Letter of Transmittal

Date: November 18, 2014

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

From: Young Jeon
ybj5001@psu.edu

Enclosed: AE 481W – Senior Thesis Structural Technical Report 4

Dear Prof. Sustersic,

The following report was prepared to be submitted for Technical Report 4 for AE 481W. This report contains revised wind and seismic calculation based on Technical Report 2 review commentaries, spot check calculations of a typical lateral resisting element, computer modeling data obtained from ETABS 2013 software and comparison with hand calculation for validity of computer model.

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 4

Hakuna Resort

Swift Water, Pennsylvania

Young Jeon
Structural Option
Advisor: Heather Sustersic
November 18th 2014

Table of Contents

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.

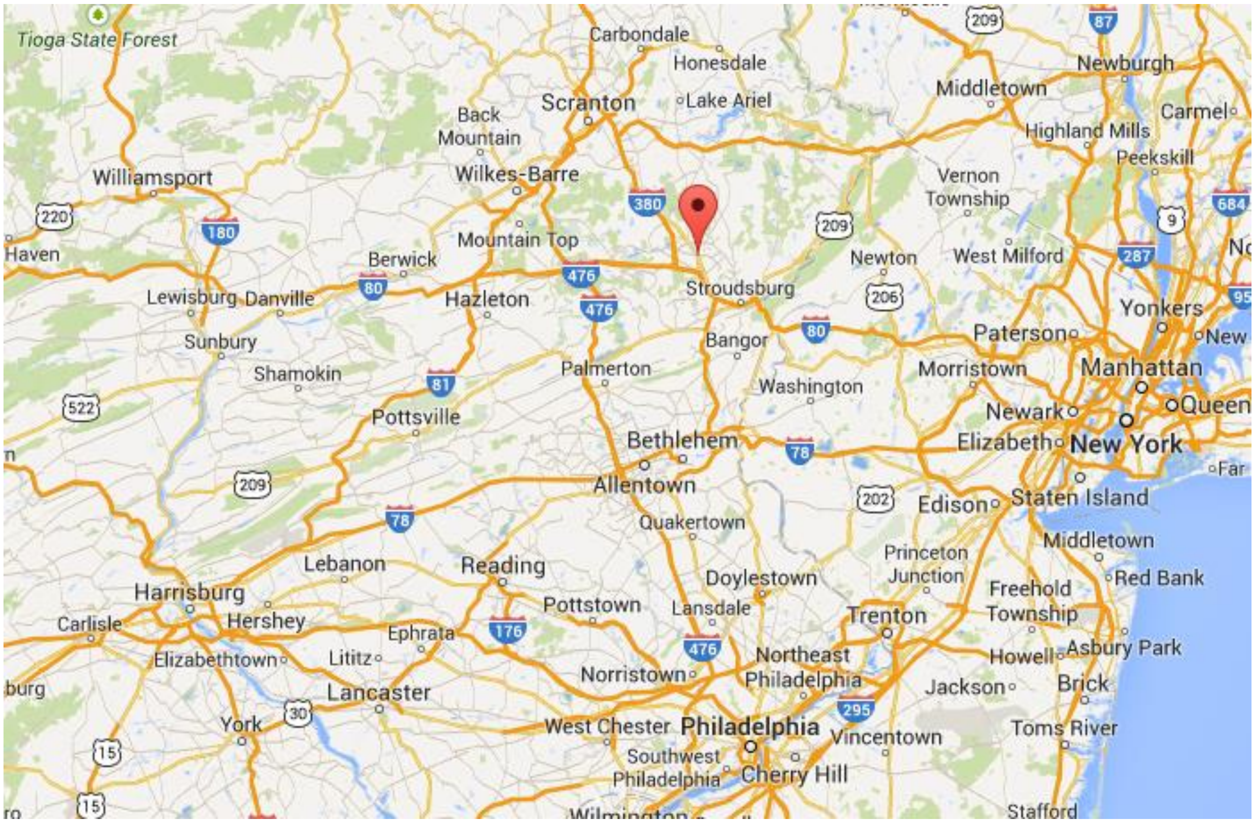
The foundation is consisted of cast in place concrete with footings and piers while north-west portion of building is partially unexcavated. The excavated portion of basement space is divided into usable rooms by concrete and masonry walls.

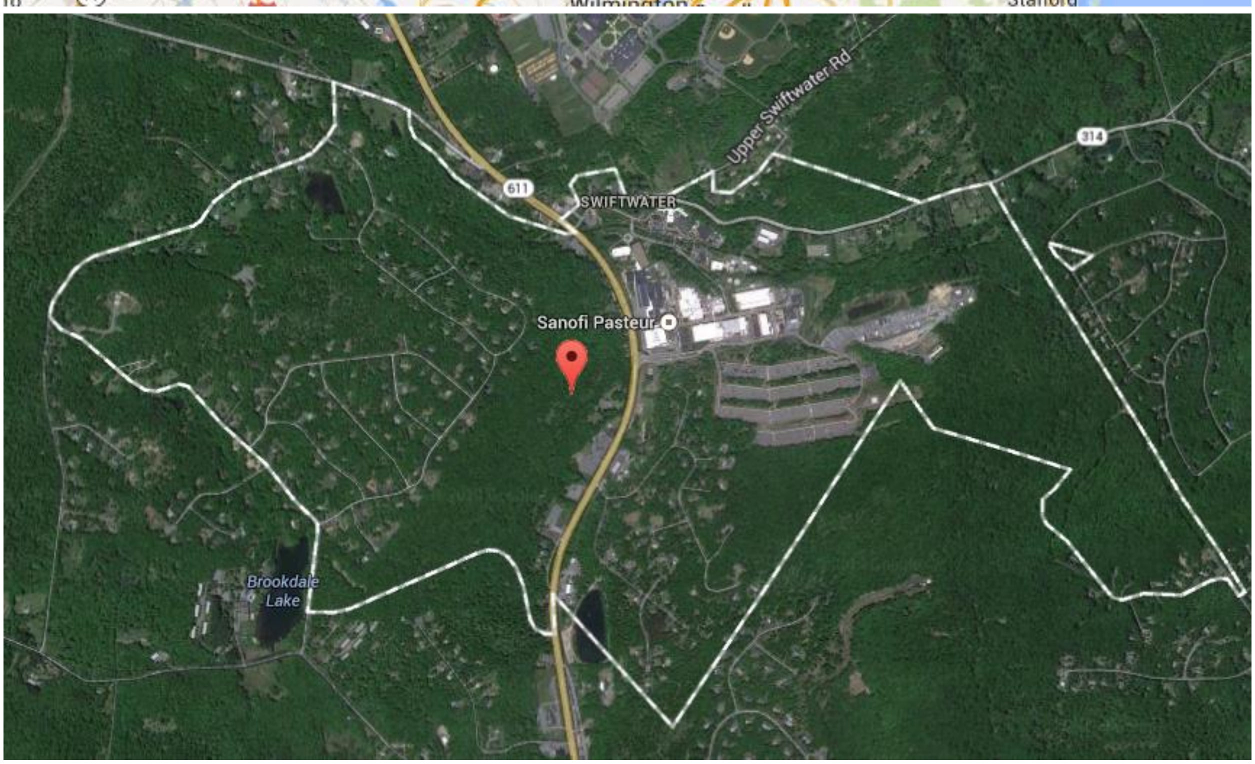
The typical elevated floor is 10" precast prestressed hollow core planks. At the excavated basement floor and first level floor above unexcavated foundation, a unique condition exists such that slab on grade concrete is used. The precast planks are supported by loadbearing masonry walls throughout the 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

Images Courtesy of LMN Development, LLC

Architecture

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

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.

Lighting and Electrical

The public area lighting with occupancy sensors is a florescent lighting system Compact florescent down lights and line voltage halogen continuous run wall graze luminaries to provide uniform grazing on the vertical surface in the corridor. The primary feed is 208Y/120V system for general use and 480Y/277 for lighting.

Mechanical

The AHU operate with a Variable Air Volume System that has hook ups in the primary spaces, allowing the end user to monitor each space. A building automation system is provided to monitor various mechanical points throughout the building.

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

Documents Used to Create This Report

Masonry Standards Joint Committee

- Building Code Requirements and Specification for Masonry Structures
    - Building Code Requirements for Masonry Structures
        - TMS 402-11 / ACI 530-11 / ASCE 5-11
    - Specification for Masonry Structures
        - TMS 602-11 / ACI 530.1-11 / ASCE 6-11

Concrete Masonry Association of California and Nevada

- 2009 Design of Reinforced Masonry Structures

American Concrete Institute

- ACI 318-08 – Building Code Requirements for Structural Concrete and Commentary

American Institute of Steel Construction

- Steel Construction Manual 14^{th} Edition

Hakuna Resort Construction Documents

- Architectural and Structural Sets

Revised Wind Load Calculation

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

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 (Section 6.5.8)

   - Building height, $h_n = 101'-5'' = 101.4167'$
   - Natural Frequency. (C 6.5.8)

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

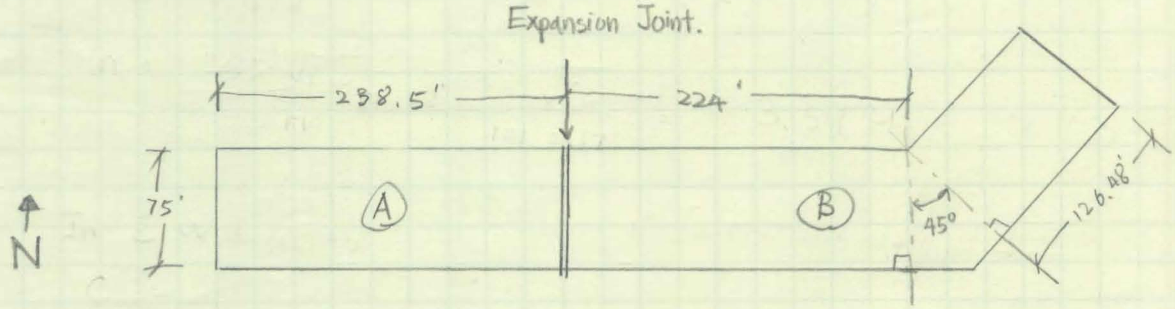


Fig. 1 Hotel Building of Hakuna Resort.

Hotel Ⓐ N-S (see attached spreadsheet)

$C_w = 0.479$

$$n_1 = \frac{385\sqrt{C_w}}{H} = \frac{385\sqrt{0.479}}{101.4167} = 2.63\text{Hz} > 1.0$$

∴ Structure is rigid.

$\bar{z} = 0.6h = 0.6(101.42) = 60.85'$

$\bar{\alpha} = 0.1538$

$\bar{b} = 0.65$

$c = 0.2$

$l = 500'$

$\bar{\epsilon} = 0.2$

$$I_z = c\left(\frac{33}{\bar{z}}\right)^{\bar{\epsilon}} = 0.2\left(\frac{33}{60.85}\right)^{0.2} = 0.177$$

$$L_z = l\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{60.85}{33}\right)^{0.2} = 565.09'$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_z}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{238.5+101.42}{565.09}\right)}} = 0.8284$$

$$\bar{V}_z = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = (0.65)\left(\frac{60.85}{33}\right)^{0.1538}(90)\left(\frac{88}{60}\right) = 94.3 \text{ mph}$$

$g_Q, g_v = 3.4$ (§ 6.5.8.1)

$$G = 0.925\left(\frac{(1+1.7g_Q I_{\bar{z}} Q)}{1+1.7g_v I_{\bar{z}}}\right) = \boxed{0.845}$$

Hotel Ⓐ E-W

$C_w = 0.027$

$n_1 = 0.63\,Hz < 1.0$ ∴ Structure is flexible

$\bar{Z} = 60.85'$
$\bar{\alpha} = 0.1538$
$\bar{b} = 0.65$
$c = 0.2$
$\ell = 500'$
$\bar{\epsilon} = 0.2$
$I_{\bar{z}} = 0.177$
$L_{\bar{z}} = 565.09'$
$\bar{V}_{\bar{z}} = 94.3$ mph

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{75 + 101.42}{565.09}\right)}} = 0.8762$$

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

$$= \sqrt{2\ln(3600 \cdot 0.63)} + \frac{0.577}{\sqrt{2\ln(3600 \cdot 0.63)}} = 4.1$$

g_Q & $g_v = 3.4$

Assume $\beta = 0.02$ (C 6.5.8)

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.63(565.09)}{94.3} = 3.77\,Hz$$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(3.77)}{(1 + 10.3(3.77))} = 0.06$$

$$\eta \text{ for } R_h = 4.6 n_1 \frac{h}{\bar{V}_{\bar{z}}} = 4.6(3.77)\frac{101.42}{94.3} = 3.109$$

$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta}) = \frac{1}{3.109} - \frac{1}{2(3.109)^2}(1 - e^{-2(3.109)}) = 0.27$$

$$\eta \text{ for } R_B = 4.6 n_1 \frac{B}{\bar{V}_{\bar{z}}} = 4.6(0.63)\frac{75}{94.3} = 9.611$$

$$R_B = \frac{1}{9.611} - \frac{1}{2(9.611)}(1 - e^{-2(9.611)}) = 0.0986$$

$$\eta \text{ for } R_L = 15.4 n_1 \frac{L}{\bar{V}_{\bar{z}}} = 15.4(0.63)\frac{238.5}{94.3} = 24.48$$

$$R_L = \frac{1}{24.48} - \frac{1}{2(24.48)}(1 - e^{-2(24.48)}) = 0.04$$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.209$$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right) = \boxed{0.884}$$

Shear Wall Contribution to the Natural Frequency

North-South Direction

	Height (ft)	Length (ft)	Width (ft)	Area (ft^2)	Individual Shear Wall Contribution
1	101.42	70	0.97	67.81	24.73
2	101.42	15.33	0.97	14.85	0.40
3	97.33	60.6	0.97	58.71	20.29
4	97.33	60.6	0.97	58.71	20.29
5	97.33	60.6	0.97	58.71	20.29
6	97.33	60.6	0.97	58.71	40.58
7	97.33	60.6	0.97	58.71	40.58
8	97.33	60.6	0.97	58.71	20.29
9	97.33	31.33	0.97	30.35	3.66
10	97.33	60.6	0.97	58.71	20.29
				Sum =	211.41
				Cw =	0.4786

East-West Direction

	Height (ft)	Length (ft)	Width (ft)	Area (ft^2)	Individual Shear Wall Contribution
11	97.33	28.5	0.97	27.645	2.81
12	97.33	9	0.97	8.73	0.10
13	97.33	12	0.97	11.64	0.23
14	97.33	12	0.97	11.64	0.23
15	97.33	42.38	0.97	41.1086	8.30
16	97.33	15	0.97	14.55	0.44
				Sum =	12.10
				Cw =	0.0274

Enclosed building ⇒ $GC_{pi} = \pm 0.18$

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

	Ⓐ N-S	Ⓐ E-W
Windward	0.8	0.8
Leeward	-0.5	-0.24
Sidewall	-0.7	-0.7

Wind Load Calculation

L	75 ft
B	239 ft
K_{zt}	1
K_d	0.85
V	90
I	1
$G_{, NS}$	0.845
$G_{f, EW}$	0.884

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	101.42	900	9.5	1.27	22.37	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)	Moment at Each Story (ft-k)
Ground	0	0	15.25	22.18	10.31	-9.37	8.2	1958	38.53	535.54	0.00
2	16.417	16.417	15.25	22.18	10.31	-9.37	16.0	3806	74.90	497.02	1229.67
3	31.917	15.5	17.54	22.18	11.86	-9.37	13.2	3140	66.66	422.11	2127.61
4	42.75	10.833	18.65	22.18	12.61	-9.37	10.8	2584	56.79	355.45	2427.79
5	53.583	10.833	19.56	22.18	13.22	-9.37	10.8	2584	58.38	298.66	3128.15
6	64.417	10.834	20.33	22.18	13.75	-9.37	10.8	2584	59.73	240.28	3847.64
7	75.25	10.833	21.01	22.18	14.20	-9.37	10.8	2584	60.91	180.55	4583.39
8	86.083	10.833	21.61	22.18	14.61	-9.37	13.1	3121	74.84	119.64	6442.78
Roof	101.42	15.337	22.37	22.18	15.12	-9.37	7.7	1829	44.80	44.80	4543.71
Base Shear	535.54										
Total Overturning Moment	28331 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)	Moment at Each Story (ft-k)
Ground	0	0	15.25	22.18	10.78	-3.92	8.2085	615.64	9.05	131.44	0.00
2	16.417	16.417	15.25	22.18	10.78	-3.92	15.9585	1196.89	17.60	122.39	288.96
3	31.917	15.5	17.54	22.18	12.40	-3.92	13.1665	987.49	16.12	104.79	514.55
4	42.75	10.833	18.65	22.18	13.19	-3.92	10.833	812.48	13.90	88.66	594.39
5	53.583	10.833	19.56	22.18	13.83	-3.92	10.8335	812.51	14.43	74.76	773.01
6	64.417	10.834	20.33	22.18	14.38	-3.92	10.8335	812.51	14.87	60.33	957.92
7	75.25	10.833	21.01	22.18	14.86	-3.92	10.833	812.48	15.26	45.46	1148.21
8	86.083	10.833	21.61	22.18	15.29	-3.92	13.085	981.38	18.85	30.21	1622.62
Roof	101.42	15.337	22.37	22.18	15.82	-3.92	7.6685	575.14	11.36	11.36	1151.68
Base Shear	131.44										
Total Overturning Moment	7051 ft-k										

Roof Wind Uplift

Roof Wind Uplift North - South Direction

Location on Roof	Cp	G	qh	Pressure (psf)
0 to 98.63 ft	-0.875	0.845	22.18	-16.40

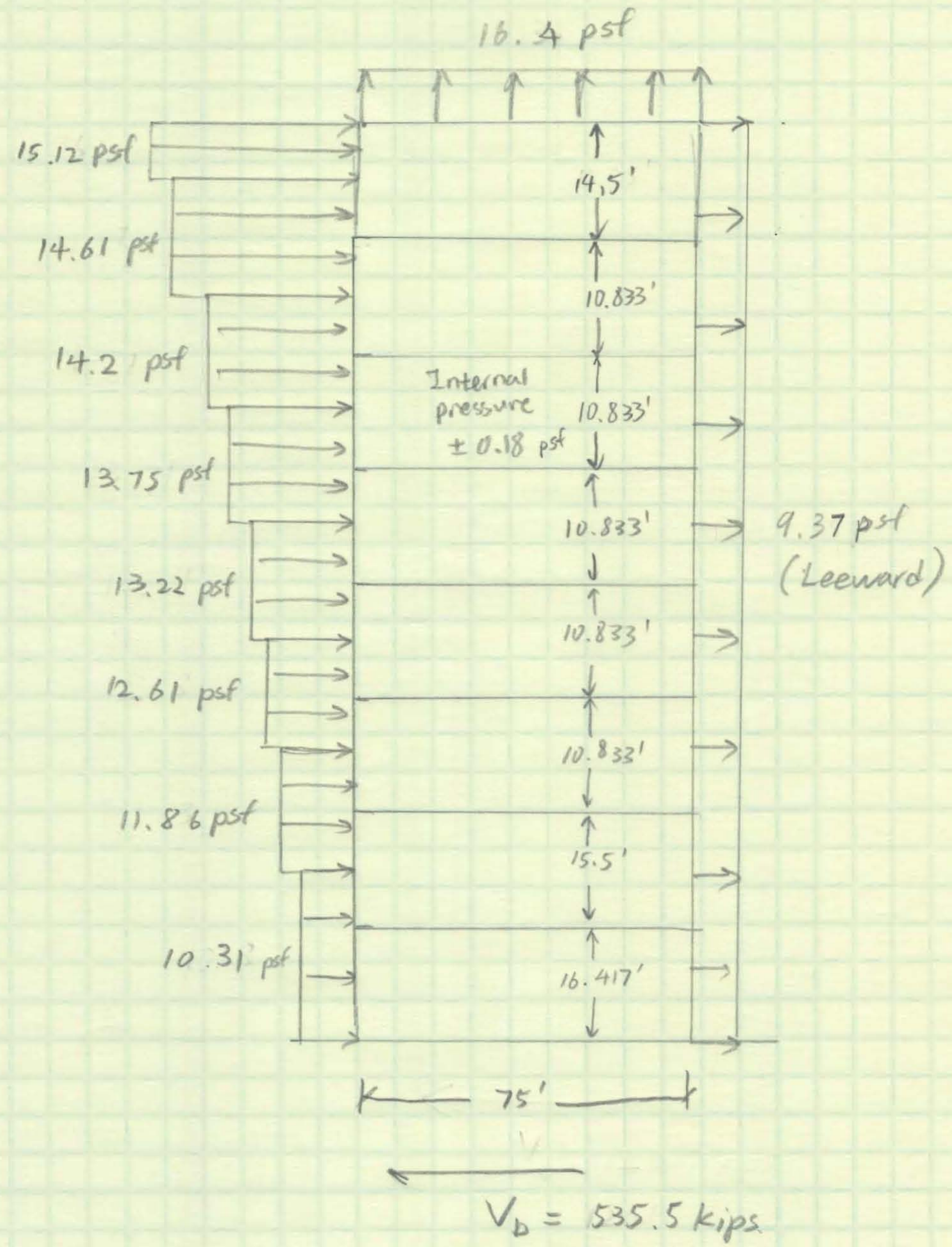
h =	101.42	
L =	75	
h/L =	1.35	>1.0

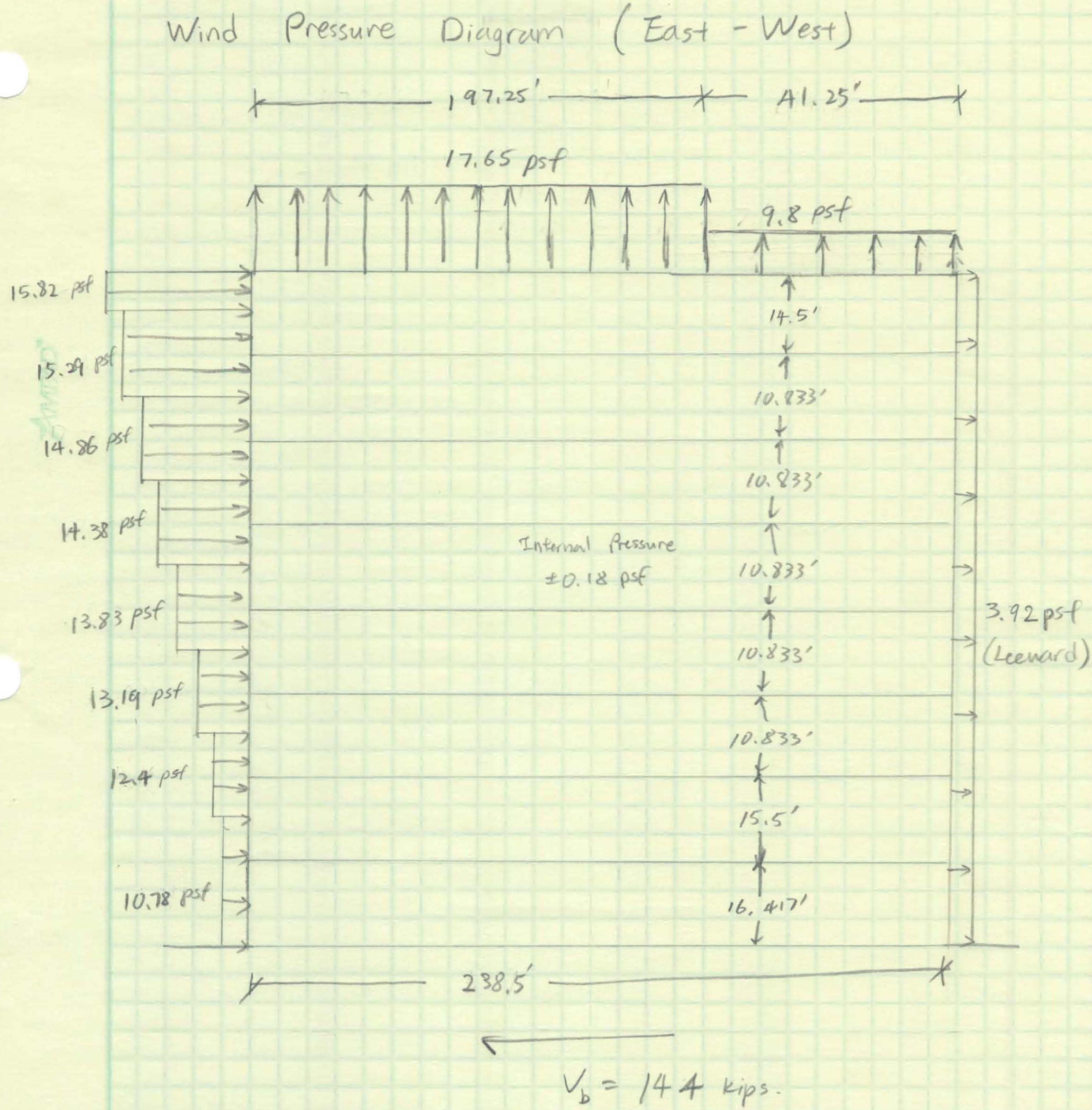
Roof Wind Uplift East - West Direction

Location on Roof	Cp	G	qh	Pressure (psf)
0 to 98.63 ft	-0.9	0.884	22.18	-17.65
98.63 to 197.25 ft	-0.9	0.884	22.18	-17.65
197.25 to 238.5 ft	-0.5	0.884	22.18	-9.80

h =	101.42	
L =	589	
h/L =	0.17	<0.5

Wind Pressure Diagram (North - South)




Wind Pressure Diagram (East - West)
197.25'
41.25'
17.65 psf
9.8 psf
15.82 psf
15.29 psf
14.86 psf
14.38 psf
13.83 psf
13.19 psf
12.4 psf
10.78 psf
14.5'
10.833'
10.833'
10.833'
10.833'
10.833'
15.5'
16.417'
Internal Pressure
±0.18 psf
3.92 psf
(Leeward)
238.5'
V_b = 144 kips.

Revised Seismic Load Calculation

Seismic Load Calculation

- Revised.

Mapped Acceleration Parameters

$S_S = 0.221$

$S_1 = 0.059$

Site Class = C - (Provided in structural drawing S13.1)

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-8 Intermediate reinforced masonry shear wall

$R = 3.5$ (Table 12.2-1)

$T_a = \frac{0.0019}{\sqrt{C_w}} h_n$ (Eq. 12.8-9)

For N-S : $C_w = 0.479$. $h_n = 101.42$.

Hotel (A).

$T_a = \frac{0.0019}{\sqrt{0.479}} (101.42)$

$T_a = 0.278$ s

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

$T_a = 0.278 < 6.0$ s $= T_L$ ∴ compare w/ Eq. 12.8-3

$C_s = \frac{S_{DS}}{R/I} = \frac{0.1768}{\left(\frac{3.5}{1.0}\right)} = 0.0505$ ← use this.

$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.0669}{0.278\,(3.5/1.0)} = 0.0688$ ← max.

$0.0505 > 0.01$ ✓

For E-W: $C_w = 0.027$

$T_a = \frac{0.0019}{\sqrt{0.027}} (101.42) = 1.17$ s. < 6s $= T_L$.

$C_s = \frac{0.0669}{1.17(3.5/1.0)} = 0.0163$ ← max. use this.

$0.0163 > 0.01$ ✓.

Hotel (A)

Building weight:

Total building floor area = 143100 ft^2 (excluding basement)

Typical floor bay dead load = 115.5 psf

W = 19113 k. ★ Please see attached spreadsheet for detailed calculation.

Base shear, V (Eq. 12.8-1)

For N-S direction:

$V = C_s W = (0.0505)(19113) = 965.2^k$

0.5 s k = 1

0.64 s k = 1.07

2.5 s k = 2

For E-W direction:

$V = (0.0163)(19113) = 311.5^k$

1.17 s k = 1.335

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	115.5	15	130.5	17888	2334
3	31.917	15.5	115.5	15	130.5	17888	2334
4	42.75	10.833	115.5	15	130.5	17888	2334
5	53.583	10.833	115.5	15	130.5	17888	2334
6	64.417	10.834	115.5	15	130.5	17888	2334
7	75.25	10.833	115.5	15	130.5	17888	2334
8	86.083	10.833	115.5	15	130.5	17888	2334
Roof	101.42	15.337	155	0	155	17888	2773
						Total Weight =	**19113**

C_s = 0.0505

k= 1.07

V_b = 965.2 k

Story Forces (North - South)

Floor Number	Height above ground	Story Height (ft)	W (k)	Wh^k	C_{vx}	Forces (k)
2	16.417	16.417	2334	46615	0.030	29.24
3	31.917	15.5	2334	94943	0.062	59.56
4	42.75	10.833	2334	129796	0.084	81.42
5	53.583	10.833	2334	165280	0.107	103.68
6	64.417	10.834	2334	201275	0.131	126.26
7	75.25	10.833	2334	237696	0.154	149.11
8	86.083	10.833	2334	274487	0.178	172.19
Roof	101.42	15.337	2773	388538	0.253	243.73
					Base Shear:	**965.20**

C_s = 0.0163

k= 1.335

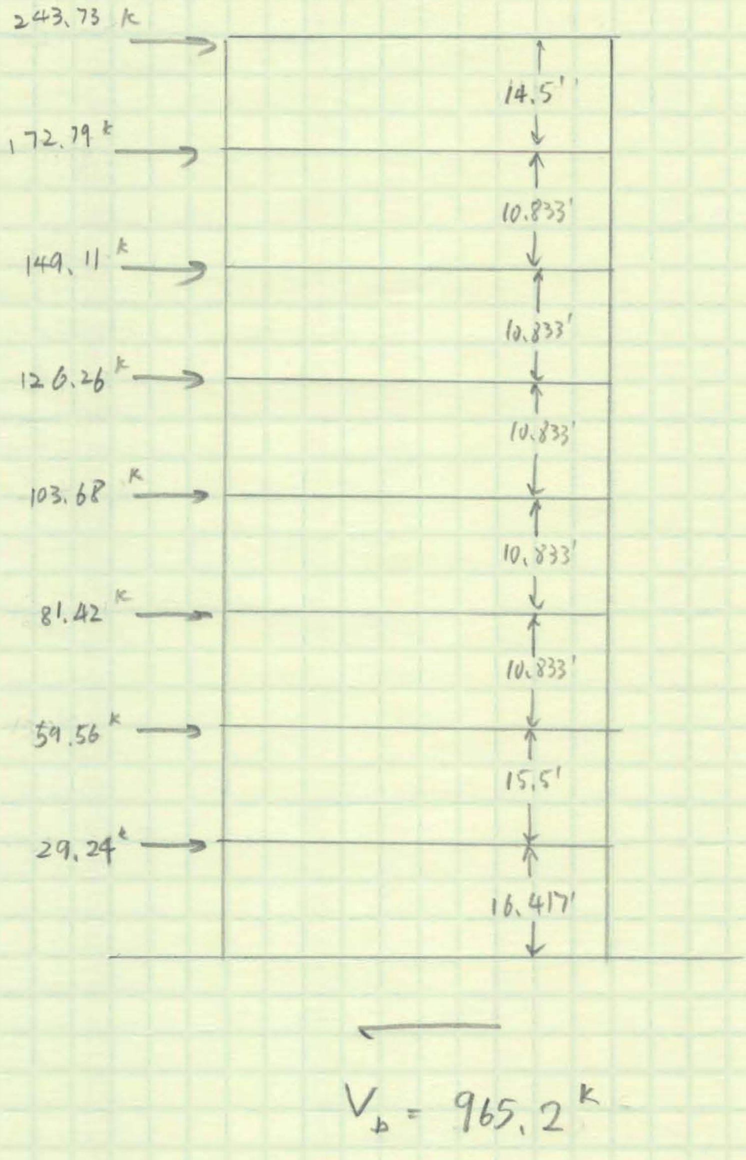
V_b = 311.5 k

Story Forces (East - West)

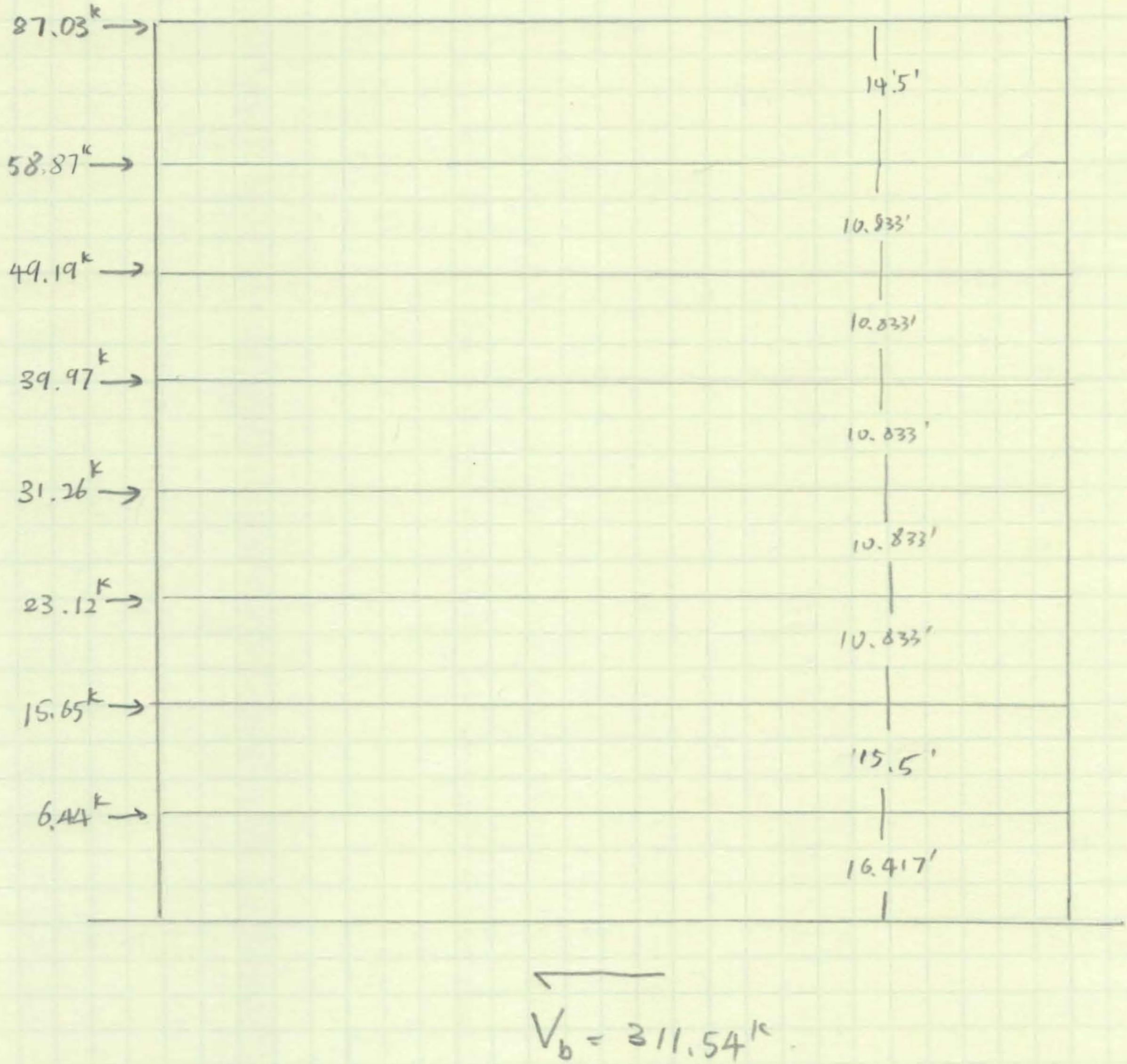
Floor Number	Height above ground	Story Height (ft)	W (k)	Wh^k	C_{vx}	Forces (k)
2	16.417	16.417	2334	97854	0.021	6.44
3	31.917	15.5	2334	237700	0.050	15.65
4	42.75	10.833	2334	351123	0.074	23.12
5	53.583	10.833	2334	474691	0.100	31.26
6	64.417	10.834	2334	606982	0.128	39.97
7	75.25	10.833	2334	746958	0.158	49.19
8	86.083	10.833	2334	893871	0.189	58.87
Roof	101.42	15.337	2773	1321467	0.279	87.03
					Base Shear:	**311.54**

Seismic Loading Diagram

(North-South)



(East - West)


87.03k
58.87k
49.19k
39.97k
31.26k
23.12k
15.65k
6.44k
14.5'
10.833'
10.833'
10.833'
10.833'
10.833'
15.5'
16.417'
V_b = 311.54k

Structural Modeling

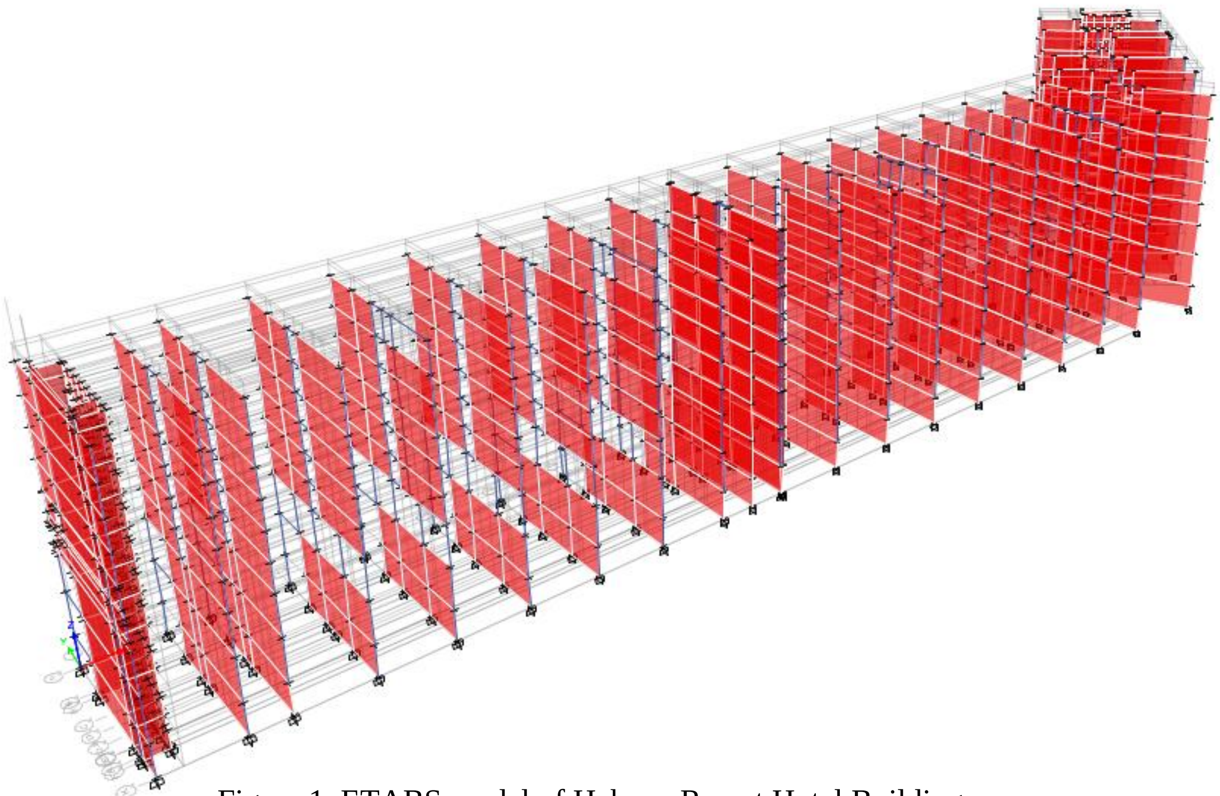


Figure 1. ETABS model of Hakuna Resort Hotel Building

Modeling Process

Hakuna Resort's hotel building is divided from convention building and indoor waterpark structure by expansion joints. The hotel building itself has yet another expansion joint that divides the building in half. For the purpose of this technical assignment, which was hinted in the lateral load analysis revision, only hotel building A will be studied.

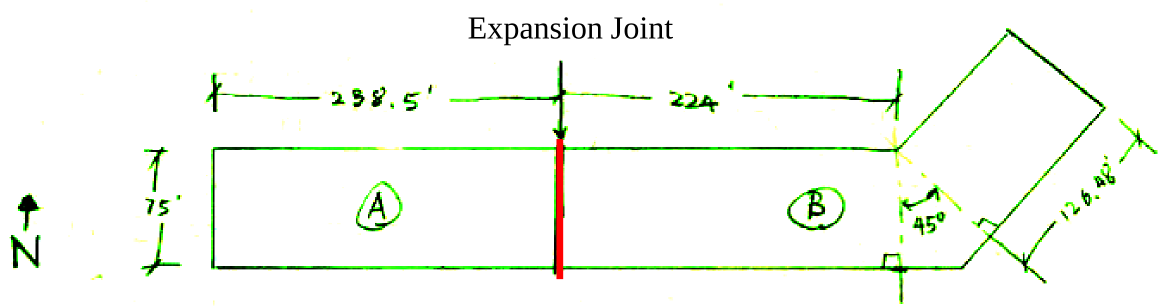


Figure 2. Expansion joint in the hotel building

One of unique features that Hakuna Resort's hotel building has when looking at its lateral system is the overwhelming number of lateral force resisting elements it contains. By having this expansion joint, a simpler model can be made to reduce the amount of time to generate and run a bigger model.

First the structural model was studied to make appropriate assumptions before making model. After making assumptions, the model was generated with lateral loads automatically calculated by ETABS's built in ASCE 7-05 provisions and the revised hand calculation to compare the differences directly.

Assumptions

- Shear walls have fixed bases because it is connected to concrete basement wall that is connected to concrete piers with spread footings.
- Moment frames have fixed bases in order to have enough stiffness to support the additional 60' tall masonry shear walls above them.
- Floor system is rigid diaphragm to keep model drift consistent and simple.
- All shear walls were modeled as membrane shell element with meshing size of roughly 12" X 18", keeping 1:1.5 ratio.
- Partially grouted masonry walls were assigned material property of weight accordingly with their grout spacing.
- Masonry piers within masonry shear walls were modeled as line elements and overlap in areas were ignored.
- Masonry piers that were within fully grouted masonry shear wall was not modeled.

Typical Lateral Resisting Element Analyzed

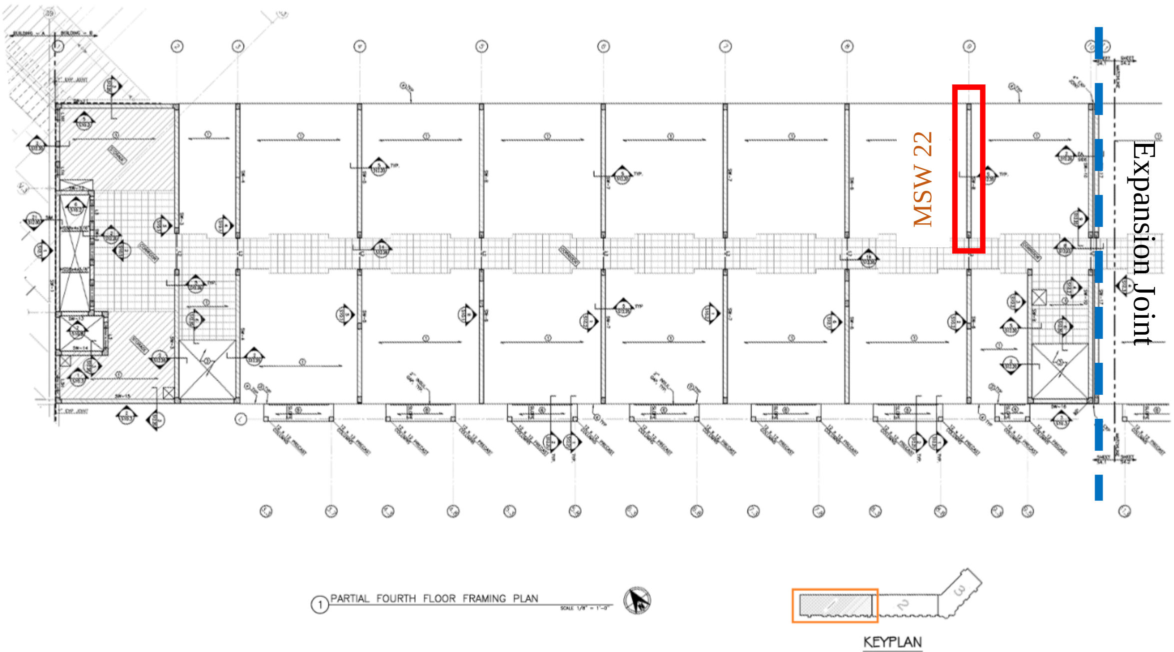


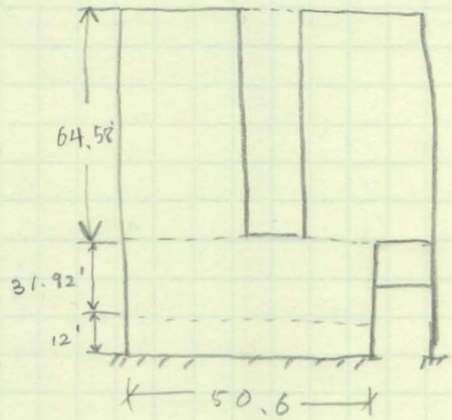
Figure 3. Hakuna Resort Hotel Building A, 4th Floor

Verifying Model with Stiffness Spot Check

Before making a 3D model of the structure, a typical shear wall was taken separately as 2D model with applied 100 kip unit load to spot check the validity of model.

Stiffness Calculation

SW on elevation #9



Masonry

$E = 2700$ ksi

$G = 1125$ ksi

$t = 11.625$ in

Conc.

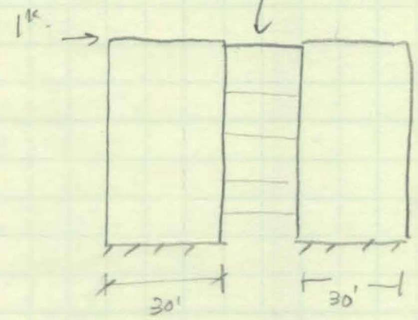
$E = 3605$ ksi

$G = 1502$ ksi

Steel

$E = 29000$ ksi

Masonry

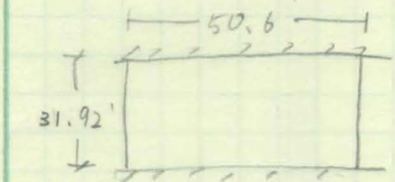


$$k = \frac{1}{\dfrac{h^3}{t\,3E\,\frac{b^3}{12}} + \dfrac{1.2h}{tbG}} \times 2$$

$$= \frac{1}{\dfrac{(64.58\times12)^3}{(11.625)3(2700)\frac{(30\times12)^3}{12}} + \dfrac{1.2(64.58\times12)}{(11.625)(30\times12)(1125)}} \times 2$$

$$= 680.8 \text{ k/in}$$

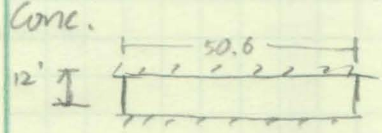
Masonry



$$k = \frac{1}{\left(\dfrac{h^3}{Eb^3t} + \dfrac{1.2h}{Gbt}\right)} = \frac{1}{\left(\dfrac{(31.92\times12)^3}{2700(50.6\times12)^3(11.625)} + \dfrac{1.2(31.92\times12)}{1125(50.6\times12)(11.625)}\right)}$$

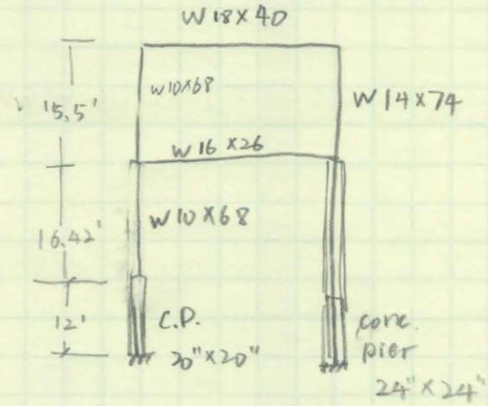
$$= 15179 \text{ k/in}$$

Conc.



$$k = \frac{1}{\left(\dfrac{(12\times12)^3}{3605(50.6\times12)^3(12)} + \dfrac{1.2(12\times12)}{1502(50.6\times12)(12)}\right)}$$

$$= 62121 \text{ k/in}$$



W10X68: $I_x = 394\ in^4$

W14X74: $I_x = 795\ in^4$

20 X 20 C.P.

$$k = \frac{1}{\left(\frac{h^3}{Eb^3t} + \frac{1.2h}{Gbt}\right)} = \frac{1}{\frac{(12\times12)^3}{3605(20)^3 20} + \frac{1.2\ (12\times12)}{1502\ (20)(20)}} = 183\ k/in.$$

24 X 24 C.P.

$$k = \frac{1}{\frac{(28.42\times12)^3}{3605(24)^3(24)} + \frac{1.2(28.42\times12)}{1502(24)(24)}} = 29.73\ k/in.$$

W 10 X68

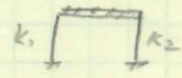
$$k = \frac{12EI}{h^3} = \frac{12(29000)(394)}{(16.42\times12)^3} = 17.9\ k/in$$

$$k = \frac{12(29000)(394)}{(15.5\times12)^3} = 21.3\ k/in.$$

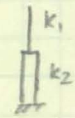
W 14X74

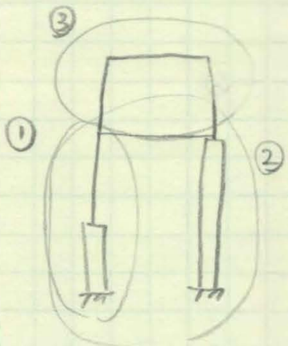
$$k = \frac{12(29000)(795)}{(15.5\times12)^3} = 43\ k/in.$$

k_{eq} in parallel: $k_1 + k_2$



k_{eq} in series: $\frac{1}{k_{eq}} = \frac{1}{k_1} + \frac{1}{k_2}$

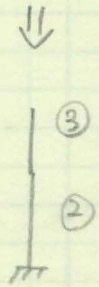




① $\frac{1}{k_{eq}} = \frac{1}{17.9} + \frac{1}{183} = 0.613$

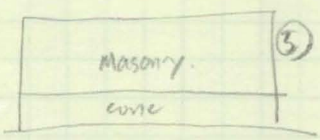
$k_{eq_{①}} = 16.3$ k/in.

② $K_{eq} = 16.3 + 29.73 = 46.03$ k/in.

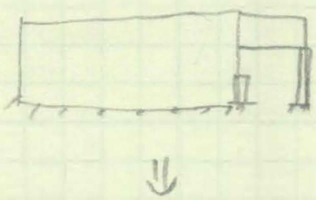


③ $k_{eq} = 21.3 + 43 = 64.3$ k/in

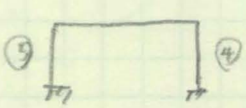
④ $\frac{1}{k_{eq}} = \frac{1}{46.03} + \frac{1}{64.3} \Rightarrow 26.8$ k/in. $= k_{eq}$



⑤ $\frac{1}{k_{eq}} = \frac{1}{62121} + \frac{1}{15179} \Rightarrow k_{eq} = 12198$ k/in



⑥ $= 12198 + 26.8 = 12224.8$ k/in.



⑦ $= \frac{1}{12224.8} + \frac{1}{680.8} =$

Total $= k_{eq} =$ **645 k/in**

$$K = \frac{P}{\Delta}$$

$$K = \frac{100}{0.146}$$

K = 685 k/in

Center of Rigidity Calculation for Typical 4th Floor

4th floor											
Element Label	**Element Direction**	**Height (ft)**	**Width (ft)**	**Length (ft)**	**Stiffness (k/in)**	**Distance from Ref. Datum**		**Rx (k/in)**	**Ry (k/in)**	**Rx*Y**	**Ry*X**
						X (ft)	**Y (ft)**				
MSW 1	Y	10.667	0.97	66.666	67512.68	0.00	-	-	67512.68	-	0.00
MSW 2	Y	10.667	0.97	37.800	37580.98	7.92	-	-	37580.98	-	297517.38
MSW 3	X	10.667	0.97	27.380	26573.58	-	66.67	26573.578	-	1771554.15	-
MSW 4	X	10.667	0.97	7.917	4961.09	-	47.80	4961.0864	-	237149.85	-
MSW 5	X	10.667	0.97	7.917	4961.09	-	17.73	4961.0864	-	87952.62	-
MSW 6	X	10.667	0.97	7.917	4961.09	-	10.00	4961.0864	-	49610.86	-
MSW 7	X	10.667	0.97	41.380	41324.31	-	0.00	41324.313	-	0.00	-
MSW 8	Y	10.667	0.97	31.100	30527.76	27.38	-	-	30527.76	-	835850.19
MSW 9	Y	10.667	0.97	31.100	30527.76	27.38	-	-	30527.76	-	835850.19
MSW 10	Y	10.667	0.97	31.100	30527.76	41.38	-	-	30527.76	-	1263238.88
MSW 11	Y	10.667	0.97	31.100	30527.76	41.38	-	-	30527.76	-	1263238.88
MSW 12	Y	10.667	0.97	31.100	30527.76	69.38	-	-	30527.76	-	2118016.28
MSW 13	Y	10.667	0.97	31.100	30527.76	69.38	-	-	30527.76	-	2118016.28
MSW 14	Y	10.667	0.97	31.100	30527.76	97.38	-	-	30527.76	-	2972793.68
MSW 15	Y	10.667	0.97	31.100	30527.76	97.38	-	-	30527.76	-	2972793.68
MSW 16	Y	10.667	0.97	31.100	30527.76	125.38	-	-	30527.76	-	3827571.08
MSW 17	Y	10.667	0.97	31.100	30527.76	125.38	-	-	30527.76	-	3827571.08
MSW 18	Y	10.667	0.97	31.100	30527.76	153.38	-	-	30527.76	-	4682348.48
MSW 19	Y	10.667	0.97	31.100	30527.76	153.38	-	-	30527.76	-	4682348.48
MSW 20	Y	10.667	0.97	31.100	30527.76	181.38	-	-	30527.76	-	5537125.88
MSW 21	Y	10.667	0.97	31.100	30527.76	181.38	-	-	30527.76	-	5537125.88
MSW 22	Y	10.667	0.97	31.100	30527.76	209.38	-	-	30527.76	-	6391903.28
MSW 23	Y	10.667	0.97	31.100	30527.76	209.38	-	-	30527.76	-	6391903.28
MSW 24	Y	10.667	0.97	31.100	30527.76	223.31	-	-	30527.76	-	6817231.35
MSW 25	Y	10.667	0.97	31.100	30527.76	238.50	-	-	30527.76	-	7280871.77
MSW 26	Y	10.667	0.97	31.100	30527.76	238.50	-	-	30527.76	-	7280871.77
MSW 27	X	10.667	0.97	14.068	11981.04	-	0.00	11981.036	-	0.00	-
							Sum =	**94762.19**	**685121.18**	**2146267.48**	**76934187.79**
										d_{Ry}	d_{Rx}
								Center of Rigidity		**22.65**	**112.29**
								Center of Rigidity (ETABS)		**15.57**	**109.04**

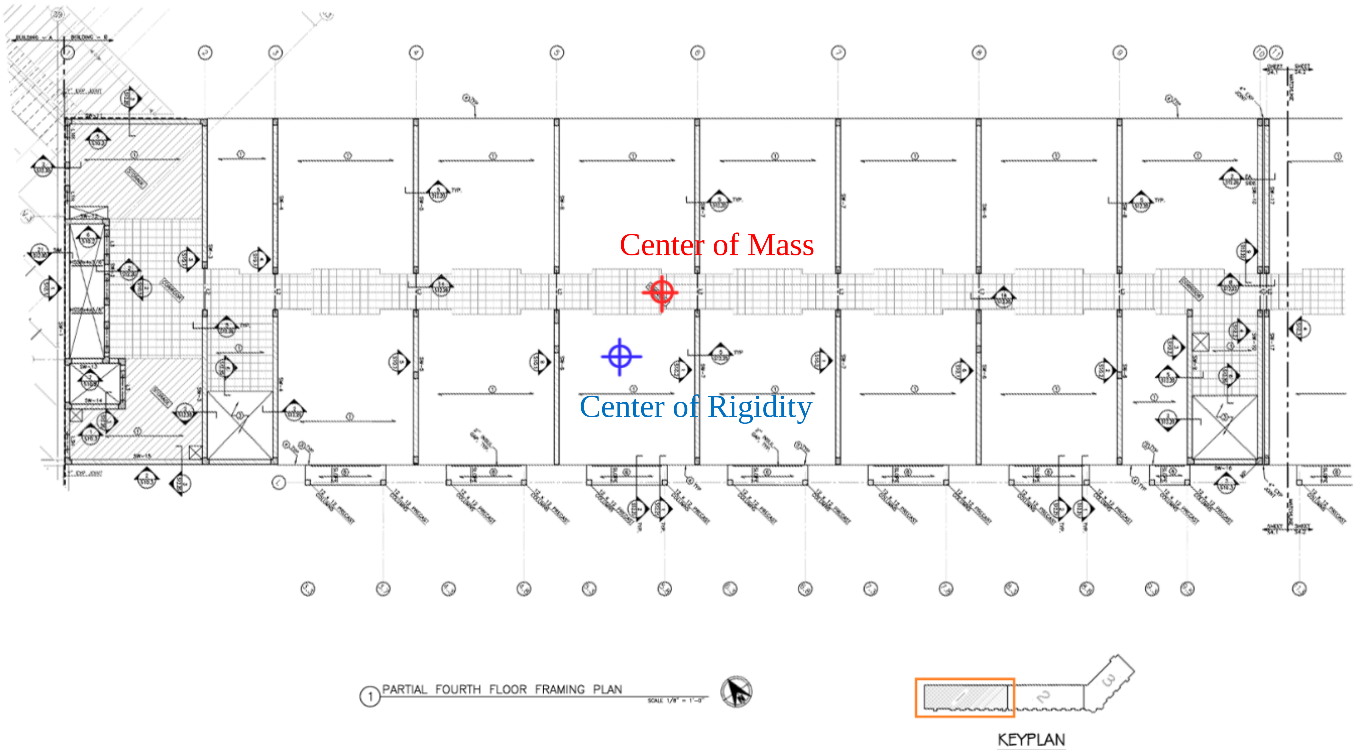


Figure 4. Partial 4th Floor Center of Mass and Rigidity

The hand calculated center of rigidity had percent error of 2.9% in X-direction. However, a larger difference was calculated in Y-direction, which resulted percent error of 31%. This may have to do with the openings in the diaphragms and masonry piers, which were not considered in the hand calculation.

Torsional Rigidity of 4th Floor

Element Label	Stiffness (k/in)	Distance from CoR		Torsional Rigidity ($k*ft^2/in$)
		X (ft)	Y (ft)	
MSW 1	67512.68	112.29	-	851313052
MSW 2	37580.98	104.38	-	409421297
MSW 3	26573.58	-	44.02	51486240
MSW 4	4961.09	-	25.15	3138751
MSW 5	4961.09	-	4.92	120114
MSW 6	4961.09	-	12.65	793758
MSW 7	41324.31	-	22.65	21198405
MSW 8	30527.76	84.91	-	220110874
MSW 9	30527.76	84.91	-	220110874
MSW 10	30527.76	70.91	-	153512759
MSW 11	30527.76	70.91	-	153512759
MSW 12	30527.76	42.91	-	56217179
MSW 13	30527.76	42.91	-	56217179
MSW 14	30527.76	14.91	-	6789134
MSW 15	30527.76	14.91	-	6789134
MSW 16	30527.76	13.09	-	5228623
MSW 17	30527.76	13.09	-	5228623
MSW 18	30527.76	41.09	-	51535647
MSW 19	30527.76	41.09	-	51535647
MSW 20	30527.76	69.09	-	145710204
MSW 21	30527.76	69.09	-	145710204
MSW 22	30527.76	97.09	-	287752297
MSW 23	30527.76	97.09	-	287752297
MSW 24	30527.76	111.02	-	376265989
MSW 25	30527.76	126.21	-	486253953
MSW 26	30527.76	126.21	-	486253953
MSW 27	11981.04	-	22.65	6145991
			Sum =	**4546104938**

Total Shears on 4th Floor

Total Shear (North-South Direction)

		e =	5.24	ft				
Seismic Story Shear at 4th floor			876.4	kips				
Seismic Shear only at 4th floor			81.42	kips				
Torsional Rigidity			4546104938	k*ft2/in				

Element Label	Stiffness (k/in)	Distance from CoR X (ft)	Direct Shear (kips)	Direct Story Shear (kips)	Torsional Shear (kips)	Torsional Story Shear (kips)	Total Shear (kips)	Total Story Shear (kips)
MSW 1	67512.68	112.29	8.02	86.36	-0.71	-7.66	7.31	78.70
MSW 2	37580.98	104.38	4.47	48.07	-0.37	-3.96	4.10	44.11
MSW 8	30527.76	84.91	3.63	39.05	-0.24	-2.62	3.38	36.43
MSW 9	30527.76	84.91	3.63	39.05	-0.24	-2.62	3.38	36.43
MSW 10	30527.76	70.91	3.63	39.05	-0.20	-2.19	3.42	36.86
MSW 11	30527.76	70.91	3.63	39.05	-0.20	-2.19	3.42	36.86
MSW 12	30527.76	42.91	3.63	39.05	-0.12	-1.32	3.50	37.73
MSW 13	30527.76	42.91	3.63	39.05	-0.12	-1.32	3.50	37.73
MSW 14	30527.76	14.91	3.63	39.05	-0.04	-0.46	3.59	38.59
MSW 15	30527.76	14.91	3.63	39.05	-0.04	-0.46	3.59	38.59
MSW 16	30527.76	13.09	3.63	39.05	0.04	0.40	3.67	39.45
MSW 17	30527.76	13.09	3.63	39.05	0.04	0.40	3.67	39.45
MSW 18	30527.76	41.09	3.63	39.05	0.12	1.27	3.75	40.32
MSW 19	30527.76	41.09	3.63	39.05	0.12	1.27	3.75	40.32
MSW 20	30527.76	69.09	3.63	39.05	0.20	2.13	3.83	41.18
MSW 21	30527.76	69.09	3.63	39.05	0.20	2.13	3.83	41.18
MSW 22	30527.76	97.09	3.63	39.05	0.28	2.99	3.91	42.04
MSW 23	30527.76	97.09	3.63	39.05	0.28	2.99	3.91	42.04
MSW 24	30527.76	111.02	3.63	39.05	0.32	3.42	3.95	42.47
MSW 25	30527.76	126.21	3.63	39.05	0.36	3.89	3.99	42.94
MSW 26	30527.76	126.21	3.63	39.05	0.36	3.89	3.99	42.94
Sum =	**685121.18**		**81.42**	**876.4**	**0.00**	**0.0**	**81.42**	**876.4**

Total Shear (East-West Direction)

		e =	10.42	ft				
Seismic Story Shear at 4th floor			289.44	kips				
Seismic Shear only at 4th floor			23.12	kips				
Torsional Rigidity			4546104938	k*ft2/in				

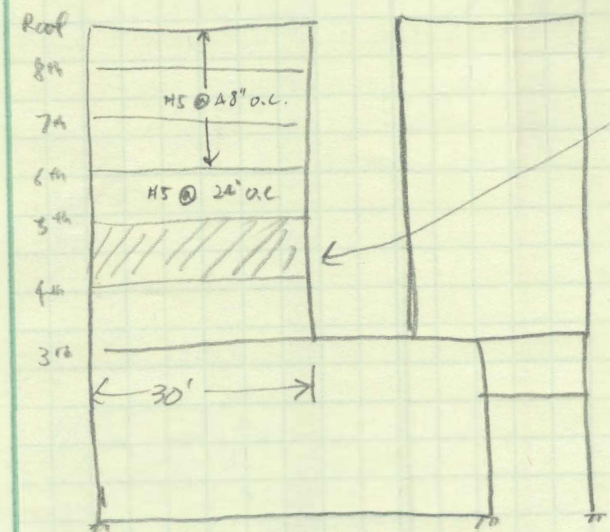
Element Label	Stiffness (k/in)	Distance from CoR Y (ft)	Direct Shear (kips)	Direct Story Shear (kips)	Torsional Shear (ft-kips)	Torsional Story Shear (ft-kips)	Total Shear (kips)	Total Story Shear (kips)
MSW 3	26573.58	44.02	6.48	81.17	-0.06	-0.78	6.42	80.39
MSW 4	4961.09	25.15	1.21	15.15	-0.01	-0.08	1.20	15.07
MSW 5	4961.09	4.92	1.21	15.15	0.00	0.02	1.21	15.17
MSW 6	4961.09	12.65	1.21	15.15	0.00	0.04	1.21	15.19
MSW 7	41324.31	22.65	10.08	126.22	0.05	0.62	10.13	126.84
MSW 27	11981.04	22.65	2.92	36.59	0.01	0.18	2.94	36.77
Sum =	**94762.19**		**23.12**	**289.44**	**0.00**	**0.00**	**23.12**	**289.44**

Verification of Lateral Force Resisting Element

Verification of Element size for LFR.

Column line 9

MSW # 23



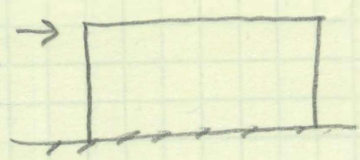
4th floor MSW # 23

12" CMU

#5 @ 16" o.c. partially grouted.

obtained from 4th floor total shear spreadsheet.

Story shear carried from the roof.



$M = 1200^{'k}$

Gravity Load: T. 3.3.3 2004 Design of Reinf. Masonry Str.

Self-weight = (93 psf.)(30)(10.667) = 29.8^k

Stories above = (86 psf)(30)(10.667) + (78 psf)(30)(43.26) = 128.7^k.

Roof = (82.25 psf)(30)(28) = 69.1^k

Tech. 2

Floor = (115.5)(30)(28) = $97^k \times 4 = 388^k$

Total DL = 29.8 + 128.7 + 69.1 + 388 = 615.6^k

D + 0.7E

D + 0.75 L + 0.75 (0.7E)

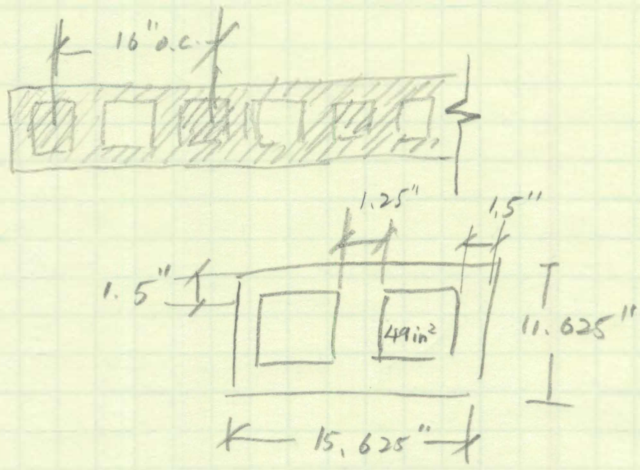
0.6D + 0.7E

Max. shear, max moment, and minimum axial.

$P = 0.6(615.6) = 369.36^k$

$V = 0.7(42) = 29.4^k$

$M = 0.7(1200) = 840^{'k}$



Applied Shear Stress

$A_{nv} \Rightarrow \frac{15.625 \times 11.625 - 49}{16} \times 12$

$99.5 \ in^2/ft$

$d = 30'$

$(99.5)(30) = 2985 \ in^2 = A_{nv}$

$f_v = \frac{29.4^k (1000^{lb}/_k)}{2985} = 9.85 \ psi$

Max. Allowable Shear Stress

$F_v = F_{vm} + F_{vs}$

$\frac{M}{Vd} = \frac{(840)(12^{in}/ft)}{29.4(30 \times 12)} = 0.95 < 1.0$

Linearly interpolate : $F_v \le 2.067\sqrt{f'_m} = 2.067\sqrt{3000} = 113.2 psi$

Allowable Shear in Masonry.

$F_{vm} = \frac{1}{2}\left(\left[4 - 1.75\frac{M}{Vd}\right]\sqrt{f'_m}\right) + \frac{1}{4}\left(\frac{P}{A_n}\right) = \frac{1}{2}\left((4 - 1.75(0.95))\sqrt{3000}\right) + \frac{1}{4}\left(\frac{369360}{2985}\right)$

$F_{vm} = 64 + 30.9 = 94.9$

$F_v = \min \begin{vmatrix} 113.2 \\ 64 \end{vmatrix} \leftarrow$

$f_v < F_v$

$9.85 < 64$ ✓

Add Shear Reinforcing.

$$A_v = \frac{V_s}{0.5 F_s d}$$

$$F_v = F_{vm} + F_{vs} \Rightarrow F_v - F_{vm} = F_{vs}$$

$$F_{vs} = 9.85 - 64 = \boxed{-54.15} \quad ?$$

No need for reinf. due to enough shear in masonry to resis lateral load?

Flexural Check

Section properties:

Strong axis: $r = \frac{b}{\sqrt{12}} = \frac{30 \times 12}{\sqrt{12}} = 103.9$

Weak axis: $r = \frac{t}{\sqrt{12}} = \frac{11.625}{\sqrt{12}} = 3.36$ ← controls.

Slenderness ratio.

$\frac{h}{r} = \frac{10.667(12)}{3.36} = 38.1 < 99$

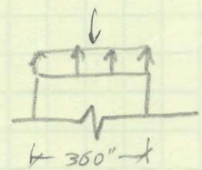
$R = \left(\frac{70}{h/r}\right)^2 = \left(\frac{70}{38.1}\right)^2 = 3.38$

Compessive limit.

$P_a = \frac{1}{4} f'_m R A_n = \frac{1}{4} \frac{(3000)(3.38)(2985)}{1000} = 7567^k$

Interaction Diagram

Point ① Pure compression

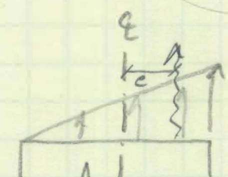


$\varepsilon = \frac{F_b}{E_m} = \frac{0.45}{900} = 0.0005$

$A_n = (30')(99.5 \text{ in}^2/\text{ft}) = 2985 \text{ in}^2$

$C_m = \varepsilon \cdot E_m A_n = 0.0005(2700)(2985) = 4034.25^k = P_1$

Point ③ No net tension.



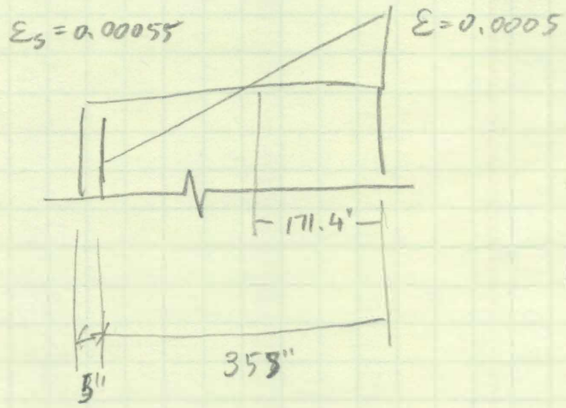
$A_n = 2985 \text{ in}^2$

$C_m = \frac{1}{2}(0.0005)(2985)(2700) = 2016^k = P_2$

$e_2 = \frac{360}{2} - \frac{1}{3}(360) = 60''$

$M_2 = C_{m_2} e_2 = 120892 \text{ k-in.}$

Point ③ 50% allowable stress. try #5 @ 16" o.c.



$k = 0.476$

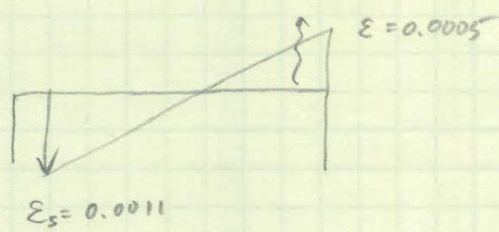
$kd = 0.476(360) = 171.4"$

(See attached spreadsheet)

$M_3 = 113560$ in-k

$P_3 = 928.4^k$

Point ④ 100% Steel allowable stress.



$k = 0.3058$

$kd = 110.1"$

$M_4 = 64132$ in-k

$P_4 = 581.6^k$

Point ⑤

$P_5 = (-0.0011)(29000)(0.31)\left(\frac{360}{16}\right) = -222.5^k$

$M_5 = 0$

4th Floor Masonry Shear Wall Interaction Diagram

f'm 3000 psi
Em 2700 ksi
Es 29000 ksi
grade 60 steel
12" nominal thickness

Point 1 Pure Compression (neglecting slenderness)

	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	180.0	2985.0	0.0005	4029.75	0.0000	0.00
Steel Layer	180.0	0.3100	0.0005	4.50	0.0000	0.00
			P1 =	4034.25	M1 =	0.00

Point 2 No net tension at outside face of masonry

	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	2.9	2985.0	0.0005	2014.88	60.0000	120892.50
Steel Layer	5.8	0.3100	0.0002	1.50	0.0000	0.00
			P2 =	2016.37	M2 =	120892.50

Point 3 50% Allowable Stress

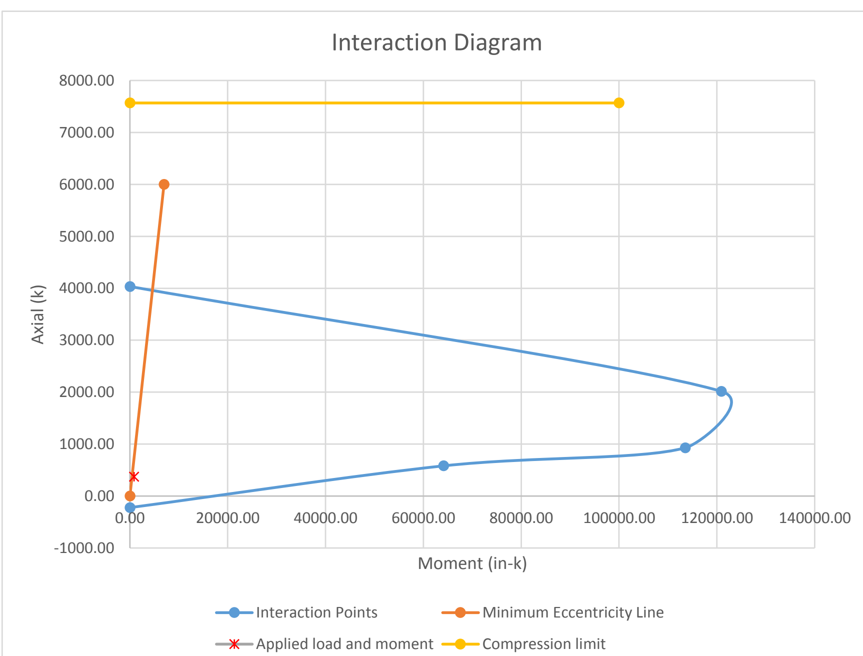
	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	171.4	1421.1917	0.00050	959.30	114.2700	109619.71
Steel Layer						
1	355.0	0.3100	0.000550	-4.94	-183.6	907.81
2	339.0	0.3100	0.000502	-4.51	-167.6	756.48
3	323.0	0.3100	0.000454	-4.08	-151.6	618.94
4	307.0	0.3100	0.000406	-3.65	-135.6	495.19
5	291.0	0.3100	0.000358	-3.22	-119.6	385.22
6	275.0	0.3100	0.000310	-2.79	-103.6	289.05
7	259.0	0.3100	0.000262	-2.36	-87.6	206.66
8	243.0	0.3100	0.000214	-1.93	-71.6	138.06
9	227.0	0.3100	0.000167	-1.50	-55.6	83.25
10	211.0	0.3100	0.000119	-1.07	-39.6	42.23
11	195.0	0.3100	0.000071	-0.64	-23.6	15.00
12	179.0	0.3100	0.000023	-0.20	-7.6	1.56
			P3 =	928.41	M3 =	113559.16

Point 4 100% Allowable Stress

	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	110.1	912.9125	0.0005	616.22	73.4	45230.25
Steel Layer						
1	355.0	0.3100	0.00110	-9.89	-244.9	2421.82
2	339.0	0.3100	0.00103	-9.24	-228.9	2115.71
3	323.0	0.3100	0.00102	-9.20	-212.9	1958.20
4	307.0	0.3100	0.00102	-9.15	-196.9	1800.81
5	291.0	0.3100	0.00101	-9.09	-180.9	1643.55
6	275.0	0.3100	0.00100	-9.01	-164.9	1486.47
7	259.0	0.3100	0.00099	-8.93	-148.9	1329.60
8	243.0	0.3100	0.00098	-8.83	-132.9	1173.03
9	227.0	0.3100	0.00097	-8.70	-116.9	1016.85
10	211.0	0.3100	0.00095	-8.54	-100.9	861.23
11	195.0	0.3100	0.00093	-8.32	-84.9	706.44
12	179.0	0.3100	0.00089	-8.03	-68.9	552.95
13	163.0	0.3100	0.00084	-7.59	-52.9	401.65
14	147.0	0.3100	0.00077	-6.90	-36.9	254.54
15	131.0	0.3100	0.00062	-5.60	-20.9	117.06
16	115.0	0.3100	0.00026	-2.32	-4.9	11.36
	99.0	1.3100	-0.00249	94.66	11.1	1050.78
			P4 =	581.56	M4 =	64132.29

Point 5 Pure Tension

	d (in)	Area (in^2)	Strain (in/in)	Force (k)	e (in)	Moment (in-k/ft)
Masonry	180	2985	0.0000	0.00	0.0	0.00
Steel Layer	180	6.9750	-0.0011	-222.50	0.0	0.00
			P5 =	-222.50	M5 =	0.00



Story Forces and Drifts Analysis (MSW 22)

Story Forces and Drifts - Wind by ETABS				
Story	VX kip	VY kip	Drift	ASCR7-05 Drift Limit
Roof	-9.996	-45.372	0.000032	0.145
Story 8	-24.085	-109.515	0.000033	0.107
Story 7	-37.536	-171.017	0.000034	0.107
Story 6	-50.703	-231.505	0.000034	0.107
Story 5	-63.551	-290.86	0.000032	0.107
Story 4	-76.037	-348.923	0.000029	0.107
Story 3	-90.737	-417.954	0.00022	0.107
Story 2	-107.53	-498.093	0.000224	0.155
Story 1	-121.37	-565.511	0.000008	0.164

Story Forces and Drifts - Seismic by ETABS				
Story	VX kip	VY kip	Drift	ASCR7-05 Drift Limit
Roof	-154.942	-203.835	0.00012	0.145
Story 8	-295.487	-388.732	0.000134	0.107
Story 7	-420.804	-553.597	0.000144	0.107
Story 6	-531.246	-698.89	0.000151	0.107
Story 5	-627.367	-825.345	0.000156	0.107
Story 4	-707.884	-931.271	0.000157	0.107
Story 3	-776.846	-1022	0.00016	0.107
Story 2	-832.336	-1095	0.000164	0.155
Story 1	-854.212	-1123.78	0.000058	0.164

Story Forces and Drifts - Wind by Hand Calculation				
Story	VX kip	VY kip	Drift	ASCR7-05 Drift Limit
Roof	-11.36	-44.8	0.00003	0.145
Story 8	-30.21	-119.64	0.000031	0.107
Story 7	-45.47	-180.55	0.000031	0.107
Story 6	-60.34	-240.28	0.000031	0.107
Story 5	-74.77	-298.66	0.000029	0.107
Story 4	-88.67	-355.45	0.000026	0.107
Story 3	-104.79	-422.11	0.001683	0.107
Story 2	-122.39	-497.01	0.001625	0.155
Story 1	-131.44	-535.54	0.00001	0.164

Story Forces and Drifts - Seismic by Hand Calculation				
Story	VX kip	VY kip	Drift	ASCR7-05 Drift Limit
Roof	-87.031	-243.73	0.0001	0.145
Story 8	-145.901	-415.92	0.000101	0.107
Story 7	-195.091	-565.03	0.000101	0.107
Story 6	-235.061	-691.29	0.000098	0.107
Story 5	-266.321	-794.97	0.000092	0.107
Story 4	-289.441	-876.39	0.00008	0.107
Story 3	-305.091	-935.95	0.000773	0.107
Story 2	-311.531	-965.19	0.000839	0.155
Story 1	-311.531	-965.19	0.000023	0.164

The output produced by the hand calculated values seems to be close compared to hand calculated outputs to believe that model is valid. However, while the wind forces by ETABS ASCE 7-05 seems to be very close to the hand calculated ASCE 7-05 wind load values, the seismic forces are very different. The seismic base shear in X-direction came out almost tripled. There must be an error in either parameter in ETABS seismic ASCE 7-05 input, or the hand calculation is wrong. Even with the very high value by ETABS seismic shear, the displacement limit test barely passed, which makes the structure almost not acceptable.

Roof Displacement (in)		H/600	
Wind - ETABS	0.02249	0.16833	OK
Wind - Hand Calc	0.01976	0.16833	OK
Seismic - ETABS	0.16172	0.16833	OK
Seismic - Hand Calc	0.07674	0.16833	OK