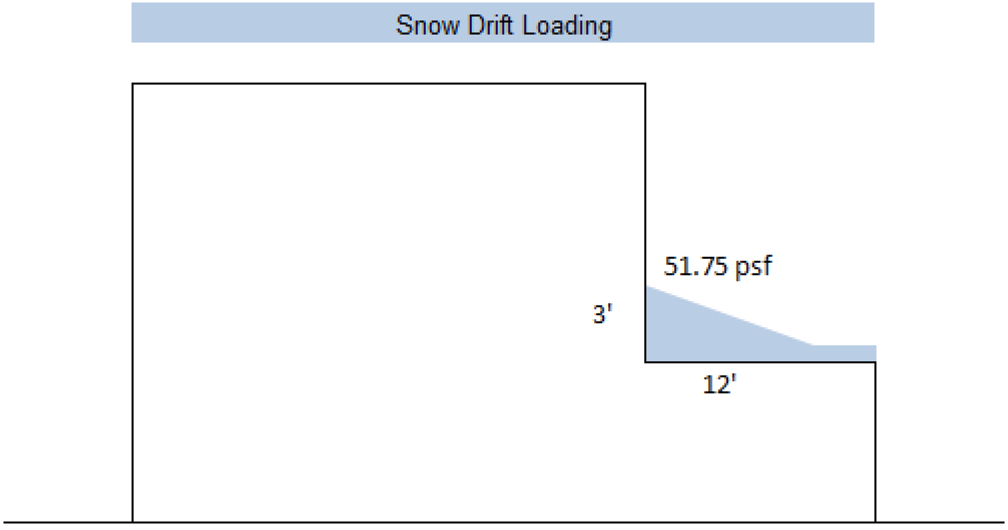


FIGURE 6: SNOW DRAFT LOAD ON ATRIUM ROOF
NOT DRAWN TO SCALE

Dead Loads

The roof dead loads for the mechanical equipment were assumed to be 150psf since there were multiple pieces of equipment weighing more than 15,000 pounds. The metal decking used for the roof did not add too much weight to the roof, only about 1.27psf. A framing allowance for the steel system was assumed to be 10psf for the roof and every other floor. Decking weight for the roof and the composite decking

FIGURE 7: TYPICAL BAY USED FOR SPOT CHECKS
COURTESY OF TURNER CONSTRUCTION

weight for the floors were taken out of the Vulcraft Steel and Roof Decking manual. Though, the decking for UMCP was manufactured by United Steel Inc. The decking was the same for all six floors, and it weighed 39.5psf. The composite beam check turned out to be the same that was designed to. The check for the girder and columns turned out to be a little different, which could be from the assumed weights or also using the newest codes and standards. The girder came out to be a W21x62, but was designed at a W24x62. This difference could be from different design practices and different loads assumed.

LIVE LOADS

Chapter 4 of ASCE7-10 provided the live loads for operating rooms, patient rooms, and corridors above first floor as 60psf, 40psf, and 80psf respectively. For the spot checks the spans crossed to different occupant rooms, so whichever occupancy had the higher live load is the one load that controlled. None of the tributary areas are big enough to use live load reduction factors.

Floor Live loads

Area	ASCE7-10 Loads
Lobby/Corridor 1st Floor	100psf
Corridors above1st Floor	80psf
Operating Rooms	60psf
Patient Rooms	40psf

Wind Loads

For the wind load calculations the MWFRS directional procedure was used to determine the lateral loads and the equations used to perform this method were taken from ASCE7-10 chapter 27. It turned out to be that the UMCP structure is flexible. Since UMCP has such a large area, with a wind speed of 120mph, the wind ended up controlling over the seismic loads. All supporting calculations can be found in Appendix 5.

A diagram showing the wind pressure coming from East/West and North/South for those facades is shown below in figure 7 and figure 8. According to ASCE7-10 the parapets also needed to be taken as a separate practice, and are not included in the figures below. Since the UMCP building is curved the structure will catch more wind, but this discrepancy will be better evaluated during the next technical report because it was assumed that the curving face will act like a perfectly horizontal face. Through these calculations, the base shear for the East/West and North/South came out to be 1372kips and 2034kips, respectively. It was proven that the greater the area the more base shear will occur in the building.

Windward Pressure East/West										
floor	z	li	kz	q	windward, p	Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	78	0.85	26.63	22.66 (+/-) 4.79	27.45	18.20	27.70	18.37	0
2	17	78	0.87	27.26	23.19 (+/-) 4.91	28.10	224.96	27.70	37.81	17
3	35	78	1.01	31.65	26.92 (+/-) 5.70	32.62	238.77	27.70	34.57	18
4	49	78	1.085	34.00	28.92 (+/-) 6.12	35.04	224.44	27.70	30.25	14
5	63	78	1.142	35.78	30.44 (+/-) 6.44	36.88	236.23	27.70	30.25	14
6	77	78	1.198	37.54	31.93 (+/-) 6.76	38.69	247.82	27.70	30.25	14
roof	91	78	1.242	38.92	33.11 (+/-) 7.01	40.11	21.90	27.70	15.12	14
						Σ	1194.12		178.25	

B	78.00
g_q	3.40
gv	3.40
c	0.20
z(bar)	54.60
L_z	552.98
b(bar)	0.65
ά	0.15
Vz(bar)	123.61
l	500.00
ε	0.20
h	91
L	457.5
V	120.00

G	1.06
n_a	0.60
I_z	0.18
Q	0.88
N_1	2.69
R_n	0.07
β	0.01
g_R	4.07
$R_{h,n}$	2.04
$R_{L,n}$	34.27
$R_{B,n}$	1.75
R_h	0.37
R_L	0.03
R_B	0.41
R	0.79

Base Shear	Over Turning Moment
1372.37 k	59136.42 k-ft

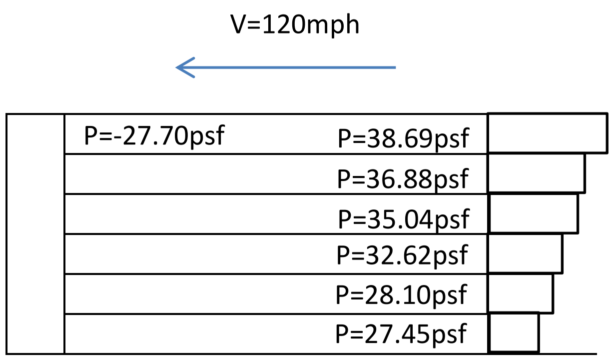


FIGURE 8: EAST/WEST WIND LOAD VARIABLES, LOADS, & PRESSURE DIAGRAM

Windward Pressure North/South										
floor	z	li	kz	q	windward, p	Windward Pressure	Windward Force	Leward, p	Leward Force	Floor Hight
1	0	78	0.85	26.63	18.85 (+/-) 4.79	23.64	91.94	24.22	94.19	0
2	17	78	0.87	27.26	19.29 (+/-) 4.91	24.20	193.75	24.22	193.91	17
3	35	78	1.01	31.65	22.40 (+/-) 5.70	28.09	205.65	24.22	177.29	18
4	49	78	1.085	34.00	24.06 (+/-) 6.12	30.18	193.31	24.22	155.13	14
5	63	78	1.142	35.78	25.32 (+/-) 6.44	31.77	203.46	24.22	155.13	14
6	77	78	1.198	37.54	26.57 (+/-) 6.76	33.32	213.44	24.22	155.13	14
roof	91	78	1.242	38.92	27.54 (+/-) 7.01	34.55	110.64	24.22	77.56	14
						Σ	1120.24		914.15	

B	457.50
g_q	3.40
gv	3.40
c	0.20
z(bar)	54.60
L_z	552.98
b(bar)	0.65
ά	0.15
Vz(bar)	123.61
l	500.00
ε	0.20
h	91
L	78
V	120.00

G	0.88
n_a	0.60
I_z	0.18
Q	0.78
N_1	2.69
R_n	0.07
β	0.01
g_R	4.07
$R_{h,n}$	2.04
$R_{L,n}$	5.84
$R_{B,n}$	10.24
R_h	0.37
R_L	0.16
R_B	0.09
R	0.39

Base Shear	Over Turning Moment
2034.39 k	59284.17 k-ft

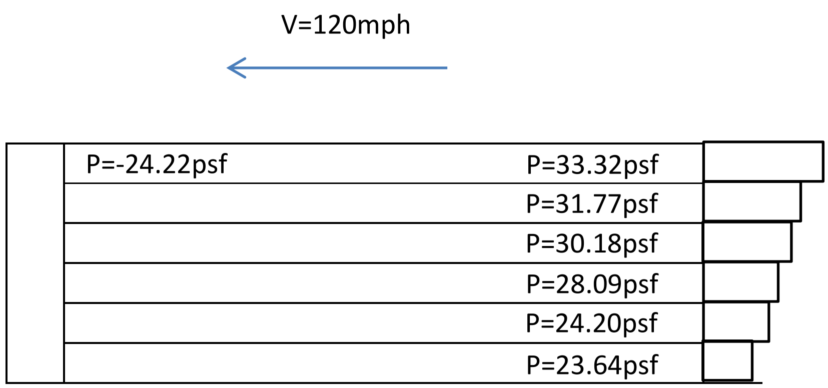


Figure 9: East/West Wind Load Variables, Loads, & Pressure Diagram

Seismic Loads

For the seismic design process, ASCE7-10 chapter 12 was applied. The USGS Earthquake Ground Motion Parameter Application was used to find the seismic response coefficients (S_1 and S_s) for Princeton, New Jersey. Since all of the floors have the same floor plans and use the same decking, each floor weighs the same. The roof weighs more due to the fact that the mechanical equipment is so heavy. Also, the response modification factor value, R, changes from 3.25 to 3.5 in the North/South and East/West direction since the framing is moment resisting in the one direction and braced in the other. Figure 9 shows the story shear forces in each direction and the calculations for determining these values are located in Appendix 6.

North-South Direction Loading

T= 0.590 s
k= 1.250
V_b= 600 kips

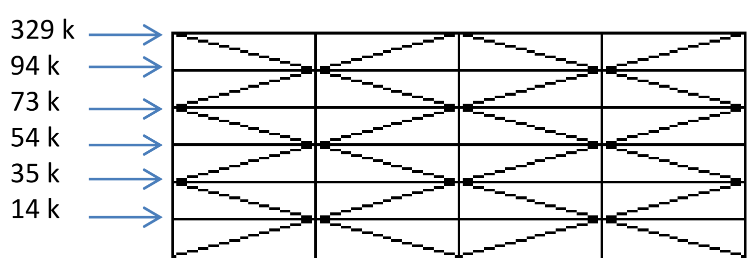
Floor	$h_{i\,(ft.)}$	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i (kips)
Roof	14	91	6374	1791488	0.548	329
6	14	77	2254	514123	0.157	94
5	14	63	2254	400064	0.122	73
4	14	49	2254	292213	0.089	54
3	18	35	2254	191884	0.059	35
2	17	17	2254	77806	0.024	14
		Σ	17644	3267578		600

East-West Direction Loading

T= 1.034 s
k= 1.630
V_b= 371 kips

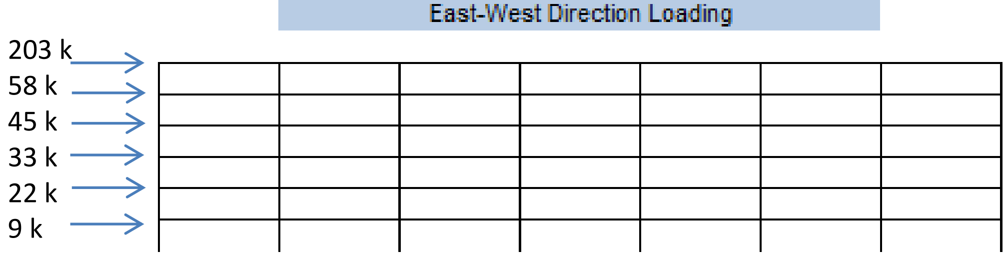
Floor	$h_{i\,(ft.)}$	h (ft)	w (kips)	$w*h^k$	C_{VX}	f_i (kips)
Roof	14	91	6374	1791488	0.548	203
6	14	77	2254	514123	0.157	58
5	14	63	2254	400064	0.122	45
4	14	49	2254	292213	0.089	33
3	18	35	2254	191884	0.059	22
2	17	17	2254	77806	0.024	9
		Σ	17644	3267578		371

North-South Direction Loading



Overturning Moment
45,937.5 k-ft.

East-West Direction Loading



Overturning Moment
28,336.5 k-ft.

FIGURE 10: NORTH/SOUTH SEISMIC LOAD VARIABLES, LOADS, & FORCE DIAGRAM
NOT DRAWN TO SCALE

Conclusion

After analyzing the framing and foundation systems, much knowledge was gained on the structural system. After calculating and examining the steel members it was determined that the author's analysis closely matches that of the original design.

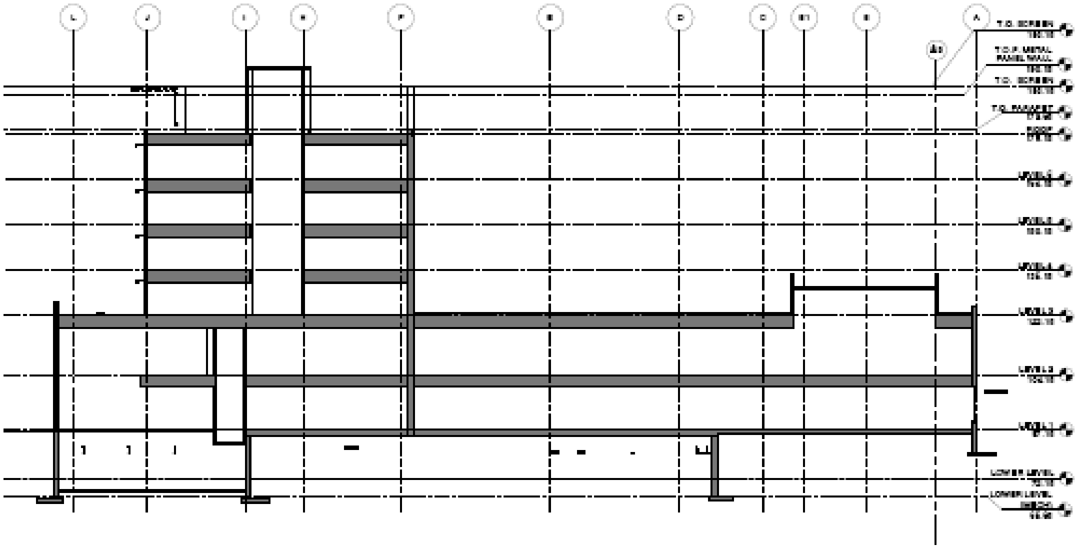
It was not a surprise when the calculations for the wind analysis came out to have a higher base shear than the seismic base shear. The wind controls on total base shear, but the lateral system must work together to take on both wind and seismic forces.

A problem with the spot checks, even though the same or close to the same size members were determined, might have be the loads assumed or the design methods were different. Also, if a beam would span into two areas, the greater load was taken into account instead of splitting the load on beam. More research will be applied on this subject in the next tech report.

Since this building utilizes operating rooms with tedious procedures, it might be a good idea to check the vibration in the floor system. If the vibrations are too high then it would be in the best interest to change the steel system to a concrete system. This would cut down on floor vibrations, making it safer for medical operation to occur.

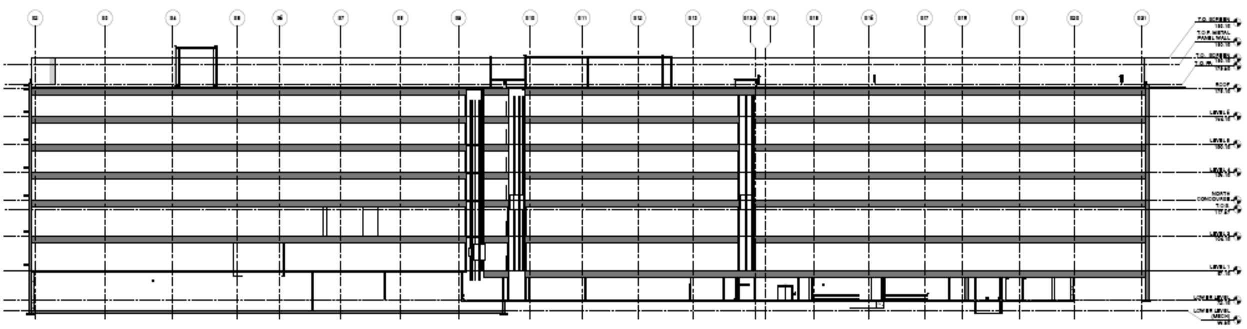
Appendices

Appendix 1: Architectural Sections & Plans



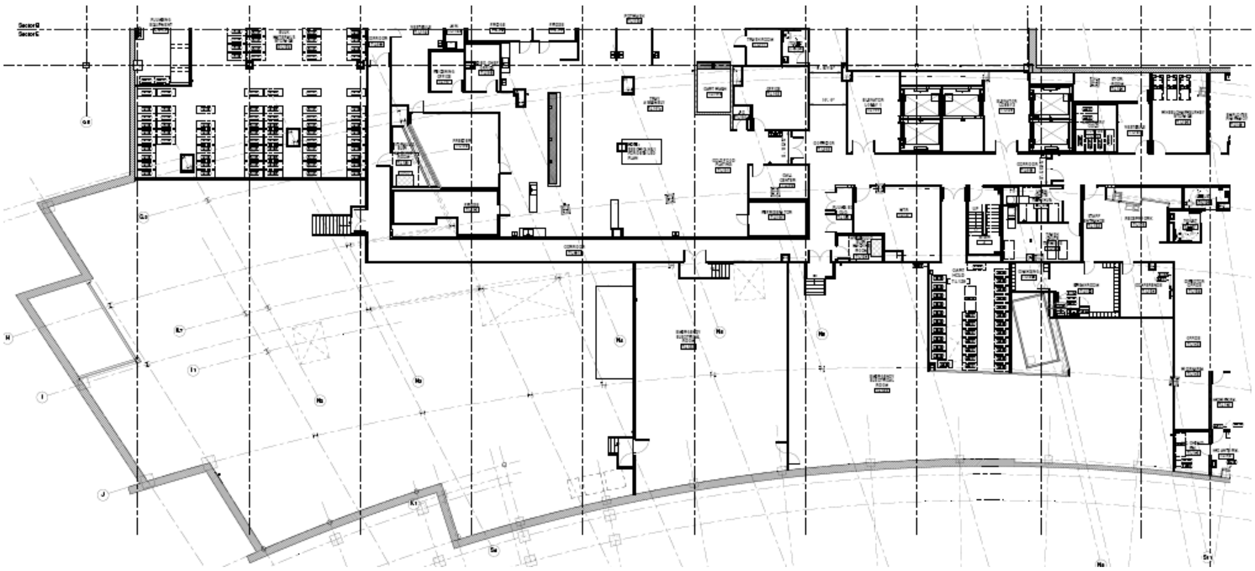
EAST/WEST SECTION

COURTESY OF TURNER CONSTRUCTION



NORTH/SOUTH SECTION

COURTESY OF TURNER CONSTRUCTION



TYPICAL WEST END FLOOR PLAN

COURTESY OF TURNER CONSTRUCTION

TYPICAL WEST END FLOOR PLAN

COURTESY OF TURNER CONSTRUCTION

Appendix 2: Snow Load

Alex Burg | Tech. Report 1 | Snow load | 1/2

Flat Roof Snow Loads, p_f

$p_f = 0.7\,C_e\,C_t\,I_s\,p_g$

$I_s = 1.10$ -> Table 1.5-2

$C_e = 1.0$ -> Table 7-2 (Exposure C, Partially Exposed Roof)

$C_t = 1.0$ -> Table 7-3

$p_g = 25$ lb/ft^2

$p_f = 0.7(1.0)(1.0)(1.10)(25)$

$\boxed{p_f = 19.25\ \text{psf}}$

Building steps back @ floor 3 -> Find Drift

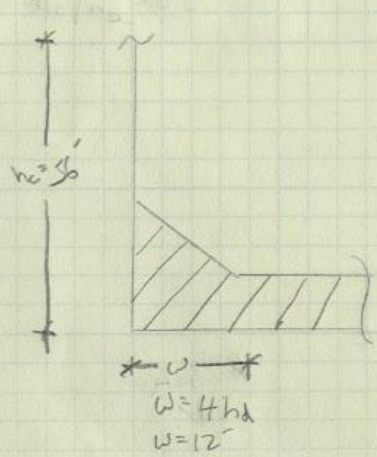
$l_u = 80$ ft

$h_d = 0.43(80)^{1/3}(25+10)^{1/4} - 1.5$

$h_d = 3'$ fig 7.9

$\gamma = 0.13\,p_g + 14$
$= 0.13(25) + 14$
$= 17.25$

$h_b = p_s/\gamma$
$= 19.25/17.25$
$= 1.1$



$p_d = h_d\,\gamma$
$= 3(17.25)$
$= \boxed{51.75\ \text{psf}}$

$h_c/h_b = 56/1.1$
$= 51 > 0.2$
∴ Calculate Drift

Alex Burg	Tech. Report 1	Snow load	2/2

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Drifts for a Parapet

l_u shall be taken as the length of the roof E-W since this is the longest length

The roof is curved as well, but just take the whole length of the roof

$l_u = 451.5'$

$h_d = 0.43\,(451.5)^{1/3}\,(25+10)^{1/4} - 1.5$

$h_d = 6.5' \times 0.75 = 4.9$

$P_d = 4.9(17.25)$

$= \boxed{84.53 \text{ psf}}$

Appendix 3: Beam & Girder Spot Checks

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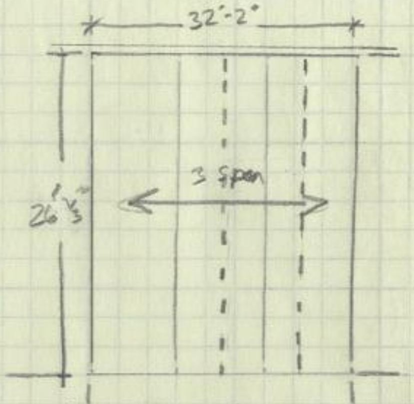
3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Deck

The tributary area was taken from the back left corner of UMCP since this is where the greatest spans are.



$\text{Span} = \frac{32.167'}{3}$

$\text{Span} = 10.72' \approx 10'\text{-}9"$

Fire Rating → 2 Hr. }
Unprotected Deck } Use: 3 1/4" LW, 1.5 VL

$A_t = 10.72' \times 26.3' = 282.3\ ft^2$

$L_L = 80$ psf

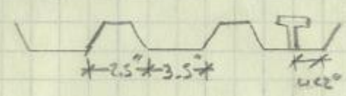
Superimposed Dead = 20 psf

MEP Load = 15 psf

Total Weight = 135 psf → Use: $\boxed{1.5VLR19 \quad t = 3"}$

Slab + Deck weight: 39 psf

$W_u = (1.2(15+20+39+\underbrace{10}_{\text{Beam weight}})+1.6(80))$
$= (260)10.72$
$= \underline{2.8^k}$



Beam

Max Moment

$M_u = \frac{w_u L^2}{8} = \frac{2.8(26.3)^2}{8} = 242.1'^k$

Max Shear

$V_u = \frac{w_u L}{2} = \frac{2.8(26.3)}{2} = 36.8^k$

Assumed 1 stud/rib weak position

$f'_c = 4000^k$, 3/4" ø

$Q_n = 17.2^k$ → Table 3-21

$Y_2 = t - a/2$
$Y_2 = 3 - 1/2$
$= 2.5"$ assume $a = 1"$

Alex Burg | Tech Report 1 | 2nd Floor Composite Beam Girder Deck | 2/4

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Try W 14x26

$M_n = 252^{'k} > 242.1^{'k}$ $V_n = 106^k > 36.8^k$

$\Sigma Q_n = 279^k$

$b_{eff} = \min \begin{cases} \dfrac{26.33 \times 12}{4} = 78.99'' \leftarrow \text{Controls} \\ 10.72 \times 12 = 128.64 \end{cases}$ $b_{eff} = 79''$

$a = \dfrac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \dfrac{279}{0.85(4)(79)} = 1.038 > 1 \therefore NG$

Try W 14x30 $I = 291 in^4$

$M_n = 273^{'k} > 242.1^{'k}$ V_n ✓

$\Sigma Q_n = 248^{'k}$

$a = \dfrac{273}{0.85(4)79} = 0.92 < 1 \therefore OK$

$I_{LB} = 572 \rightarrow$ Table 3-20

$\Delta_{LL} = 1/360$
$= \dfrac{26.3 \times 12}{360}$
$= 0.88$

$\Delta_{LL} = 5 \cdot w_{LL} L^4 / (384 (E) I_{LB})$ $w_{LL} = 100 \times 10.9 = 1090 \approx 1.1^k$

$= \dfrac{5(1.1)(26.3)^4 \times 1728}{384(29000)572}$

$= 0.72 < 0.88 \therefore ok$

$\Delta_{wc} = 1/240$
$= \dfrac{26.3 \times 12}{240}$
$= 1.32$

$I_{req} = 5 w_{CDL} l^4 / 384 E \Delta$ $w_{CDL} = 39_{psf} \times 10.72 = 418.08 \approx 0.42^k$

$= \dfrac{5(0.42)(26.3)^4 \times 1728}{384(29000)(1.32)}$

$= 118.1 < 291 \therefore$ No Camber

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Alex Burg | Tech. Report 1 | 2nd Floor Composite Deck Beam Girder | 3/4

Try W16x26

$\emptyset M_n = 270$
$\Sigma Q_n = 242$

$w_u = 1.2(15+20+39+26) + 1.6(100)$
$w_u = 280 \times 10.72$
$w_u = 3.0$ K/LF

$b_{eff} = \min \begin{cases} (26.33 \times 12)/4 = 79'' \leftarrow \text{controls} \\ 10.72 \times 12 = 128.6'' \end{cases}$

$M_u = \frac{3(26.3)^2}{8} = 259 < 270$ ✓

$a = \frac{\Sigma Q_n}{0.85(f'_c)(b_{eff})} = \frac{242}{0.85(4)\,79} = 0.9 < 1'' \therefore$ OK

Studs

$\frac{242}{17.2} \times 2 = 28$ Studs

$I_b = 619 \text{ in}^4 \rightarrow$ table 3-20 $I = 301 \text{ in}^4$

Live Load Deflection Check.

$\Delta_{LL} = l/360 = \frac{26.3 \times 12}{360} = 0.88''$

$\Delta_{LL} = \frac{5 \times w_{LL}\, l^4}{384(E)(I_{LB})} = \frac{5(1.1)(26.3)^4 \times 1728}{384(29000)\,619}$

$w_{LL} = 100 \times 10.9 = 1090 \approx 1.1$ K

$= 0.66 < 0.88 \therefore$ OK

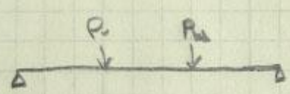
Deflection Due to Wet Concrete Check

$\Delta_{wc} = l/240 = \frac{26.3 \times 12}{240} = 1.32''$

$w_{DL} = 39 \times 10.72 = 418.08 \approx \underline{0.42}^K$

$I_{req} = \frac{5\, w_{DL}\, l^4}{384(E)\Delta} = \frac{5(0.42)(26.3)^4 \times 1728}{384\,(29000)(1.32)} = 118.1 < 301 \text{ in}^4 \therefore$ No Camber

Girder



$P_u = 3$ K/LF $\times 26.3/2$
$= 39.5^K$

$V_u = 39.5^K$

$M_u = \frac{P_u\, l}{3} = \frac{39.5(32.167)}{3} = 423.5'^K$

$Q_n = 21.2$
$Y_2 = 2.5''$
$a = 1''$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Alex Burg | Tech. Report 1 | 2nd Floor Composite Deck Beam Girder | 4/4

Try W18x40

$\phi M_n = 459^{k'}$ $\quad M_u = 423.5 + \frac{1.2(0.04)(32.167)^2}{8}$

$\Sigma Q_n = 353^k$ $\quad = 429.7 < 459$ ✓

$b_{eff} = \min \begin{cases} (32.167 \times 12)/4 = 96.5'' \Leftarrow \text{controls} \\ 26.3 \times 12 = 315.6'' \end{cases}$

$a = \frac{353}{0.85(4)\,96.5} = 1.08'' > 1''$ NG

Try W21x62

$\phi M_n = 759$ $\quad M_u = 423.5 + \frac{1.2(0.062)(32.167)^2}{8}$

$\Sigma Q_n = 318$ $\quad = 433.1 < 759$

$a = \frac{318}{0.85(4)(96.5)} = 0.97'' > 1''$ ∴ OK

Studs

$\frac{353}{21.2} \times 2 = 34$ Studs

$I_{lb} = 2070\,in^4$ $\quad I = 1330\,in^4$

↳ (3-20)

Live Load Deflection Check

$\Delta_{LL} = l/360 = (32.167 \times 12)/360 = 1.1''$ $\quad P_{LL} = 1.6(100) \times \frac{26.3}{2} \times 10.72$

$\Delta_{LL} = P_{LL} L^3/(28 E I_{LB}) =$ $\quad = 22.6^k$

$= \frac{22.6 \times (32.167)^3 \times 1728}{28 \times 29000 \times 2070} = 0.77'' < 1.1''$ OK

Check Deflection Due to Wet Concrete

$\Delta_{wc} = l/240 = (32.167 \times 12)/240 = 1.61''$ $\quad w_{cal} = 39\left(\frac{26.33}{2}\right) = 0.51$

$P_{col} = (0.42)\left(\frac{26.33}{2}\right) = 5.5$

$I_{req} = (35 w_{col} l^4 + 96 P_{col} l^3)/2688 E\Delta =$

$= \frac{(35(0.51)(32.3)^4 + 96(5.5)(32.3)^3)1728}{2688(29{,}000)(1.61)}$

$= 245 < 1330$ ∴ OK

Appendix 4: Column Spot Checks

Alex Burg | Tech. Report 1 | Interior + Exterior Columns | 1/3

Roof Loads

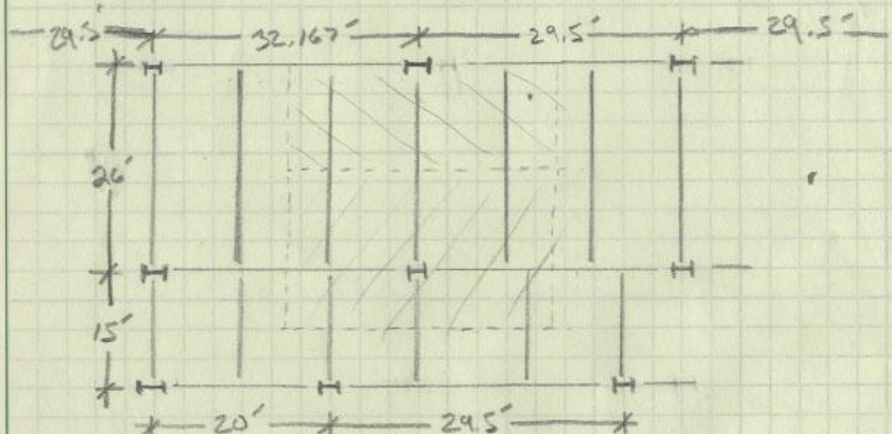
Decking + Framing = (0.78 + 10) psf

Snow Load = 25 psf ← figure 7-1

Mech Equip = 150 psf ← Assumed

$W_u = 1.2(10.78 + 150) + 1.6(25)$

$= \underline{233 \text{ psf}}$



Tributary Area (For an Interior column on the 6th floor to Roof)

$A_T = (32.167/2 + 29.5/2) \times (26/2 + 15/2)$

$= 632.1 \text{ ft}^2$

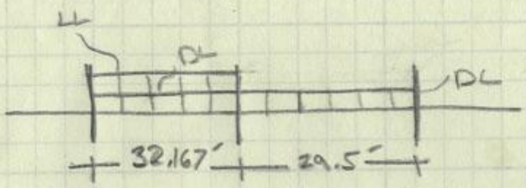
$P_u = W_u A_T$

$= 233(632.1)$

$= \underline{147.3}^{k}$

Alex Burg | Tech. Report 1 | Interior + Exterior Column 2/3

Pattern Loading



$L_L = 80$

$D_L = 39\,\text{psf (Decking)} + 10\,\text{psf (Metal Framing)}$

$W_{uDL} = 1.2(49)(29/2 + 15/2) = 1.2^k$

$W_{uLL} = 1.6(80)(29/2 + 15/2) = 2.6^k$

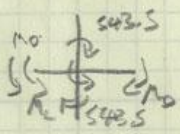
Fixed End Moment

$\frac{w_u l^2}{12}$

$M_{DL} = \frac{1.2(32.167)^2}{12} = 103.5'^k$

$M_{DL} = \frac{1.2(29.5)^2}{12} = 87.0'^k$

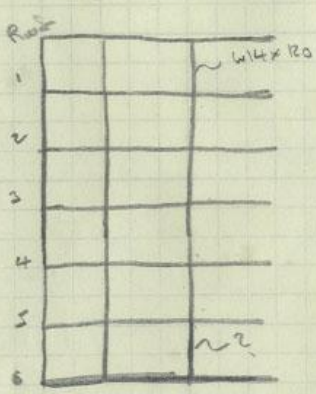
$M_{LL} = \frac{2.6(32.167)^2}{12} = 224.2^k$



$M_u = \frac{(103.5 + 224.2) - 87}{2} = 120.4'^k$

$P_{eq} = P_u + 24 M_u / d$
$= 147.3^k + 24(120.4)/14$
$= 353.7^k$

$\boxed{\text{Table 4-1 use } W14\times48 \text{ assuming } k=1}$



$P_u = 147.3 + 5(1.2 + 2.6)\left(\frac{32.167 + 29.5}{2}\right)'$

$P_u = 733.1^k$

Since the spans + Loading are the same as the 6th floor

$M_u = 120.4$

$P_{eq} = 733.1 + 24(120.4)/14$
$= 939.5$

$\boxed{\text{Table 4-1 use a } W14\times90}$

Alex Burg | Tech. Report 1 | Interior & Exterior Column | 3/3

Tributary Area (for an Exterior column on the 6th Floor to roof)

$A_t = (32.167/2 + 29.5/2) \times (26/2)$

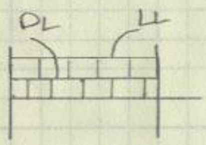
$= 400.8\ ft^2$

$P_u = W_u A_t$

$= 233\,(400.8)$

$= \underline{93.4^k}$

Pattern Loading



$L_L = 80$

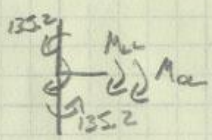
$D_L = 49\,psf$

$W_{u_{DL}} = 1.2(49)(26)$
$= 1.5^k$

$M_{DL} = \frac{1.5\,(26)^2}{12} = 84.5$

$W_{LL} = 1.6\,(80)(26)$
$= 3.3^k$

$M_{LL} = \frac{3.3(26)^2}{12} = 185.9$



$M_u = \frac{185.9 + 84.5}{2} = \underline{135.2'^k}$

$P_{eq} = P_u + 24 M_u/d$

$= 93.4 + 24(135.2)/14$

$= \underline{325.2^k} \rightarrow \boxed{\text{use } \underline{W14\times48}}$

1st floor exterior column

$P_u = 93.4 + 5(1.5+3.3)(26/2)$
$= \underline{405.4^k}$

$M_u = \underline{135.2'^k}$

$P_{eq} = 405.4 + 24(135.2)/14$
$= 637.2$

$\boxed{\text{Table 4-1 use a } \underline{W14\times68}}$

Appendix 5: Lateral Wind Loads

Alex Burg | Tech. Report 1 | Wind loads | 1/2

Constants

$V = 120$ mph (Figure 26.5-1B)

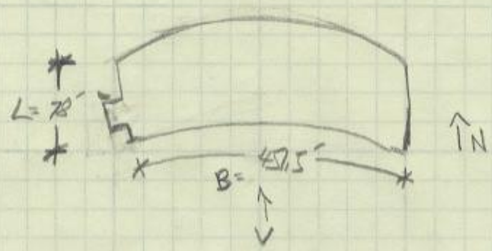
$I = 1.0$ (Table 1.5-2)

$K_d = 0.85$ (Table 26.6-1)

$K_{zt} = 1.0$ (Table 26.8-1)

$GC_{pi} = \pm 0.18$ (Enclosed Building)

Exposure C



Approximate Natural Frequency

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = \frac{7098}{91} = 78$$

$h = 91 < 4(78) = 312$ ✓ approximate frequency Applies

$$n_a = \frac{22.2}{h^{0.8}} \quad (26.9\text{-}2)$$

$$n_a = \frac{22.2}{91^{0.8}} = 0.6 < 1 \Rightarrow \text{Flexible}$$

$R_h = 0.37$
$R_L = 0.16$
$R_B = 0.09$

$R_{h,n} = 4.6 n_a h / \bar{V}_{\bar{z}}$
$= 2.04$

$R_{L,n} = 15.4 n_a L / \bar{V}_{\bar{z}}$
$= 5.84$

$R_{B,n} = 4.6 n_1 B / \bar{V}_{\bar{z}}$
$= 10.24$

Gust Factor

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

$$\boxed{G = 0.88}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .2\left(\frac{33}{54.6}\right)^{1/6} = 0.18$$

Constants
$g_Q = 3.4$
$g_v = 3.4$
$c = .2$ Table (26.9-1)
$\bar{z} = .6h = .6(91) = 54.6$

$$g_R = \sqrt{2\ln 3{,}600 n_1} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}}, \quad n_1 = n_a$$

$g_R = 4.07$

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

$Q = 0.8$

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

$\bar{\epsilon} = 1/5$ $\bar{b} = 0.65$
$l = 500$ $\bar{\alpha} = 1/6.5$
$L_{\bar{z}} = 553$

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

$R = 0.39$

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

$\beta = 1\%$ $R_n = 0.07$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = 2.69$$

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

$\bar{V}_{\bar{z}} = 123.61$

Alex Burg | Tech. Report 1 | Wind loads | 2/2

<u>Velocity pressure exposure, K_z</u> $\Rightarrow$ 1st Floor

$z = 1.242$ (interpolated from table 27.3-1)

$Z_g = 900$

$K_z = 2.01 (15/900)^{2/9.5}$

$\boxed{K_z = 0.85}$

<u>Velocity Pressure, q_z</u> $\rightarrow$ 1st floor only

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

$= 0.00256 (0.85)(1)(0.85)(120)^2$

$\boxed{q_z = 26.6 \frac{lb}{ft^2}}$

<u>Wind Pressure, p</u>

$p = q\, G C_p - q_i (G C_{pi})$

$= 26.6 (0.68)(0.8) - 26.6(\pm 0.18)$

$= 14.5 \pm 4.8$

$C_p = .8$ windward 27.4-1
$C_p = -0.5$ leeward wall (long wall)
$C_p = -0.2$ leeward wall (short wall)

See spreadsheet for additional work

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Appendix 6: Lateral Seismic Loads

Alex Burg | Tech. Report 1 | Seismic Load for lateral System | 1/6

$S_s = 0.310$ (USGS) $F_a = 1.552$ 11.4-1 Site Class D

$S_1 = 0.064$ (USGS) $F_v = 2.4$ 11.4-2

$S_{ms} = F_a S_s = 1.552(0.31) = \underline{0.481}$

$S_{m1} = F_v S_1 = 2.4(0.064) = \underline{0.154}$

$S_{DS} = (2/3) S_{ms} = 2/3(0.481) = \underline{0.321}$

$S_{D1} = 2/3 S_{m1} = 2/3(0.154) = \underline{0.1024}$

Design Category, 11.6-1 => $\underline{B}$

I => 1.25 (Table 1.5-2)

R = 3.25 (Table 12.2-1) Steel ordinary concentrically braced frames N-S Direction

$C_t = 0.02$ $x = .75$ (Table 12.8-2)

$h_n = 91'$

$T_a = C_t h_n^x = 0.02(91)^{.75} = 0.59$

$T_L = 6$ sec (Fig 22-12) $C_u = 1.7$ (Table 12.8-1)

$T = C_u T_a = 1.7(0.59) = 1.003$

$T_L > T$ use $C_s = \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{0.1024}{1.003\left(\frac{3.25}{1.25}\right)}$

$C_s = \dfrac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \dfrac{0.321}{\left(\frac{3.25}{1.25}\right)} = 0.123$

$C_s = 0.039 > 0.01$ ✓

$C_s = 0.039 < 0.123$

So

$C_s = 0.039$

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Alex Burg | Tech Report 1 | Seismic Design for lateral System | 2/6

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Base Shear

$V = C_s \cdot W$

Assuming That Floors 2 - 6 are equal weight
+ 1st Floor is its own weight

Constants Floor Weights

Slab + Decking = 39 psf MEP = 15psf

Steel Beam + Column Allowance = 10psf

Total Dead Load = 64psf

Constant Roof Weights

Decking = 1.78psf Snow = 19.25

Steel Beam + Column Allowence = 10psf

MEP Equipment = 150 psf

Total, W_h = 181psf

Floor + Roof Area

Floors 2-6 + Roof are all equal areas

Area = $451.5' \times 78' = 35,217$ ft²

$W_{floor} = 35,217 \text{ ft}^2 \times 64 \frac{lbs}{ft^2}$

$= 2,254^k$

$W_{roof} = 35,217 \text{ ft}^2 \times 181 \frac{lb}{ft^2}$

$= 6,374^k$

$W_{total} = 6,374 + 2,254 \times 5$

$= 17,644^k$

Alex Burg | Tech. Report 1 | Seismic Design for Lateral Load | 7/6

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET

Base Shear

$V = C_s W$

$V = 0.039(17{,}644)$

$= 600^k$

Vertical Distribution

$F_{roof} = C_{vr} V$

$= 0.55(600)$

$= \underline{330}$

$F_6 = 0.16(600)$

$= \underline{96}$

$F_5 = 0.12(600)$

$= \underline{72}^k$

$F_4 = 0.09(600)$

$= \underline{54}^k$

$F_3 = 0.06(600)$

$= \underline{36}^k$

$F_2 = 0.02(600)$

$= \underline{12}^k$

$C_{vr} = \dfrac{W_{roof} h_r^k}{\sum_{i=1}^{n} w_i h_i^k}$

$C_{vr} = \dfrac{6{,}374(91)^{1.25}}{3{,}267{,}578}$

$= \underline{0.55}$

$C_6 = \dfrac{2254(77)^{1.25}}{3{,}267{,}578}$

$= \underline{0.16}$

$C_5 = \dfrac{2254(63)^{1.25}}{3{,}267{,}578}$

$= \underline{0.12}$

$C_4 = \dfrac{2254(49)^{1.25}}{3{,}267{,}578}$

$= \underline{0.09}$

$C_3 = \dfrac{2254(35)^{1.25}}{3{,}267{,}578}$

$= \underline{0.06}$

$C_2 = \dfrac{2254(17)^{1.25}}{3{,}267{,}578}$

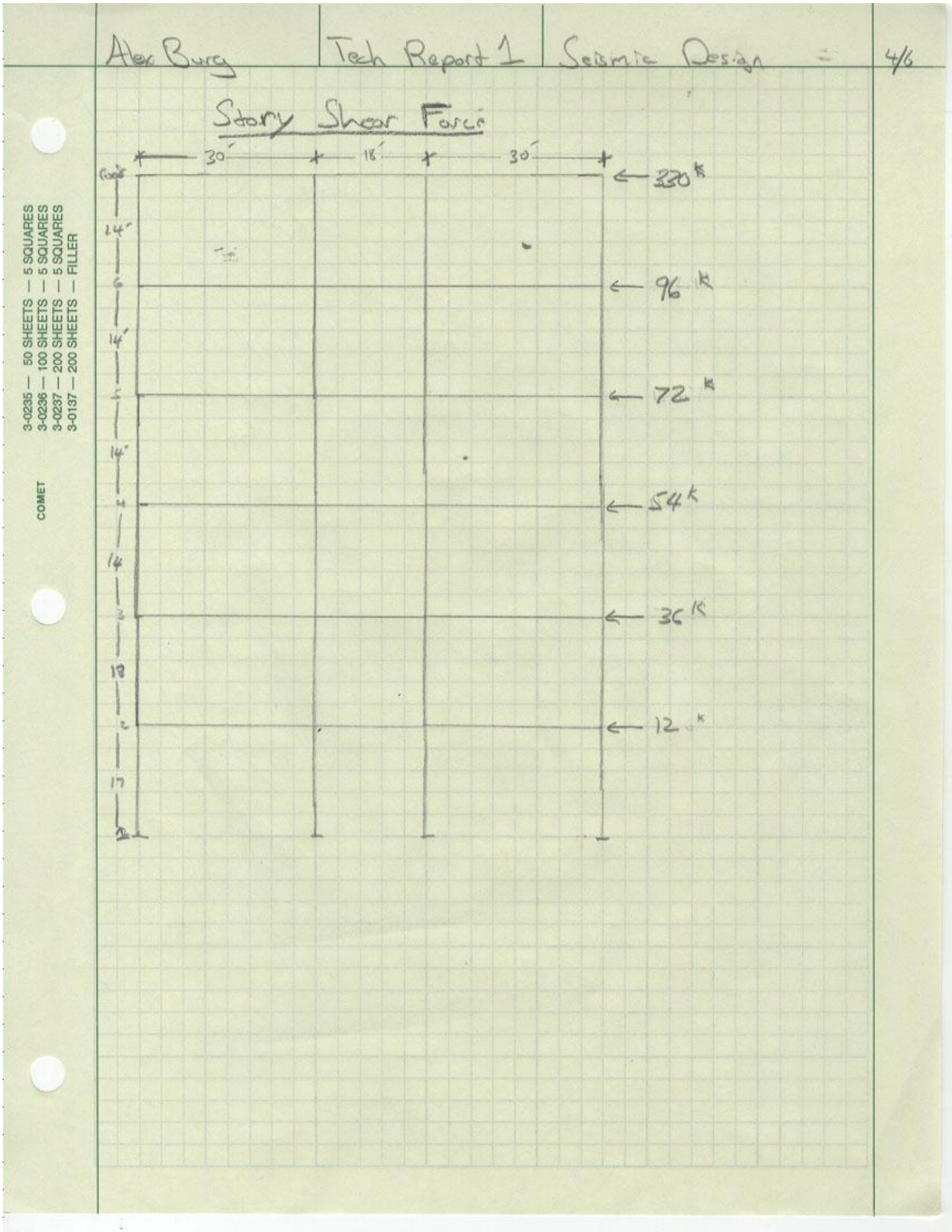
$= \underline{0.02}$

$\Sigma C = 1 \quad \therefore$ OK

$k = 1.25$ interpolated from eq. 12.8-12

$\sum_{i=1}^{n} w_i h_i^k = 6{,}374(91)^{1.25}$
$+ 2{,}254(77)^{1.25}$
$+ 2{,}254(63)^{1.25}$
$+ 2{,}254(49)^{1.25}$
$+ 2{,}254(35)^{1.25}$
$+ 2{,}254(17)^{1.25}$

$3{,}267{,}578$


Alex Burg
Tech Report 1
Seismic Design
4/6
Story Shear Force
30'
18'
30'
14'
14'
14'
14
18
17
330k
96k
72k
54k
36k
12k

Alex Burg | Tech Report 1 | Seismic Load | 5/6

The west to East Frames are Steel Ordinary moment Frames

Having an $R = 3.5$ (Table 12.2-1)

$C_t = 0.028$, $x = .8$ } (table 12.8-2)

$T_a = C_t h_n^x$ $C_u = 1.7$ (table 12.8-1)

$= 0.028(91)^{.8}$

$= 1.034$

$T = C_u T_a$

$= 1.7(1.034)$

$= 1.76 < T_L = 6$ use $C_s = \frac{S_{D1}}{T(R/I_e)} = \frac{0.1024}{1.76(3.5/1.25)} = 0.021 > 0.01$ ✓

$C_s = \frac{S_{DS}}{(R/I_e)} = \frac{0.321}{(3.5/1.25)} = 0.115 > 0.021$ ← controls

$C_s = 0.021$

Base Shear

$V = C_s \cdot W_{total}$

$= 0.021(17,644)$

$= 370.5^k$

Alex Burg | Tech Report 1 | Seismic Load | 6/6

Vertical Distribution

$K = 1.63$ interpolated from eq 12.8-12

$\sum_{i=1}^{n} = w_i h_i^{k} = 6{,}374(91)^{1.63} + 2{,}254(77)^{1.63} + 2254(63)^{1.63} + 2{,}254(49)^{1.63} + 2{,}254(35)^{1.63} + 2{,}254(17)^{1.63}$

$16{,}807{,}752$

$F_{roof} = 0.59(370.5) = \underline{218.7^{k}}$

$F_6 = 0.16(370.5) = \underline{59.3^{k}}$

$F_5 = 0.12(370.5) = \underline{44.5^{k}}$

$F_4 = 0.08(370.5) = \underline{29.6^{k}}$

$F_3 = 0.04(370.5) = \underline{14.8^{k}}$

$F_2 = 0.01(370.5) = \underline{3.7^{k}}$

$C_{vr} = \frac{6{,}374(91)^{1.63}}{16{,}807{,}752} = \underline{0.59}$

$C_6 = \frac{2{,}254(77)^{1.63}}{16{,}807{,}752} = \underline{0.16}$

$C_5 = \frac{2254(63)^{1.63}}{16{,}807{,}752} = \underline{0.12}$

$C_4 = \frac{2254(49)^{1.63}}{16{,}807{,}752} = \underline{0.08}$

$C_3 = \frac{2254(35)^{1.63}}{16{,}807{,}752} = \underline{0.04}$

$C_2 = \frac{2254(17)^{1.63}}{16{,}807{,}752} = \underline{0.01}$

$\Sigma C = 1$ ✓

3-0235 — 50 SHEETS — 5 SQUARES
3-0236 — 100 SHEETS — 5 SQUARES
3-0237 — 200 SHEETS — 5 SQUARES
3-0137 — 200 SHEETS — FILLER
COMET