James W. & Frances G. McGlothlin Medical Education Center
Virginia Commonwealth University
Richmond, VA

November 20, 2013

Professor Linda Hanagan
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Dear Professor Hanagan,

I am formally submitting Structural Technical Report #4 – Lateral System Analysis Study. As the name suggests, this report is a more thorough investigation of the lateral loads, calculated in Technical Report #2, that are applied in both the x- and y-directions to this building. Once wind and seismic loads were distributed by floor and frames in hand calculations, a computer generated model was created using RAM Structural System to have a better picture of the results. The building, specifically the structural steel concentrically braced frames, was checked for both standard strength and serviceability requirements. A table of contents and numbering of pages has been provided for ease of navigating this report. Calculations for distribution of loads, building properties, and strength checks have been done by hand, and therefore have been scanned to be inserted in to this report. While creating the computer model and analyzing the wind & seismic loads some approximation methods were used, but all pertinent information has been provided. I look forward to presenting my findings to you, other notable faculty, and my fellow classmates in the near future.

Sincerely,

Marissa Delozier

Enclosure: Report of Findings Related to Wind and Seismic Loadings on Structural Steel Concentrically Braced Frames for the James W. & Frances G. McGlothlin Medical Education Center

General Information

The James W. & Frances G. McGlothlin Medical Education Center is a 13-story building that has both a basement and small sub-basement located below ground level, which is at an elevation of 153 feet. Since the building was constructed following the demolition of the A.D. Williams Building, the foundation system is designed to accommodate existing conditions. The superstructure of the building is composed of a braced moment frame system with concrete slabs on metal decking. Both the 13th Floor and the rooftop are homes to mechanical equipment, requiring added strength. A bridge traveling over E. Marshall Street connects the new building on the 2nd Floor with the existing Main Hospital 1st Floor. Further information about the building and its location in downtown Richmond, Virginia can be found on the following pages.

NOTE: To decrease confusion and provide easier reading, from this point in the report and forward the James W. & Frances G. McGlothlin Medical Education Center will be referred to as VCU SOM project, short for Virginia Commonwealth University School of Medicine project.

Building Abstract

James W. & Frances G. McGlothlin Medical Education Center

Virginia Commonwealth University – Richmond, VA

Project Information

Type of Building :	Multipurpose Education Facility
Functions :	Administrative/Classrooms/Research
Size :	220,000 GSF
Height :	13 stories
Time Frame :	Oct. 2009 – March 2013
Cost :	$159 million
Delivery :	Design–Assist–Build

Project Team

Owner :	Virginia Commonwealth University
CM :	Gilbane Building Company
Architect :	Ballinger
Structural + MEP :	Ballinger
Exterior Façade :	Pei Cobb Freed & Partners
Civil :	Draper Aden Associates
Geotechnical :	Geotech, Inc.

Architectural

– Erected following demolition of 8-story A.D. Williams Building, which previously housed VCU School of Medicine

– Exterior façade was designed by internationally acclaimed design firm Pei Cobb Freed & Partners

Sustainability

– Climate Wall System: double-layered glass walls on South & West facades trap & exhaust heated air

– Recovery Wheels: recover exhausted air & use contained energy to heat & cool building

– Storm Water Retention: collect water from roof to be used in toilets/urinals

Structural

– Drilled pier/slab-on-grade system works in conjunction with pre-existing caissons

– Structural steel braced moment frame system

– Bridge connects 2nd Floor of building to adjacent Main Hospital 1st Floor across E. Marshall Street

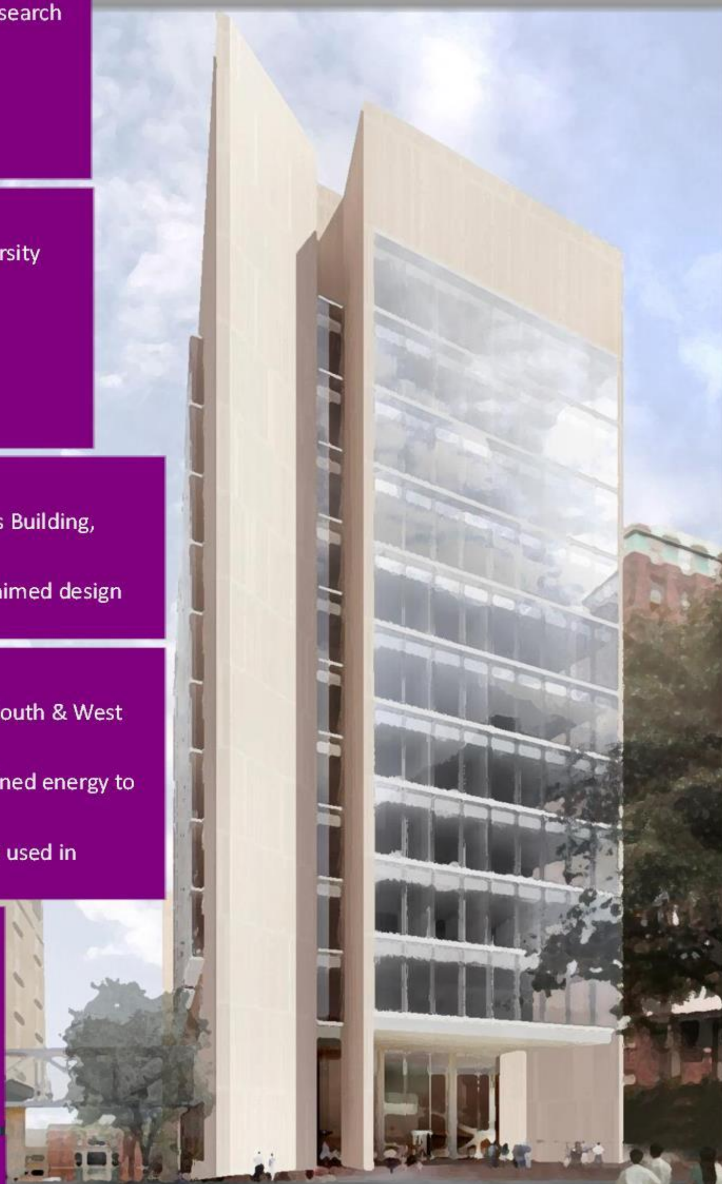
MEP

– 6 Air Handling Units serve the Lobby, Student Forum, Auditorium, and Chilled Beam system

– Cooling Tower on roof removes heat from 3 Chillers

– Use of Recovery Wheels saves 450 tons of cooling

– Daylighting sensors throughout building ensure energy is conserved



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Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2013/mnd5036/

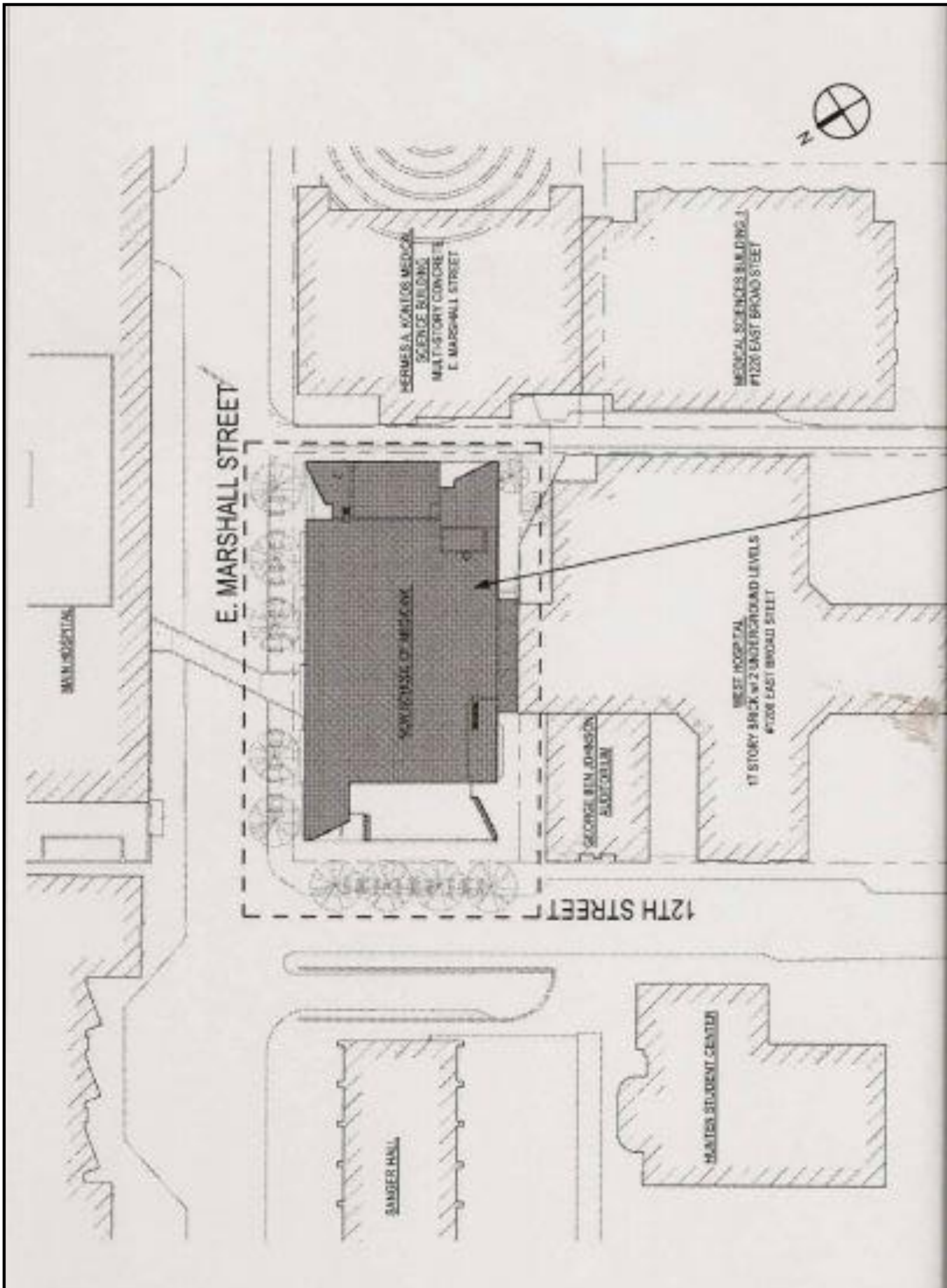
Executive Summary from Technical Report 1

The following technical report is a thorough overview of the existing conditions of the structural system found in the newly constructed James W. & Frances G. McGlothlin Medical Education Center. This report is composed of detailed descriptions of the drilled pier/slab-on-grade system, floor framing, braced moment frame system, roof scheme, bridge connecting to an adjacent structure, and all other components that contribute to the strength of the structure.

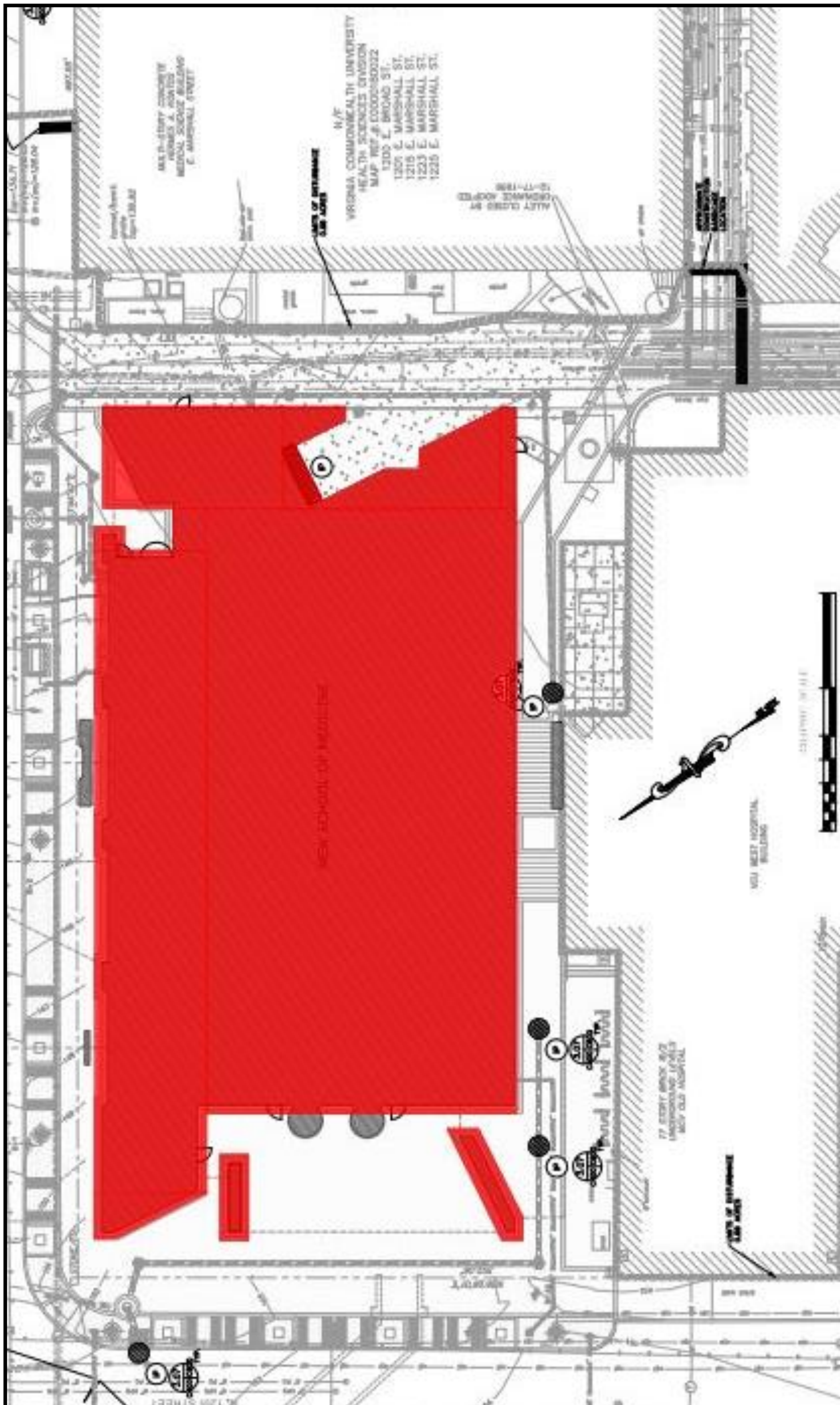
Though it is said that the sum is greater than its parts, the structural apparatuses that compose this project are diverse, complicated systems that must be thoroughly examined to fully appreciate the building. Many challenges exist surrounding the project: the site location, building size, intended function, connection to existing structures, and many more. This report is only the first investigation in to the structure of the James W. & Frances G. McGlothlin Medical Education Center – further analysis and study will be necessary to fully comprehend the magnitude of these systems.

In order to provide background information, floor plans, bays, columns, and other elements from the structure are referenced throughout the report and can be found in the appendices for further examination. State and national codes used in the design of the structure are also cited in the following report; these codes, more specifically loading values, will be utilized in further research and subsequent technical reports.

Location Plan



Site Plan



Reference Documents

In the preparation of the calculations found on the following pages, several documents outside the construction drawings and specifications were referenced. The main source of information was the American Society of Civil Engineers (ASCE) 7-05 code, specifically for both wind and seismic loads. All of the necessary variables, equations, and values needed to calculate the loadings and base shears were found from this document. A document utilized in the calculation of both roof and floor loadings was the Vulcraft Steel Roof and Floor Deck catalog. The American Institute of Steel Construction (AISC) 2005 code was also used for gravity loadings, to estimate size and weight.

Gravity Loads

Live Loads			
Floor/Area	Design (psf)	ASCE 7-05 (psf)	Typical Use
Sub-basement	250	150	Mechanical†
Loading Dock	350	—	—
Basement	100	100	Offices + Storage
1st	↑	100	Lobby
2nd		60	Assembly (fixed seat)
3rd		60	Assembly (fixed seat)
4th		80	Offices + Corridors
5th		80	Classrooms + Corridors
6th		↑	↑
7th			
8th			
9th			
10th		↓ 80	↓ Classrooms + Corridors
11th	↓	80	Offices + Corridors
12th	100	80	Offices + Corridors
13th	150	150	Mechanical†
Roof	45	20	Flat Roof

† This value was assumed.

Dead Loads°	
System	Assumed Loads (psf)
Decking	2
Insulation	2
Roofing	20
Misc. DL	10

°No known dead loads were referenced in the contract documents, so values were assumed based on common practice.

Snow Loads		
Area	Design (psf)	ASCE 7-05 (psf)
Ground	20	20
Roof	30 + drift	22*

* Value found with $P_f = 0.7 C_e C_t I P_g$

P_g = ground snow load

C_t = thermal factor

C_e = snow exposure factor

I = snow load importance factor

$P_f = 0.7(0.9)(1.0)(1.1)(20) = 14 \text{ psf}$

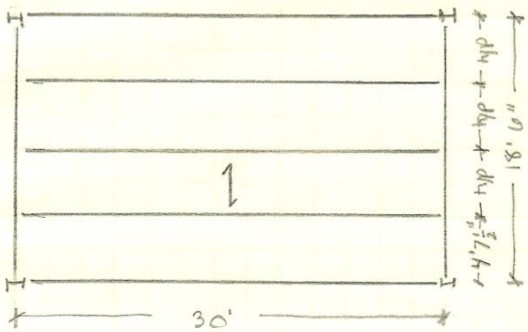
$14 \text{ psf} < 20 \text{ psf} \therefore$ NOT OK

$P_f = I P_g = 1.1(20) = 22 \text{ psf}$

Gravity Loads (cont.)

• Roof Construction

- Typical Roof Bay



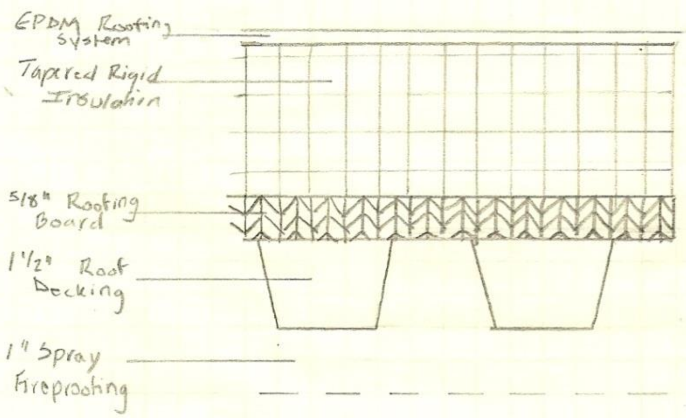
Design: 1½" wide rib galvanized steel deck

LL = 45 psf
DL = 20 (roofing) + 10 (misc. DL) + 2 (insulation) + 2 (decking) = 34 psf
SL = 30 + drift

$1.2DL + 1.6LL + 0.5SL = 1.2(34) + 1.6(45) + 0.5(30) = 128$ psf

clear span ~ 5'0" → using max const. span of 5'10" (> 5' ∴ ok✓)
assume wide rib — allowable total load = 154 psf > 128 psf ∴ ok✓
(excellent load carrying capacity)

Design: 1½" wide rib ∴ ok✓
1.46 psf < 2 psf assumed ∴ ok✓



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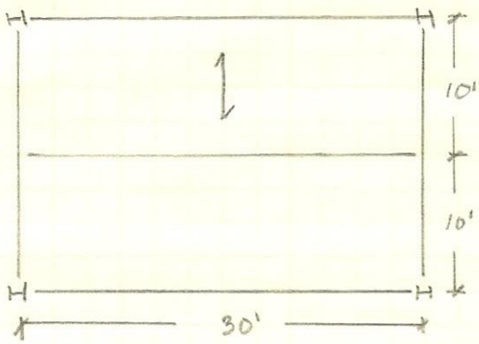
Gravity Loads (cont.)

• Floor Construction

-Typical Floor Bay → assume 4^{th} Floor



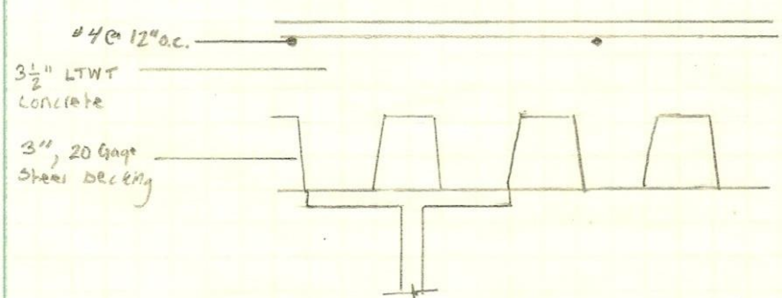
Design: 3", 20 gage decking

LL = 80 psf
DL = 15 (framing) + 10 (misc. DL)
(assumed)
$1.2DL + 1.6LL = 1.2(15+10) + 1.6(80)$
$= 158$ psf

Assume: 2 hr spray fireproofing → ≥ 3 1/4" LTWT conc. topping

t = 3.25 → total slab depth = 6.25
clr span = 10', 2 spans
max span = 14' 2"
3VLI19 decking → 168 psf > 158 psf ∴ OK ✓

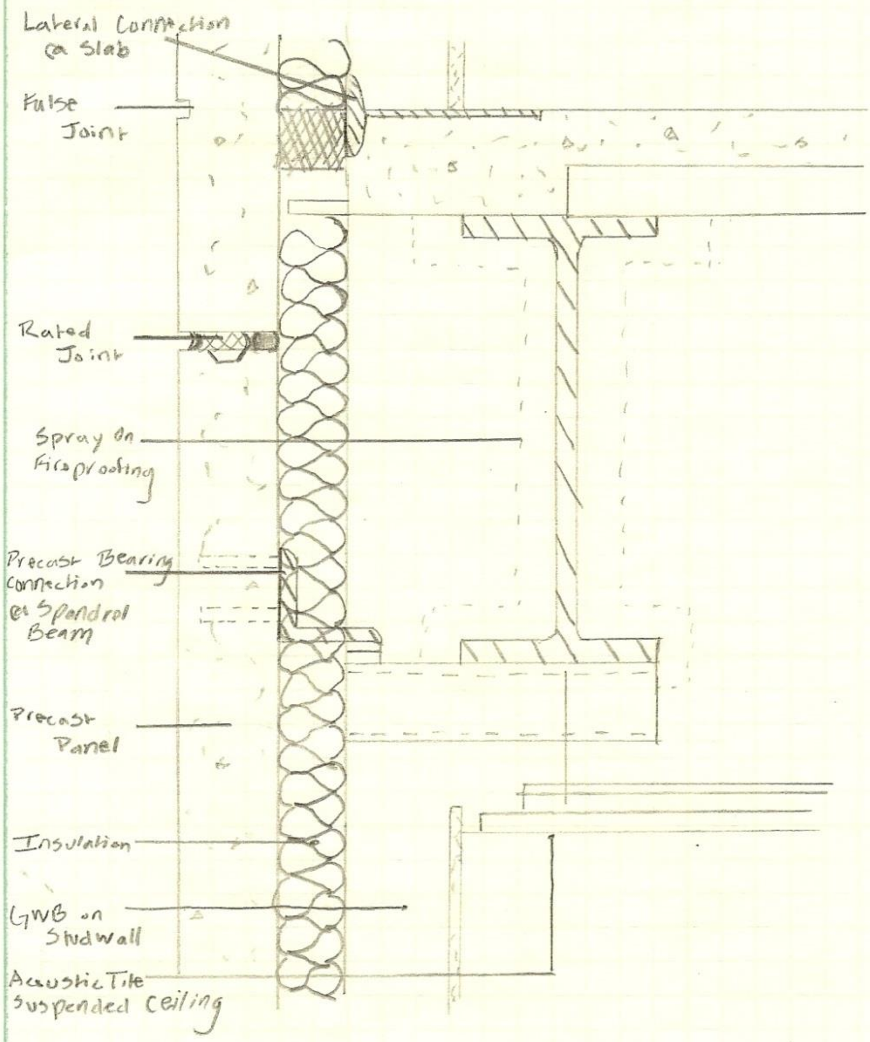
Design: 3", 20 gage + 3 1/2" LTWT conc ∴ OK ✓



Floor Construction Typical to Floors 4-12

Gravity Loads (cont.)

- Exterior Wall Construction - Pre-Cast Concrete Panels



Assumptions

- Precast panels → 130 pcf concrete
- Insulation → 4 pcf

Known

- Precast panels → 6" thick
- Insulation → 3 1/2" thick
- Panels cover roughly 60% of the building exterior

Exterior Wall Surface Areas

Side 1 = 34,800 sf
Side 2 = 17,400 sf
Side 3 = Side 1
Side 4 = Side 2   Total Surface Area = 104400 sf

$$\text{Weight} = \left[(130\,\text{pcf})(6''/12^{\text{in/ft}}) + (4\,\text{pcf})(3.5''/12^{\text{in/ft}})\right](104{,}400\,\text{sf})(0.6) = 4{,}145{,}00$$

Total Weight of Panels for Entire Building = $\boxed{4{,}145\ \text{k}}$

Gravity Loads (cont.)

· Exterior Wall Construction - (cont.)

- Glass

Assumptions
· Glass → 15psf for ~3/4" thick glass used
· Covers roughly 40% of building exterior

Weight = (15 psf)(104,400 sf)(0.4) = 626,400 lb
Total Weight of Glass for Entire Building = $\boxed{626k}$

Note: For later use in seismic calculations, weight of exterior system has been found by floor

Floor	Wall Surface Area (sf)	% Panels	% Glass	Weight (kips)
2nd	7,716	67	33	380
3rd		67	33	380
4th		50	50	313
5th				
6th				
7th				
8th				
9th				
10th				
11th				
12th	7,716	50	50	313
13th	10,520	100	0	696

Weight = {[(130 pcf)(0.5') + (4 pcf)(0.29')](SF)(% Panels) + (15 psf)(SF)(% Glass)} / 1000

Gravity Loads (cont.)

· Non-Typical Loads

Floor/Area	Design/Assumption	Justification
Sub-basement	250/150 psf	- 150 psf was assumed based on the following: maintenance, vibration, movement, etc.
Loading Dock	350 psf	- This value was used for design. Due to the high delivery traffic and possibility of heavy point loads, it is better to be conservative.
13th Floor	150 psf	- Once again, 150 psf was assumed, but was also the design value.
Elevators @ Roof	75 psf	- Additional equipment and concrete on metal decking is required in this roughly 15' x 30' area. The value of 75 psf is an estimate based on live loads only, caused by some equipment and light maintenance.

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Wind Loads

Variables	Value
Wind Speed (V)	90 mph
Wind Importance Factor (I)	1.15
Wind Exposure	B
Wind Directionality Factor (K_d)	0.85
Topographic Factor (K_{zt})	1.0
Velocity Pressue Exposure Coefficient (K_z)	varies w/ height
Building Category	III
Gust - Effect Factor (G)	0.85
Enclosure Factor	±0.18

$$q_s = 0.00256\,(V)^2 = 0.00256(90)^2 = 20.7 \text{ psf}$$

$$q_z = q_s I K_z K_{zt} K_d = (20.7)(1.15)K_{zt}(1.0)(0.85) = (20.23)K_{zt}$$

Height Above Ground Level (ft)	K_{zt}	q_z (psf)
0 - 15	0.57	11.5
20	0.62	12.5
25	0.66	13.4
30	0.70	14.2
40	0.76	15.4
50	0.81	16.4
60	0.85	17.2
70	0.89	18.0
80	0.93	18.8
90	0.96	19.4
100	0.99	20.0
120	1.04	21.0
140	1.09	22.1
160	1.13	22.9
180	1.17	23.7
200	1.21*	24.5

Assume: rigid structure, ∴ G = 0.85
ground level is at 154'6"
(street level)

Internal Pressures

$$p = q_z G C_p = 24.5(0.85)(\pm 0.18) = \pm 4 \text{ psf}$$

Wind Loads (cont.)

Windward Wind Pressures

Level	Height Above Ground Level (ft)	q_z (psf)	p (psf)
Ground	0	11.5	7.82
2nd	14' 8"	11.5	7.82
3rd	29' 4"	14.2	9.66
4th	44' 0"	15.8	10.7
5th	58' 8"	17.1	11.6
6th	73' 4"	18.3	12.4
7th	88' 0"	19.3	13.1
8th	102' 8"	20.1	13.7
9th	117' 4"	20.9	14.2
10th	132' 0"	21.7	14.8
11th	146' 8"	22.4	15.2
12th	161' 4"	22.9	15.5
13th	176' 0"	23.5	16.0
Roof	196' 0"	24.3	16.5
Parapet	200' 8"	24.5	16.7

$G = 0.85$
$C_p = 0.8$
$p = q_z G C_p = 0.68 q_z$

For wind NW-SE
(building sits at an angle to N-S direction)

Side Wall Wind Pressures

$C_p = -0.7$ $\quad q_z = 24.5$ (at parapet)
$G = 0.85$

$$p = q_z G C_p = 24.5(0.85)(-0.7) = \boxed{-14.6 \text{ psf}}$$

Leeward Wind Pressure

L/B = dimension of building parallel to wind / dimension of building normal to wind
$\approx 177'/86' \sim 2$

$\therefore C_p = -0.3$
$G = 0.85$
$q_z = 24.5$ (at parapet)

$$p = q_z G C_p = 24.5(0.85)(-0.3) = \boxed{-6.25 \text{ psf}}$$

Roof Wind Pressure

$h/L = 200'8''/177' \sim 1.13$ and $\theta < 10°$ $\longrightarrow h/L = 1.13 > 1.0$

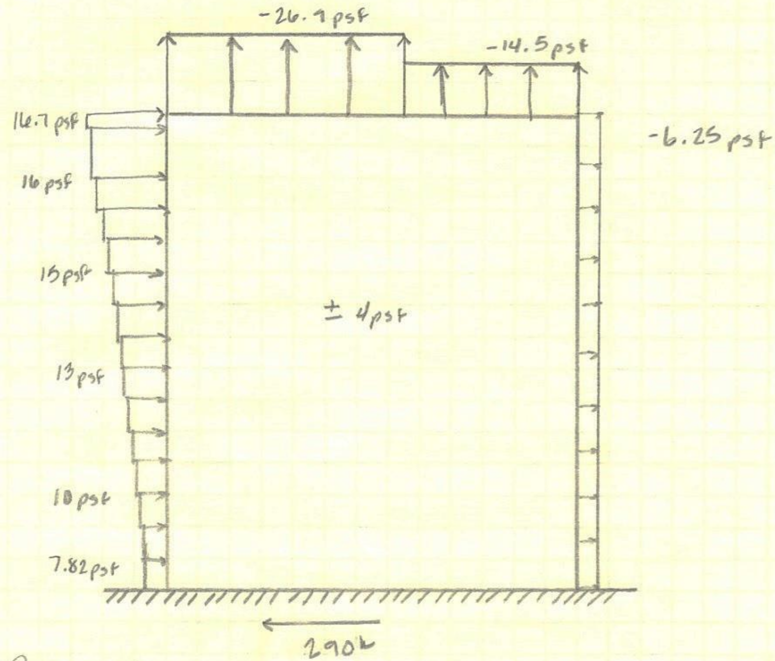
0 to $h/2 \longrightarrow C_p = -1.3$
$> h/2 \longrightarrow C_p = -0.7$

for 0 to $h/2 = 100'$:
$$p = q_{roof} G C_p = 24.3(0.85)(-1.3) = \boxed{-26.9 \text{ psf}}$$

for $> 100'$:
$$p = 24.3(0.85)(-0.7) = \boxed{-14.5 \text{ psf}}$$

Wind Loads (cont.)

For Wind NW-SE (cont.)



Base shear

Floor	Contributing Height (ft)	Length (ft)	Resultant Wind force (kip/ft)	Resultants (k)
2	14' 8"	86'	0.21	17.75
3	↑	↑	0.22	18.91
4			0.24	20.73
5			0.26	21.95
6			0.27	23.02
7			0.28	23.97
8			0.29	24.79
9			0.30	25.48
10			0.30	26.18
11	↓		0.31	26.81
12	14' 8"		0.32	27.25
13	(14'8" + 20')/2	↓	0.38	32.86
Roof	20'/2	86'	0.11	4.78
			Total =	$\boxed{290\ K}$

Resultant wind force = [(Floor below's force)(Floor below Ht)/2 + (Floor above's force)(Floor above Ht)/2]/1000

Wind Loads (cont.)

For wind NE - SW

Windward Wind Pressures

- will remain the same values as those found in other direction

Side Wall Wind Pressures

- will remain the same values as those found in other direction

Leeward Wind Pressures

$L/B \approx 86'/177' = 0.5$

$\therefore C_p = -0.5$

$G = 0.85$

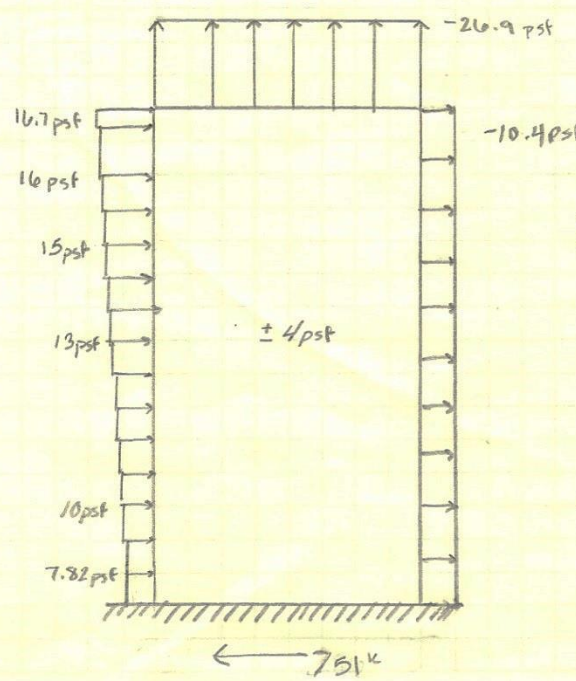
$q_z = 24.5$ (at parapet)

$p = q_z G C_p = 24.5(0.85)(-0.5) = \boxed{-10.4 \text{ psf}}$

Roof Wind Pressure

$h/L = 200'8''/86' \approx 2.33 > 1.0$

- will remain the same values as those found in other direction



Base Shear

- calculations found on following page

Wind Loads

Base Shear (for Wind NE - SW)

Floor	Contributing Ht (ft)	Length (ft)	Resultant Wind Force (k/ft)	Resultants (k)
2	14'8"	177'	0.27	47.3
3			0.28	49.7
4			0.30	53.4
5			0.32	56.0
6			0.33	58.2
7			0.34	60.1
8			0.35	61.8
9			0.36	63.2
10			0.37	64.7
11			0.37	66.0
12	14'8"		0.38	66.9
13	(14'8" + 20')/2		0.4	80.4
Roof	20'/2	177'	0.13	23.8

Total = 751 K

Seismic Loads

Variable	Value
Occupancy Category	III
Seismic Importance Factor (I)	1.25
Spectral Response Coefficients	
(S_s)	0.215
(S_1)	0.059
Site Class	D
Response Modification Factor	3
Seismic Design Category	B
Resisting System	Braced Frames
Site Coefficients	
(F_a)	1.6
(F_v)	2.4

$S_{MS} = F_a S_S = 1.6(0.215) = 0.344 \longrightarrow S_{DS} = 2/3\, S_{MS} = 2/3\,(0.344) = 0.23$

$S_{M1} = F_v S_1 = 2.4(0.059) = 0.142 \longrightarrow S_{D1} = 2/3\, S_{M1} = 2/3\,(0.142) = 0.095$

$T = C_t h_n^x = (0.028)(196)^{0.8} = 1.9\,s$

- C_t & x are values based on resisting system

$k = 1 + \frac{T - 0.5}{2} = 1 + \frac{1.9 - 0.5}{2} = 1.7$

$V = \frac{S_{D1}}{T(R/I)} W = \frac{0.095}{1.9(3/1.25)} 28157 = \boxed{587 \text{ kips}}$

Floor	DL (psf)	Weight (kips)	Height (ft)	wh^k	Force (kips)
2nd	150	2663	14' 8"	256,037	1.5
3rd		2663	29' 4"	831,384	4.9
4th		2596	44' 0"	1,615,000	9.6
5th			58' 8"	2,433,980	15.6
6th			73' 4"	3,848,430	22.8
7th			88' 0"	5,247,170	31.1
8th			102' 8"	6,819,400	40.4
9th			117' 4"	8,556,580	50.7
10th			132' 0"	10,453,950	62.0
11th			146' 8"	12,505,034	74.1
12th		2596	161' 4"	14,703,490	87.1
13th	150	2979	176' 0"	19,563,300	116.
Roof	100	1522	196' 0"	12,001,900	71.1

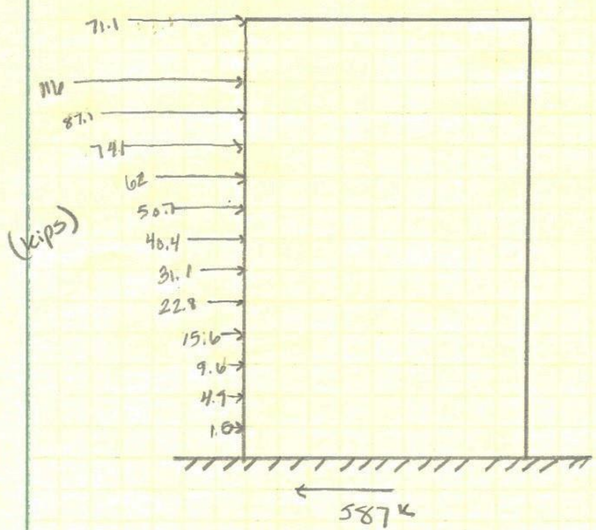
W = 33,191 kips

$\Sigma wh^k = 99,035,850$

$F = \frac{w_i h_i^k}{\Sigma wh^k} V$

Weight = (DL × SF) + $\underbrace{\text{Ext. Wall Weight}}_{\text{found earlier in report}}$

Seismic Loads (cont.)



Wind and Seismic Loads Comparison

For NW-SE → Forces

Floor	Height Above Ground Level (ft)	Wind (k)	Seismic (k)	Controls (W or S)
Ground	0	8.9	0	W
2	14' 8"	17.8	1.5	W
3	29' 4"	18.9	4.9	W
4	44' 0"	20.7	9.6	W
5	58' 8"	22.0	15.6	W
6	73' 4"	23.0	22.8	W
7	88' 0"	24.0	31.1	S
8	102' 8"	24.8	46.4	S
9	117' 4"	25.5	50.7	S
10	132' 0"	26.2	62.0	S
11	146' 8"	26.9	74.1	S
12	161' 4"	27.3	87.1	S
13	176' 0"	32.9	116	S
Roof	196' 0"	9.8	71.1	S
	Totals =	290k	587k	S

For NE-SW → Forces

Floor	Height Above Ground Level (ft)	Wind (k)	Seismic (k)	Controls (W or S)
Ground	0	23.7	0	W
2	14' 8"	47.3	1.5	W
3	29' 4"	49.7	4.9	W
4	44' 0"	53.4	9.6	W
5	58' 8"	56.0	15.6	W
6	73' 4"	58.2	22.8	W
7	88' 0"	60.1	31.1	W
8	102' 8"	61.8	46.4	W
9	117' 4"	63.2	50.7	W
10	132' 0"	64.7	62.0	W
11	146' 8"	66.0	74.1	S
12	161' 4"	66.9	87.1	S
13	176' 0"	80.4	116	S
Roof	196' 0"	23.8	71.1	S
	Totals =	751k	587k	W

Wind and Seismic Loads Comparison (cont.)

For NW-SE → Moments

Floor	Height Above Ground Level (ft)	Wind (ft·k)	Seismic (ft·k)	Controls (W or S)
Ground	0	0	0	—
2	14' 8"	260	22	W
3	29' 4"	555	144	W
4	44' 0"	912	422	W
5	58' 8"	1288	915	W
6	73' 4"	1688	1672	S
7	88' 0"	2109	2737	S
8	102' 8"	2545	4148	S
9	117' 4"	2990	5949	S
10	132' 0"	3456	8184	S
11	146' 8"	3927	10853	S
12	161' 4"	4396	14052	S
13	176' 0"	5783	20416	S
Roof	196' 0"	1917	13936	S
	Totals =	31,826	83,450	S

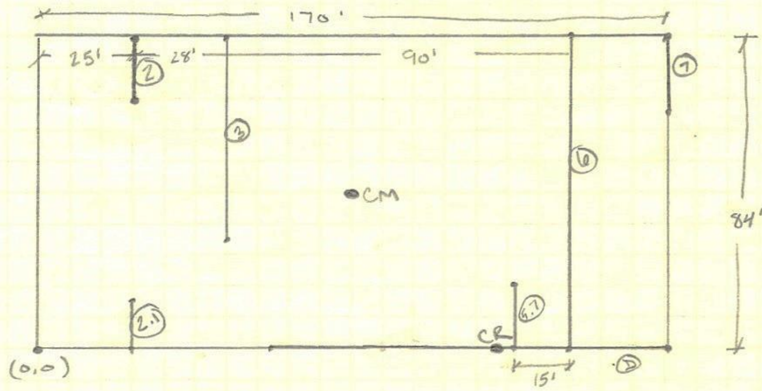
For NE-SW → Moments

Floor	Height Above Ground Level (ft)	Wind (ft·k)	Seismic (ft·k)	Controls (W or S)
Ground	0	0	0	—
2	14' 8"	694	22	W
3	29' 4"	1457	144	W
4	44' 0"	2351	422	W
5	58' 8"	3283	915	W
6	73' 4"	4265	1672	W
7	88' 0"	5290	2737	W
8	102' 8"	6345	4148	W
9	117' 4"	7419	5949	W
10	132' 0"	8535	8184	W
11	146' 8"	9660	10853	S
12	161' 4"	10787	14052	S
13	176' 0"	14142	20416	S
Roof	196' 0"	4667	13936	S
	Totals =	78,895	83,450	S

Marissa Delozier | AE Senior Thesis | Tech Report #4

Building Properties

Braced Frame	Δ (in)	K (k/in)
Line 2.1	0.09290	10.8
Line 2	0.09290	10.8
Line 3	0.03081	32.5
Line 5.7	0.0085	117.6
Line 6	0.0105	95.2
Line 7	0.0191	52.4
Line D	0.0040	250.0



Center of Rigidity

$$X_R = \Sigma K_i x_i / \Sigma K = \frac{[(25)(10.8) + (25)(10.8) + (53)(32.5) + (128)(117.6) + (143)(95.2) + (170)(52.4)]}{(10.8 + 10.8 + 32.5 + 117.6 + 95.2 + 52.4)}$$

$X_R = 124'6"$

$Y_R = 0$ *NOTE: Values found at roof level of the structure

Center of Mass

Assumptions: Due to uniformity of building and layout, CM will be relatively close to actual center on each floor

$X_M = 85'$

$Y_M = 42'$

Building Properties (cont.)

Computer Model CR & CM (at Roof Level)

$X_R = 121' 6''$ $X_M = 85'$
$Y_R = 0$ $Y_M = 41' 4''$

Compared to calculated results, the numbers are extremely close
∴ ok ✓

However, found from the RAM model, the CR does differ by level due to changes in frames and bracing.

Floor	X_R	Y_R
Roof	121' 6"	0
13	117' 8"	↑
12	116' 0"	
11	114' 4"	
10	112' 7"	
9	110' 7"	
8	108' 6"	
7	106' 1"	
6	103' 5"	
5	100' 4"	
4	97' 5"	
3	97' 10"	↓
2	93' 10"	0

Assumptions:
- Minor changes in Y_R can be neglected (insignificant)
- Since Roof Level CR was confirmed with hand calculations, other Floor Level CRs generated by the RAM will be assumed valid

Distribution of Forces

Using the RAM model created, a 100K story force was placed at the Roof Level to determine the amount of shear transferred to each frame by level. The results can be found on the following page.

Building Properties (cont.)

Distribution of Forces

% of Shear by Braced Frame

Floor	2.1	2	3	5.7	6	7
Roof	7.5	25.0	4.5	2.5	51	9.5
13	3.6	19.5	38.6	6.2	31.3	2.9
12	5.8	21.1	33.8	6.2	32.4	3.6
11	5.7	22.5	32.3	7.3	33.3	2.2
10	8.2	20.5	32.3	7.3	33.0	2.4
9	5.9	20.7	35.1	7.8	32.6	1.9
8	7.7	19.5	34.2	7.7	33.1	1.8
7	5.5	20.0	35.9	8.0	33.1	1.4
6	9.3	19.1	32.0	8.3	33.2	1.7
5	8.5	18.3	31.7	7.2	34.7	2.2
4	4.5	14.0	65.9	1.6	11.8	2.1
3	3.3	11.0	47.2	5.2	30.1	4.5
2	4.5	9.8	47.9	5.6	27.8	4.8

In the opposing direction, Frame D will carry 100% of the shear force since it is the only lateral force resisting member that runs in that direction.

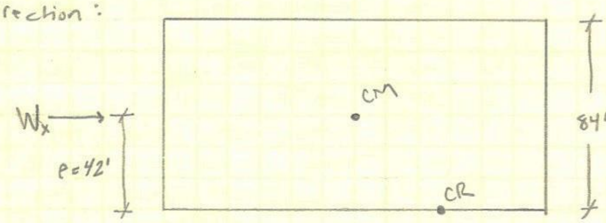
Wind Analysis

Utilizing Method 2 from ASCE-07, there are 4 possible wind cases to consider. All drawings below utilize the Roof Level.

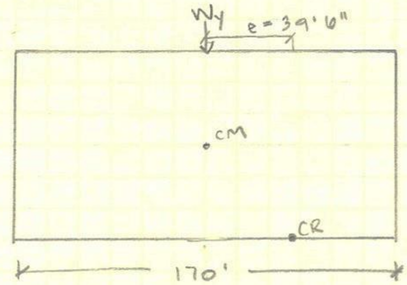
Note: Since building site does not align with N-S or E-W, from this point and forward the directions will be called universal X-Direction and Y-Direction. In actuality, X corresponds with NW-SE and Y with NE-SW.

Case #1

For X-direction:



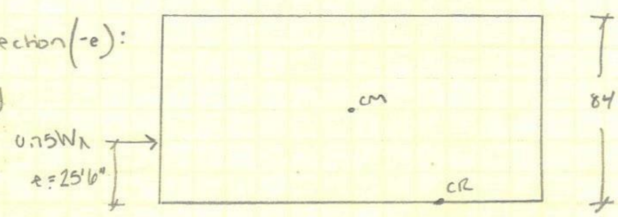
For Y-direction:



Case #2

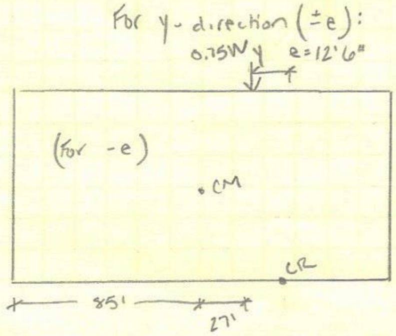
For X-direction (-e):

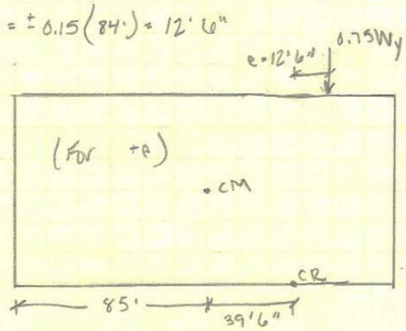
e = ±0.15(170')
e = ±25'6"



For Y-direction (±e):

e = ±0.15(84') = 12'6"

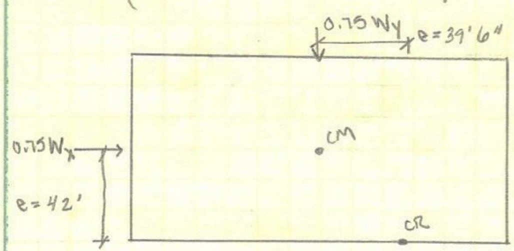




Wind Analysis (cont.)

Case #3

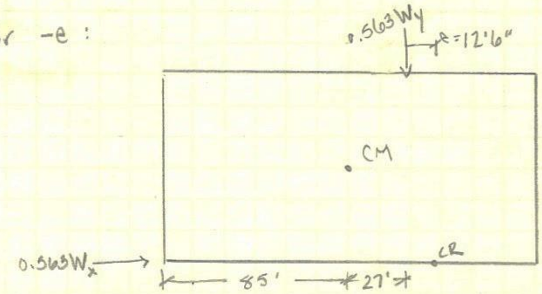
(Similar to Case #1, but at 75% of Wind loads)



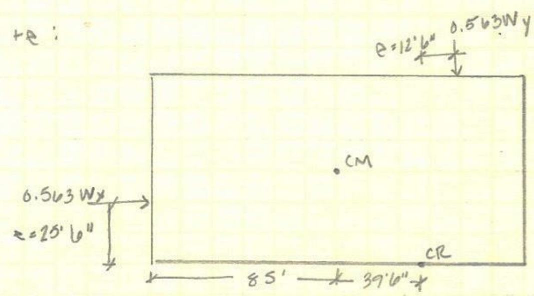
Case #4

(Similar to Case #2, but at 75% of Case 2 Wind Loads)

For -e :



For +e :



∴ All applicable load cases are :

Case #1 : $\begin{cases} W_x \\ W_y \end{cases}$

Case #2 : $\begin{cases} 0.75 W_x \, (+e_x) \\ 0.75 W_x \, (-e_x) \\ 0.75 W_y \, (+e_y) \\ 0.75 W_y \, (-e_y) \end{cases}$

Case #3 : $\begin{cases} 0.75 W_x + 0.75 W_y \\ 0.75 W_x - 0.75 W_y \end{cases}$

Case #4 : $\begin{cases} 0.563 W_x + 0.563 W_y & (CW) \\ 0.563 W_x + 0.563 W_y & (CCW) \\ 0.563 W_x - 0.563 W_y & (CW) \\ 0.563 W_x - 0.563 W_y & (CCW) \end{cases}$

Wind Analysis (cont.)

Using the RAM model, the displacement (Δ) and drift values were found for each case at every floor level.

Allowable Drift = H/400

@ Roof = (20)(12)/400 = 0.6
all other stories = (14.67)(12)/400 = 0.4

NOTE: all values in tables below are in inches

Case 1									
	X-Dir				Y-Dir.				Allowable Drift
Floor	Δx	Δy	Drift x	Drift y	Δx	Δy	Drift x	Drift y	
Roof	4.0	1.0	0.38	0.15	3.6	4.0	0.56	0.47	0.6
13	3.6	0.8	0.33	0.10	3.0	3.5	0.38	0.38	0.4
12	3.3	0.7	0.35	0.10	2.6	3.1	0.38	0.39	
11	2.9	0.6	0.37	0.10	2.2	2.7	0.37	0.39	
10	2.6	0.5	0.37	0.10	1.9	2.3	0.35	0.39	
9	2.2	0.4	0.37	0.10	1.5	1.9	0.33	0.38	
8	1.8	0.3	0.36	0.08	1.2	1.6	0.3	0.35	
7	1.5	0.2	0.34	0.07	0.9	1.2	0.27	0.32	
6	1.1	0.2	0.31	0.06	0.6	0.9	0.23	0.29	
5	0.8	0.1	0.28	0.05	0.4	0.6	0.18	0.24	
4	0.5	0.05	0.22	0.02	0.2	0.4	0.09	0.14	
3	0.3	0.03	0.19	0.02	0.1	0.2	0.08	0.14	
2	0.1	0.01	0.12	0.01	0.03	0.1	0.03	0.08	0.4

Case 1 passes for serviceability drift requirements.

Cases 2 thru 4 were also completed with the assistance of RAM. All loadings produced drift values that met the maximum allowable drift, meaning the entire structure meets all serviceability, in regards to drift requirements. The results from Cases 2 thru 4 can be found in Appendix A.

Seismic Analysis

Four load cases were considered while analyzing the seismic force resisting members:

$$E_x\ (+e)$$
$$E_x\ (-e)$$
$$E_y\ (+e)$$
$$E_y\ (-e)$$

Allowable drift = $0.015h = 0.015(196) = 2.9$ in.

X-Direction

Floor	+e Δ_x (in)	+e Δ_y (in)	+e Drift$_x$ (in)	+e Drift$_y$ (in)	−e Δ_x (in)	−e Δ_y (in)	−e Drift$_x$ (in)	−e Drift$_y$ (in)	Allowable Drift (in)
Roof	9.14	3.94	0.91	0.66	5.38	1.02	0.43	0.15	2.9
13	8.23	3.23	0.82	0.44	5.00	0.88	0.44	0.11	
12	7.41	2.84	0.87	0.43	4.52	0.77	0.48	0.11	
11	6.54	2.41	0.90	0.42	4.04	0.66	0.51	0.11	
10	5.64	1.99	0.90	0.40	3.53	0.55	0.53	0.10	
9	4.74	1.59	0.88	0.37	3.00	0.45	0.52	0.10	
8	3.86	1.23	0.84	0.33	2.48	0.34	0.50	0.09	
7	3.03	0.89	0.76	0.29	1.97	0.25	0.47	0.08	
6	2.26	0.60	0.68	0.24	1.50	0.17	0.43	0.07	
5	1.58	0.36	0.58	0.19	1.10	0.10	0.37	0.05	
4	1.00	0.17	0.45	0.08	0.70	0.05	0.31	0.02	
3	0.55	0.10	0.36	0.08	0.39	0.02	0.25	0.02	
2	0.19	0.03	0.19	0.02	0.14	0.004	0.14	0.004	2.9

Y-Direction

Floor	+e Δ_x (in)	+e Δ_y (in)	+e Drift$_x$ (in)	+e Drift$_y$ (in)	−e Δ_x (in)	−e Δ_y (in)	−e Drift$_x$ (in)	−e Drift$_y$ (in)	Allowable Drift (in)
Roof	1.65	4.43	0.29	0.53	3.46	5.88	0.52	0.78	2.9
13	1.35	3.90	0.19	0.44	2.93	5.10	0.37	0.61	
12	1.16	3.46	0.18	0.45	2.56	4.47	0.37	0.61	
11	0.98	3.00	0.18	0.46	2.19	3.88	0.36	0.61	
10	0.80	2.54	0.17	0.45	1.83	3.26	0.35	0.66	
9	0.63	2.09	0.15	0.43	1.48	2.67	0.33	0.57	
8	0.48	1.66	0.14	0.40	1.15	2.10	0.30	0.52	
7	0.34	1.26	0.12	0.36	0.85	1.58	0.26	0.46	
6	0.22	0.90	0.10	0.31	0.59	1.12	0.22	0.39	
5	0.13	0.59	0.07	0.26	0.37	0.72	0.17	0.32	
4	0.05	0.34	0.03	0.15	0.20	0.40	0.10	0.17	
3	0.03	0.19	0.02	0.13	0.10	0.23	0.07	0.16	
2	0.004	0.016	0.004	0.016	0.03	0.07	0.03	0.07	2.9

Overturning Moment Impacts

Although wind loads control at almost every floor for both directions, the overturning moment caused by the applied seismic forces controls ultimately. To find the resisting moment, the building weight calculated in Technical Report #2 is multiplied by 1/2 the length of the building in the direction of analysis. The seismic overturning moment is the same for both directions, so the shorter length of the building will be considered (in order to be conservative).

Total Building Weight = $33{,}191^{k}$

$33{,}191^{k}\,(84' / 2) = 1{,}394{,}022$ ft·k

$1{,}394{,}022$ ft·k $\gg$ $83{,}450$ ft·k

∴ building structural system is capable of resisting overturning moments caused by lateral (wind + seismic) forces

Strength Check

As shown on page 22 of this report, wind loadings tend to control in the Y-Direction, while seismic loads are slightly more controlling in the X-Direction.

Possible Load Combinations:

$$1.2D + 1.6W + L + 0.5(L_r \text{ or } S)$$
$$1.2D + 1.0E + L + 0.2S$$

X-Direction Frame Strength Check

Frame D (as shown in Appendix B)

Checks: Column D.5.7
Brace to right of Column D.5.7

Column D.6

Seismic Controls $\longrightarrow$ $1.2D + 1.0E + L + 0.2S$
(where S = 30 psf)
L > S, ∴ use L @ each level

Trib. Area = (12.25 × 8.67) + (7.5 × 6) = 151 ft²

Floor	DL* (psf)	LL* (psf)	P_u (k)
Roof	100	20	21.1
13	150	150	49.8
12	150	80	39.3
11	150	80	39.3
10	150	80	39.3
9	150	80	39.3
8	150	80	39.3
7	150	80	39.3
6	150	80	39.3
5	150	80	39.3
4	150	80	39.3
3	150	60	36.2
2	150	60	36.2
1	150	100	42.3

*As calculated in Tech Report #2

$$P_u = \frac{[1.2DL + LL](A_{TRIB})}{1000}$$

Total P_u = 539.3k

M_u = 1011 ft-k

Strength check (cont.)

X-Direction: Column D.5.7 (cont.)

$K = 0.7$ (for fixed-pinned connection)

$L = 14.67'$ $KL = 10.3' \rightarrow$ use $KL = 12'$

Using AISC Table 6-1 $\rightarrow$ W14 x 233

$p \times 10^3 = 0.355$

$b \times 10^3 = 0.544$

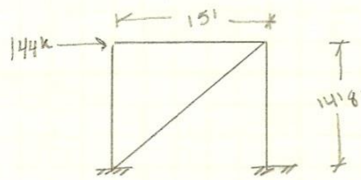
$$pP_r = \frac{(0.355)(539.3)}{1000} = 0.19 < 0.2$$

$\therefore$ use $\frac{1}{2} pP_r + \frac{9}{8} b_x M_x < 1.0$

$$\frac{1}{2}(0.19) + \frac{9}{8}\left[(0.544)(1011)/1000\right] = 0.72 < 1.0$$

$\therefore$ column D.5.7 is adequate for combined flexure and axial loads

Brace



W12 x 65 (290k)

Available Tension Strength = 697k

" Compression " = 747k

Available >> Actual Force

$\therefore$ brace is adequate for applied loads

Strength check (cont.)

Y-Direction Frame Strength Check

Frame 6 (as shown in Appendix B)

Checks: Column B-6
Brace

Column B-6

Wind controls $\longrightarrow 1.2D + 1.6W + (L \text{ or } 0.5L_r)$

Trib Area $= [9.875 \times (15+13)] + (13 \times 9.25) + [(9.25 \times 15) - 104]$
$= 432 \text{ ft}^2$

Floor	DL* (psf)	LL* (psf)	Pu (k)
Roof	100	20	60.5
13	150	150	143
12	150	80	112
11	150	80	112
10	150	80	112
9	150	80	112
8	150	80	112
7	150	80	112
6	150	80	112
5	150	80	112
4	150	80	112
3	150	60	104
2	150	60	104
1	150	100	121

*As calculated in Tech Report #2

Total $P_u = 1541^k$

$M_u = 2038^{ft \cdot k}$

$K = 0.7$ (for fixed-pinned)

$KL = 10.3' \rightarrow$ use $12'$

using AISC Table 4-4

W14x455

$p \times 10^3 = 0.179$

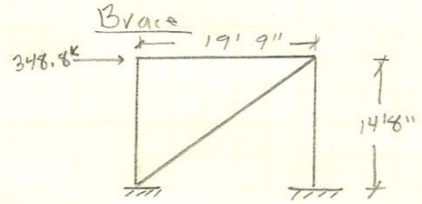
$b \times 10^3 = 0.253$

$pP_r = (0.179)(1541)/1000 = 0.276 > 0.2$

$\therefore$ use $pP_r + b_x M_x = (0.276) + [(0.253)(2038)/1000] = 0.79 < 1.0$

$\therefore$ column B-6 is adequate for combined flexure and axial loads

Brace



W12x120 (700^k)

Available Tension Strength $= 1290^k$

" Compression " $= 942^k$

Available >> Actual Force

$\therefore$ brace is adequate for applied loads

Torsional Irregularities

Due to the design of the structure, very little torsion exists when considering forces acting in the Y-direction. The frames traveling in the Y-Direction are spaced evenly and are of average size throughout the majority of the structure.

However, there is an inherent torsion that occurs for loads traveling in the X-direction. Only one frame travels in this direction, causing a much greater mass and stiffness along the side of the building that sits. Eccentricity was considered during both wind and seismic loading analysis; no major issues were found. Further exploration of the effects of this irregularity could be beneficial in future analysis.

While the computer generated model was created to be a perfectly rectangular shape, this is not completely true for the actual building. On all four corners of the structure there exists interesting shapes and wall positioning. The shape vaguely resembles a pinwheel. By nature, pinwheels are designed to catch air and spin (an idea that is not wanted in buildings). The effects of the actual shape of the building and cutouts could be potentially reviewed in future work; for this report though, the building will remain a rectangular shape.

Conclusions

After analyizing the James W. and Frances C. McGlothin Medical Education Center for both wind and seismic loadings, it has been determind that all standard strength and serviceability requirements have been met. While distributing forces, it was found that seismic controlled in the X-direction and wind in the Y-. This seems plausible since wind loadings would tend to control when applied to a larger surface area. An inherent eccentricity was found to exist in the structure due to the distance between the center of mass and center of rigidity, so special attention was applied to the loadings multiplied by ±e.

Even with the high loadings and ±e, the structure safely met serviceability drift requirements for all seismic (1 case) and wind (4 cases) loadings. Overturning Moment was also calculated for wind and seismic, but the resisting moment at the foundation was more than sufficient to counteract. Frames in both the X- and Y- directions were considered when evaluating strength. Members, both columns and braces, at base level were checked for combined axial and flexure forces. The four members checked met or exceeded the necessary strengths.

The building was slightly simplified for this analysis; irregularities throughout the structure could be used for potential analysis in the future to confirm the results found in this report. However, for the moment, the building meets all necessary requirements for both strength and serviceability.

Appendix A: Additional Wind Analysis Results

CASE 2									
Floor	X-Direction (+e)				X-Direction (-e)				Allowable
	Δx	Δy	Drift x	Drift y	Δx	Δy	Drift x	Drift y	Drift (in)
Roof	2.8	1.2	0.25	0.19	1.7	0.29	0.12	0.04	0.60
13	2.5	0.97	0.23	0.12	1.5	0.25	0.12	0.03	0.40
12	2.3	0.84	0.23	0.12	1.4	0.22	0.13	0.03	0.40
11	2	0.72	0.25	0.12	1.3	0.19	0.14	0.03	0.40
10	1.8	0.6	0.26	0.11	1.1	0.16	0.15	0.03	0.40
9	1.5	0.49	0.26	0.11	0.98	0.13	0.15	0.03	0.40
8	1.3	0.38	0.25	0.1	0.83	0.1	0.15	0.03	0.40
7	1	0.28	0.23	0.09	0.68	0.08	0.15	0.02	0.40
6	0.79	0.2	0.22	0.07	0.54	0.05	0.14	0.02	0.40
5	0.57	0.12	0.19	0.06	0.4	0.03	0.13	0.02	0.40
4	0.38	0.06	0.16	0.02	0.27	0.02	0.11	0.01	0.40
3	0.22	0.04	0.14	0.03	0.16	0.01	0.1	0.007	0.40
2	0.08	0.01	0.08	0.01	0.06	0.001	0.06	0.001	0.40

CASE 2									
Floor	Y-Direction (+e)				Y-Direction (-e)				Allowable
	Δx	Δy	Drift x	Drift y	Δx	Δy	Drift x	Drift y	Drift (in)
Roof	0.34	2.1	0.09	0.21	2.5	3.8	0.36	0.5	0.60
13	0.25	1.9	0.05	0.19	2.2	3.3	0.26	0.38	0.40
12	0.21	1.7	0.04	0.2	1.9	2.9	0.26	0.38	0.40
11	0.16	1.5	0.04	0.21	1.7	2.6	0.26	0.38	0.40
10	0.12	1.3	0.04	0.21	1.4	2.2	0.25	0.37	0.40
9	0.08	1.1	0.03	0.21	1.2	1.8	0.24	0.36	0.40
8	0.05	0.9	0.03	0.2	0.93	1.4	0.22	0.34	0.40
7	0.02	0.71	0.02	0.18	0.71	1.1	0.2	0.31	0.40
6	0.003	0.53	0.02	0.16	0.51	0.81	0.17	0.27	0.40
5	0	0.36	0.01	0.14	0.33	0.54	0.14	0.23	0.40
4	0	0.23	0	0.09	0.19	0.31	0.09	0.13	0.40
3	0	0.13	0	0.08	0.1	0.19	0.07	0.12	0.40
2	0	0.05	0	0.05	0.03	0.06	0.03	0.06	0.40

CASE 3									
Floor	X + Y				X - Y				Allowable
	Δx	Δy	Drift x	Drift y	Δx	Δy	Drift x	Drift y	Drift (in)
Roof	3.7	3.7	0.41	0.47	0.76	-2.3	-0.04	-0.24	0.60
13	3.3	3.2	0.33	0.36	0.8	-2	0.02	-0.21	0.40
12	2.9	2.9	0.34	0.37	0.78	-1.8	0.04	-0.21	0.40
11	2.6	2.5	0.35	0.37	0.74	-1.6	0.05	-0.22	0.40
10	2.2	2.1	0.35	0.36	0.69	-1.4	0.06	-0.22	0.40
9	1.9	1.7	0.34	0.35	0.63	-1.1	0.07	-0.22	0.40
8	1.5	1.4	0.33	0.33	0.57	-0.93	0.08	-0.2	0.40
7	1.2	1.1	0.3	0.3	0.49	-0.73	0.08	-0.19	0.40
6	0.92	0.79	0.27	0.26	0.4	-0.54	0.08	-0.17	0.40
5	0.64	0.53	0.24	0.22	0.33	-0.37	0.09	-0.14	0.40
4	0.41	0.31	0.17	0.12	0.24	-0.23	0.1	-0.09	0.40
3	0.23	0.18	0.15	0.12	0.14	-0.14	0.08	-0.09	0.40
2	0.08	0.06	0.08	0.06	0.06	-0.05	0.06	-0.05	0.40

CASE 4									
Floor	X + Y CW				X + Y CCW				Allowable Drift (in)
	Δx	Δy	Drift x	Drift y	Δx	Δy	Drift x	Drift y	
Roof	4	3.7	0.46	0.52	1.5	1.8	0.15	0.19	0.60
13	3.5	3.2	0.37	0.38	1.3	1.6	0.13	0.16	0.40
12	3.2	2.8	0.38	0.38	1.2	1.5	0.13	0.17	0.40
11	2.8	2.5	0.38	0.38	1.1	1.3	0.14	0.18	0.40
10	2.4	2.1	0.38	0.37	0.94	1.1	0.14	0.18	0.40
9	2	1.7	0.37	0.35	0.8	0.93	0.14	0.18	0.40
8	1.7	1.4	0.36	0.33	0.66	0.76	0.13	0.17	0.40
7	1.3	1	0.33	0.29	0.53	0.59	0.13	0.15	0.40
6	0.97	0.75	0.3	0.26	0.4	0.43	0.12	0.14	0.40
5	0.68	0.49	0.25	0.21	0.29	0.3	0.1	0.12	0.40
4	0.43	0.28	0.18	0.11	0.19	0.18	0.08	0.07	0.40
3	0.24	0.17	0.16	0.11	0.11	0.11	0.07	0.07	0.40
2	0.09	0.05	0.09	0.05	0.04	0.04	0.04	0.04	0.40

CASE 4									
Floor	X - Y CW				X - Y CCW				Allowable Drift (in)
	Δx	Δy	Drift x	Drift y	Δx	Δy	Drift x	Drift y	
Roof	1.8	-0.72	0.12	-0.02	-0.68	-2.7	-0.18	-0.35	0.60
13	1.7	-0.71	0.13	-0.05	-0.5	-2.3	-0.11	-0.26	0.40
12	1.6	-0.66	0.15	-0.06	-0.4	-2	-0.1	-0.26	0.40
11	1.4	-0.6	0.16	-0.07	-0.3	-1.8	-0.09	-0.26	0.40
10	1.3	-0.53	0.17	-0.07	-0.22	-1.5	-0.08	-0.26	0.40
9	1.1	-0.46	0.17	-0.07	-0.14	-1.3	-0.07	-0.25	0.40
8	0.92	-0.39	0.17	-0.07	-0.07	-1	-0.05	-0.23	0.40
7	0.75	-0.32	0.16	-0.07	-0.02	-0.78	-0.04	-0.21	0.40
6	0.59	-0.24	0.15	-0.07	0.02	-0.56	-0.02	-0.19	0.40
5	0.44	-0.18	0.14	-0.06	0.05	-0.38	-0.01	-0.16	0.40
4	0.3	-0.12	0.13	-0.05	0.06	-0.22	0.02	-0.09	0.40
3	0.17	-0.07	0.11	-0.04	0.04	-0.13	0.02	-0.09	0.40
2	0.07	-0.03	0.07	-0.03	0.02	-0.05	0.02	-0.05	0.40

Appendix B: Detailed Drawings of Braced Frames