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September 26, 2016

Dr. Linda Hanagan
The Pennsylvania State University
212 Engineering Unit A
University Park, PA 16802

Dear Dr. Hanagan,

I submit the follow document, Technical Report II – Building Codes, Specifications, and Loads, in support of my senior thesis program. This report has been written to identify the load determination process and all building codes applied to the analysis and design of 706 Madison Avenue. Due to the complexity of my building, hand calculations with graphs has been chosen as a major way of illustrating the relevant structural loads imposed on this building.

In addition, the report consists of an executive summary, site plan and location plan, and a brief introduction in order to provide a better understanding of the building and the purpose of this report. The hand calculations presents the determination of structural gravity, wind, and seismic loads imposed on the building.

Thank you for your consideration and evaluation of this report.

Sincerely,

Yong Yue

The Pennsylvania State University

Architectural Engineering Class of 2017

706 Madison Avenue | New York, USA

Building Codes, Specifications, and Loads

Structural Notebook Submission A

Submitted to: Dr. Linda Hanagan, Advisor

Prepared by: Yong Yue [Structural Option] Prepared

on: September 26th, 2016

Executive Summary

706 Madison Avenue is a 48,500 square-foot, high-end retail building located on the southwest corner of Madison Avenue and 63rd Street of the upper east side of Manhattan, New York. The building consists of a 3-story existing landmarked building and a five-story horizontal extension on two sides.

The existing landmarked building was built in 1920 and was initially constructed with masonry walls, steel columns, cinder concrete slabs, and marble and brick façades. Back to 1920s, building codes didn't require seismic design for structures. So the old building wasn't designed to resist seismic load; however, the masonry walls and core stairwells in the building have been designed for wind.

The addition took place on March 2015. It is still under construction and scheduled to be done in January 2016. The structural system consists of steel columns, concrete slab with composite metal deck, a mat foundation and moment frames for a lateral load resisting system. However, the addition's lateral load resisting system is independent from the existing building.

The building was designed based on the 2008 New York City Building Code. The exterior of building also needs to meet the historical requirements, which are regulated by Landmark Preservation Commission (LPC).

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[1] Introduction

1.1 Purpose

This report has been written in order to develop a detailed description of the loading conditions on 706 Madison Avenue. The loads described and analyzed in this report will serve as a foundation of technical knowledge for an investigation of the existing structural system of 706 Madison Avenue in the following reports.

1.2 Scope

The content of this report is comprised of three major sections: gravity loads, wind loads, and seismic loads. The structural loads imposed on this building are shown by hand calculations as well as graphs.

1.3 Site Location and Plan

As shown in the figure above (Figure 1 & 2), 706 Madison Avenue is located at the southwest corner of Madison Avenue and East 63rd Street, which is in a historical district at the upper east side of Manhattan, New York. The shape of the site is basically a rectangular, with a demension 90’x100’.

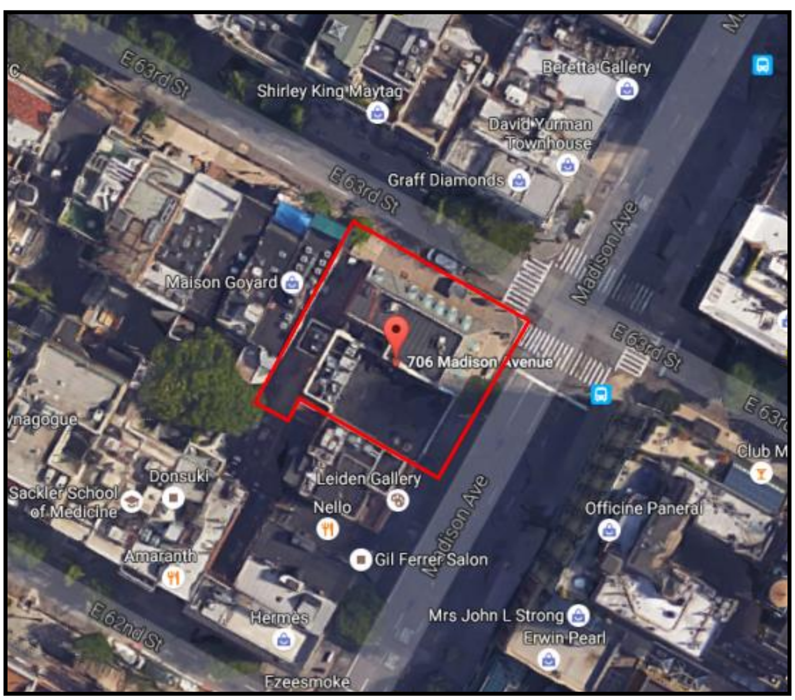


Figure 1
Site Context

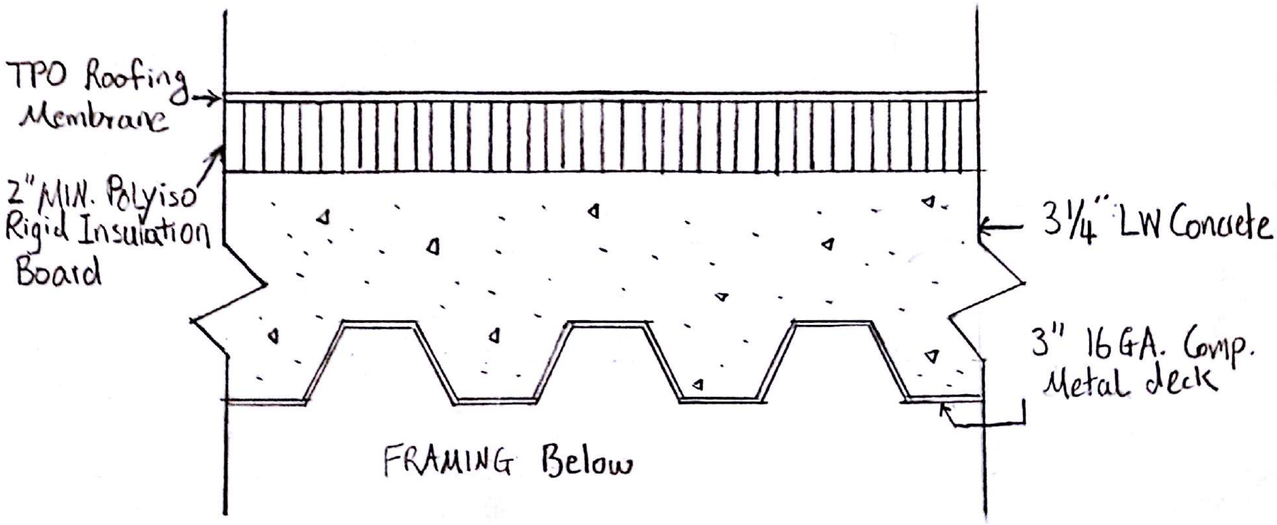
Figure 2 Site Context (Google Map)

1.4 Building Codes & Reference Standards

A. New York City Building Codes (NYCBC), 2008 with Current Revisions
B. ASCE 7 – 02 Minimum Design Loads for Buildings and Other Structures

[2] GRAVITY LOAD

2.1 Cross section of roof constuction



- Roof Loading
  - Dead Load: (according to ASCE7-02 Table C3-1)

Roofing Membrane:	1 psf
2" Rigid Insulation:	3 psf
3¼" LW Concrete } 3" 16GA. Comp. deck } :	46 psf
Framing:	7 psf
Miscellaneous:	10 psf
	$DL_r = 67$ psf

  - Live Load:

    $LL_r = 20$ psf (according to ASCE7-02 C4.9.1 Min. Roof Live Loads)

  - Snow Load:

    Ground Snow Load; $P_g = 25$ psf (ASCE7-02 Figure 7-1)

    Exposure Factor; $C_e = 0.9$ (ASCE7-02 Table 7-2 for Terrain Category B and a fully exposed roof)

    Thermal Factor; $C_t = 1.0$ (ASCE7-02 Table 7-3)

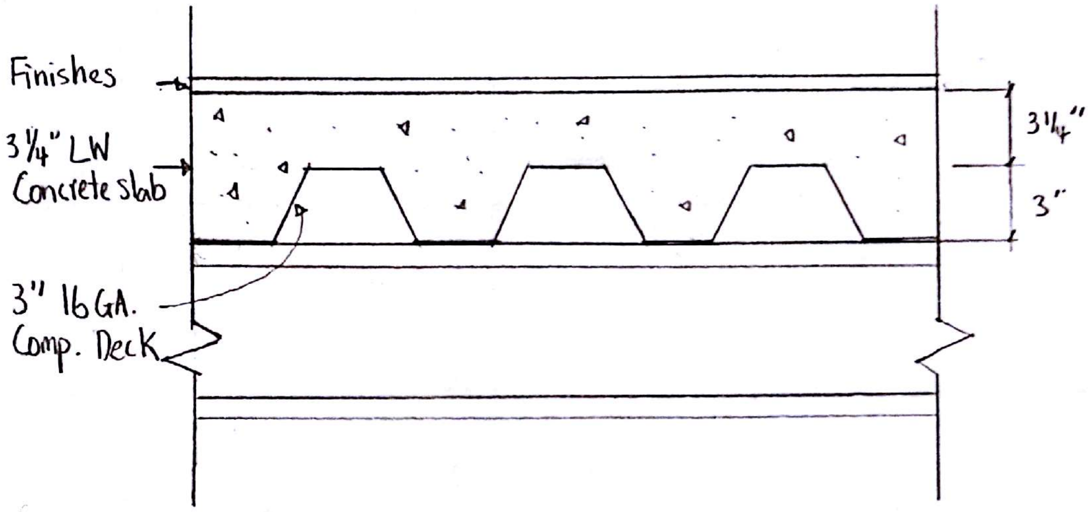
    Important Factor; $I_s = 1.0$ (ASCE7-02 Table 7-4)

    Flat Roof Snow Load; $P_f = 0.7\, C_e C_t I_s P_g$
    $= 0.7(0.9)(1)(I)(25) = 16$ psf < 20 psf (Min)

    Thus, use $P_f = 20$ psf.

GRAVITY LOAD (cont)

2.2 Cross section of floor construction



- Floor Loading
  - Dead Load

Finishes:	2 psf
Concrete Slab + deck:	46 psf
Framing:	7 psf
Columns:	1 psf
Miscellaneous:	10 psf

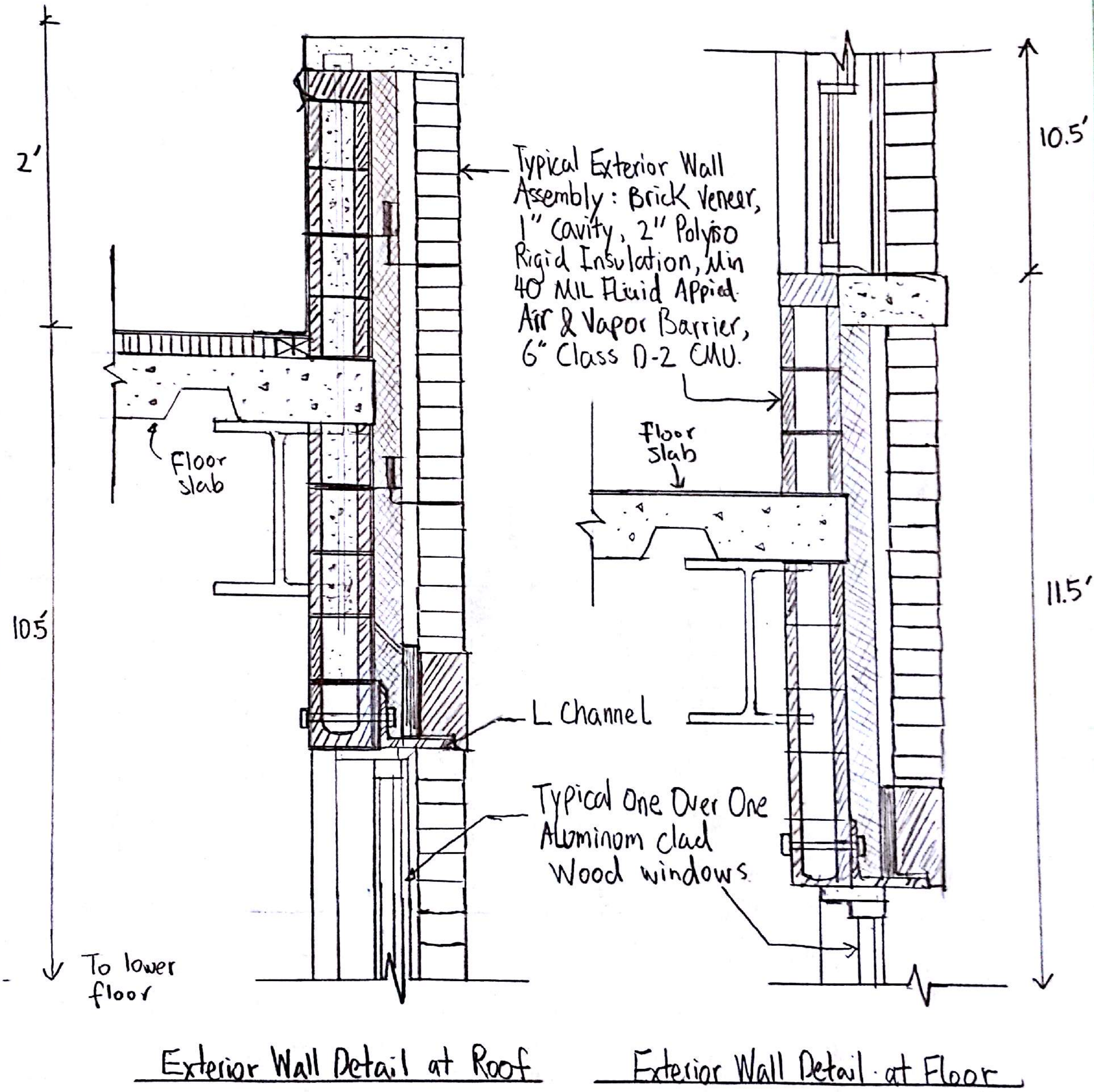
$DL_f = 66$ psf

  - Live load:

	Number Noted in Drawings	Code Minimum (ASCE7-02)
Retail – 1st Floor	105 psf	100 psf
Retail – Upper Floors	75 psf	75 psf
Public Assembly Space	100 psf	100 psf
Stairs and Exits	100 psf	100 psf
Storage	125 psf	125 psf
Side walk	600 psf	250 psf
Elevator Machine Room	125 psf	150 psf

GRAVITY LOAD (cont.)

2.3 Cross section of typical wall details with load path description and dead Load.



• Load Path:

Wall loads act on the L channels, L channels transfer Loads into 6" Concrete masonry unit (CMU) throught the bolts, CMUs transfer loads to the concrete slab with composite metal deck, concrete slabs transfer load to the transverse beams, the transverse beams transfer loads to the columns, and the columns transfer loads down to foundation.

GRAVITY LOAD (Cont.)

- Dead Load of Wall (From ASCE7-02 Table C3-1)

Hollow CMU wythes, 6", full grout:	55 psf
Clay brick wythes, 4":	39 psf
Rigid Insulation, 2":	1½ psf
Vapor Barrier:	½ psf
	$DL_w = 96$ psf

For Roof:

$$W_{wall} = 96\left(\frac{10.5}{2} + 2\right) = 696 \text{ Plf}$$

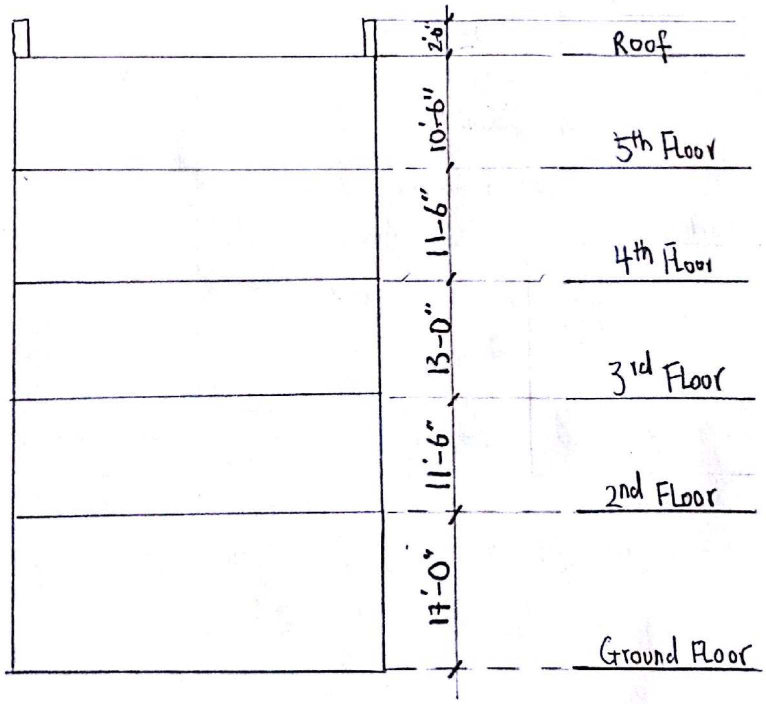
For floors:

$$W_{wall(5)} = 96\left(\frac{10.5}{2} + \frac{11.5}{2}\right) = 1{,}056 \text{ Plf}$$

$$W_{wall(4)} = 96\left(\frac{11.5}{2} + \frac{13}{2}\right) = 1176 \text{ Plf}$$

$$W_{wall(3)} = 1176 \text{ Plf}$$

$$W_{wall(2)} = 96\left(\frac{11.5}{2} + \frac{17}{2}\right) = 1368 \text{ Plf}$$



East Elevation

2.4 Snow Load

- Drift @ Parapet (Windward drift)

$P_g = 25$ psf
$P_f = 20$ psf   (From previous cals.)

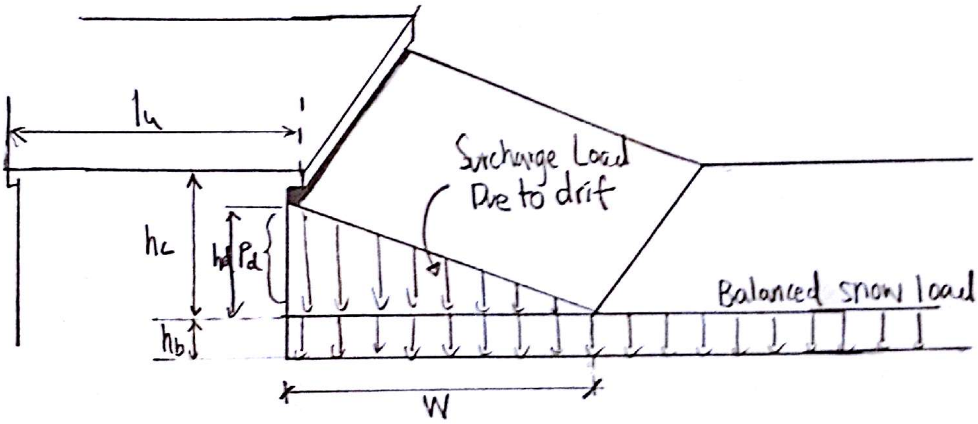
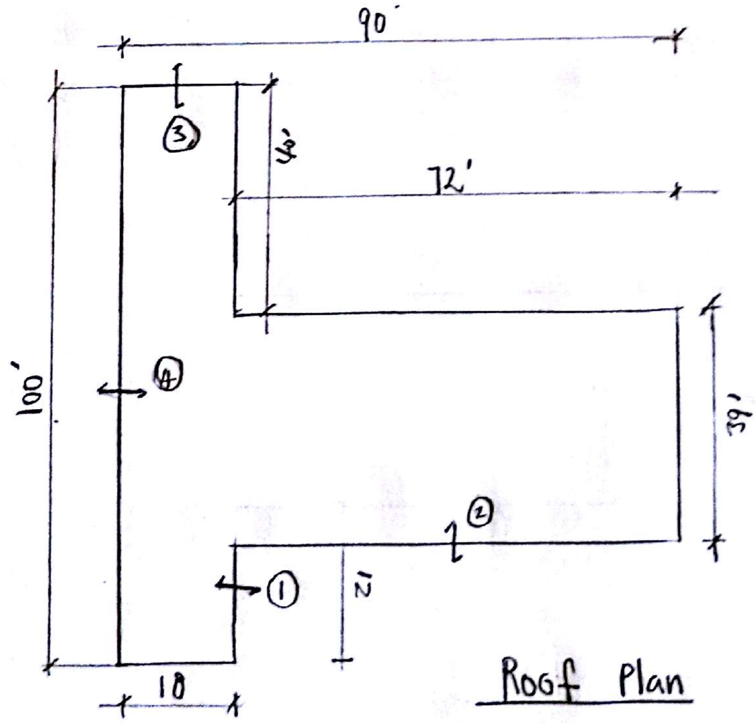


Figure 7.8 Snow Drifts on Lower Roof (ASCE 7-02)

$\gamma = 0.13 P_g + 14 = 0.13(25) + 14 = 17.25$ PCF (but no more than 30 PCF)

$h_b = 20/17.25 = 1.16$ ft

$h_c = 2' - 1.16' = 0.84'$; $\frac{h_c}{h_b} = \frac{0.84}{1.16} = .72 > 0.2 \Rightarrow$ drift load must to be considered.



Snow Load (cont.)

Parapet Section ①:

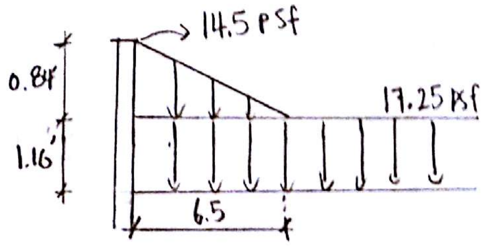
$l_u = 18' < 25 \therefore$ use $l_u = 25'$

$h_d = 0.75[0.43\sqrt[3]{25}\sqrt[4]{25+10} - 1.5] = 1.17 > h_c \therefore$ Same drift for ②, ③, ④

$W = 4h_d^2/h_c$ & $h_d = h_c$

$W = \frac{4(1.17)^2}{0.84} = 6.5' < 8h_c = 8(0.84) = 6.72' \therefore W = 6.5'$

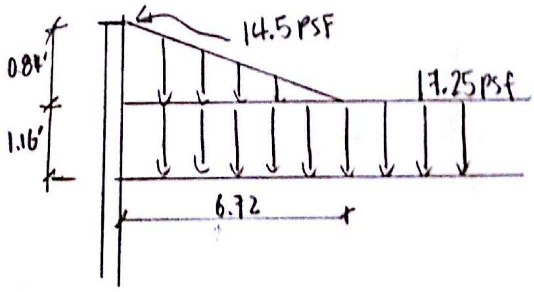
$P_d = \gamma h_d = \gamma h_c = 17.25(0.84) = 14.5$ psf



Section 1

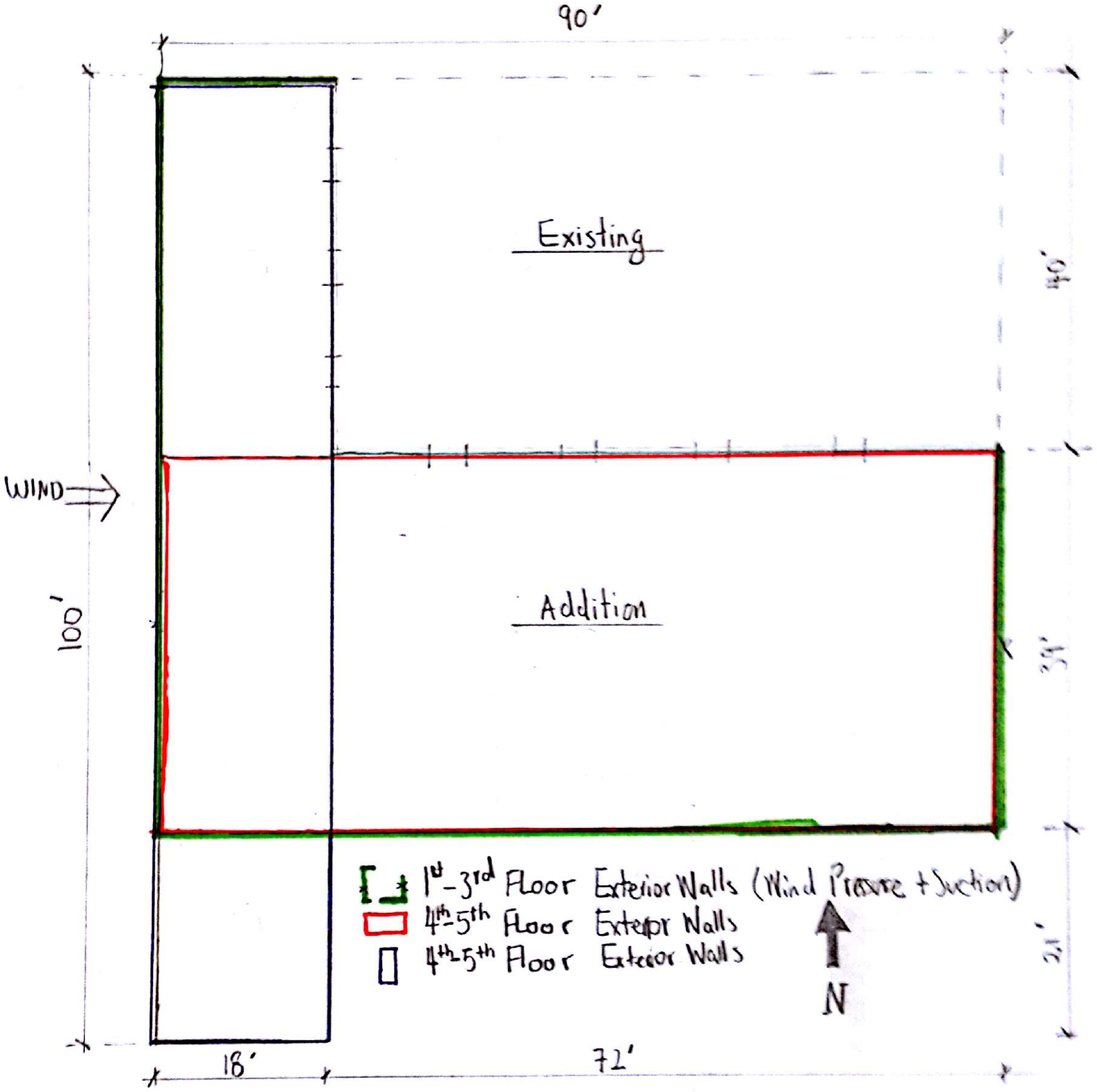
②: $h_d = 0.75[0.43\sqrt[3]{39}\sqrt[4]{25+10} - 1.5] = 1.54 > h_c$

$W = \frac{4(1.54)^2}{0.84} = 11.2' > 8h_c = 6.72' \therefore W = 6.72'$ for ②, ③, ④



$P_d = \gamma h_c = 17.25(0.84) = 14.5$ psf

[3] WIND LOAD



- Risk Category: II (ASCE7-02 Table 1-1)
- Wind Speed: 98 mph (Figure 6-1, 3-sec gust)
- Wind Directionality Factor: $K_d = 0.85$ (Table 6-4)
- Wind Important Factor: $I_w = 1.0$ (Table 6-1)
- Exposure Category: B (Section 6.5.6.3)
- Topograhic Factor: $K_{zt} = 1$ (1.0 except for isolated escarpments, ridges & hills Section 6.5.7)

WIND LOAD (cont.)

- Velocity Pressure Coefficients, K_z (Table 6-3)

Story	Ht. z (ft)	Story Ht (ft)	K_z	I	K_{zt}	K_d	V^2	q_z (psf)
1	0	17	0.7	1	1	.85	9604	14.6
2	17	11.5	0.7	1	1	.85	9604	14.6
3	28.5	13	0.7	1	1	.85	9604	14.6
4	41.5	11.5	0.77	1	1	.85	9604	16.1
5	53	10.5	0.82	1	1	.85	9604	17.1
Roof	63.5		.86	1	1	.85	9604	18.0

* $q_z = 0.00256\, K_z K_{zt} K_d V^2 I$ (Eq. 6-15)

- Gust Effect Factor:

For structure steel moment resisting frame buildings:

$n_a = 22.2/h^{0.8} = 22.2/(63.5)^{.8} = .802 < 1$ (ASCE7-10, Eq. 26.9-2)

$\therefore$ Not rigid.

$\beta = 0.01$; Conservative for steel (ASCE7-10, Structural damping)

Since ASCE7-02 doesn't have this section.

Exposure B $\Rightarrow$ $\hat{\alpha} = 1/4$, $\hat{b} = 0.45$, $\ell = 320'$, $\bar{\epsilon} = 1/3$, $c = 0.3$ (ASCE7-02 Table 6-2)

$\bar{z} = 0.6(63.5) = 38.1' > 30'$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{38.1}{33}\right)^{1/3} = 336$ (Eq. 6-7)

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{38.1}\right)^{1/6} = .293$ (Eq. 6-5)

Section	h (ft)	Bew (ft)	Lew (ft)
①	63.5	79	90
②	63.5	100	18
③	63.5	39	90

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WING Load (cont.)

For ① Wind E-W →

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{79+63.5}{336}\right)^{0.63}}} = .855$$

For ②

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{100+63.5}{336}\right)^{0.63}}} = .845$$

③

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{39+63.5}{336}\right)^{.63}}} = .818$$

} Eq. 6-6

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.45\left(\frac{38.1}{33}\right)^{1/4}(98)\left(\frac{88}{60}\right) = 67.0 \text{ ft/s} \quad \text{(Eq. 6-14)}$$

R, the resonant response factor, $R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$ (Eq. 6-10)

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{.802 * 336}{67.0} = 4.02$$

$$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}} = \frac{7.47 \times 4.02}{(1+10.3 * 4.02)^{5/3}} = .058 \quad \text{(Eq. 6-11)}$$

$$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) \quad \text{(Eq. 6-13a)}$$

$R_h = .245$ — $\eta = 4.6 n_1 h/\bar{V}_{\bar{z}} = 3.50$

$R_B = .203$ — $\eta = 4.6 n_1 B/\bar{V}_{\bar{z}} = 4.35$ ①
$R_B = .165$ — $\eta = 5.5$ ②
$R_B = .359$ — $\eta = 2.15$ ③
$R_L = .181$ — $\eta = 4.6 n_1 L/\bar{V}_{\bar{z}} = 4.96$ ①
$R_L = .570$ — $\eta = .99$ ②
$R_L = .181$ — $\eta = 4.96$ ③

$$R(①) = \sqrt{\frac{1}{0.01}(.058)(.245)(.203)(0.53+0.47 \times .181)} = .421$$

$$R(②) = \sqrt{\frac{1}{0.01}(.058)(.245)(.165)(0.53+0.47 \times .570)} = .433$$

$$R(③) = \sqrt{\frac{1}{0.01}(.058)(.245)(.359)(0.53+0.47 \times .181)} = .560$$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{.577}{\sqrt{2\ln(3600 n_1)}} = 4.14 \quad \text{(Eq. 6-9)}$$

WING Load (cont.)

(Eq. 6-8)

$$G_f = 0.925\left(\frac{1+1.7 I_z\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_z}\right) = \left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.855)^2+(4.14)^2(.421)^2}}{1+1.7(3.4)(.293)}\right) = .9232 \approx .92 \quad ①$$

$$G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.845)^2+(4.14)^2(.433)^2}}{1+1.7(3.4)(.293)}\right) = .9227 \approx .92 \quad \text{For } ②$$

$$G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.878)^2+(4.14)^2(.560)^2}}{1+1.7(3.4)(.293)}\right) = .99 \quad \text{For } ③$$

- Enclosed Building ⇒ Internal Pressure Coefficient, $GC_{pi} = \pm 0.18$ (Fig. 6-5)
- External Pressure Coefficient, C_p (Wind E-W) (Fig. 6-6)

For ①: $L/B = 1.14$; $h/L = .706$ $A = 79 * 100 = 7900\ ft^2 \Rightarrow$ Red. factor = 0.8

- Walls: C_p, windward, ew = 0.8
  C_p, leedward, ew = -0.47
  C_p, sidewall, ew = -0.7

1	-0.5
1.14	-0.47
2	-0.3

For ②: $L/B = .18$, $h/L = 3.53$ $A = 100 \times 18 = 1800\ ft^2 \Rightarrow R.f = .8$

- Walls: C_p, windward, ew = .8
  C_p, leedward, ew = -0.5
  C_p, sidewall, ew = -0.7
- Roofs: C_p, roof, ew (0-9) = -1.3 × 0.8 = -1.04
  C_p, roof, ew (9-18) = -0.7

For ③: $L/B = 2.31$, $h/L = .706$, $A = 39 * 100 = 3900\ ft^2 \Rightarrow Rf = .8$

- Walls: C_p. w, ew = .8
  C_p. L, ew = -0.28
  C_p, s, ew = -0.7

2	-0.3
2.31	
4	-0.2

- Roofs: C_p. roof, ew (0-31.75) = -.96
  C_p, roof, ew (31.75-63.5) = -.82
  C_p, roof, ew (63.5-90) = -.58

h/L	Cp 0-h/2	h/2-h	h-2h
0.5	-0.9	-0.9	-0.5
0.706	-0.96	-0.82	-0.58
1.0	-1.04	-0.7	-0.7

WIND LOAD (cont.)

- Pressure on the parapets K_z @ 65.5 = 0.87

$GC_{pi} = +1.5$ (Windward)
$= -1.0$ (leeward)

$q_p = 0.00256\,(0.87)(0.85)(1)(9604)(1) = 18.2$ psf

P_p. windward = $18.2 * (1.5) = 27.3$

P_p, leedward = $18.2 * (-1.0) = -18.2$

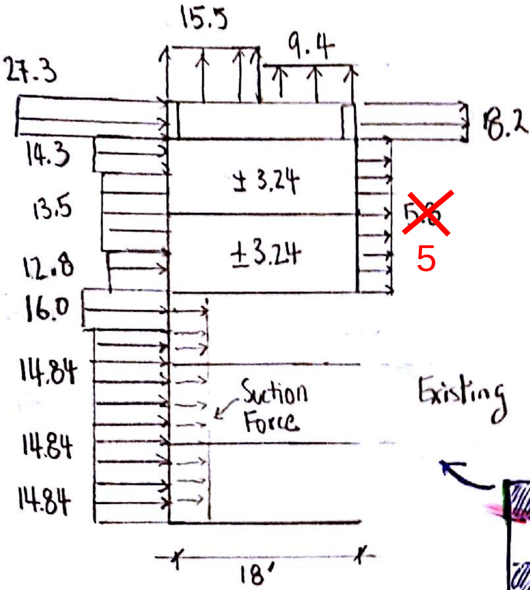
- Pressure on each surface : $P = qGC_p - q_i(GC_{pi})$ (Eq. 6-23)

Use $G_f = 0.99$ (Conservative)

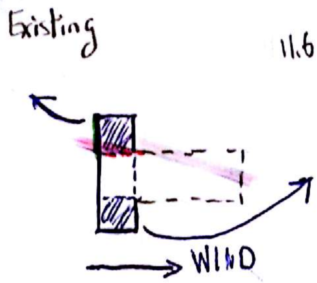
Location		z (ft)	q_z psf	C_p	q_zGC_p (psf)	GC_{pi}	q_hGC_{pi} (psf)	Net Pressure (psf): $q_zGC_p - q_h(+GC_p)$ (psf)	Net Pressure (psf): $q_zGC_p - q_h(-GC_p)$ (psf)
Windward		0	14.6	0.8	11.6	0.18	3.24	8.36	14.84
		17	14.6	↓	11.6	↓	↓	8.36	14.84
		28.5	14.6	↓	11.6	↓	↓	8.36	14.84
		41.5	16.1	↓	12.8	↓	↓	9.56	16.04
		53	17.1	↓	13.5	↓	↓	10.3	16.74
		63.5	18.0	↓	14.3	↓	↓	11.1	17.54
Leeward	②	ALL	18.0	-0.5	-9.0	0.18	3.24	-12.2	-5.8
	③	ALL	18.0	-0.28	-5.0			-8.2	-1.8
Parapet	(W)	65.5	18.2			1.5			27.3
	(L)	65.5	18.2			-1.0			-18.2
Roof ②	0-9	63.5	18.0	-1.04	-18.7	0.18	3.24	-21.9	-15.5
	9-18			-0.7	-12.6			-15.8	-9.4
Roof ③	0-31.75	63.5	18.0	-0.96	-17.3			-20.5	-14.1
	31.75-63.5			-.82	-14.8			-18	-11.6
	63.5-90			-.58	-10.4			-13.6	-7.2

WIND LOAD (cont.)

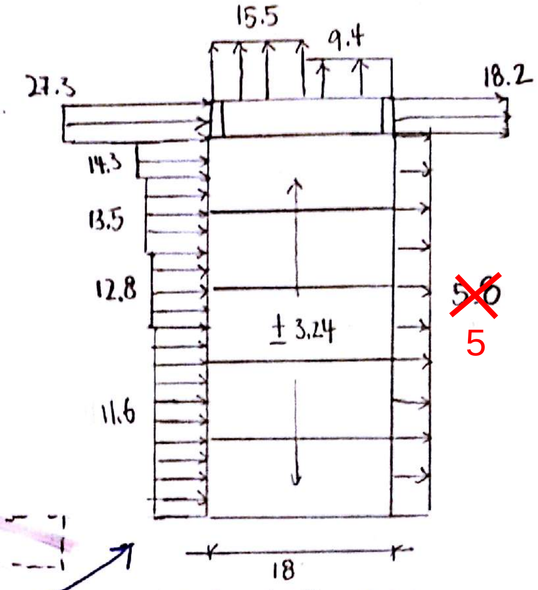
• North Part of ② (with Suction) Wind E-W



Trib. With = 40'



• South Part of ② (without suction) Wind E-W



Trib. Width = 21'

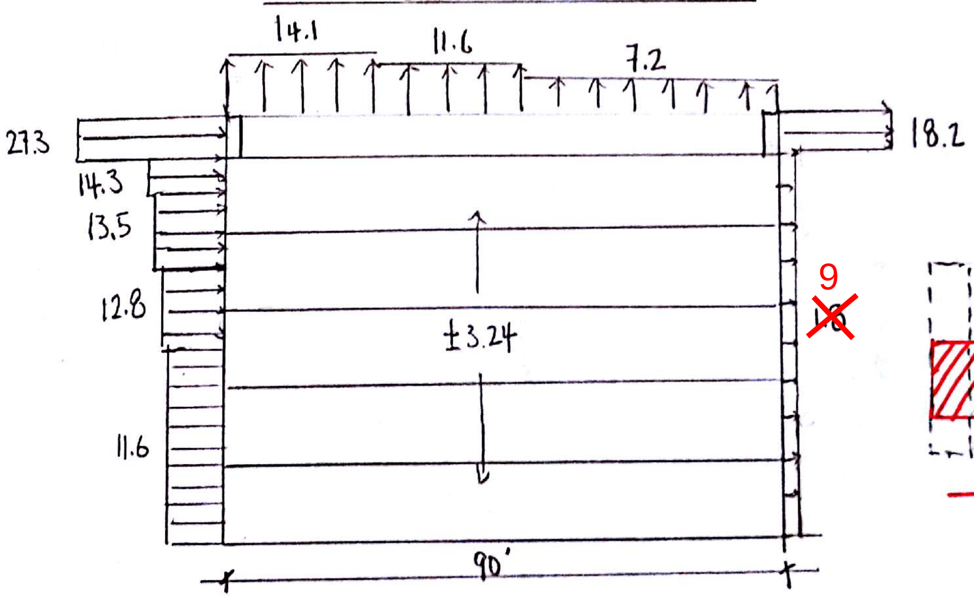
$V = [14.8(17+11.5+6.5) + 16(6.5) + (12.8+5.8)(5.75) + (13.5+5.8)(5.75+5.25) + (14.3+5.8)(5.25) + (27.3+18.2)(2)] \times 40$

$= 45.5^{K}$

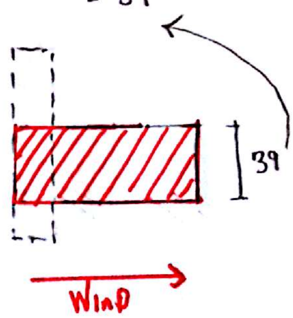
$V = [(11.6+5.8)(17+11.5+6.5) + (12.8+5.8)(6.5+5.75) + (13.5+5.8)(5.75+5.25) + (14.3+5.8)(5.25) + (27.3+18.2)(2)] \times 21$

$= 26.0^{K}$

③ Wind E-W



Trib. Width = 39'



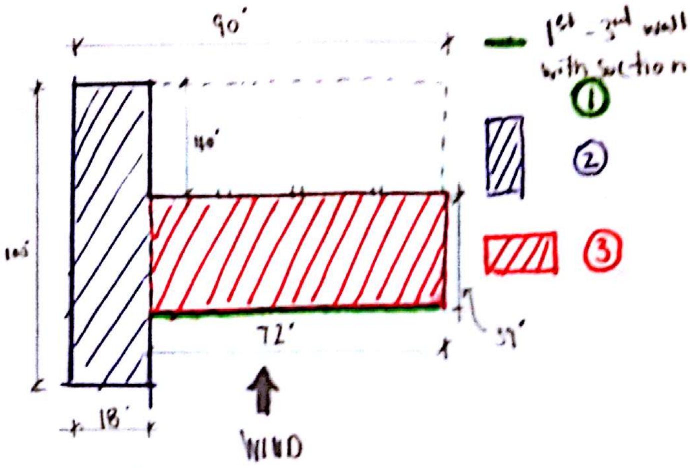
$V = [(11.6+1.8)(17+11.5+6.5) + (12.8+1.8)(6.5+5.75) + (13.5+1.8)(5.75+5.25) + (14.3+1.8)(5.25) + (27.3+18.2)(2)] \times 39 = 38.4^{K}$

* Base Shear (E-W) $= 45.5 + 26 + 38.4 = 110^{K}$

WIND LOAD (cont.)

* N-S Direction

Section	h (ft)	B_{NS} (ft)	L_{NS} (ft)
①	63.5	72	79
②	63.5	18	100
③	63.5	72	39



For ① & ③:

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{72+63.5}{336}\right)^{0.63}}} = .859$$

(Eq. 6-6)

For ②:

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{18+63.5}{336}\right)^{0.63}}} = .892$$

R, the resononant response factor, $R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.58 + 0.47 R_L)}$ (Eq. 6-10)

$N_1 = 4.02$, $R_n = .058$, $\bar{V}_{\bar{z}} = 67.0'$ [from previous calcs.]

$R_L = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$ (Eq. 6-13a)

$R_h = .245$

$R_B = .220$ $\eta = 4.6 n_1 B / \bar{V}_{\bar{z}} = 3.96$ for ① & ③

$R_B = .570$ $\eta = .991$ ②

$R_L = .203$ $\eta = 4.6 n_1 L / \bar{V}_{\bar{z}} = 4.35$ for ①

$R_L = .165$ $= 5.50$ ②

$R_L = .359$ $= 2.15$ ③

$$R(①) = \sqrt{\frac{1}{0.01}(.058)(.245)(.220)(.58 + 0.47 * .203)} = .460$$

$$R(②) = \sqrt{\frac{1}{0.01}(.058)(.245)(.57)(.58 + 0.47 * .165)} = .730$$

$$R(③) = \sqrt{\frac{1}{0.01}(.058)(.245)(.220)(.58 + 0.47 \times .359)} = .484$$

WIND Load (cont.)

- $g_r = 4.14$ (From previous calcs.)
- $G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.859)^2+(4.14)^2(.46)^2}}{1+1.7(.293)(3.4)}\right) = .9398 \approx .94$ ①

$G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.892)^2+(4.14)^2(.73)^2}}{1+1.7(3.4)(.293)}\right) = 1.076 \approx 1.08$ ② ⇐ Governs

$G_f = 0.925\left(\frac{1+1.7(.293)\sqrt{(3.4)^2(.859)^2+(4.14)^2(.484)^2}}{1+1.7(3.4)(.293)}\right) = .9478 \approx .95$ ③

- External Pressure Coefficient, C_p (Wind N-S) [Fig. 6-6]

For ①: $L/B = 1.1$, $h/L = .804$

- Walls: $C_{p,W,NS} = 0.8$
  $C_{p,L,NS} = -0.48$
  $C_{p,S,NS} = -0.7$

1	-0.5
1.1	-0.48
2	-0.3

For ②: $L/B = 5.56$, $h/L = .635$ $A = 1800 \Rightarrow R_f = 0.8$

- Walls: $C_{p,W,NS} = .8$
  $C_{p,L,NS} = -.2$
  $C_{p,S,NS} = -0.7$

- Roofs: $C_{p,roof,NS}(0-31.75) = -.94$
  $C_{p,roof,NS}(31.75-63.5) = -.85$
  $C_{p,roof,NS}(63.5-100) = -.55$

h/L	0-h/2	h/2-h	h-2h
0.5	-0.9	-0.9	-0.5
0.635	-.94	-.85	-.55
1.0	-1.04	-0.7	-0.7

For ③: $L/B = .542$, $h/L = 1.63$ $A = 39 \times 72 = 2808 \Rightarrow R_f = 0.8$

- Walls: $C_{p,W,NS} = 0.8$
  $C_{p,L,NS} = -0.5$
  $C_{p,S,NS} = -0.7$

- Roofs: $C_{p,rf}(0-31.75) = -1.04$
  $C_{p,rf}(>31.75) = -0.7$

WIND LOAD (cont.)

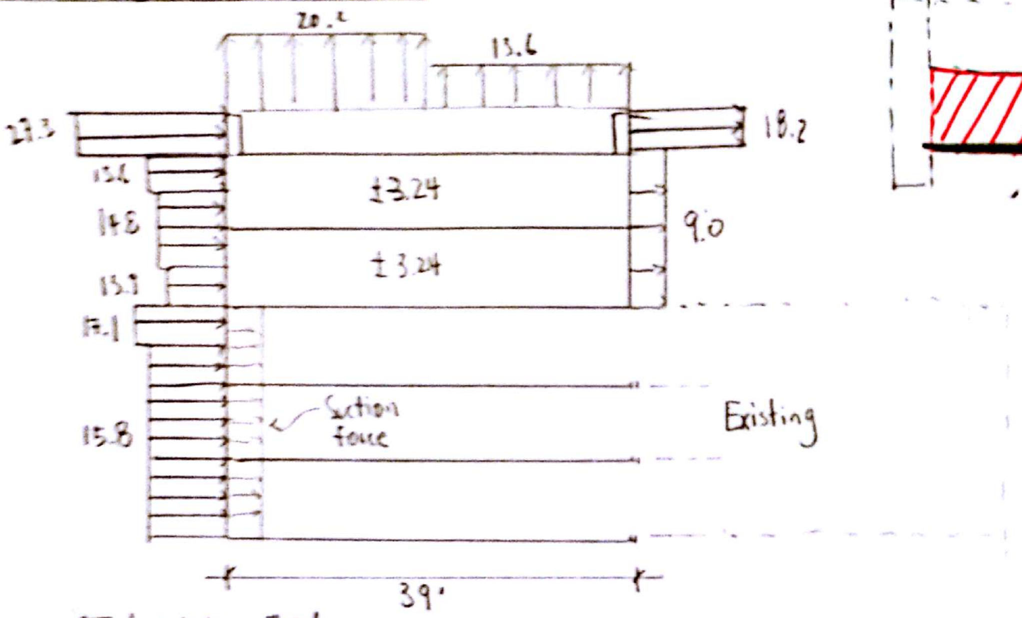
- Pressure on each surface: $P = q G C_p - q_i (G C_{pi})$ (Eq. 6-23)

  Use $G_f = 1.08$

Location	z (ft)	q_z (psf)	C_p	$q_z G C_p$ (psf)	$G C_{pi}$	$q_h G C_{pi}$ (psf)	$q_z G C_p - q_h(+G C_{pi})$ (psf)	$q_z G C_p - q_h(-G C_{pi})$ (psf)
Windward	0	14.6	0.8	12.6	0.18	3.24	9.4	15.8
	17	14.6		12.6			9.4	15.8
	28.5	14.6		12.6			9.4	15.8
	41.5	16.1		13.9			10.7	17.1
	53	17.1		14.8			11.6	18
	63.5	18.0		15.6			12.4	18.8
Leedward ②	ALL	18.0	-0.2	-3.6			-6.8	-0.4
③	ALL	18.0	-0.5	-9.0			-12.2	-5.8
Parapet (W)	65.5	18.2			1.5			27.3
(L)	65.5	18.2			-1.0			-18.2
Roof ② 0–31.5			-.94	-18.3			-21.5	-15.1
② 31.5–63.5	63.5	18.0	-.85	-16.5	0.18	3.24	-19.7	-13.3
② 63.5–100			-.55	-10.7			-13.9	-7.5
③ 0–31.5			-1.04	-20.2			-23.4	-17.0
③ 31.5–39			-0.7	-13.6			-16.8	-10.6

WIND LOAD (cont.)

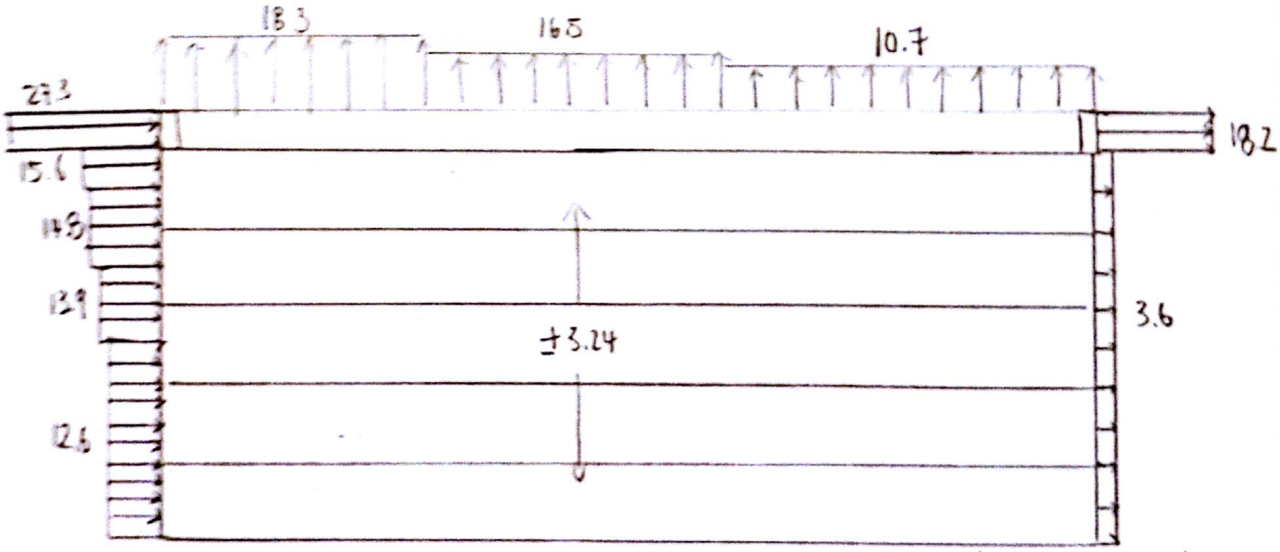
• ① & ③ Wind N-S



Trib. Width = 72'

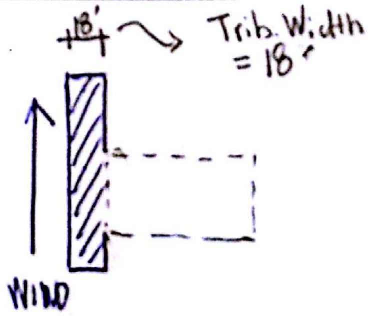
$$V = [15.8(17+11.5+6.5) + 17.1(6.5) + (13.9+9)(5.75) + (14.8+9)(5.75+5.25) + (15.6+9)(5.25) + (27.3+18.2)(2)] * 72 = 92.0^{K}$$

• ② Wind N-S



$$V = [(12.6+3.6)(17+11.5+6.5) + (13.9+3.6)(6.5+5.75) + (14.8+3.6)(5.75+5.25) + (15.6+3.6)(5.25) + (27.3+18.2)(2)] * 18$$

$$= 21.2^{K}$$



* Base Shear (N-S) = 92 + 21.2 = 113.2^{K}

[4] SEISMIC LOAD

- Structure not exempt (ASCE 7-02 §9.1.2)
- Site Class D (§9.4.1.2.1)
- $S_s = 0.365$ $S_{DS} = 0.367$
  $S_1 = 0.071$ $S_{D1} = 0.114$ (From USGS Design Maps Report)
- Seismic design category

  Table 9.4.2.1a, $0.33 \leq S_{DS} < 0.5 \rightarrow C$
  Table 9.4.2.1b, $0.067 \leq S_{D1} < 0.133 \rightarrow B$ ∴ SDC C.
- Table 9.5.2.5.1, ELF is permitted, use LEF Procedure
- Response Modification Factor, R (Table 9.5.2.2)
  - Ordinary Steel Moment Frames
    - No height limit
    - $R = 3.5$, $C_d = 3$, $W_o = 3$ (noted $R=3$ in design)
- Risk Category II ⇒ Seismic Use Group I (Table 9.1.3)

  ⇒ Seismic Impotant Factor, $I_e = 1.0$ (Table 9.1.4)
- Foundamental Period of the Building, T_a

  $T_a = C_t \cdot h_n^x$ (Eq. 9.5.5.3.2-1)

  $C_t = 0.028$, $x = 0.8$ (Table 9.5.5.3.2)

  $T_a = 0.028(63.5)^{0.8} = 0.775$
- Seismic Response Coefficient, C_s

  $C_s = \frac{S_{DS}}{R/I_e} = \frac{0.367}{3/1} = 0.122$ (Eq. 5.5.5.2.1-1)

  $\geq C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.114}{.775 * 3} = .049$ ⇐ Controls (Eq. 5.5.5.2.1-2)

  ∴ $C_s = 0.049$

  check: Max $\left| \begin{array}{l} 0.044 S_{DS} I = .016 \\ 0.01 \checkmark \end{array} \right.$ $0.049 > 0.01$

  ∴ $C_s = 0.049$

SEISMIC LOAD

- Effective Total Seismic Weight

$P_f = 20\,psf < 30\,psf \Rightarrow$ Snow load in not considered (§9.5.3)

~~$W_{roof} = (18\times100+72\times39)(67+20) + 2(100+90)(696)$~~

~~$= 665.4^K$~~

~~$W_{Floor} = 4(18\times100+72\times39)(100) + 2(100+90)(1056+1176+1176+1368)$~~

~~$= 3658.1^K$~~

~~$W_{Total} = W_{roof} + W_{floors}$~~
~~$= 3658.1 + 665.4$~~
~~$= 4323.5^k$~~

W (R) =(18x100+72x39)(67psf)+2(100+90)(696plf) = 573k
W (5th) =(18x100+50x9)(67psf)+2(100+90)(1056plf) = 552k
W (4th) =(18x100+72x39)(67psf)+2(100+90)(1176plf)= 757k
W (3th) =(18x100+72x39)(67psf)+2(100+90)(1176plf)= 757k
W (2th) =(18x100+23x72)(67psf)+2(100+90)(1368plf)= 758k

3397k

- Seismic Base Shear

$V = C_s W = 0.049 * 3397k = 166.5k$ (Eq. 9.5.5.2.1)

- Vertical Distribution of Seismic Forces (F_x)

$F_x = C_{vx} \cdot V = \left[\frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}\right] V$ (Eq. 9.5.4.4)

$V_x = \sum_{i=x}^{n} F_i$ [Eq. 9.5.5.5]

Ta	K
0.5	1
0.775	1.18 ✓
2.5	2

Level	h_x (ft)	W_x (Kip)	$W_x h_x^k$	C_{vx}	F_x (Kip)	V_x (Kip)
Roof	63.5	573	76812	0.297	49.4	49.4
5th	53	552	59784	0.231	38.4	87.8
4th	41.5	757	61432	0.237	39.5	127.3
3th	28.5	757	39429	0.152	25.4	152.7
2th	17	758	21458	0.083	13.8	166.5
Σ		3397	258915	1	166.5	

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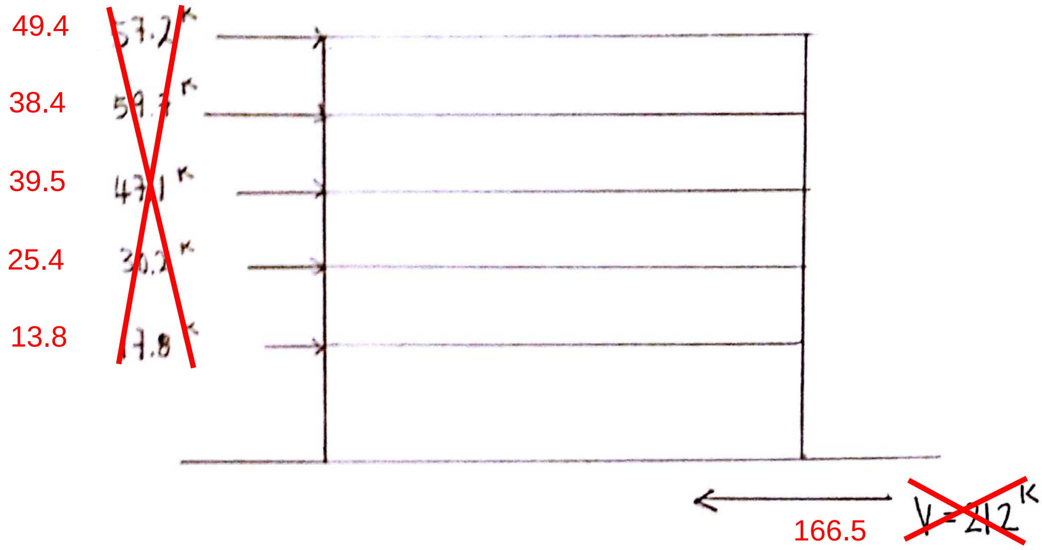
SEISMIC LOAD



Seismic loading vs. height

[5] Conclusion

After extensive analysis of the loading conditions imposed on 706 Madison Avenue, it is able to conclude that the wind forces will control the lateral design of the building. Snow loads could be not considered due to region's climate. Minimum design loads and load determination processes are specified using ASCE 7-02.