Letter of Transmittal

Date: September 26, 2014

To: Prof. Heather Sustersic
had132@engr.psu.edu

From: Young Jeon
ybj5001@psu.edu

Enclosed: AE 481W – Senior Thesis Structural Technical Report 2

Dear Prof. Sustersic,

The following report was prepared to be submitted for Technical Report 2 for AE 481W. This report contains a detailed structural analysis and calculation of dead and live gravity loads, wind loads and seismic loads of Hakuna Resort at Swiftwater, Pennsylvania. It is created using a combination of hand written calculations and excel spreadsheets. Wind and seismic load calculations concludes their sections with loading diagrams.

Thank you very much for taking your time to review this report. I look forward to discussing it with you in the future.

Sincerely,

Young Jeon

Technical Report 2

Hakuna Resort

Swift Water, Pennsylvania

Young Jeon
Structural Option
Advisor: Heather Sustersic
September 26th 2014

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Executive summary

Hakuna Resort is a jungle/safari theme hotel that includes a 217,703 square feet indoor water park as well as outdoor pool. The other side of the resort is convention centers which provides multiple meeting spaces. Divided into three distinctive spaces, the hotel is in between the indoor water park and convention space. These spaces are connected with expansion joints, therefore, can be looked at as three separate buildings.

The hotel building has total of eight stories above ground with total height of 101'-5" to the top of roof excluding the basement. With each floor having approximately 45,000 SF, the hotel portion of the resort has 395,938 SF by itself. The scope of this thesis project is limited to the hotel portion of the site; however, future assignment may incorporate an impactful design of hotel to improve cohesiveness of adjacent buildlings.

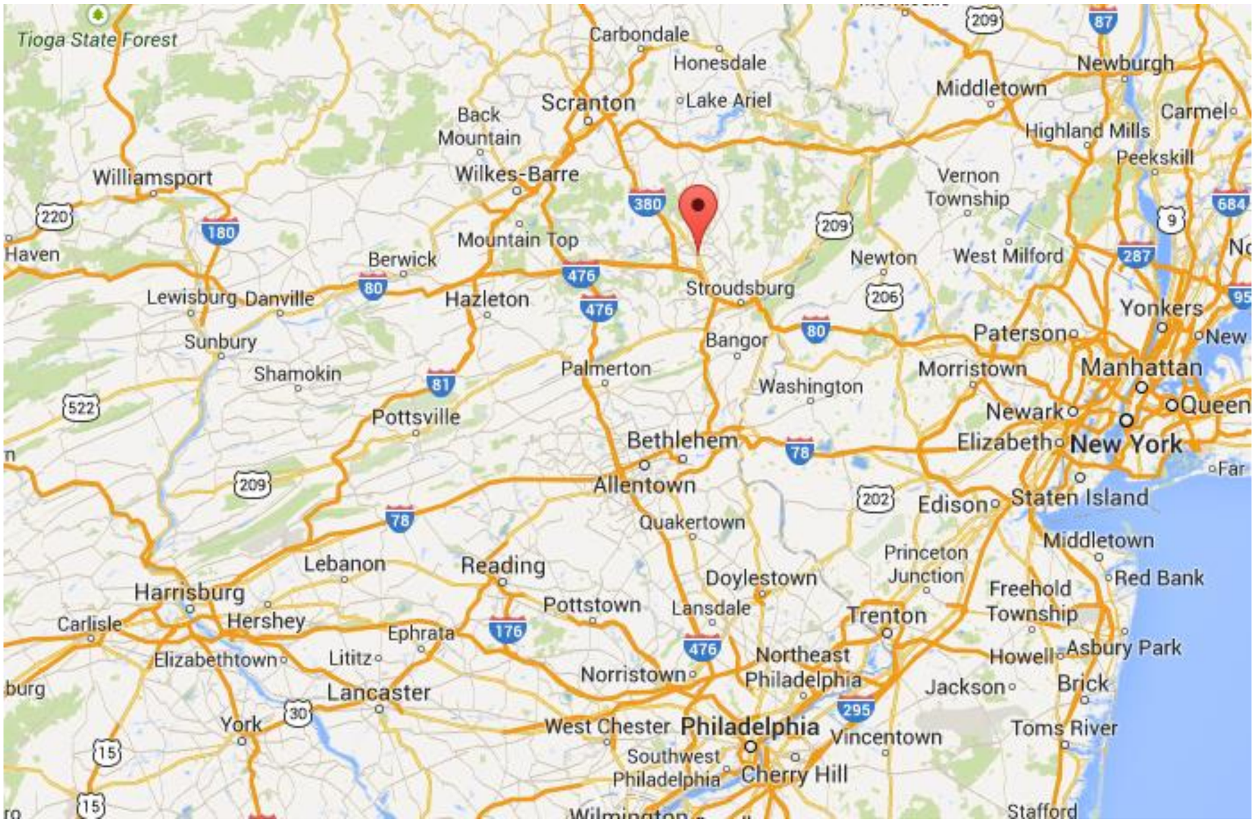
The foundation is consisted of cast in place concrete with footings and piers while north-west portion of building is partially unexcavated. The excavated portion of basement space is divided into usable rooms by concrete and masonry walls.

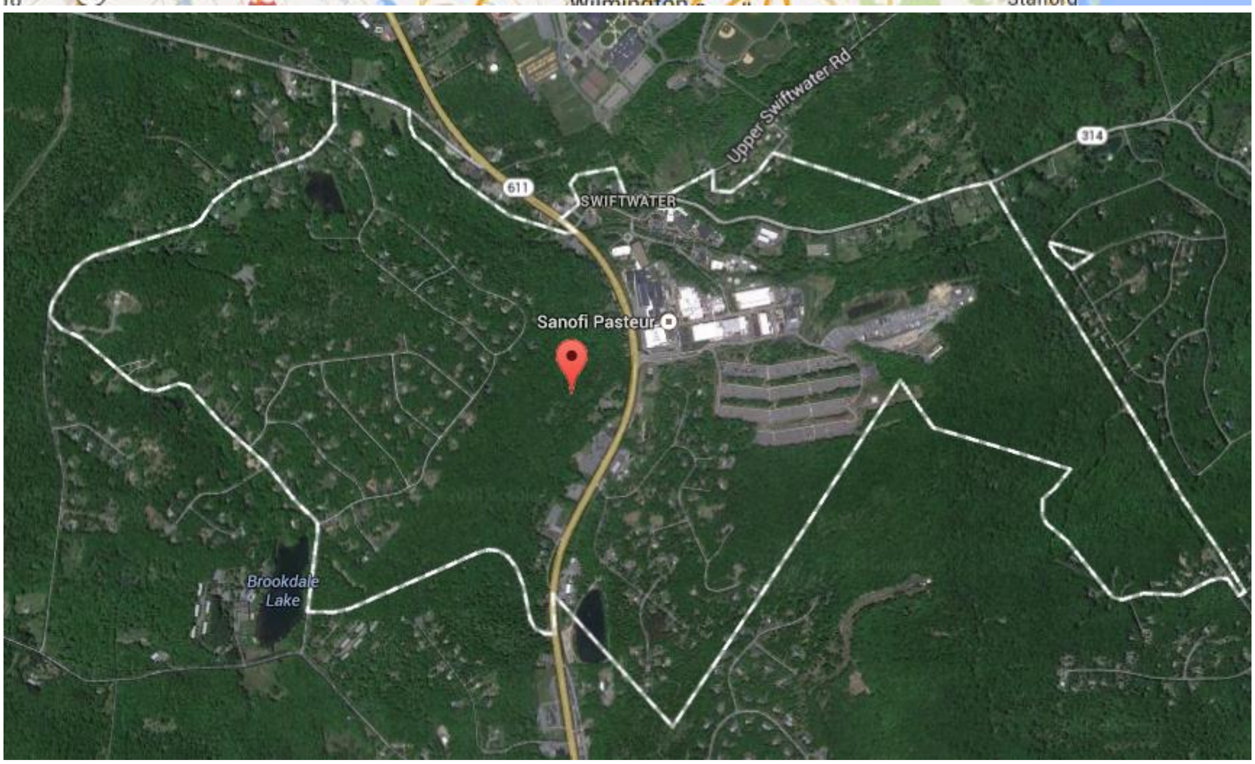
The typical elevated floor is 10" precast prestressed hollow core planks. At the excavated basement floor and first level floor above unexcavated foundation, a unique condition exists such that slab on grade concrete is used. The precast planks are supported by loadbearing masonry walls throughout the structure. However, in service areas like sauna, message and treatment on second floor, steel framing system is used to take advantage of opened frame system compared to solid shear wall that may block the view or pedestrian flow.

The nature of repetitive and typical hotel room floor layout allows the structural system to be simple and typical as well. The need of privacy also enabled the usage of masonry shear walls in between almost every room. Like mentioned earlier, these shear walls are supporting precast planks, therefore resisting gravity load.

In conclusion, while dominant structural system is masonry shear walls with precast planks, there are also structural wide flange steel framing in appropriate spaces, as well as reinforced concrete walls in lower levels. This usage of multiple structural systems will be analyzed throughout this report.

Building Site Information





Abstract

Hakuna Resort

Swiftwater, Pennsylvania

Project Team

Owner: LMN Development, LLC
Architect: Architectural Design Consultant
General Contractor: Kraemer Brothers, LLC
MEP/Structural: Harwood Engineering Consulants, LTD
Civil: Pennoni Associates, INC

General Building Data

Construction Dates: March 2014 - Summer of 2015
Building Cost: (Information Requested)
Delivery Method: Design Bid Build
Size: 395.938 SF

Architecture

The façade of hotel building has color tone of brown, red, and grey to give earth-like feeling. At the corners of building, architectural finish will be done to resemble ancient stone. Also little more distinctive color finishes will be used at the top of hotel façade to give tribal character to the building. The interior designs are also jungle theme. Most of the furniture in hotel have bark surface finishes.

Structural

Hakuna Resort is composed with three major components: indoor waterpark, hotel, and convention center. These components are connected by expansion joints, which allows each section to be looked at as separate independent buildings. As stated before, only hotel building will be described in this report due to its size. The main structural system used in this building is masonry shear walls and precast planks. There are also concrete piers, spread and strip footings, walls and masonry walls in the foundation and steel framing system in areas that require more flexible open spaces. The roof system is also precast hollow core planks.

Mechanical

(Not enough information - Material requested)

Lighting and Electrical

(Not enough information - Material requested)

Sustainability

(Not enough information - Material requested)

Young Jeon - Structural Option https://www.engr.psu.edu/ae/thesis/portfolios/2015/ybj5001/index.html

Documents Used to Create This Report

International Code Council

- IBC 2009 – International Building Code 2009

American Society of Civil Engineers

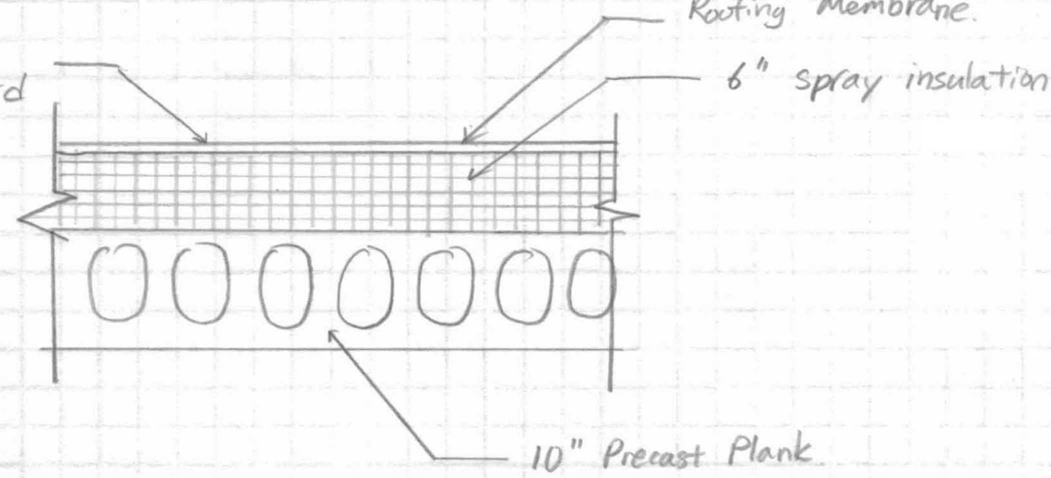
- ASCE 7-05 – Minimum Design Loads for buildings and Other Structures

Hakuna Resort Construction Documents

- Architectural and Structural Sets

Gravity Load Calculations

Typical Roof Bay Loading



Roof dead load:

Adhered Roofing Membrane = 2 psf

1/2" Deck Cover Board = 2 psf

6" Spray Insulation = 6 psf

10" precast plank

$(150\ pcf)(10/12) = 125\ psf$

Superimposed

Ceiling = 5 psf

Mechanical = 10 psf

Sprinkler = 5 psf

Total = 155 psf

Roof Live Load:

$L_r = 20$ psf (ASCE 7-05 Table 4-1)

* Roof live load for the design not shown in the drawing.

Snow Load:

According to Figure. 7-1 in ASCE 7-05, Case Studies are required. Therefore, used suggested load on the structural drawing.

$P_g = 40$ psf

$C_e = 1$

$I = 1$

$C_t = 1.1$

$P_f = P_g(0.7)(C_e)(I)(C_t) = 0.7(40)(1)(1)(1.1) = 30.8$ psf

* The structural design document used 35 psf for conservativity.

$\gamma = 0.13 P_g + 14 < 30$ pcf.

$\gamma = 0.13(40) + 14 = 19.2$ pcf < 30 pcf ✓

$h_b = \frac{P_f}{\gamma} = \frac{35}{19.2} = 1.82$ ft.

$h_r = 4$ ft.

$h_c = h_r - h_b = 4 - 1.8 = 2.18$ ft.

$$\frac{h_c}{h_b} = \frac{2.18}{1.82} = 1.19 > 0.2$$ ∴ Drift loads are required.

For Roof Projection Condition. (East-West)

$$h_d = 0.75\left((0.43)(L_{u_{lower}})^{1/3}(P_g+10)^{1/4} - 1.5\right)$$

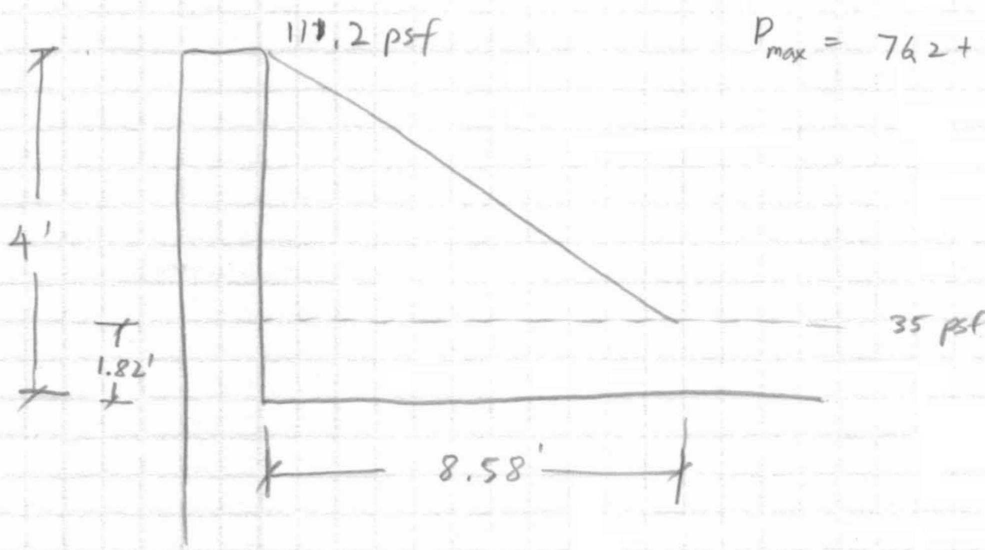
$$h_d = 0.75\left((0.43)(589')^{1/3}(40+10)^{1/4} - 1.5\right)$$

$$h_d = 5.69' > h_c \rightarrow h_d = h_c = 2.18.$$

$$w = \frac{4h_d^2}{h_c} = \frac{4(2.18)^2}{2.18} = 8.71 \text{ ft} \leq 8h_c = 17.4 \text{ ft} ✓$$

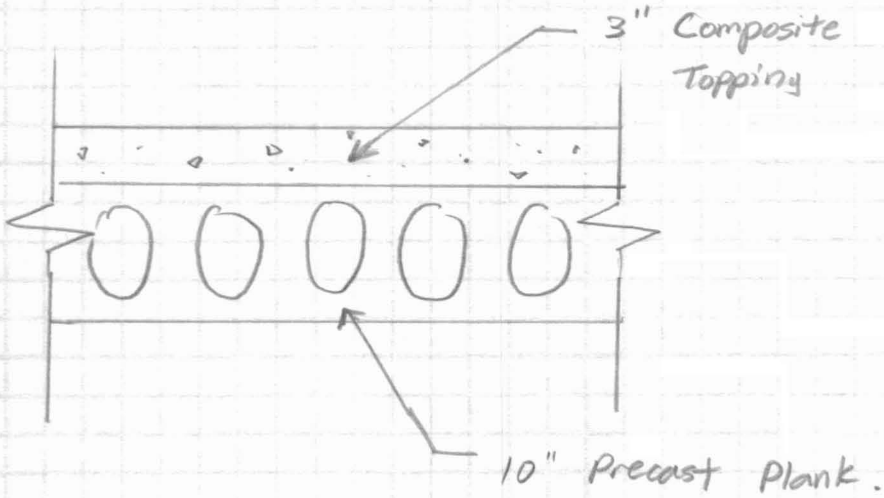
$$P_d = P_f\, h_d = 35(2.18) = 76.2 \text{ psf}$$

$$P_{max} = 76.2 + 35 = 111.2 \text{ psf}$$



* Same snow drif load in North-South direction due to large $L_{u_{lower}}$.

Typical Floor Bay Loading



Floor dead load:

10" Precast Plank = 125 psf

3" Composite Topping =

(150 pcf)(3"/12") = 37.5 psf

Superimposed

Ceiling = 5 psf

Mechanical = 10 psf

Sprinkler = 5 psf

Total: 182.5 psf

Floor bay live load:

ASCE 07-05 Table 4-1.

Lobbies = 100 psf
Corridors = 100 psf
Hotel room (private) = 40 psf
Hotel rooms (public) = 100 psf
Partition (minimum) = 15 psf.

"Typical floor above grade including partition" from structural drawing = 55 psf

ASCE 7-05

Hotel Room (Private) Partition

40 psf + 15 psf = 55 psf ✓

Non - Typical Dead Load

- Balcony roof. (Above balconies)
  - Plywood deck attachment - 5 psf
  - light gage steel truss - 15 psf

  20 psf.

- 12" precast plank w/ 3" Topping. (Luxury suite, longer floor span)

  12" plank. =

  (150 pcf) (12/12) = 150 psf

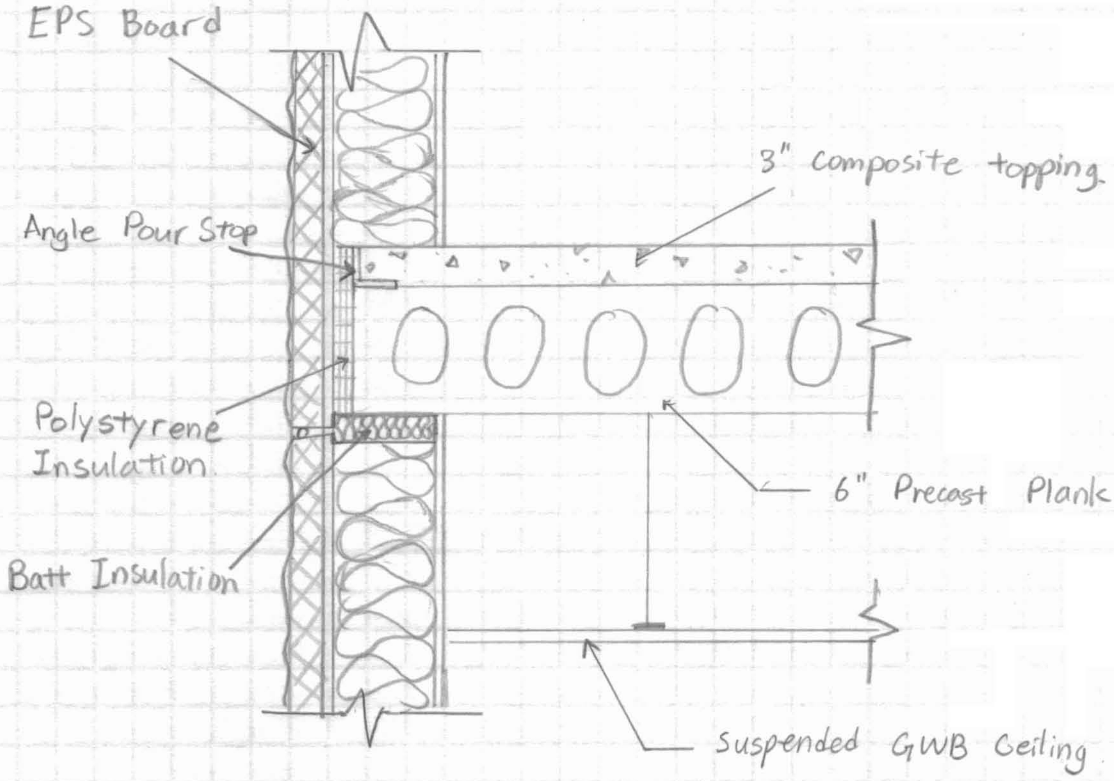
  3" Composite topping = 37.5 psf

  187.5 psf.

Non- Typical Live Load

- High density file room = 250 psf
- Mechanical Room = 175 psf
- Storage = 125 psf.

Typical Exterior Wall Load



Wall dead load:

EPS Board : 8 psf × 2"/12" = 1.33 plf

Insulations : 5 psf × 6"/12" = 2.5 plf

Wall Board : 6 psf × 1"/12" = 0.5 plf

4.33 plf

Load path:

Typical exterior walls are not load bearing. The plank shown above spans in and out of the page, which is than supported by interior masonry shear walls that are normal to the shown exterior wall. Therefore, the weight of exterior wall is carried through floor planks to shear walls.

Wind Load Calculations

Wind Load Calculation - ASCE 7-05 § 6.5 Analytical Method.

1. Basic wind speed (Fig. 6-1)

   $V = 90$ mph

   Wind directionality factor (Table 6-4)

   $K_d = 0.85$

2. Importance factor (Table 6-1)

   $I = 1.0$

3. Exposure Category (Section 6.5.6.3)

   Exposure C.

4. Topographic factor K_{zt} (Section 6.5.7)

   $K_{zt} = 1.0$

5. Gust effect factor G.

   - Building height, $h_n = 197.25'$

   Avg. building natural frequency $(n_1 = \frac{100}{H}) = \frac{100}{197.25} = 0.51$ Hz (Eq. C6-17)

   Lower bound natural frequency $(n_1 = \frac{75}{H}) = \frac{75}{197.25} = 0.38$ Hz (Eq. C6-18)

The ASCE 7-05 Recommendations can be rewritten in the same form as the approximate empirical equations provided in the seismic section.

Height from base to highest level, $h_n = 197.25'$
Approx. period parameter, $C_t = 0.013$
Approx. period parameter, $x = 1.00$

Table 12.8-2 modified for wind parameters of non-moment frame systems.

$T_a = C_t h_n^x$ Approximate Fundamental Period (Eq. 12.8-7)

$T_a = (0.013)(197.25)^1 = 2.564\ s.$

Lowest Natural Frequency of the Building (Fundamental Frequency)

$n_1 = \frac{1}{T_a} = \frac{1}{2.564} = \boxed{0.39\ Hz} < 1.0\ Hz$

$\therefore$ Flexible Structure. (section 6.5.8.2)

$$G_f = 0.925\left(\frac{1+1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_z}\right)$$

- North - South

$\bar{z} = 0.6h = 0.6(197.25) = 118.35'$

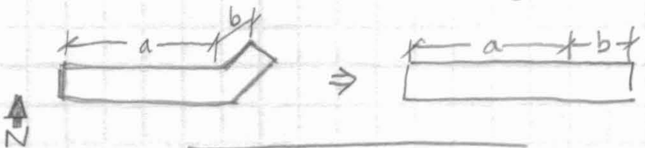
$\bar{\alpha} = 0.1538$
$\bar{b} = 0.65$
$c = 0.2$
$\ell = 500$
$\bar{\epsilon} = 0.2$

} (Table 6.2)

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{118.35}\right)^{1/6} = 0.162$ (Eq. 6-5)

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{118.35}{33}\right)^{0.2} = 645.51'$ (Eq. 6-7)

* Assume building is rectangle



	N-S	E-W
	L = 75'	L = 589'
	B = 589'	B = 75'

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{589+197.25}{645.51}\right)^{0.63}}} = 0.764$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = 0.65\left(\frac{118.35}{33}\right)^{0.1538}(90)\left(\frac{88}{60}\right) = 104.4\ mph$$

2

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

$$g_R = \sqrt{2\ln(3600 \times 0.38)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.38)}}$$

$$g_R = 4.0$$

$$N_1 = \frac{n_1 L_z}{\bar{V}_z} = \frac{0.38(645.51)}{104.4} = 2.37\ Hz$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(2.37)}{(1+10.3(2.37))^{5/3}} = 0.08$$

η for $R_h = 4.6 n_1 \frac{h}{\bar{V}_z} = 4.6(0.38)\frac{197.25}{104.4} = 3.336$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{3.336} - \frac{1}{2(3.336)^2}(1-e^{-2(3.336)}) = 0.255$$

η for $R_B = 4.6 n_1 \frac{B}{\bar{V}_z} = 4.6(0.38)\frac{589}{104.4} = 9.96$

$$R_B = \frac{1}{9.96} - \frac{1}{2(9.96)^2}(1-e^{-2(9.96)}) = 0.0954$$

η for $R_L = 15.4 n_1 \frac{L}{\bar{V}_z} = 15.4(0.38)\frac{75}{104.4} = 4.246$

$$R_L = \frac{1}{4.246} - \frac{1}{2(4.246)^2}(1-e^{-2(4.246)}) = 0.2078$$

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

Assume:
★ $\beta = 0.02$
Typ. 1-2% in U.S. (C6.5.8)

$$= \sqrt{\frac{1}{0.02}(0.08)(0.255)(0.0954)(0.53+0.47(0.2078))}$$

$$R = 0.248$$

3

$$G_f = 0.925\left(\frac{1+1.7 I_z\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I_z}\right)$$

$g_Q, g_v = 3.4$ (6.5.8.2)

$$G_f = 0.925\left(\frac{1+1.7(0.162)\sqrt{(3.4)^2(0.764)^2+4^2(0.248)^2}}{1+1.7(3.4)(0.162)}\right)$$

$$\boxed{G_f = 0.843 \text{ for North - South}}$$

- East - West

* Assume masonry shear wall is considered conc. shearwall system for Fundamental Frequency calculation.

NUMBER	HEIGHT	LENGTH	WIDTH	AREA	INDIVIDUAL SHEAR WALL CONTRIBUTION
n	h_i	D_i	w_i	A_i	$\Sigma (H / h_i)^2 (A_i / (1 + 0.83(h_i / Di)^2))$
1	197.25 ft	28.25 ft	1.00 ft	28.25 sf	0.68
2	197.25 ft	42.60 ft	1.00 ft	42.60 sf	2.27
3	197.25 ft	9.00 ft	1.00 ft	9.00 sf	0.02
4	197.25 ft	12.00 ft	1.00 ft	12.00 sf	0.05
5	197.25 ft	12.00 ft	1.00 ft	12.00 sf	0.05
6	197.25 ft	14.60 ft	1.00 ft	14.60 sf	0.10
7	197.25 ft	15.00 ft	1.00 ft	15.00 sf	0.10
8	197.25 ft	12.00 ft	1.00 ft	12.00 sf	0.05
9	197.25 ft	12.00 ft	1.00 ft	12.00 sf	0.05
10	197.25 ft	9.00 ft	1.00 ft	9.00 sf	0.02
					3.41

$$C_w = (100 / A_B) \sum_{i=1}^{n} (H / h_i)^2 \frac{A_i}{[1 + 0.83(h_i / D_i)^2]}$$

$C_w =$ 0.008

$n_1 = (385)(C_w)^{0.5} / H$ Building Natural Frequency *Equation C6-16*

$n_1 =$ 0.17 Hz < 1.0 Therefore this is a flexible structure

4

$\bar{Z} = 118.35'$

$I_{\bar{z}} = 0.162$

$L_{\bar{z}} = 645.51'$

$\bar{V}_{\bar{z}} = 104.4$ mph

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{75+197.25}{645.51}\right)^{0.63}}} = 0.8557$$

$$g_R = \sqrt{2\ln(3600n_1)} + \frac{0.577}{\sqrt{2\ln(3600n_1)}} = \sqrt{2\ln(3600\times0.17)} + \frac{0.577}{\sqrt{2\ln(3600\times0.17)}}$$

$g_R = 3.7$

$$N_1 = \frac{n_1 L_z}{\bar{V}_z} = \frac{0.17(645.51)}{104.4} = 1.06\ Hz$$

$$R_n = \frac{7.47 N_1}{(1+10.3 N_1)^{5/3}} = \frac{7.47(1.06)}{(1+10.3(1.06))^{5/3}} = 0.13$$

$$\eta \text{ for } R_h = 4.6 n_1 \frac{h}{\bar{V}_z} = 4.6(0.17)\frac{197.25}{104.4} = 1.489$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1-e^{-2\eta}) = \frac{1}{1.489} - \frac{1}{2(1.489)^2}(1-e^{-2(1.489)}) = 0.4575$$

$$\eta \text{ for } R_B = 4.6 n_1 \frac{B}{\bar{V}_z} = 4.6(0.17)\frac{75}{104.4} = 1.268$$

$$R_B = \frac{1}{1.268} - \frac{1}{2(1.268)^2}(1-e^{-2(1.268)}) = 0.5022$$

$$\eta \text{ for } R_L = 15.4 n_1 \frac{L}{\bar{V}_z} = 15.4(0.17)\frac{589}{104.4} = 14.886$$

$R_L = 0.0649$

5

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

$$R = \sqrt{\frac{1}{0.02} (0.13)(0.4575)(0.5022)(0.53 + 0.47(0.0649))}$$

$$R = 0.9057$$

$$G_f = 0.925 \left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R R^2}}{1 + 1.7 g_v I_z} \right)$$

$$= 0.925 \left(\frac{1 + 1.7(0.162)\sqrt{(3.4)^2(0.8557)^2 + (3.7)^2(0.9057)^2}}{1 + 1.7(3.4)(0.162)} \right)$$

$G_f = 1.066$ for East - West

Enclosed Building.

$GC_{pi} = \pm 0.18$

Ext. Pressure Coeff., C_p. (Fig. 6-6)

N-S.		E-W	
Windward	0.8	WW	0.8
Leeward	-0.5	LW	-0.2
Side Wall	-0.7	SW	-0.7

6

Wind Load Calculation

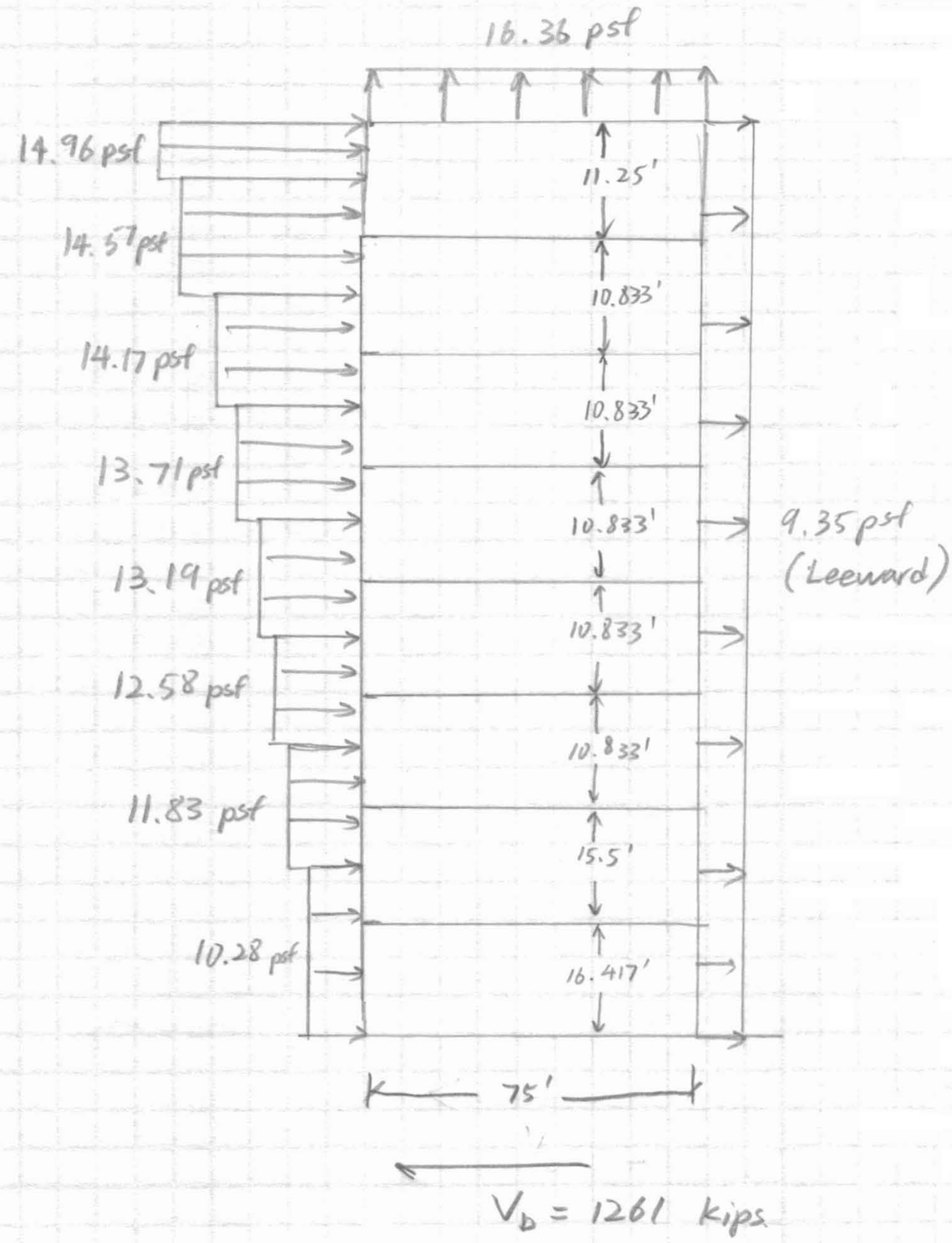
K_{zt}	1
K_d	0.85
V	90
I	1
$G_{f, NS}$	0.843
$G_{f, EW}$	1.066

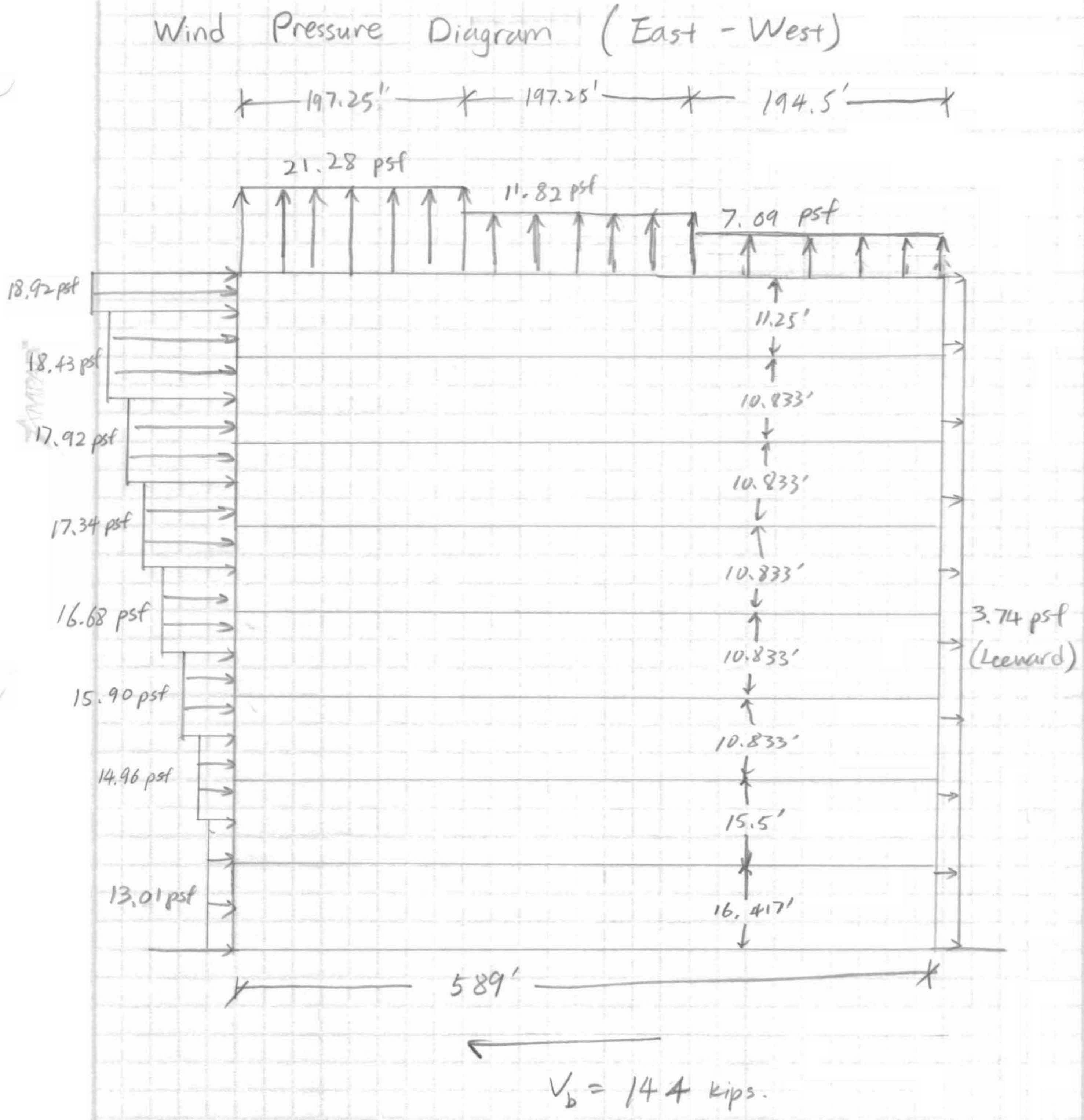
Floor Number	Height above ground	Z_g	α	k_z	q_z	q_h
2	16.417	900	9.5	0.87	15.25	22.18
3	31.917	900	9.5	1.00	17.54	22.18
4	42.75	900	9.5	1.06	18.65	22.18
5	53.583	900	9.5	1.11	19.56	22.18
6	64.417	900	9.5	1.15	20.33	22.18
7	75.25	900	9.5	1.19	21.01	22.18
8	86.083	900	9.5	1.23	21.61	22.18
Roof	97.333	900	9.5	1.26	22.18	22.18

Wall Pressure North - South Direction											
Floor Number	**Height above ground**	**Story Height (ft)**	**q_z**	**q_h**	**Windward (psf)**	**Leeward (psf)**	**Tributary Height (ft)**	**Tributary Area (ft^2)**	**Force (k)**	**Story Shear (k)**	**Overturning Moment (ft-k)**
Ground	0	0	15.25	22.18	10.28	-9.35	8.2085	4834.81	94.91	1260.69	0.00
2	16.417	16.417	15.25	22.18	10.28	-9.35	15.9585	9399.56	184.54	1165.78	3029.56
3	31.917	15.5	17.54	22.18	11.83	-9.35	13.1665	7755.07	164.23	981.25	5241.69
4	42.75	10.833	18.65	22.18	12.58	-9.35	10.833	6380.64	139.90	817.02	5980.68
5	53.583	10.833	19.56	22.18	13.19	-9.35	10.8335	6380.93	143.82	677.12	7706.39
6	64.417	10.834	20.33	22.18	13.71	-9.35	10.8335	6380.93	147.14	533.30	9478.00
7	75.25	10.833	21.01	22.18	14.17	-9.35	10.833	6380.64	150.05	386.16	11291.59
8	86.083	10.833	21.61	22.18	14.57	-9.35	11.0415	6503.44	155.57	236.11	13392.27
Roof	97.333	11.25	22.18	22.18	14.96	-9.35	5.625	3313.13	80.53	80.53	7838.45
Base Shear	1261 kips										
Total Overturning Moment	63959 ft-k										

Wall Pressure East - West Direction											
Floor Number	**Height above ground**	**Story Height (ft)**	**q_z**	**q_h**	**Windward (psf)**	**Leeward (psf)**	**Tributary Height (ft)**	**Tributary Area (ft^2)**	**Force (k)**	**Story Shear (k)**	**Overturning Moment (ft-k)**
Ground	0	0	15.25	22.18	13.01	-3.74	8.2085	615.64	10.31	144.01	0.00
2	16.417	16.417	15.25	22.18	13.01	-3.74	15.9585	1196.89	20.04	133.70	329.03
3	31.917	15.5	17.54	22.18	14.96	-3.74	13.1665	987.49	18.46	113.65	589.32
4	42.75	10.833	18.65	22.18	15.90	-3.74	10.833	812.48	15.96	95.19	682.33
5	53.583	10.833	19.56	22.18	16.68	-3.74	10.8335	812.51	16.59	79.23	889.06
6	64.417	10.834	20.33	22.18	17.34	-3.74	10.8335	812.51	17.13	62.64	1103.18
7	75.25	10.833	21.01	22.18	17.92	-3.74	10.833	812.48	17.60	45.51	1324.10
8	86.083	10.833	21.61	22.18	18.43	-3.74	11.0415	828.11	18.36	27.92	1580.35
Roof	97.333	11.25	22.18	22.18	18.92	-3.74	5.625	421.88	9.56	9.56	930.25
Base Shear	144.01										
Total Overturning Moment	7428 ft-k										

Wind Pressure Diagram (North - South)




Wind Pressure Diagram (East - West)
197.25'
197.25'
194.5'
21.28 psf
11.82 psf
7.09 psf
18.92 psf
18.43 psf
17.92 psf
17.34 psf
16.68 psf
15.90 psf
14.96 psf
13.01 psf
11.25'
10.833'
10.833'
10.833'
10.833'
10.833'
15.5'
16.417'
3.74 psf
(Leeward)
589'
V_b = 144 kips.

Seismic Load Calculations

Seismic Load Calculation

Mapped Acceleration Parameters

$S_S = 0.221$

$S_1 = 0.059$

Site Class = C

Site Coefficients and Adjusted MCE Spectral Response Acceleration Parameters

$F_a = 1.2$ (Table 11.4-1, 11.4-2)

$F_v = 1.7$

$S_{MS} = F_a S_S = 1.2(0.221) = 0.2652$

$S_{M1} = F_v S_1 = 1.7(0.059) = 0.1003$

Design Spectral Acceleration Parameters

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.2652) = 0.1768$

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.1003) = 0.0669$

Importance Factor, I = 1.0

Occupancy Category, = II.

Seismic Design Category :

$S_{DS} = 0.1768$, II ⇒ SDC : B

$S_{D1} = 0.0669 \approx 0.067$, II ⇒ SDC : B

Equivalent Lateral Force Procedure

A.9.: Ordinary reinforced masonry shear walls. (Table 12.14-1)

↳ $R = 2$.

$T_a = C_t h_n^x$ (Section 12.8.2)

$C_t = 0.02$
$x = 0.75$
$h_n = 197.75$ ft.

$T_a = 0.02(197.75)^{0.75} = 1.055$ s

$T_L = 6$ s (Fig. 22-15)

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.1768}{\left(\frac{2}{1.0}\right)} = 0.0884$ (Section 12.8.1.1)

$T_a = 1.055 < 6 = T_L$

↳ $C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.0669}{1.055\left(\frac{2}{1}\right)} = 0.0317$

$C_s = \min \begin{vmatrix} 0.0884 \\ 0.0317 \end{vmatrix} = 0.0317 > 0.01$ ✓

Building weight:

Total building floor area = 360,893 ft^2 (excluding basement)

Typical floor bay dead load = 182.5 psf

W = 69357^k. ★ Please see attached spreadsheet for detailed calculation.

Base shear, V (Eq. 12.8-1)

$V = C_s W = (0.0317)(69357) = 2198.6^k$.

0.5 s $k = 1$

1.055 s $k = 1.277$

2.5 s $k = 2$

Roof Wind Uplift

Roof Wind Uplift North - South Direction

Location on Roof	Cp	G	qh	Pressure (psf)
0 to 98.63 ft	-0.875	0.843	22.18	-16.36

h =	197.25	
L =	75	
h/L =	2.63	>1.0

Roof Wind Uplift East - West Direction

Location on Roof	Cp	G	qh	Pressure (psf)
0 to 98.63 ft	-0.9	1.066	22.18	-21.28
98.63 to 197.25 ft	-0.9	1.066	22.18	-21.28
197.25 to 394.5 ft	-0.5	1.066	22.18	-11.82
394.5 to 589 ft	-0.3	1.066	22.18	-7.09

h =	197.25	
L =	589	
h/L =	0.335	<0.5

Seismic Load Calculation

Floor Number	Height above ground	Story Height (ft)	Dead Load	Partition Load	Total Weight (psf)	Floor Area (ft^2)	Weight (k)
2	16.417	16.417	182.5	15	197.5	46001	9085
3	31.917	15.5	182.5	15	197.5	45694	9025
4	42.75	10.833	182.5	15	197.5	44807	8849
5	53.583	10.833	182.5	15	197.5	44807	8849
6	64.417	10.834	182.5	15	197.5	44807	8849
7	75.25	10.833	182.5	15	197.5	44807	8849
8	86.083	10.833	182.5	15	197.5	44807	8849
Roof	97.333	11.25	155	0	155	45163	7000
						Total Weight =	**69357**

k= 1.277

V_b = 2198.6

Story Forces (North - South)

Floor Number	Height above ground	Story Height (ft)	W (k)	Wh^k	C_{vx}	Story Forces (k)
2	16.417	16.417	9085	323791	0.026	56.39
3	31.917	15.5	9025	751731	0.060	130.92
4	42.75	10.833	8849	1070578	0.085	186.44
5	53.583	10.833	8849	1428501	0.113	248.78
6	64.417	10.834	8849	1807202	0.143	314.73
7	75.25	10.833	8849	2204001	0.175	383.83
8	86.083	10.833	8849	2616993	0.207	455.76
Roof	97.333	11.25	7000	2421713	0.192	421.75
					Base Shear:	**2198.60**

Seismic Loading Diagram