Jonathan Ebersole
235 E. Fairmount Ave. Apt. 102
State College, PA 16801

November 20, 2013
Dr. Linda Hanagan
Associate Professor
212 Engineering Unit A
University Park, Pa 16802

Dear Dr. Hanagan,

This document is developed to help guide you through the evaluation of alternative lateral load cases for the Oklahoma University Children's Medical Office Building. The purpose of this assignment is to understand the methods used to distribute the lateral forces throughout the lateral force resisting system. Strength and serviceability requirements for the lateral systems are also addressed in this assignment. The document contains a site plan of the building along with a list of codes and documents used during the analysis. This document is accompanied with calculations that derive the gravity loads as well as the lateral loads used in the analysis. Calculations for the distribution of the loads and the strength and serviceability checks are also contained within.

Sincerely,

Jonathan Ebersole

Table of Contents

OU Children's Medical Office Building

Jonathan Ebersole | Structural Option

http://www.engr.psu.edu/ae/thesis/portfolios/2014/jme5193/index.html

Project Team

- Owner: University Hospitals Trust
- Construction Manager: Flintco, Inc.
- Project Architect: Miles Associates
- Design Architect: Hellmuth, Obata, and Kassabaun, Inc.
- Structural Engineer: Zahl-Ford
- MEP Engineer: ZRHD, P.C.
- Civil Engineer: Smith, Roberts, Baldischwiler, Inc.

General Information

- Location: 1200 North Children's Avenue, Oklahoma City, Oklahoma
- Occupancy: Office
- Size: 320,000 sq. ft.
- Height: 12 Stories for a total of 172 ft.
- Construction Dates: February 2007-Spring of 2009
- Building Cost: $59,760,000
- Delivery Method: Design-Bid-Build

Architecture

- Exterior Façade comprised of brick Veneer with large glass curtain wall on the front face of the building
- Supports Hospital with additional office space, exam rooms, and labs
- Membrane roof system with rigid insulation and light weight insulating concrete

Structural Design

- Reinforced concrete columns and beams
- 10" thick flat slab system with drop panels
- Concrete shear walls located in elevator shafts and stairwells
- Drilled pier foundation with a minimum bearing capacity of 45 KSF

Mechanical Design

- 7,500 CFM Air Handling unit occupies each floor
- Heat Exchanger is used to heat water before entering the heating coil
- 850 CFM fans are used to pressurize the stairwells

Lighting/Electrical Design

- Service voltage is 480/277 V, three phase, with 4 wires
- Voltage reduced to 120/208V, three phase, with 4 wires and supplied to each panel box
- Fluorescent lamps are used throughout the building to save energy costs

General Information

Executive Summary

OU Children's Medical Office Building is an office building located in Oklahoma. It is situated next to an existing hospital and parking garage. The building houses offices, examination rooms and labs for the expanding OU Children's Hospital. It is the largest free standing clinical office in the state and provides much needed medical services to the children of Oklahoma and their families.

The structure of the building is reinforced concrete. The building uses a flat slab system supported by columns and exterior beams. Drop panels are used at the column locations to provide extra shear and moment capacity to the slab. The columns are supported on piers that transfer the loads to bedrock underneath the building. The building also uses shear walls and moment frames to resist the lateral forces.

This building provides several unique challenges that a typical office building would not otherwise have. These include a parking garage located on the first floor, a future helicopter pad positioned on the roof, and impact loads on lower levels for vehicle collisions with the building. These design parameters will increase the difficulty of future design assignments as all load cases must be analyzed.

Site Plan

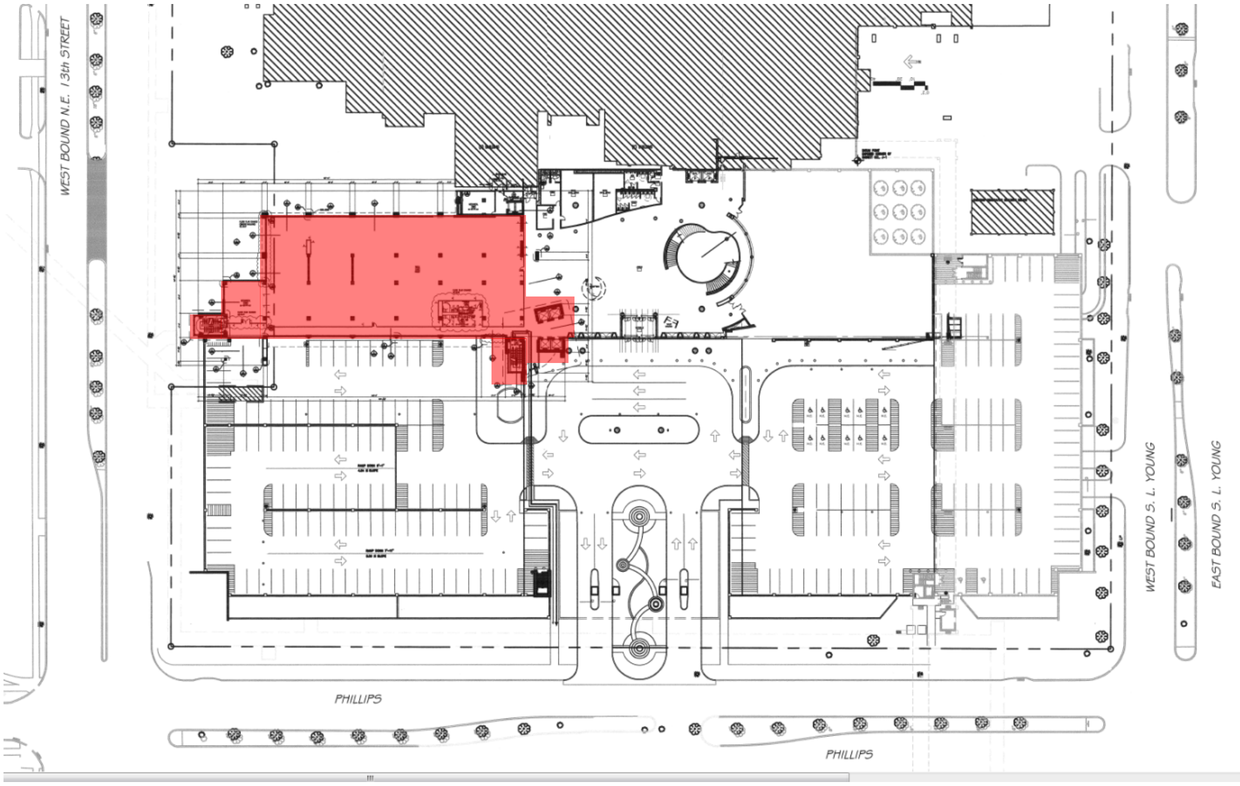


Figure A. Building outlined in red.

OU Children's Medical Office Building is located on 1200 N. Children's Avenue Oklahoma City, Oklahoma between Stanton L. Young Blvd and N.E. 13th Street. (Refer to figure A for site and building footprint). The building is twelve stories above grade and is approximately 180 feet tall. The building footprint is 22,820 square feet with a total area of 320,000 square feet. The building is positioned between an existing hospital and existing parking structure. A large atrium connects the hospital to the office building and parking structure but it is a future addition and not part of the original office construction. The building is located in an urbanized area which will later impact the design for the lateral loads.

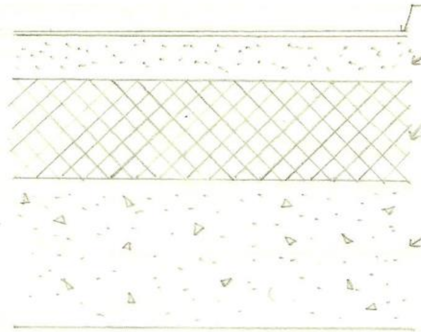
List of Documents

For this assignment, several documents where used in order to evaluate the lateral system members. The ACI 318-02 code was used to analyze the existing shear walls. I also used examples and design aids from the sixth edition of *Reinforced Concrete Mechanics and Design* written by James Wight and James MacGregor.

2-1

1. Gravity Loads

1.a. Roof



cross section of Typical Roof Construction

Loads

Live Load - 20 psf (IBC 2003)

Dead Load

Materials

Membrane Roof	ASCE 7-02	-	~1 psf
Light Weight Insulating Concrete (2" thk)	ASCE 7-02	$115\text{pcf} \cdot \frac{2\text{in}}{12\text{in}}$	~19 psf
Rigid Insulation (4" thk)	AISC 14th ed.	$1.5\text{psf/in} \cdot 4\text{in}$	~6 psf
Superimposed Dead Load			~15 psf

Structure

slab (10" thk)	$150\text{pcf} \cdot \frac{10\text{in}}{12\text{in}}$	~125 psf
drop panels (4" thk)	$150\text{pcf} \cdot \frac{4\text{in}}{12\text{in}} \cdot \frac{10.67' \cdot 7.33'}{26' \cdot 32'}$	~5 psf
column (22"x22")	$150\text{pcf} \cdot \frac{(144\text{in} - 14\text{in})}{12\text{in}} \cdot \frac{22\text{in} \cdot 22\text{in}}{12^2 \cdot 26' \cdot 32'}$	~7 psf

2-2

Snow Loads

Flat Roof Snow Loads (ASCE 7-02)
Slope = 1/4" per foot < 5° slope ⇒ fits criteria

$p_f = 0.7 C_e C_t I p_g$

$C_e = 1.0$
$C_t = 1.1$
$I = 1.0$
$p_g = 10$ psf

$p_f = 0.7 \cdot 1.0 \cdot 1.1 \cdot 1.0 \cdot 10$ psf $= 7.7$ psf

but not less than:

p_g is 20 psf or less

$p_f = I(p_g) = 1.0 \cdot 10 = 10$ psf → controls

Snow Drift Loads

Parapet

$l_u = 91.29$ ft
$h_d = 2.5$ ft
$h_c = 4.683$ ft
$\gamma = 0.13 p_g + 14 = 0.13 \cdot 10 + 14 = 15.3$ pcf
$h_b = p_f / \gamma = 10 / 15.3 = 0.65$ ft
$w = 4 h_d = 4 \cdot 2.5 = 10$ ft

$h_c / h_b = 4.683 / 0.65 = 7.2 > 0.2$ ∴ must be applied

$p_d = 3/4 \, h_d \gamma = 3/4 \cdot 2.5 \cdot 15.3 = 28.69$ psf (windward)

$p_d \cdot 0.5 \cdot w = 28.69 \cdot 0.5 \cdot 10 = 143.45$ plf

Load Combinations

$1.2D + 0.5L_r$ or S
$1.2D + 1.6L_r$ or S

Component

Slab

Live Load - 20 psf > S = 10 psf
Dead Load - 166 psf

2-3

$w_u = 1.2(166) + 1.6 \cdot (20) =$ **231.2 psf**

$w_u = 1.2(166) + 0.5(20) = 209.2$ psf

Column

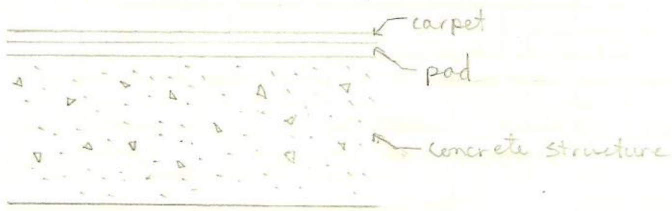
Live Load - 20 psf (unreducable
Dead Load - 178 psf

$w_u = 1.2(178) + 1.6(20) =$ **245.6 psf**

$w_u = 1.2(178) + 0.5(20) = 223.6$ psf

2-4

1. b. Floor



Cross Section of Typical Floor Construction

Loads

Live Loads

Office - 50 psf + 20 psf = 70 psf (IBC 2003)
Corridor - 80 psf (IBC 2003) → used to design for building flexibility

Dead Loads

Materials

Carpet with Pad (Boise Cascade - Weights of Materials) - 2 psf
Superimposed Dead Load - 15 psf

Structure

slab (10" thk) $150\text{ pcf} \cdot \frac{10\text{ in}}{12\text{ in}}$ - 125 psf

drop panels (4" thk) $150\text{ pcf} \cdot \frac{4\text{ in}}{12\text{ in}} \cdot \frac{10.67' \cdot 7.33'}{24' \cdot 32'}$ - 5 psf

column (30" x 30") $150\text{ pcf} \cdot \frac{(148\text{ in} - 14\text{ in})}{12\text{ in}} \cdot \frac{30\text{ in} \cdot 30\text{ in}}{12\text{ in}^2 \cdot 24' \cdot 32'}$ - 15 psf

Load Combinations

1.2D + 1.6L

Components

Slab

Live Load - 80 psf
Dead Load - 142 psf

$w_u = 1.2(142) + 1.6(80) = 298.4$ psf

column

Live Load - 80 psf

$L = 80 \cdot \max \left| \begin{array}{l} 0.4 \\ 0.25 + \frac{15}{\sqrt{4 \cdot 26 \cdot 32}} \end{array} \right. = 0.51 \cdot 80 = 40.8$ psf

$\hookrightarrow 3328 > 400$ ok ✓

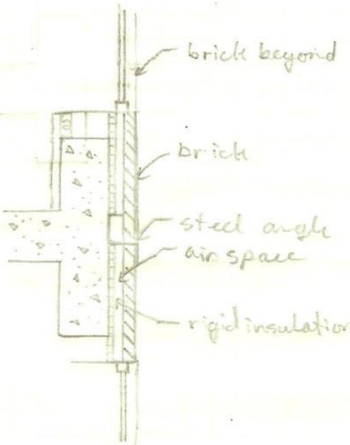
Dead Load - 162 psf

$w_u = 1.2 \cdot 162 + 1.6 \cdot 40.8 = 259.7$ psf

2-6

Exterior Wall

Brick Wall



Dead Load

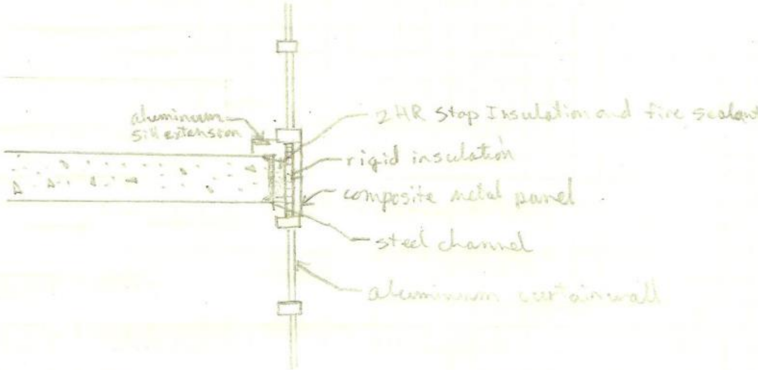
Materials
brick - 42 psf
rigid insulation (2" Thk.) - 3 psf

Total Dead Load - 45 psf · 14 ft = 630 plf (for floors 1-3)
- 45 psf · 14.67 ft = 660 plf (for floor 4)
- 45 psf · 13.115 ft = 590.2 plf (for floor 5)
- 45 psf · 12 ft = 540 plf for floor 6-Roof)

Load transfer
- brick → steel angle → structure

2-7

aluminum and Glass wall



Materials
aluminum - 4 psf
rigid insulation (2" thk) - 3 psf
glass (2 pane @ 1/4" Thk ea.) - 8 psf

Total dead load - 15 psf · 14 ft = 210 plf (for floors 1-3)
- 15 psf · 14.67 ft = 220 plf (for floor 4)
- 15 psf · 13.15 ft = 197 plf (for floor 5)
- 15 psf · 12 ft = 180 plf (for floors 6-roof)

Load Transfer
- composite metal panel → steel channel → structure

Non-Typical Loads

- vehicle impact load - 6 kips at 18" above finish floor
  - 6 kips is approximately the weight of a large vehicle

- helicopter pad load - 60 psf on roof between grid 6 and 7 and Q and P.2
  - weight includes the weight of the helicopter and the weight of the pad.

- ambulance load - 480 plf of load lane - designed as HS15 from AASHTO - located at the first floor at joists

2-54

Lateral Loads

Wind (East-West Direction)

analytical Procedure

Risk category II
V = 90 mph
Kd = 0.85
Exposure category B

Topographic Factor

$K_{zt} = 1.0$ (no nearby hill, ridge, or escarpment)

Determine Fundamental Frequency

$T_a = C_t h_n^x = 0.02 \cdot (176)^{0.75} = 0.966$

$C_t = 0.02$
$h_n = 176'$
$x = 0.75$

$T_a = \frac{1}{n_a}$ $\quad 0.966 = \frac{1}{n_a}$ $\quad n_a = 1.04$ ∴ considered rigid

Gust Effect Factor

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

$I_{\bar{z}} = c(33/\bar{z})^{1/6} = 0.30(33/99.6)^{1/6} = 0.25$

$c = 0.30$
$\bar{z} = 0.6 \cdot 166 = 99.6$
$g_Q = 3.4$
$g_v = 3.4$

$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{282+166}{462.45}\right)^{0.63}}} = 0.706$

$B = 282$
$h = 166$
$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}} = 320 \cdot (99.6/33)^{1/3} = 462.45$

$\ell = 320$
$\bar{z} = 99.6$
$\bar{\epsilon} = 1/3$

2-55

$$G = 0.925\left(\frac{(1+1.7\cdot 3.4\cdot 0.25\cdot 0.786)}{1+1.7\cdot 3.4\cdot 0.25}\right) = 0.809$$

Building is Enclosed

$GC_{pi} = \pm 0.18$

Find K_z

For $z < 15$ ft, $K_z = 2.01(15/z_g)^{2/\alpha}$

For $15\text{ft} < z < z_g$, $K_z = 2.01(z/z_g)^{2/\alpha}$

$z_g = 1200$
$\alpha = 7.0$

z	K_z
0'	0.575
7'	0.575
14'	0.575
21'	0.633
28'	0.687
35'	0.732
42'	0.771
49'	0.806
56'	0.837
63'	0.866
70'	0.892
76'	0.914
82'	0.934
88'	0.953
94'	0.971
100'	0.988
106'	1.000
112'	1.021
118'	1.036
124'	1.051
130'	1.065
136'	1.079
142'	1.092
148'	1.105
154'	1.118
160'	1.130
166'	1.142

Find q_z

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

z	Kz	qz (psf)
0'	0.575	0.00256·0.575·1.0·0.85·90²·1.0 = 10.13
7'	0.575	10.13
14'	0.575	10.13
21'	0.633	11.16
28'	0.687	12.11
35'	0.732	12.90
42'	0.771	13.59
49'	0.806	14.21
56'	0.837	14.75
63'	0.866	15.26
70'	0.892	15.72
76'	0.914	16.11
82'	0.934	16.46
88'	0.953	16.80
94'	0.971	17.11
100'	0.988	17.41
106'	1.000	17.63
112'	1.021	18.00
118'	1.036	18.26
124'	1.051	18.52
130'	1.065	18.77
136'	1.079	19.02
142'	1.092	19.23
148'	1.105	19.48
154'	1.118	19.71
160'	1.130	19.92
166'	1.142	20.13

$p = qGC_p - q_i(GC_{pi})$

$q = q_z$ @ z

$G_p = 0.808$

C_p - windward wall - 0.80

C_p - Leeward wall - 0.50
L/B = 152/282 = 0.539

C_p - side wall - -0.70

C_p, roof
h/L = 166/152 = 1.09 > 1.0

from 0' to 83' → C_p = -1.3
→ area reduction - 83·152 = 12616 > 1000 ∴ 0.8
from 83' to 152' → C_p = -0.7

2-57

$q_i = q_h = 20.13$

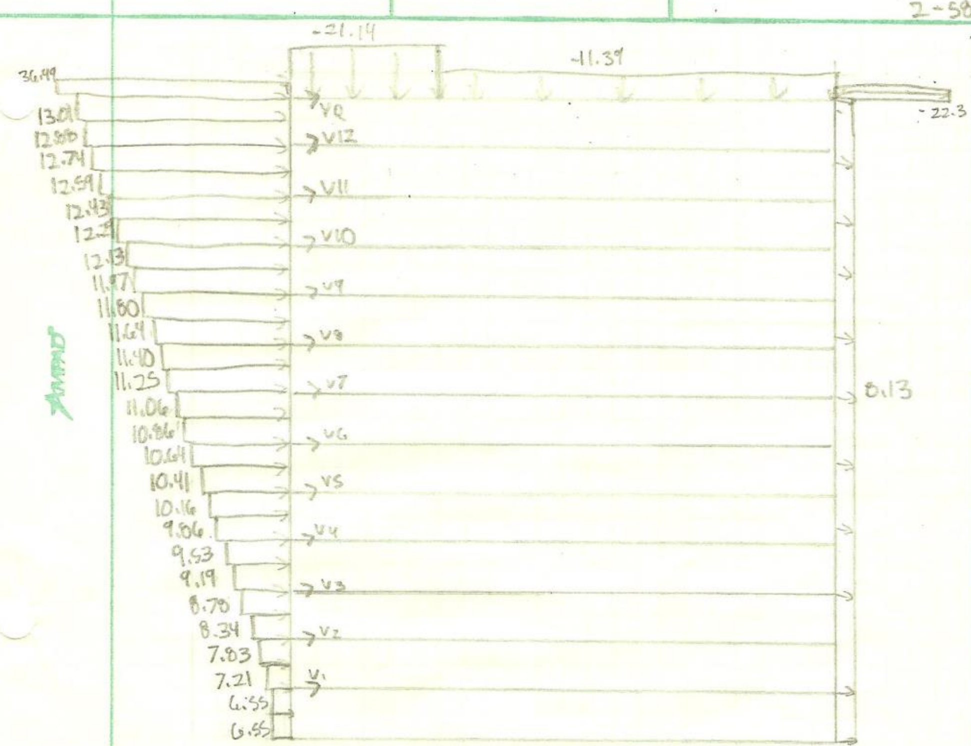
$(GC_{pi}) = \pm 0.18$

Parapets

$P_p = q_p G C_{pn}$

$q_p = 20.27 \cdot 1.8 = 36.49$ windward
$q_p = 20.27 \cdot -1.1 = -22.3$ leeward

					Net Pressure	
Location	z	q	q GCP	qi(GCPi)	+ GCpi	- GCPi
	0'	10.13	6.55	± 3.62	10.17	2.93
	7'	10.13	6.55	± 3.62	10.17	2.93
	14'	10.13	6.55	± 3.62	10.17	2.93
	21'	11.16	7.21	± 3.62	10.83	3.59
	28'	12.11	7.83	± 3.62	11.45	4.21
	35'	12.90	8.34	± 3.62	11.96	4.72
	42'	13.59	8.78	± 3.62	12.40	5.16
	49'	14.21	9.19	± 3.62	12.81	5.57
	56'	14.75	9.53	± 3.62	13.15	5.91
	63'	15.26	9.86	± 3.62	13.48	6.24
	70'	15.72	10.16	± 3.62	13.78	6.54
windward	76'	16.11	10.41	± 3.62	14.03	6.79
	82'	16.46	10.64	± 3.62	14.26	7.02
	88'	16.80	10.86	± 3.62	14.48	7.24
	94'	17.11	11.06	± 3.62	14.68	7.44
	100'	17.41	11.25	± 3.62	14.87	7.63
	106'	17.63	11.40	± 3.62	15.02	7.78
	112'	18.00	11.64	± 3.62	15.26	8.02
	118'	18.26	11.80	± 3.62	15.42	8.18
	124'	18.52	11.97	± 3.62	15.59	8.36
	130'	18.77	12.13	± 3.62	15.75	8.51
	136'	19.02	12.29	± 3.62	15.91	8.67
	142'	19.23	12.43	± 3.62	16.05	8.81
	148'	19.48	12.59	± 3.62	16.21	8.97
	154'	19.71	12.74	± 3.62	16.36	9.12
	160'	19.92	12.88	± 3.62	16.50	9.26
	166'	20.13	13.01	± 3.62	16.63	9.39
leeward	166'	20.13	8.13	± 3.62	11.75	4.51
side wall	166'	20.13	-11.39	± 3.62	-7.77	-15.01
Roof (0' to 83')	166'	20.13	-21.14	± 3.62	-17.52	-24.76
Roof (83' to 152')	166'	20.13	-11.39	± 3.62	-7.77	-15.01



$V = B \cdot (p_{wind} + p_{lee})(\text{trib area})$

$V_1 = 282 \cdot (6.55 + 8.13)(7) + 282 \cdot (7.21 + 8.13) \cdot (7) = 59.3\,k$

$V_2 = 282 \cdot (7.83 + 8.13)(7) + 282 \cdot (8.34 + 8.13) \cdot (7) = 64\,k$

$V_3 = 282 \cdot (8.78 + 8.13)(7) + 282 \cdot (9.19 + 8.13) \cdot (7) = 67.6\,k$

$V_4 = 282 \cdot (9.53 + 8.13)(7) + 282 \cdot (9.86 + 8.13) \cdot (7) = 70.4\,k$

$V_5 = 282 \cdot (10.16 + 8.13)(7) + 282 \cdot (10.41 + 8.13)(6) = 67.5\,k$

$V_6 = 282 \cdot (10.64 + 8.13)(6) + 282(10.86 + 8.13)(6) = 63.9\,k$

$V_7 = 282 \cdot (11.06 + 8.13)(6) + 282(11.25 + 8.13)(6) = 65.3\,k$

$V_8 = 282 \cdot (11.40 + 8.13)(6) + 282(11.64 + 8.13)(6) = 66.5\,k$

$V_9 = 282 \cdot (11.80 + 8.13)(6) + 282(11.97 + 8.13)(6) = 67.7\,k$

2-59

$V_{10} = 282 \cdot (12.13 + 8.13)(6) + 282(12.29 + 8.13)(6) = 68.8k$

$V_{11} = 282 \cdot (12.43 + 8.13)(6) + 282(12.59 + 8.13)(6) = 69.8k$

$V_{12} = 282 \cdot (12.74 + 8.13)(6) + 282(12.86 + 8.13)(6) = 70.9k$

$V_R = 282 \cdot (13.01 + 8.13)(6) + 282(36.49 + 8.13)(6) = 111.3k$

$V_{hs} = 59.3 + 64 + 67.6 + 70.4 + 62.3 + 63.9 + 65.3 + 66.5 + 67.7$
$+ 68.8 + 69.8 + 70.9 + 111.3 = 907.8k$

2-60

Wind (North - South Direction)

Analytical Procedure

Risk Category - II
$V = 90$ mph
$K_d = 0.85$
Exposure Category B

Topographic Factor

$K_{zt} = 1.0$ (no nearby hill, ridge, or escarpment)

Determine Fundamental Frequency

$T_a = C_t h_n^x = 0.02 \cdot (176)^{0.75} = 0.966$

$C_t = 0.02$
$h_n = 176$
$x = 0.75$

$T_a = \frac{1}{n_a}$ $\quad 0.996 = \frac{1}{n_a}$ $\quad n_a = 1.04$ ∴ considered rigid

Gust Effect Factor

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

$I_{\bar{z}} = c(33/\bar{z})^{1/6} = 0.30(33/99.6)^{1/6} = 0.25$

$c = 0.30$
$\bar{z} = 0.6 \cdot 166 = 99.6$
$g_Q = 3.4$
$g_v = 3.4$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{152+166}{462.45}\right)^{0.63}}} = 0.817$$

$B = 152$
$h = 166$
$L_{\bar{z}} = \ell(\bar{z}/33)^{\bar{\epsilon}} = 320 \cdot (99.6/33)^{1/3} = 462.45$

$\ell = 320$
$\bar{z} = 99.6$
$\bar{\epsilon} = 1/3$

2-61

$G = 0.925\left(\frac{(1+1.7\cdot 3.4\cdot 0.25\cdot 0.917)}{1+1.7\cdot 3.4\cdot 0.25}\right) = 0.825$

Building is Enclosed

$GC_{pi} = \pm 0.18$

Find K_z

for $z < 15$ ft, $K_z = 2.01(15/z_g)^{2/\alpha}$ $z_g = 1200$

For $15\text{ ft} < z < z_g$, $K_z = 2.01(z/z_g)^{2/\alpha}$ $\alpha = 7.0$

z	K_z
0'	0.575
7'	0.575
14'	0.575
21'	0.633
28'	0.687
35'	0.732
42'	0.771
49'	0.806
56'	0.837
63'	0.866
70'	0.892
76'	0.914
82'	0.934
88'	0.953
94'	0.971
100'	0.988
106'	1.000
112'	1.021
118'	1.036
124'	1.051
130'	1.065
136'	1.079
142'	1.092
148'	1.105
154'	1.118
160'	1.130
166'	1.147

Find q_z

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

2-62

z	kz	qz (psf)
0'	0.575	0.00256 · 0.575 · 1.0 · 0.85 · 90² · 1.0 = 10.13
7'	0.575	10.13
14'	0.575	10.13
21'	0.633	11.16
28'	0.687	12.11
35'	0.732	12.90
42'	0.771	13.59
49'	0.806	14.21
56'	0.837	14.75
63'	0.866	15.26
70'	0.892	15.72
76'	0.914	16.11
82'	0.934	16.46
88'	0.953	16.80
94'	0.971	17.11
100'	0.988	17.41
106'	1.000	17.63
112'	1.021	18.00
118'	1.036	18.26
124'	1.051	18.52
130'	1.065	18.77
136'	1.079	19.02
142'	1.092	19.23
148'	1.105	19.48
154'	1.118	19.71
160'	1.130	19.92
166'	1.142	20.13

$p = qG_iC_p - q_i(GC_{pi})$

$q = q_z$ @ z

$G_p = 0.825$

C_p - windward wall - 0.80

C_p - leeward wall = 0.328

L/B = 282/152 = 1.86

C_p - side wall - -0.70

C_p; roof

h/L = 166/282 = 0.589

- from 0' to 83' → C_p = -0.972
- from 83' to 166' → C_p = -0.864
- from 166' to 282 → C_p = -0.536

2-63

$q_i = q_h = 20.13$

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

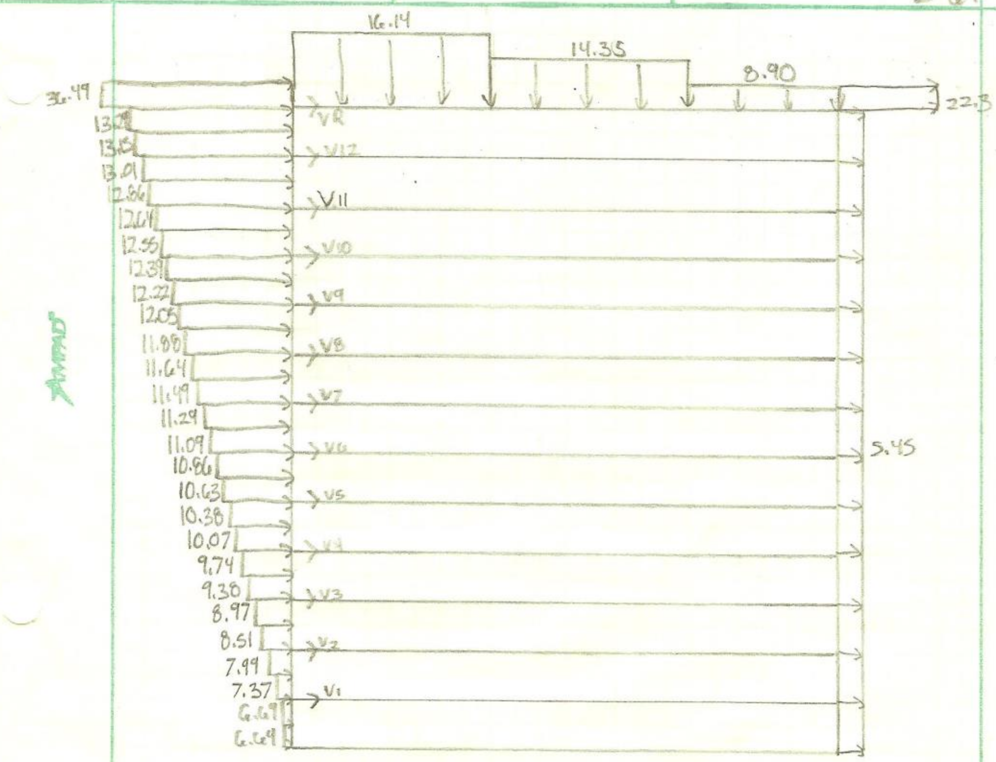
Parapets

$P_p = q_p GC_{pn}$

$q_p = 20.27 \cdot 1.8 = 36.49$ windward
$q_p = 20.27 \cdot -1.1 = -22.3$ leeward

Location	z	q	qGCP	qi(GCPi)	Net Pressure +GCPi	Net Pressure -GCPi
Windward	0'	10.13	6.69	±3.62	10.31	3.07
	7'	10.13	6.69	±3.62	10.31	3.07
	14'	10.13	6.69	±3.62	10.31	3.07
	21'	11.16	7.37	±3.62	10.99	3.75
	28'	12.11	7.99	±3.62	11.61	4.37
	35'	12.90	8.51	±3.62	12.13	4.89
	42'	13.59	8.97	±3.62	12.59	5.35
	49'	14.21	9.38	±3.62	13.00	5.76
	56'	14.75	9.74	±3.62	13.36	6.12
	63'	15.26	10.07	±3.62	13.69	6.45
	70'	15.72	10.38	±3.62	14.00	6.76
	76'	16.11	10.63	±3.62	14.25	7.01
	82'	16.46	10.86	±3.62	14.48	7.24
	88'	16.80	11.09	±3.62	14.71	7.47
	94'	17.11	11.29	±3.62	14.91	7.67
	100'	17.41	11.49	±3.62	15.11	7.87
	106'	17.63	11.64	±3.62	15.26	8.02
	112'	18.00	11.88	±3.62	15.50	8.26
	118'	18.26	12.05	±3.62	15.67	8.43
	124'	18.52	12.22	±3.62	15.84	8.60
	130'	18.77	12.39	±3.62	16.01	8.77
	136'	19.02	12.55	±3.62	16.17	8.93
	142'	19.23	12.69	±3.62	16.31	9.07
	148'	19.48	12.86	±3.62	16.48	9.24
	154'	19.71	13.01	±3.62	16.63	9.39
	160'	19.92	13.15	±3.62	16.77	9.53
	166'	20.13	13.29	±3.62	16.91	9.67
Leeward	166'	20.13	-5.45	±3.62	9.07	1.83
Side wall	166'	20.13	-11.63	±3.62	-8.01	-15.25
Roof (0'-83')	166'	20.13	-16.14	±3.62	-12.52	-19.76
Roof (83'-166')	166'	20.13	-14.35	±3.62	-10.73	-17.97
Roof (166'-282')	166'	20.13	-8.90	±3.62	-5.28	-12.52

2-64



$V = B \cdot (p_{wind} + p_{lee})(\text{trib area})$

$V_1 = 152 \cdot (6.69 + 5.45)(7) + 152 \cdot (7.37 + 5.45)(7) = 26.6k$

$V_2 = 152 \cdot (7.99 + 5.45)(7) + 152 \cdot (8.51 + 5.45)(7) = 29.2k$

$V_3 = 152 \cdot (8.97 + 5.45)(7) + 152 \cdot (9.38 + 5.45)(7) = 31.1k$

$V_4 = 152 \cdot (9.74 + 5.45)(7) + 152 \cdot (10.07 + 5.45)(7) = 32.7k$

$V_5 = 152 \cdot (10.38 + 5.45)(7) + 152 \cdot (10.63 + 5.45)(6) = 31.5k$

$V_6 = 152 \cdot (10.86 + 5.45)(6) + 152 \cdot (11.09 + 5.45)(6) = 30.0k$

$V_7 = 152 \cdot (11.29 + 5.45)(6) + 152 \cdot (11.49 + 5.45)(6) = 30.7k$

$V_8 = 152 \cdot (11.64 + 5.45)(6) + 152 \cdot (11.88 + 5.45)(6) = 31.4k$

$V_9 = 152 \cdot (12.05 + 5.45)(6) + 152 \cdot (12.22 + 5.45)(6) = 32.1k$

2-65

$$V_{10} = 152 \cdot (12.39 + 5.45)(6) + 152 \cdot (12.55 + 5.45)(6) = 32.7k$$

$$V_{11} = 152 \cdot (12.64 + 5.45)(6) + 152 \cdot (12.86 + 5.45)(6) = 33.2k$$

$$V_{12} = 152 \cdot (13.01 + 5.45)(6) + 152 \cdot (13.15 + 5.45)(6) = 33.8k$$

$$V_R = 152 \cdot (13.29 + 5.45)(6) + 152 \cdot (36.49 + 5.45)(6) = 55.3k$$

$$V_{ns} = 26.6 + 29.2 + 31.1 + 32.7 + 29.1 + 30.0 + 30.7 + 31.4 + 32.1 + 32.7 + 33.2 + 33.8 + 55.3 = 427.9k$$

2-66

Seismic Loads

Site Class C - from Geotechnical Report
-soft rock conditions

S_s - 0.408g

S_1 - 0.089g

S_{MS} - 0.490g

S_{M1} - 0.152g

S_{DS} - 0.327g

S_{D1} - 0.101g

Seismic Design Catagory

$0.167g < S_{DS} < 0.33g \rightarrow B$ } use B

$0.067g < S_{D1} < 0.133g \rightarrow B$

Use Equivalent Lateral Force Procedure

Ordinary Reinforced Concrete Shear Walls

$R = 5$
$\Omega_0 = 2\frac{1}{2}$
$C_d^b = 4\frac{1}{2}$

$I_e = 1.0$

$T_a = C_t h_n^x = 0.02 \cdot 166^{0.75} = 0.925$ secs

$C_t = 0.02$
$x = 0.75$
$h_n = 166$ ft

$C_s = \frac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \frac{0.101}{0.925\left(\frac{5}{1.0}\right)} = 0.022$

$C_s = \frac{S_{DS}}{R/I} = \frac{0.327}{\left(\frac{5}{1.0}\right)} = 0.0654$

$C_s = 0.044 S_{DS} I = 0.044 \cdot 0.327 \cdot 1.0 = 0.0144$

$C_s = 0.022$

2-67

Determine Building Weights

Roof = 178 psf · 22,500 ft^2 = 4005 k
Floor 2-12 = 162 psf · 22,500 ft^2 = 3645 k
Floor 1 = 162 psf · 24,000 ft^2 = 3888 k

Exterior Walls
Floor 1-3 630 plf · 508.12 ft + 210 plf · 206.29 ft = 363.44 k
Floor 4 – 660 plf · 508.12 ft + 220 plf · 206.29 ft = 380.74 k
Floor 5 – 590 plf · 508.12 ft + 197 plf · 206.29 ft = 340.43 k
Floor 6-12 – 540 plf · 508.12 ft + 180 plf · 206.29 ft = 311.52 k

Shear Walls
Floor 1-2 — 3209.5 ft^3 · 150 pcf = 481.4 k
Floor 3 — 2984.3 ft^3 · 150 pcf = 447.6 k
Floor 4 — 2808.7 ft^3 · 150 pcf = 421.3 k
Floor 5 — 2631.4 ft^3 · 150 pcf = 394.7 k
Floor 6-8 — 2407.5 ft^3 · 150 pcf = 361.1 k
Floor 9 — 2323 ft^3 · 150 pcf = 348.5 k
Floor 10-12 — 2407.5 ft^3 · 150 pcf = 361.1 k
Roof — 2407.5 ft^3 · 150 pcf = 361.1 k

$F_x = C_{vx} V$

$C_{vx} = \frac{w_x h_x^k}{\sum w_i h_i^k}$ $V = C_s W = 57394.3 \cdot 0.022 = 1262.67$ k

$k = \frac{0.925 - 0.5}{2.5 - 0.5}(2-1) + 1 = 1.21$

Level	h_x	W_x	$W_x h_x^k$	C_{vx}	F_x	V_x
Roof	166	4677.62	2.27×10^6	0.161	203.29	203.29
12	154	4317.62	1.91×10^6	0.135	170.46	373.75
11	142	4317.62	1.74×10^6	0.123	155.31	529.06
10	130	4317.62	1.56×10^6	0.11	138.89	667.95
9	118	4305.02	1.38×10^6	0.10	126.27	794.22
8	106	4317.62	1.22×10^6	0.09	113.64	907.86
7	94	4317.62	1.05×10^6	0.07	88.39	996.25
6	82	4317.62	8.93×10^5	0.06	75.76	1072.01
5	70	4380.13	7.48×10^5	0.05	63.13	1135.14
4	56	4447.04	5.80×10^5	0.04	50.51	1185.65
3	42	4456.04	4.1×10^5	0.03	37.88	1223.53
2	28	4489.84	2.53×10^5	0.02	25.25	1248.78
1	14	4732.84	1.15×10^5	0.01	12.63	1261.41

Base Shear = 1262.67 k

Distribution of Loads

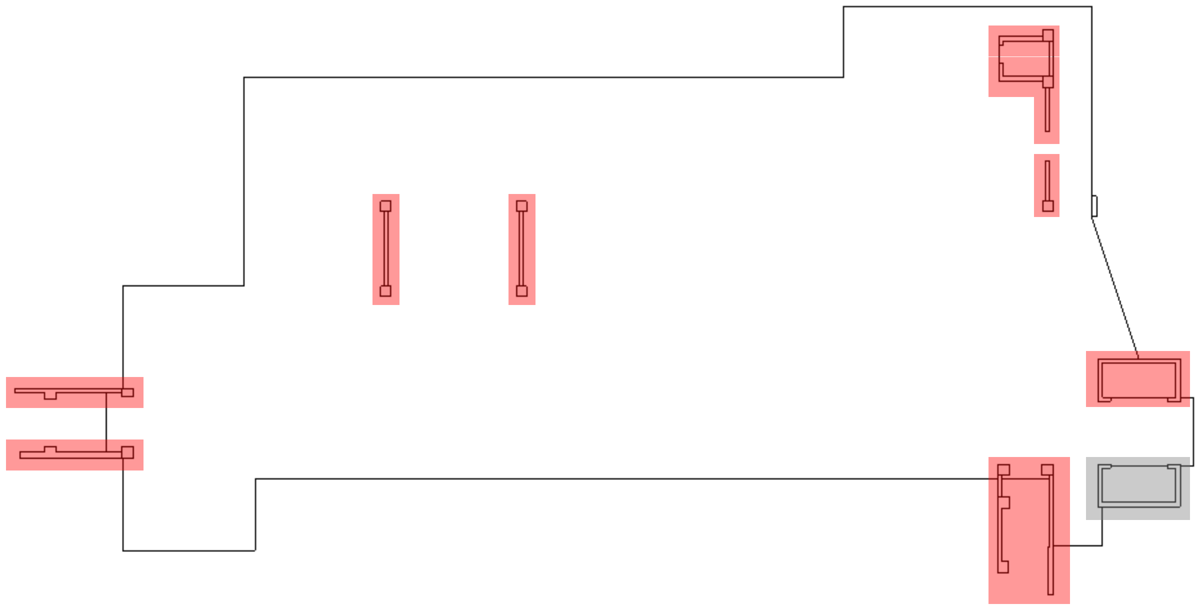


Figure B. Location of Shear Walls

The first step in determining how the loads are distributed to the different shear walls is to determine whether a wall is actually a shear wall or not. In order for a wall to be considered a shear wall, a large portion of the diaphragm must be able to transfer the lateral forces into the wall. A majority of the walls in this building are considered shear walls but due to the geometry of the building, the walls outlined in grey are not shear walls. These walls surround an elevator shaft which makes for a good location to put shear walls; however, the floor slab is not a large enough area to transfer the loads. The walls that were considered are highlighted in red in Figure B. The small walls that are part of the other two elevator shafts are also not considered due to their small size. These walls will only carry an extremely small amount of load compared to the nearby walls so they can be neglected. To aid in dividing up the loads, I named each wall as shown in Figure 3.

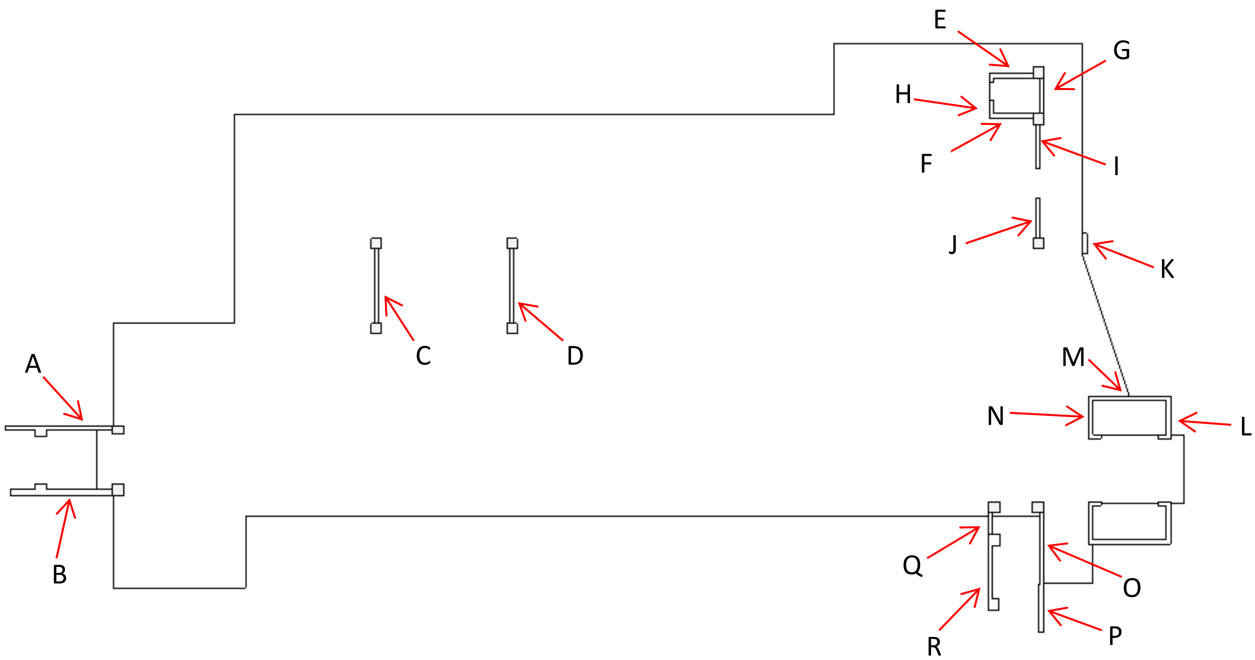


Figure C. Wall Designations

Before the loads can be distributed, the center of mass and the center of rigidity must be determined. In order to find the center of mass, the floor can be divided into several areas that have simple geometries; therefore, simplifying the calculation. Figure D shows the breakdown of the different floor areas. A sample calculation of the center of mass and center of rigidity is located in Appendix B.

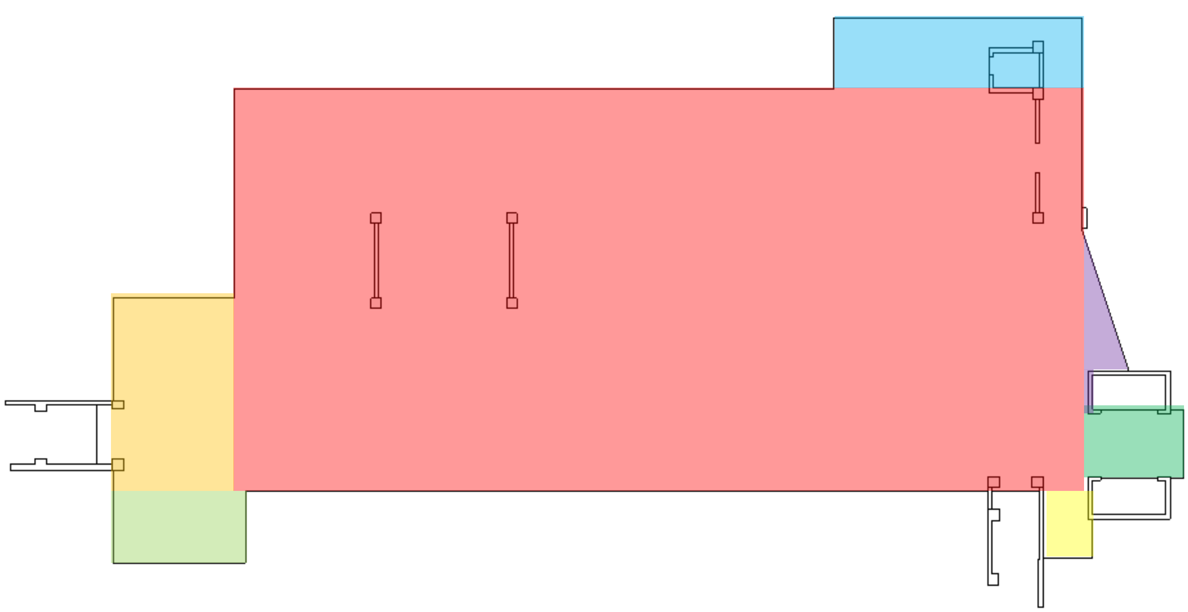


Figure D. Floor Area Breakdown by Color

Since the lateral system comprises of shear walls, the rigidities of the walls can be easily determined from a few calculations. These calculations can be viewed in the Appendix B. Based on the rigidity calculations, the critical wall in the North/South direction will be wall M. The critical wall in the East/West direction will be wall C. Both of these walls have a high stiffness ratio due to their longer lengths in relationship to the other walls. Figure E shows a chart of the rigidities of all of the walls on the fourth floor.

Wall	R (kip/in)	Reletive Rigidity
A	1285.97	9.17
B	1285.97	9.17
C	1535.72	10.95
D	1535.72	10.95
E	664.38	4.74
F	664.38	4.74
G	460.29	3.28
H	49.23	0.35
I	697.89	4.98
J	569.97	4.06
K	121.28	0.86
L	635.83	4.53
M	1496.68	10.67
N	635.83	4.53
O	1447.81	10.32
P	784.86	5.60
Q	154.49	1.10

Figure E. Rigidities of the Elements on Fourth Floor

The lateral forces that where calculated as part of the second technical assignment, can now be distributed to the lateral elements. As part of the distribution of the forces, a torsional force must be considered in the calculation if the building's center of mass and center of rigidity do not coincide. In the case of the Children's Medical Office Building, the center of mass and center of rigidity do not align. Figure F shows the locations of the center of mass and the center of rigidity. The difference in the distances between the center of mass and the center of rigidity creates an eccentricity that produces a moment that can add or subtract to the force of an element. Figure G shows the wind story shear distributed to the different lateral elements for the fourth floor. Figure H shows the seismic story shear distributed to the different lateral elements for the fourth floor.

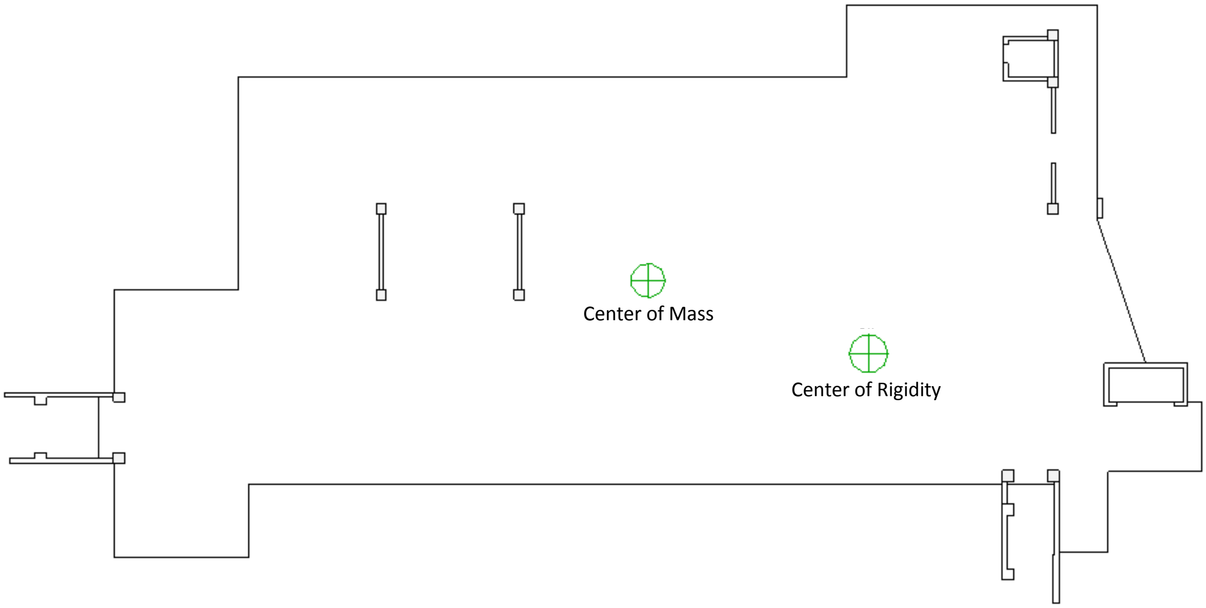


Figure F. Centers of Mass and Rigidity

Wall	Shear	Direct Shear	Torsional Shear	Total Force
A	343.4			
B	343.4			
C	343.4	61.13	54.99	116.12
D	343.4	61.13	38.91	100.04
E	343.4			
F	343.4			
G	343.4	18.31	-7.12	11.19
H	343.4	1.95	-0.57	1.38
I	343.4	27.80	-10.64	17.16
J	343.4	22.67	-8.67	13.99
K	343.4	4.80	-2.27	2.54
L	343.4	25.29	-16.05	9.24
M	343.4			
N	343.4	25.29	-12.24	13.05
O	343.4	57.61	-22.44	35.18
P	343.4	31.26	-12.13	19.13
Q	343.4	6.14	-1.78	4.36

Figure G. Distribution of Wind Forces

Wall	Shear	Direct Shear	Torsional Shear	Accidental Shear	Total Force
A	1185.65	282.47	-6.24	-10.31	265.93
B	1185.65	282.47	-11.67	-19.28	251.53
C	1185.65				
D	1185.65				
E	1185.65	146.01	12.02	19.87	177.90
F	1185.65	146.01	10.29	17.00	173.30
G	1185.65				
H	1185.65				
I	1185.65				
J	1185.65				
K	1185.65				
L	1185.65				
M	1185.65	328.68	-4.41	-7.29	316.98
N	1185.65				
O	1185.65				
P	1185.65				
Q	1185.65				

Figure H. Distribution of Seismic Forces

Strength Checks

As shown in Figures G and H, the seismic forces are much higher than the wind forces of a given story. This means that the seismic cases are going to control the design of the shear wall. In the case of wall C, the seismic case with a negative accidental torsional moment controls the design of the shear wall, as shown in Figure I. The shear wall has enough reinforcement to resist both the moment and the shear since both the design moment and shear are greater than the actual moment and shear.

Shear Wall Strength Check			
Mu (kip ft.)	ϕMn (kip ft.)	Vu (kips)	ϕVn (kips)
19920	20321	292.6	296.1

Figure I. Shear Wall Check

This seismic case also controls the story drift and the overturning moment. The story drift is shown in Figure J along with the code maximum story drift. The story drifts are well below the code maximum drifts. The overturning moment is determined from the story shears and the height of the building. The building's foundation counteracts the overturning moment along with the building's weight. In the case of the Children's Medical Office Building, the foundations consist of drilled piers and large footings to resist the overturning moment. The foundations underneath the elevators in the south western corner are large footings that are significantly wider than the shears walls that they support. The width of the footing is determined by the overturning moment that the footing has to counteract. The building's weight is high enough to counteract the overturning moment as shown in Figure K. For the drift of the building, the wind load case 1 applied in the East/West direction controls. The calculated drift is below the code maximum. The building drift is shown in Figure L.

Floor Height	Max. Drift Allowable by Code	Δ
166	3.32	0.059
154	3.08	0.050
142	2.84	0.045
130	2.6	0.041
118	2.36	0.047
106	2.12	0.033
94	1.88	0.026
82	1.64	0.022
70	1.4	0.045
56	1.12	0.018
42	0.84	0.014
28	0.56	0.009
14	0.28	0.005

Figure J. Story Drifts

Overturning Moment	Moment Produced by Building Weight
496375.5 ft-kips	4225837 ft-kips

Figure K. Overturning Moment

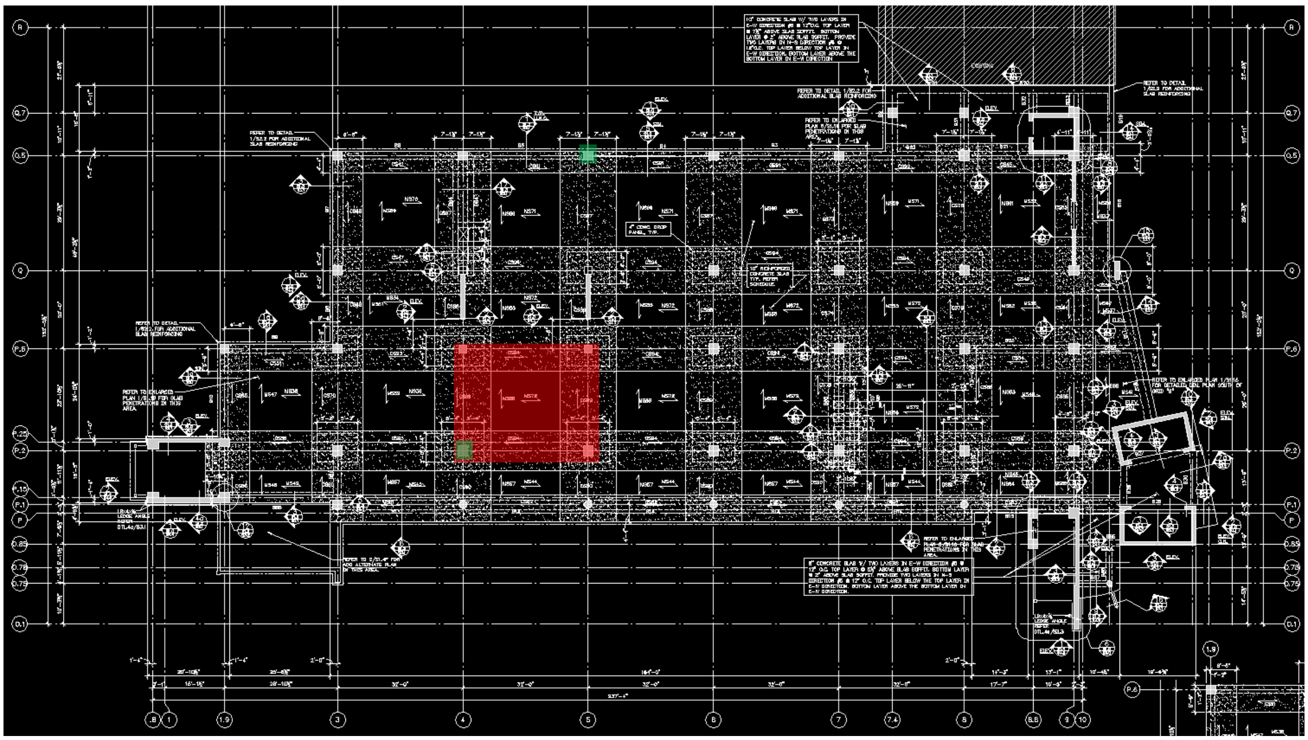
Code Maximum	Calculated Drfit
4.98 in.	4.77 in.

Figure L. Building Drift

Building Torsional Irregularities

Since the building's seismic design category is B, the only torsional irregularity is the out-of-plane offsets. My building does not have any discontinuities in the lateral force resisting path; therefore, there are no torsional irregularities in this building. For vertical structural irregularities, my building is exempt from applying the torsional factors since there is no reduction in stiffness in the elements and there story lateral strength is not less than 80% of the story above.

Appendix A - Floor Plan with Typical Bay and Columns



Typical bay is outlined in red. Interior and exterior columns are outlined in green.

Appendix B Sample Calculations

Sample Center of Mass Calculation

Story 1

Floor A

$A = 1300\ ft^2$
$w = 0.142\ ksf$

$W = 0.142 \cdot 1300 = 184.6\ K$

$\bar{x} = 31.4$
$\bar{y} = 51.8$

$W\bar{x} = 184.6 \cdot 31.4 = 5796.44$
$W\bar{y} = 184.6 \cdot 51.8 = 9564.59$

$$\bar{x} = \frac{\Sigma W}{\Sigma W\bar{x}} = \frac{3355.74}{457278} = 136.27\ ft.$$

$$\bar{y} = \frac{\Sigma W}{\Sigma W\bar{y}} = \frac{3355.74}{247077.} = 73.63\ ft.$$

Sample Wall Rigidity Calculation

Story 1

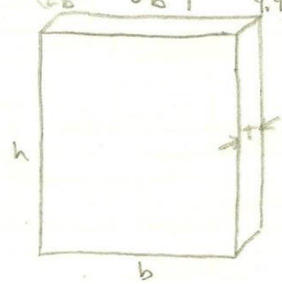
Wall A

$E = 57000 \cdot \sqrt{6000} = 4.415 \times 10^6 \quad G \approx 0.4 \cdot E = 1766080$

$P = K\Delta$

$P = 100\ Kips$

$$K = \frac{1}{\left(\frac{h^3}{Eb^3} + \frac{1.2h}{Gb}\right)} = \frac{1}{\frac{14^3}{4.415\times10^6 \cdot 23.95} + \frac{1.2 \cdot 14}{1766\times10^6 \cdot 23.95}} = 2261.23\ K/in$$



$$\text{Relative Rigidity} = \frac{R_i}{\Sigma R_i} = \frac{2261.23}{16603} = 13.62$$

Sample Center of Rigidity Calculation

$R_{xy} = R_x \cdot \bar{y} = 13.62 \cdot 46.83 = 637.87$

$R_{yx} = R_y \cdot \bar{x}$

$\bar{x} = \frac{\Sigma R_y}{\Sigma R_y \bar{x}} = \frac{57.58}{11054.35} = 191.98$

$\bar{Y} = \frac{\Sigma R_x}{\Sigma R_{xy}} = \frac{42.41}{2493.5} = 58.80$

Sample Wind Load Calculation

Story 1

Wall C

$V_i^d = \frac{R_y}{\Sigma R_y} \cdot V = \frac{9.25}{57.58} \cdot 430.3 = 69.13$ Kips

$V_i^T = \frac{V \cdot e\, d_y \cdot R_y}{J} = \frac{430.3 \cdot -50.98 \cdot 9.25 \cdot -117.89}{311617.9} = 73.51$ K

$J = \Sigma R_i d_i^2 = 311617.9$

total Force $= V_i^d + V_i^t = 142.6$ K

Sample drift Calculation

Story 1

Wall C

$P = 6.94$
$K = 1535.72$

$\Delta = P/K = \frac{6.94}{1535.72} = 0.004519$ in

Sample Overturning Moment Calculation

Wall C

Story 1 $\quad M = h \cdot P = 14 \cdot 752.89 = 10540.46$

total Moment $= \Sigma M = 496375.5$ ft·kips

$W \cdot \bar{y} = 57394.3 \cdot 73.63 = 4225838$ ft·kips > 496375.5 ft·Kips

Shear Wall Strength Check of Wall C

$U = 0.9D + 1.0E$

$M_u = 1.0 \cdot 180 \cdot 2/3 \cdot 166 = 19920$ Kip·ft

$T = A_s \cdot f_y = 12 \cdot 1.27 \cdot 60 = 914.4$ Kips

$d = l_w - (1.5in + 2 \cdot 9in) = 212 - 19.5 = 192.5$ in

$N_u = 0.9 N_D = 0.9(440 + 80 \cdot 12) = 1260$ kips

$\frac{150 \text{ lbs}}{ft^3} \cdot 17.67ft \cdot 166ft \cdot 1.0ft = 440$ Kips

$\frac{150 \text{ lbs}}{ft^3} \cdot 32ft \cdot 20ft \cdot \frac{10}{12} = 80$ Kips per floor

$a = \frac{T + N_u}{0.85 \cdot f'_c \cdot b} = \frac{914.4 + 1260}{0.85 \cdot 6 \cdot 12in} = 35.53in$

$\phi M_n = \phi\left[T\left(d - \frac{a}{2}\right) + N_u\left(\frac{l_w - a}{2}\right)\right]$

$0.9\left[914.4\left(192.5 - \frac{35.53}{2}\right) + 1260\left(\frac{212 - 35.53}{2}\right)\right]$

$\phi M_n = 20321.5 > 19920$ OK

$N_{pr} = N_D + N_L = 1260$ kips $+ 300$ kips $= 1560$ kips

$a = \frac{T + N_{pr}}{0.85 f'_c \cdot b} = \frac{914.4 + 1560}{0.85 \cdot 6 \cdot 12} = 40.4$ in

$M_{pr} = T\left(d - \frac{a}{2}\right) + N_{pr}\left(\frac{l_w - a}{2}\right)$

$= 914.4\left(192.5 - \frac{40.4}{2}\right) + 1560\left(\frac{212 - 40.4}{2}\right)$

$= 24283$ kip·ft

V_u (cap-based) $= \frac{M_{pr}}{0.5 h_w} = \frac{24283}{0.5 \cdot 166} = 292.6$ kips

$A_{cv} = h l_w = 12in \cdot 212 = 2544 in^2$

$\alpha_c = 2.0$ (slender wall)

$$p_t = \frac{A_{v,horiz}}{h s_2} = \frac{2 \cdot 0.31}{12in \cdot 12in} = 0.00431 > 0.0025$$

$$V_n = A_{cv}(\alpha_c \lambda \sqrt{f'_c} + p_t f_y)$$

$$= 2544 \cdot (2.0 \cdot 1.0 \cdot \sqrt{6000} + 0.00431 \cdot 60)$$

$$= 394.78\ k$$

$$8 \cdot 2544 \cdot \sqrt{6000} = 1576.5\ \text{Kips}$$

$$\emptyset V_n = 0.75 \cdot 394.78k = 296.1\ \text{kips} > 292.6\ \text{Kips}$$