LETTER OF TRANSMITTAL

DATE: September 27, 2013

TO: Dr. Linda Hanagan
Lhanagan.engr.psu.edu

FROM: Alyssa Stangl
ams6158@psu.edu

ENCLOSED: AE 481W – Senior Thesis | Structural Technical Report 2

Dear Dr. Hanagan,

This report was prepared to be submitted for Technical Report 2 for AE 481W – Senior Thesis. It includes a thorough calculation and analysis of all dead and live gravity loads, wind loads, and seismic loads. The report was created using a combination of hand written calculations and excel spreadsheets. The calculations were summarized for the lateral loads in loading diagrams at the end of each section.

Thank you for your time reviewing this report. I look forward to discussing it with you in the near future.

Sincerely,

Alyssa Michelle Stangl

September 27, 2013

Technical Report 2

La Jolla Commons Phase II Office Tower

San Diego, California

Alyssa Stangl | Structural Option | Advisor: Dr. Linda Hanagan

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Executive Summary

La Jolla Commons Phase II Office Tower is a 13 story office building in San Diego, California. Each floor is about 40,320 square feet, and the structure reaches 198 feet from ground level to the top of the penthouse. With two levels of underground parking, the building extends about 20 feet below grade. Acting as an office building for LPL Financial, the building has open floor plans and large areas of glass curtain wall. La Jolla Commons Tower II received a LEED-CS Gold Certification and is the nation's largest and most advanced net-zero office building.

The building's gravity system begins with a mat foundation, two stories below grade. The mat foundation was chosen for its constructability, when compared to a system of footers and grade beams. The super structure consists of two way, flat plate, concrete slabs on a rectangular column grid. A typical bay is 30 feet by 40 feet. Each level varies in thickness – 18, 14, or 12 inches, reinforcing was used as required by code. Camber was used for the slab at each level (except Lower Level 2 where the mat foundation serves as the floor). This was done because large construction loads crack the slab, causing considerable deflections after construction and finishes are completed. Camber ranges from ¼ inch at the exterior edge of a bay to 2 ¼ inches at the center of the bay, creating an essentially flat slab after building loads have been applied.

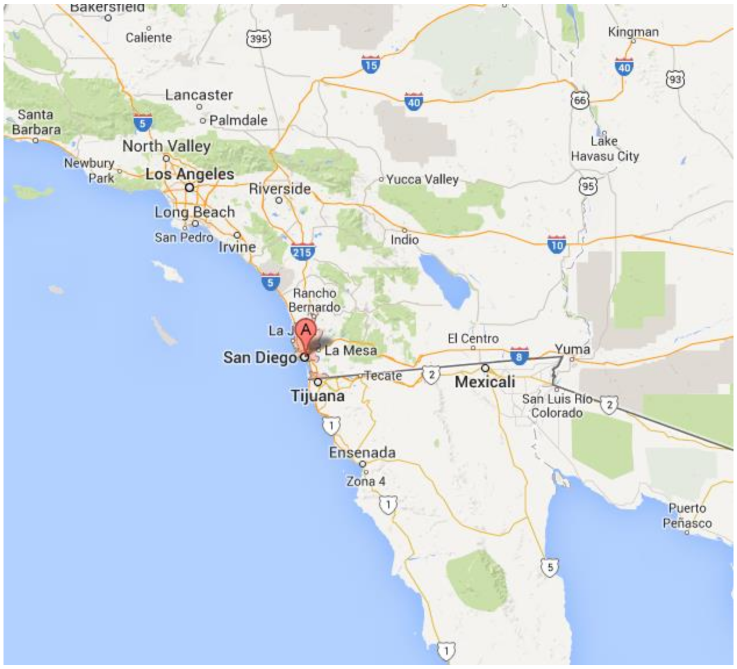
Laid out at the core of the building, the lateral system of La Jolla Commons Tower II consists of reinforced concrete shear walls. Due to the high shear forces associated with earthquake loading in this Seismic Category D structure, the diaphragm is not relied upon to transfer lateral loads to the shear wall system; therefore, collector beams are used to aid in load transfer.

La Jolla Commons Tower II has two unique structural and architectural features. The north and south sides of the building feature 15 foot cantilevers that start at Level 3 and continue up to the roof level. The structure is similar to that of the rest of slab; however, it does have additional reinforcement and a thickened slab edge, creating a back-span for the cantilever. Also, the building has a plaza area on the Ground Level which essentially carves out a portion of the Ground Level and Level 2. Main building columns are exposed here, and additional 18 inch columns are added to support the slab edge above.

La Jolla Commons Tower II was designed using the 2010 California Building Code which corresponds to ASCE 7-05 and ACI 318-08. CBC 2010 and ASCE 7-05 were used to calculate live, wind, and earthquake loads. ACI 318 – 08, Chapter 21, references the design of concrete Earthquake-Resistant structures, and ASCE 7-05, Chapter 12, details the Seismic Design Requirements for Building Structures. Both of these documents were used heavily in the design of LJC II in order to account for seismic loading and detailing.

La Jolla Commons Phase II Office Tower is full of educational value. It has several structural challenges and unique conditions– punching shear, seismic loading and detailing, concrete shear wall design, and computer modeling.

Building Site Information



San Diego California *(Google Maps)*

Building Site Plan *(Courtesy of Hines)*

La Jolla Commons Phase II Office Tower

San Diego , California | LPL Financial Office Tower

Primary Project Team

Owner | Hines
Tenant | LPL Financial
Architect | AECOM
Structural Engineer | Nabih Youssef Associates
MEP Engineer | WSP Flack + Kurtz
Civil Engineer | Leppert Engineering

General Building Data

Construction Dates | April 2012 – May 2014
Building Cost | $78,000,000
Delivery Method | Design-Bid-Build
Height| 198' – 8" | 13 Stories
Size | 462,301 SF

Architecture

Featuring a glass curtain wall system, the building is very modern in style. The floor-to-floor height is 12 feet, and each level is very open, creating a spacious and inviting office space for the tenant, LPL Financial.

Sustainability Features

This building is the first Class A NetZero Office Building in the United States – the building returns more energy to the grid than it uses on an annual basis. La Jolla Commons Tower II also received a LEED – CS Gold Certification upon completion.

Structural

The building structure is comprised of a two-way, flat-plate, reinforced concrete slab. This slab is supported by concrete columns on a rectangular column grid. The lateral system is made up of reinforced concrete shear walls. The structural system is also supported by a mat foundation system.

Mechanical

The mechanical system consists of chilled water floor-by-floor VAV dual path air handling units – two per floor. Each AHU will provide ventilation and cooling through underfloor air distribution. Each AHU will supply overhead air distribution to perimeter zones.

Lighting and Electrical

The lighting system utilizes high efficiency, low glare fixtures, with high power factor electronic ballasts in all fluorescent fixtures. The lighting control system will be integrated with the Building Management System, with local override switches at each floor.

Two 400 Amp, 480/277V, 3-phase, 4 wire switchboards are required to service the building. One services the lower level tenant bus riser, and the second services the upper level tenant bus riser. The building has one radiator cooled, diesel fuel standby engine generator.

Alyssa Stangl [Structural Option] http://www.engr.psu.edu/ae/thesis/portfolios/2014/ams6158

Documents Used to Create This Report

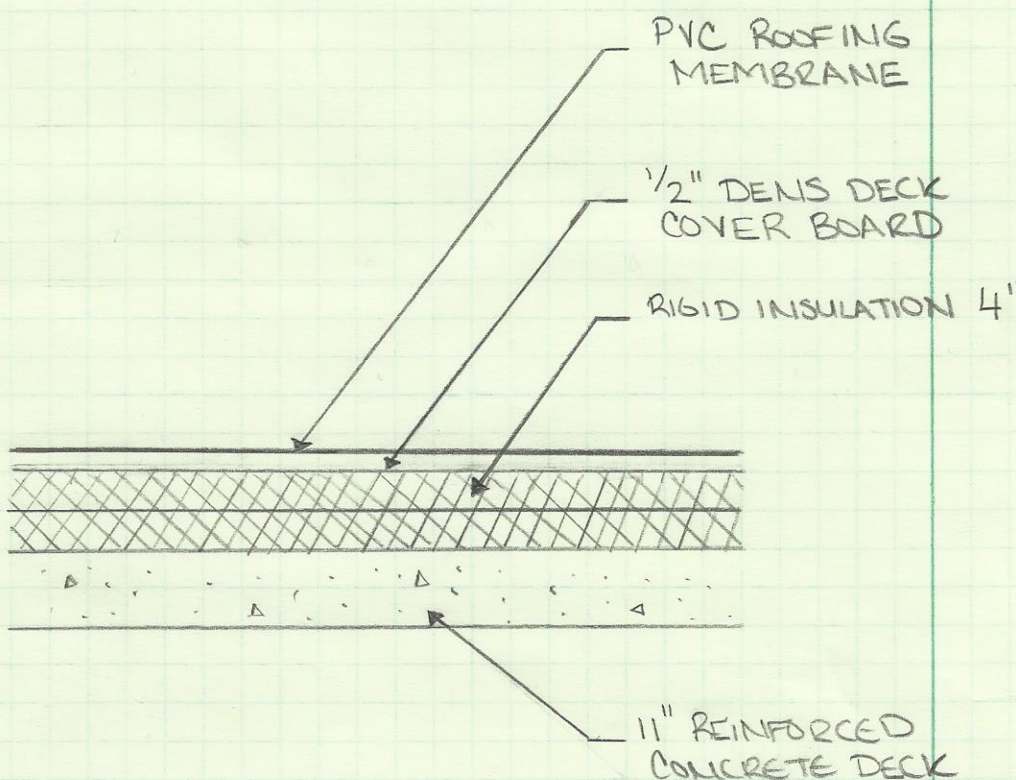
- *California Building Code 2010*
  - Adopts IBC 2009 with some modifications
- *American Society of Civil Engineers*
  - ASCE 7-05 – Minimum Design Loads for Buildings
- *La Jolla Commons Phase II Office Tower*
  - Construction Documents
  - Technical Specifications

GRAVITY LOAD

CALCULATIONS

Typical Roof Bay Loading

Cross Section of Roof Construction (A470-B2)



ROOF DEAD LOAD:

Adhered PVC Membrane = 2 PSF

1/2" Dens Deck Cover Board = 2 PSF

4" Rigid Insulation = 6 PSF

Concrete Slab, 11"

$= (150\text{ PCF})(11''/12'') = 137.5\text{ PSF}$

Superimposed/Misc

Ceilings = 5 PSF
MEP = 15 PSF
Sprinklers = 3 psf

= 23 PSF

+

$\boxed{\text{ROOF DEAD LOAD} = 171\text{ PSF}}$

ROOF LIVE LOAD:

ASCE 7-05 : Ch.4 Table 4-1

$L_r = 20$ PSF

Construction Documents - S001

$\boxed{L_r = 20 \text{ PSF}}$

* Roof live load used for design is equal to the code minimum value

SNOW LOAD:

ASCE 7-05 : Ch.7

Below 1500 ft elevation → 0 PSF
Elevation from 2000-1500 ft → 5 PSF

Site elevation is about 330 ft (C103)

$P_g = 0$ PSF

$P_f = 0.7 C_e C_t I P_g$

$\boxed{P_f = 0 \text{ PSF}}$

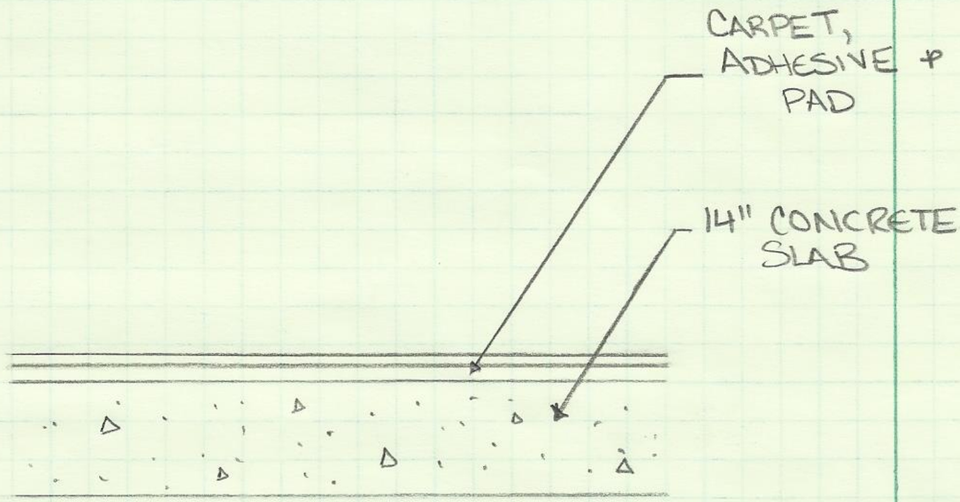
From Figure 7-9 ASCE 7-05:

$h_d = 0$ ft for $P_g = 0$ PSF

$\boxed{\text{Drift Calculation will yield no drift load because } P_g = 0 \text{ PSF}}$

Typical Floor Bay Loading

Cross Section of floor Construction



FLOOR DEAD LOAD:

14" Concrete slab

$= (150 \text{ PCF})(14''/12'') = 175 \text{ PSF}$

Carpet + Adhesive + Pad = 1.5 PSF

Superimposed / MISC

Ceilings = 5 PSF

MEP = 15 PSF

FULLY SPRINKLED = 3 PSF

23 PSF

Raised access floor (by tenant) = 15 PSF (allowance)

\+

214.5

$\boxed{\text{Typical floor bay dead load} = 215 \text{ PSF}}$

Typical Floor Bay Live load:

ASCE 7-05 Chapter 4

Office live load = 50 PSF
Interior partitions = + 20 PSF

70 PSF

Offices, Corridors above 1st floor = 80 PSF

→ Apply 80 PSF to entire office area to allow for future layout flexibility.

$\boxed{\text{Typical Bay Floor Live load = 80 PSF}}$

→ This matches the design value from sheet S001 for office spaces = 80 PSF

Non-Typical Dead Loads:

Floors and Roofs:

- Ground to Level 13, 18" slab edges

  $= (150\ \text{PCF})(18''/12'') = \underline{225\ \text{PSF}}$

- Ground to Level 13, 10" core slab

  $= (150\ \text{PCF})(10''/12'') = \underline{125\ \text{PSF}}$

- Roof / Penthouse floor, 11" slab

  $= (150\ \text{PCF})(11''/12'') = \underline{137.5\ \text{PSF}}$

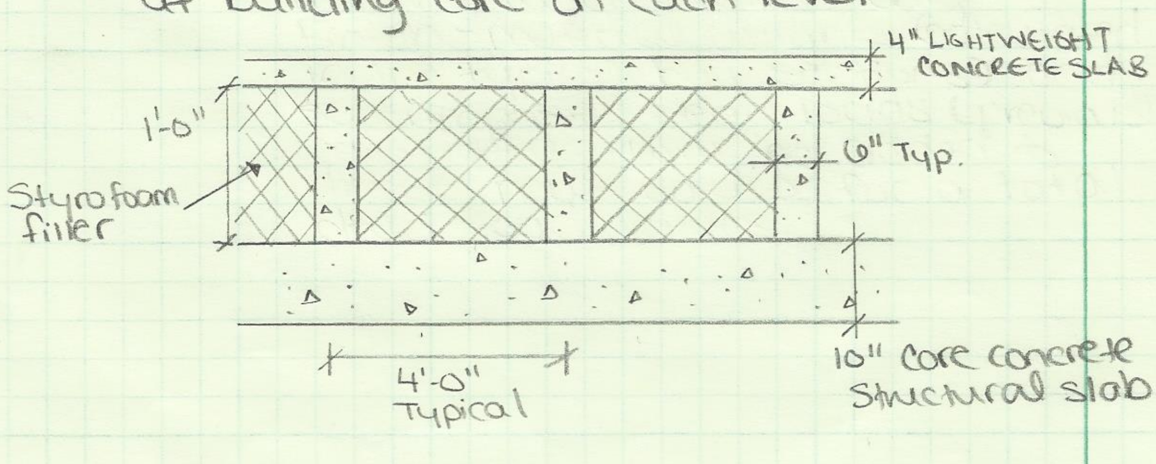
- Roof of Penthouse, 8" slab

  $= (150\ \text{PCF})(8''/12'') = \underline{100\ \text{PSF}}$

- 1/4" x 2" - Roof metal bar grating = $\underline{15\ \text{PSF}}$

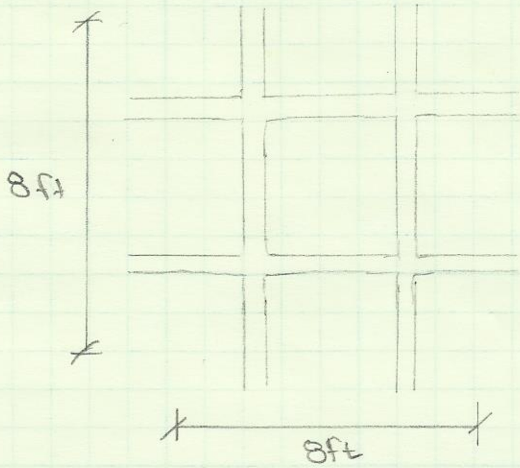
  * From Grating Pacific Catalogue derivision was done.

- Built-up slab at several locations at building core on each level



see next page for calculation

Built up Slab Continued:



- Lets consider an 8 ft x 8 ft segment

LGTWT Concrete

$= (4/12)(115 \text{ PCF}) = \underline{38.3 \text{ PSF}}$

Structural Slab

$= (10''/12'')(150 \text{ PCF}) = \underline{125 \text{ PSF}}$

Pedistals

$(12''/12'')(6''/12)(32')(115 \text{ PCF}) = 1840 \text{ lb}$

$1840 \text{ lb} / 64 \text{ ft}^2 = \underline{28.7 \text{ PSF}}$

TOTAL at built up slab = $\underline{\underline{192 \text{ PSF}}}$

Special Note For
<u>NON-Typ. DEAD LOADS</u>:

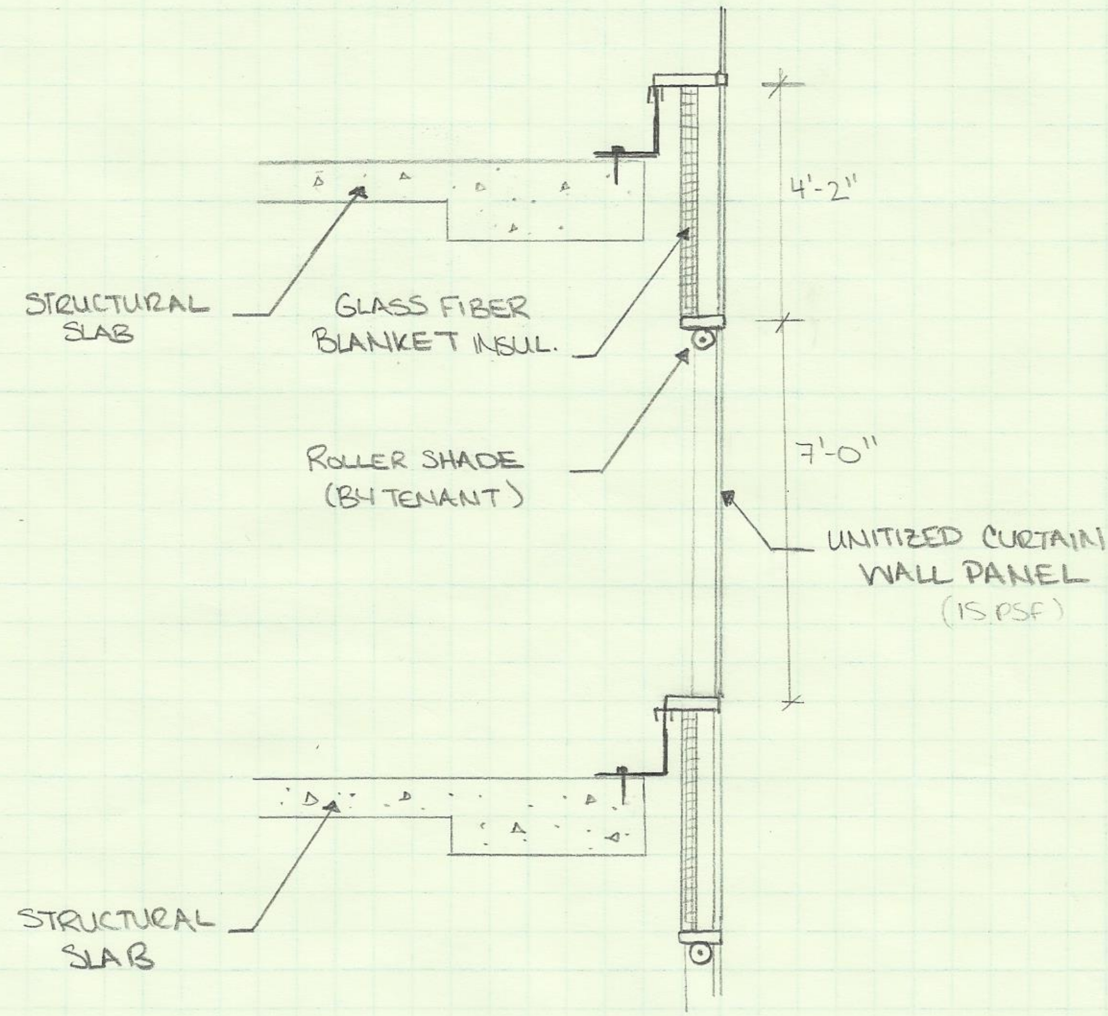
The values provided are modifications to structural components only. The finishes load and superimposed loads (calculated for a typical bay) needs to be added to these values for a total dead load value.

Non-Typical Live Loads:

Use	Location in Building	Design Value	ASCE 7-5 Value	Explaination (if necessary)
Lobby + Corridor	Ground level at core	100 PSF	100 PSF (first floor corridor)	
Lobby + Corridor	Above ground level around core	100 PSF	80 PSF	20 PSF was added for partitions in design to allow flexibility in layout for tenant
Core/ Egress	Above ground level at core	250 PSF	N/A	Value based on egress requirements for possible future multi-tenant conditions
Exit Stairs	At building core	100 PSF	100 PSF	
Cafeteria	In lease space *	100 PSF	100 PSF (Dining and resturants)	
Fitness Center	In lease space *	100 PSF	100 PSF (gymnasiums)	
Conference Center	In lease space *	100 PSF	100 PSF	ASCE 7-10 has an office for heavier anticipated occupancy
Data Center	In lease space *	250 PSF	100 PSF (computer rooms)	PSF load determined from known equipment weights
Mech. Areas	Mechanical Rooms at core on each level	200 PSF	N/A	Value based on industry standard and actual equipment loads if known.

Typical Exterior Wall Load:

Typical Curtain Wall Section:



Wall Load Path - Gravity

The curtain wall is essentially hung by the top mullion of each unit from the slab edge. The unit then ties into the unit at the level below. The wall load goes into the slab which transfers the load into the edge columns. The columns will transfer this load down to the mat foundation, which will spread out the load to meet the bearing capacity of the soil.

Typical Curtain Wall Dead Load:

Line load at slab edge

Fiber blanket Insulation

1.0 PCF, 1" → 1 PCF $(1''/12'')(4' + 2/12) = 0.35$ PLF

Roller Shade

Allowance for tenant selection = 5 PLF

Curtain Wall Units

10 PSF $(4' + 2''/12'' + 7') = 112$ PLF

\+

117.35 PLF

$\boxed{\text{Curtain Wall Assembly Dead Load} = 118 \text{ PLF}}$

WIND LOAD CALCULATION

WIND LOAD CALCULATIONS

- ASCE 7-05 Section 6.5 - Method 2 - Analytical Procedure

1. Occupancy Category (Table 1-1)

   II, All buildings except those in I, III, IV

2. Wind Load Importance Factor (Table 6-1, § 6.5.5)

   $I = 1.00$, for Category II Non-Hurricane prone

3. Basic Wind Speed (Figure 6-1)

   $V = 85$ mph

4. Wind Load Parameters

   a. Wind Directionality Factor, K_d (Table 6-4)

   $K_d = 0.85$

   b. Exposure Category (§ 6.5.6.3)

   Exposure C

   c. Topographic Factor, K_{zt} (Figure 6-4)

   No hill, $K_{zt} = 1.0$

d. Gust Effect Factor (§ 6.5.8)

i. Building Natural Frequency (§ C6.5.8)

- 26.9.2.1 Limitations for approx. natural frequency:

① $h = 198'-8'' < 300'$ ✓
② $4(115') = 460' > 198'-8''$ ✓

Limits are met.

- Approx. natural period for concrete shearwall systems:

$$n_1 = 385(C_w)^{0.5} / H$$

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

$H = 198.67$ ft
$A_B = (315')(123.67')$
$= 39{,}000$ SF

SW 4,6: $h_i = 198.67$ ft
$D_i = 30$ ft
$A_i = (30')(14''/12'') = 35$ ft^2

$$\left(\frac{198.67}{198.67}\right)^2 \frac{35}{1 + 0.83\left(\frac{198.67}{30}\right)^2} = 5.60$$

SW S,R,K,J: $h_i = 174.34$ ft
$D_i = 30$ ft
$A_i = (30')(18''/12'') = 45$ ft^2

$$\left(\frac{198.67}{174.34}\right)^2 \frac{(45)}{\left[1 + 0.83\left(\frac{174.34}{30}\right)^2\right]} = 2.01$$

SW O,N: $h_i = 198.67$ ft
$D_i = 20$ ft
$A_i = (20')(12''/12'') = 20$ ft^2

$$\left(\frac{198.67}{198.67}\right)^2 \frac{20}{\left[1 + 0.83\left(\frac{198.67}{20}\right)^2\right]} = 0.241$$

SW 5 NORTH & 4 SOUTH : $h_i = 198.67$ ft
$D_i = 17$ ft
$A_i = (17 \text{ ft})(18''/12'') = 25.5 \text{ ft}^2$

$$\left(\frac{198.67}{198.67}\right)^2 \frac{25.5}{\left[1+0.83\left(\frac{198.67}{17}\right)^2\right]} = 0.223$$

SW 5 SOUTH & 4 NORTH : $h_i = 198.67$ ft
$D_i = 17$ ft
$A_i = (17 \text{ ft})(26''/12'') = 36.83 \text{ ft}^2$

$$\left(\frac{198.67}{198.67}\right)^2 \frac{36.83}{\left[1+0.83\left(\frac{198.67}{17}\right)^2\right]} = 0.322$$

SW 4 and 4.7: $h_i = 174.34$ ft
$D_i = 30$ ft
$A_i = (30')(14''/12'') = 35 \text{ ft}^2$

$$\left(\frac{198.67}{174.34}\right)^2 \frac{35}{\left[1+0.83\left(\frac{174.34}{30}\right)^2\right]} = 1.57$$

NORTH-SOUTH:

$$\Sigma \left(H/h_i\right)^2 \frac{A_i}{\left[1+0.83(h_i/D_i)^2\right]}$$

$$= 2(0.223) + 2(0.322) + 2(1.57) = 4.23$$

$$C_W = \frac{100}{39000 \text{ SF}}(4.23) = 0.01085$$

$$n_{N\text{-}S} = 385(0.01085)^{0.5}/198.67$$

$$\boxed{n_{N\text{-}S} = 0.202 \text{ Hz}}$$

EAST-WEST:

$$\Sigma \left(H/h_i\right)^2 \frac{A_i}{\left[1+0.83(h_i/D_i)^2\right]}$$

$$= 2(5.6) + 4(2.01) + 2(0.241) = 19.72$$

$$C_W = \frac{100}{39000 \text{ SF}}(19.72) = 0.05057$$

$$n_{E\text{-}W} = 385(0.05057)^{0.5}/198.67$$

$$\boxed{n_{E\text{-}W} = 0.436 \text{ Hz}}$$

$\therefore$ Flexible ($n_a < 1$ Hz) in both directions

ii. Flexible Structures (§6.5.8.2)

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

NORTH - South : $g_Q = g_v = 3.4$

$$\bar{z} = \max \left| \begin{array}{l} 0.6h = 0.6(198.67\text{ ft}) = 119.2\text{ ft} \\ z_{min} = 15\text{ ft} \end{array} \right.$$

$\bar{z} = 119.2$ ft

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6}, \quad c = 0.20$$

$$= 0.20\left(\frac{33}{119.2}\right)^{1/6}$$

$I_{\bar{z}} = 0.161$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}}, \quad \bar{\epsilon} = 1/5,\ \ell = 500\text{ ft}$$

$$= (500)\left(\frac{119.2}{33}\right)^{1/5}$$

$L_{\bar{z}} = 646.43$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right), \quad \bar{b} = 0.65,\ \bar{\alpha} = 1/6.5$$

$$= 0.65\left(\frac{119.2}{33}\right)^{1/6.5}(90)(88/60)$$

$\bar{V}_{\bar{z}} = 104.54$ ft/s

$$Q_{NS} = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}, \quad B = 115\text{ ft},\ h = 198.67\text{ ft}$$

$$= \frac{1}{1 + 0.63\left(\frac{115 + 198.67}{646.43}\right)^{0.63}}$$

$Q_{NS} = 0.715$

North-South Gust Factor Cont.

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

$$= \sqrt{2\ln(3600 \times 0.202)} + \frac{0.577}{\sqrt{2\ln(3600 \times 0.202)}}$$

$g_R = 3.79$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.202)(646.43)}{104.54}$$

$N_1 = 1.25$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(1.25)}{(1+10.3(1.25))^{5/3}}$$

$R_n = 0.117$

$$\eta(R_h) = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.202)(198.67)/104.54$$
$$= 1.766$$

$$\eta(R_B) = 4.6 n_1 B / \bar{V}_{\bar{z}} = 4.6(0.202)(115)/104.54$$
$$= 1.022$$

$$\eta(R_L) = 15.4 n_1 L / \bar{V}_{\bar{z}} = 15.4(0.202)(279)/104.54$$
$$= 8.30$$

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

$$= \frac{1}{1.766} - \frac{1}{2(1.766)^2}(1 - e^{-2(1.766)})$$

$R_h = 0.411$

$$R_B = \frac{1}{1.022} - \frac{1}{2(1.022)^2}(1 - e^{-2(1.022)})$$

$R_B = 0.562$

North-South Gust Factor Cont.

$$R_L = \frac{1}{8.30} - \frac{1}{2(8.3)^2}\left(1-e^{-2(8.3)}\right)$$

$R_L = 0.113$

β = damping ratio, unknown however from AE 538 most buildings have a β of 5-7%.

$\beta = 0.05$

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

$$= \sqrt{\frac{1}{0.05}(0.117)(0.411)(0.562)(0.53+0.47(0.113))}$$

$R = 0.561$

$$G_{f,N\text{-}S} = 0.925\left[\frac{1 + 1.7(0.161)\sqrt{(3.4)^2(.715)^2 + (3.79)^2(0.561)^2}}{1 + 1.7(3.4)(0.161)}\right]$$

$$\boxed{G_{f,N\text{-}S} = 0.903}$$

EAST-WEST GUST FACTOR :

$\bar{z} = 119.2 \text{ ft}$

$I_{\bar{z}} = 0.161 \text{ ft}$

$L_{\bar{z}} = 646.43$

$\bar{V}_{\bar{z}} = 104.54 \text{ ft/s}$

$$Q_{EW} = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$$

$B = 279 \text{ ft}$
$h = 198.67 \text{ ft}$

$$= \sqrt{\frac{1}{1 + 0.63\left(\frac{279 + 198.67}{646.43}\right)^{0.63}}}$$

$Q_{EW} = 0.811$

$$g_R = \sqrt{2 \ln(3600 \times 0.436)} + \frac{0.577}{\sqrt{2 \ln(3600 \times 0.436)}}$$

$g_e = 3.99$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.436)(646.43)}{104.54}$$

$N_1 = 2.70$

$$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{7.47(2.7)}{(1 + 10.3(2.7))^{5/3}}$$

$R_n = 0.0745$

$\eta(R_B) = 4.6(0.436)(279) / 104.54$

$= 5.35$

$\eta(R_L) = 15.4(0.436)(115) / 104.54$

$= 7.39$

$\eta(R_h) = 4.6 n_1 h / \bar{V}_{\bar{z}} = 4.6(0.436)(198.67) / 104.54$

$= 3.81$

East west gust factor cont.

$$R_n = \frac{1}{3.81} - \frac{1}{2(3.81)^2}\left(1-e^{-2(3.81)}\right)$$

$$\underline{R_n = 0.228}$$

$$R_B = \frac{1}{5.355} - \frac{1}{2(5.35)^2}\left(1-e^{-2(5.35)}\right)$$

$$\underline{R_B = 0.169}$$

$$R_L = \frac{1}{7.39} - \frac{1}{2(7.39)^2}\left(1-e^{-2(7.39)}\right)$$

$$\underline{R_L = 0.126}$$

$\beta = 0.05$ (for same reason as N/S direction)

$$R = \sqrt{\frac{1}{0.05}(0.0745)(0.228)(0.169)(0.53+0.47(0.126))}$$

$$\underline{R = 0.184}$$

$$G_{f,E\text{-}W} = 0.925\left[\frac{1+1.7(0.161)\sqrt{(3.4)^2(0.811)^2+(3.99)^2(0.184)^2}}{1+1.7(3.4)(0.161)}\right]$$

$$\boxed{G_{f,EW} = 0.853}$$

e. Enclosure Classification (§6.5.9)

Enclosed (§6.2)

f. Internal pressure Coefficient (Fig. 6-5)

$GC_{pi} = \pm 0.18$

Interpolation for Roof Cp Values: (Fig. 6-6)

NORTH - SOUTH $h/L = 0.712$

0 to 99.34 ft:

-0.9	0.5
?	0.712
-1.3(0.8) = 1.04	1.0

(99.34)(115) = 11424.1 > 1000 → 0.8 reduction

$C_p = -1.2194$

99.34 ft to 198.67 ft:

-0.9	0.5
?	0.712
-0.7	1.0

$C_p = -0.8152$

198.67 ft to 279 ft:

-0.5	0.5
?	0.712
-0.7	1.0

$C_p = -0.5848$

EAST - WEST $h/L = 1.728 > 1.0$

0 to 99.34 ft :

$C_p = -1.3$

$A = (99.34)(279) = 27716\ ft^2 > 1000\ ft^2$

$C_p = -1.3(0.8) = \underline{-1.04}$

99.34 ft to 115 ft :

$C_p = -0.7$

Interpolation for Wall Cp Values: (Fig. 6-6)

NORTH - SOUTH

$L/B = 2.43$

-0.3	2
?	2.43
-0.2	4

$C_p = -0.279$

* No other Cp-values were interpolated

WIND LOADING CALCULATIONS

Equations Utilized:

$Kz = 2.01\ (z/z_g)^{2/\alpha}$

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

$p = qG_fC_p$ (MWFRS for Flexible Buildings)

Constants Previously Calculated by hand:

k_{zt} = 1.00
k_d = 0.85
V = 85.0
I = 1.00
$G_{f,\ NS}$= 0.903
$G_{f,\ EW}$= 0.853

Caclulating k_z and q_z - NS and SW						
Floor Number	**Height above ground (z)**	z_g	α	k_z	q_z	q_h
2	15.00	900	9.5	0.85	13.36	22.99
3	28.17	900	9.5	0.97	15.24	22.99
4	41.34	900	9.5	1.05	16.52	22.99
5	54.51	900	9.5	1.11	17.51	22.99
6	67.68	900	9.5	1.17	18.33	22.99
7	80.85	900	9.5	1.21	19.03	22.99
8	94.02	900	9.5	1.25	19.64	22.99
9	107.19	900	9.5	1.28	20.19	22.99
10	120.36	900	9.5	1.32	20.69	22.99
11	133.53	900	9.5	1.35	21.15	22.99
12	146.70	900	9.5	1.37	21.57	22.99
13	159.87	900	9.5	1.40	21.96	22.99
Penthouse Floor	173.04	900	9.5	1.42	22.33	22.99
Penthouse Roof	198.67	900	9.5	1.46	22.99	22.99

Wall Pressures | NORTH-SOUTH DIRECTION

Wind Pressures | North-South Direction										
Floor Number	**Height above ground (z)**	**q_z**	**q_h**	**Windward (PSF)**	**Leeward (PSF)**	**Trib Height**	**Trib Area (SF)**	**Force (k)**	**Story Shear (K)**	**Overturning Moment (ft-k)**
Ground	0.00	13.36	22.99	9.65	-5.78	7.50	2092.50	32.30	421.44	0.00
2	15.00	13.36	22.99	9.65	-5.78	14.09	1619.78	25.00	389.14	375.03
3	28.17	15.24	22.99	11.01	-5.78	13.17	1514.55	25.43	364.14	716.38
4	41.34	16.52	22.99	11.93	-5.78	13.17	1514.55	26.83	338.71	1109.27
5	54.51	17.51	22.99	12.65	-5.78	13.17	1514.55	27.92	311.87	1521.73
6	67.68	18.33	22.99	13.24	-5.78	13.17	1514.55	28.81	283.96	1949.83
7	80.85	19.03	22.99	13.74	-5.78	13.17	1514.55	29.57	255.15	2391.09
8	94.02	19.64	22.99	14.19	-5.78	13.17	1514.55	30.25	225.57	2843.77
9	107.19	20.19	22.99	14.59	-5.78	13.17	1514.55	30.85	195.33	3306.58
10	120.36	20.69	22.99	14.95	-5.78	13.17	1514.55	31.39	164.48	3778.51
11	133.53	21.15	22.99	15.28	-5.78	13.17	1514.55	31.89	133.08	4258.76
12	146.70	21.57	22.99	15.58	-5.78	13.17	1514.55	32.36	101.19	4746.68
13	159.87	21.96	22.99	15.87	-5.78	13.84	1591.03	34.44	68.83	5506.40
Penthouse Floor	173.04	22.33	22.99	16.13	-5.78	19.42	1196.30	26.22	34.39	4536.56
Penthouse Roof	198.67	22.99	22.99	16.61	-5.78	12.17	365.10	8.17	8.17	1624.11

Base Shear [k] = 421
Total Overturning Moment [ft-k] = 38665

Windard Wall C_p= 0.800 L= 279.00
Leeward Wall C_p= -0.279 B= 115.00
(interpolate) L/B= 2.43

Wall Pressures | EAST-WEST DIRECTION

Wind Pressures | East-West Direction										
Floor Number	**Height above ground (z)**	**q_z**	**q_h**	**Windward (PSF)**	**Leeward (PSF)**	**Trib Height**	**Trib Area (SF)**	**Force (k)**	**Story Shear (K)**	**Overturning Moment (ft-k)**
1	0.00	13.36	22.99	9.12	-10.38	7.50	862.5	16.82	1166.73	0
2	15.00	13.36	22.99	9.12	-10.38	14.09	3929.7	76.63	1149.91	1149.43
3	28.17	15.24	22.99	10.40	-10.38	13.17	3674.4	76.35	1073.28	2150.90
4	41.34	16.52	22.99	11.27	-10.38	13.17	3674.4	79.57	996.93	3289.34
5	54.51	17.51	22.99	11.95	-10.38	13.17	3674.4	82.05	917.36	4472.62
6	67.68	18.33	22.99	12.51	-10.38	13.17	3674.4	84.10	835.31	5691.76
7	80.85	19.03	22.99	12.98	-10.38	13.17	3674.4	85.85	751.21	6941.05
8	94.02	19.64	22.99	13.40	-10.38	13.17	3674.4	87.39	665.36	8216.50
9	107.19	20.19	22.99	13.78	-10.38	13.17	3674.4	88.77	577.97	9515.16
10	120.36	20.69	22.99	14.12	-10.38	13.17	3674.4	90.02	489.20	10834.74
11	133.53	21.15	22.99	14.43	-10.38	13.17	3674.4	91.17	399.18	12173.40
12	146.70	21.57	22.99	14.72	-10.38	13.17	3674.4	92.23	308.01	13529.62
13	159.87	21.96	22.99	14.99	-10.38	13.84	3860.0	97.92	215.79	15654.63
Penthouse Floor	173.04	22.33	22.99	15.24	-10.38	19.42	3300.3	84.55	117.87	14631.08
Penthouse Roof	198.67	22.99	22.99	15.69	-10.38	12.17	1277.9	33.31	33.31	6618.37

Base Shear [k]= 1167
Total Overturning Moment [ft-k] = 114869

Windard Wall C_p= 0.800 L= 115.00
Leeward Wall C_p= -0.500 B= 279.00

Roof Wind Uplift | NORTH-SOUTH DIRECTION

Wind Pressures - Roof Uplift | North South					
Location on Roof	**Cp**	**G**	**q_h**	**Pressure [PSF]**	
0 to 99.34 ft	-1.2194	0.903	22.99	-25.32	<- Area Reduction Applies for Cp = 99.34*115=11424.1 ft^2
99.34 to 198.67 ft	-0.8152	0.903	22.99	-16.92	
198.67 to 279 ft	-0.5848	0.903	22.99	-12.14	

h= 198.67
L= 279
h/L= 0.712

NOTE: Interpolation between h/L=0.5 and h/L=1.0 can be seen on Page 18 of hand calculations. Also, Area reduction calculation can be seen on Page 18.

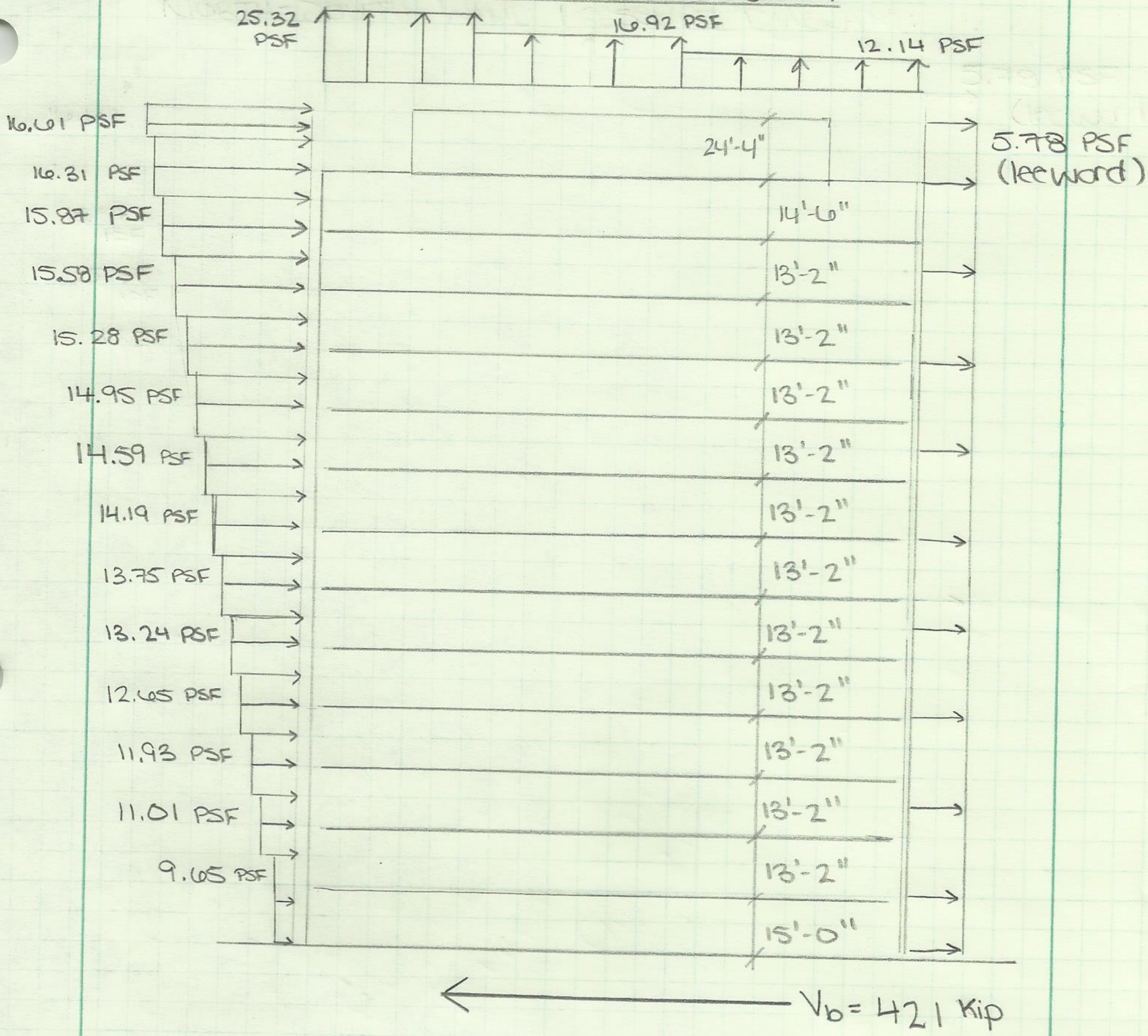
Roof Wind Uplift | EAST-WEST DIRECTION

Wind Pressures - Roof Uplift | East West					
Location on Roof	**Cp**	**G**	**q_h**	**Pressure [PSF]**	
0 to 99.34 ft:	-1.040	0.853	22.99	-20.40	<- Area Reduction Applies for Cp= 99.34*279=27716 ft^2
99.34 to 115 ft:	-0.700	0.853	22.99	-13.73	

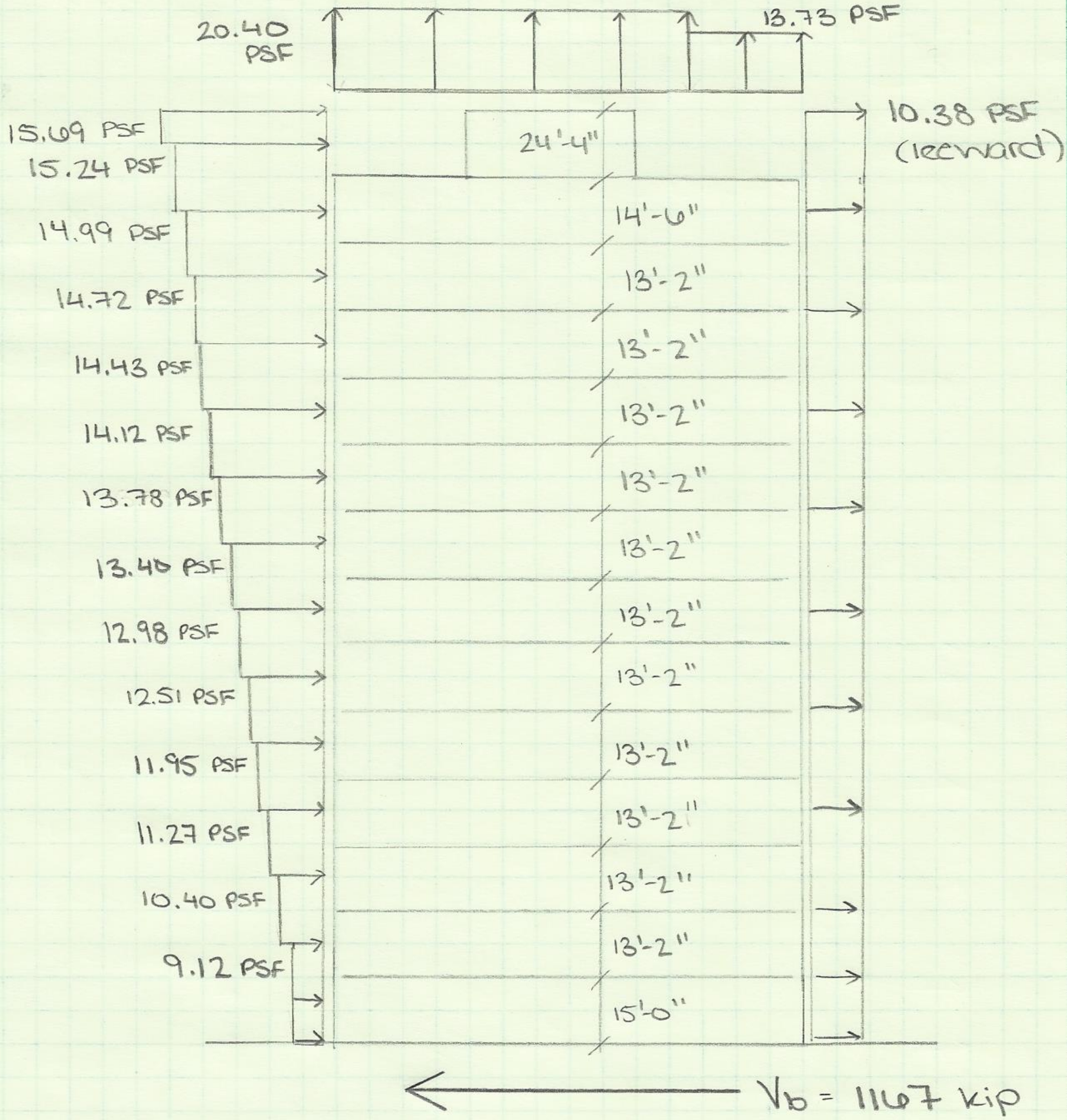
h= 198.67
L= 115
h/L= 1.728

NOTE: Area reduction application calculation can be seen on Page 19 of hand calculations.

NORTH-SOUTH WIND PRESSURE DIAGRAM



EAST-WEST WIND PRESSURE DIAGRAM



SEISMIC LOAD CALCULATION

SEISMIC LOAD CALCULATIONS

1. Building not exempt. (§ 11.1.2)
2. Design Spectral Response Acceleration (§11.4)
   a) Site Class Definition = C
   b) Acceleration Parameters

$S_S = 1.418\,g$

$S_1 = 0.527\,g$

   c) Site Class Effects (§11.4.3)

$F_a = 1.0$
$F_v = 1.3$

$S_{MS} = (1.0)(1.418g) \rightarrow \underline{S_{MS} = 1.418g}$

$S_{M1} = (1.3)(0.527g) \rightarrow \underline{S_{M1} = 0.6851g}$

   d) Determine Spectral Acceleration Parameters (§11.4.4)

$S_{DS} = 2/3\, S_{MS}$
$= 2/3(1.418g) \rightarrow \underline{S_{DS} = 0.9453}$

$S_{D1} = 2/3\, S_{M1}$
$= 2/3(0.6851g) \rightarrow \underline{S_{D1} = 0.4567}$

3. Find Seismic Design Category (§11.5 + §11.6)

Occupancy Category = II

$I = 1.0$

$S_{DS} = 0.9453 \geq 0.5 \rightarrow \underline{SDC = D}$

4. Analysis Procedure Selection

(Table 12.6-1)

Equivalent Lateral Force Procedure

5. Determine Response Modification Factor

B.5 - Special reinforced concrete shear walls

$R = 6$

6. Find Period (T) (§ 12.8.2)

$$T_a = C_t h_n^{x}$$

$C_t = 0.02$
$h_n = 198.17 \text{ ft}$
$x = 0.75$

$$T_a = 0.02(198.17)^{0.75} \rightarrow T_a = 1.056\text{ s}$$

7. Determine, T_L (Fig. 22-15)

$T_L = 8\text{ s}$

8. Find C_s (seismic response coefficient) (§12.8.1.1)

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{0.9453}{(6/1.0)} \rightarrow C_s = 0.1576$$

$T_a = 1.056\text{ s} < T_L = 8\text{ s}$

$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.4561}{(1.056)(6/1.0)} = 0.0720$$

$$C_s = \min \left| \begin{array}{l} 0.1576 \\ 0.0720 \end{array} \right. = 0.0720 > 0.01 \checkmark$$

$C_s = 0.0720$

9. Weight Calculation: See excel sheet Page. 28 of this document.

10. Calculate Base Shear (Eq. 12.8-1)

$$V = C_s W$$

$$= (0.0720)(88979\ k)$$

$$V = 6406.5\ k$$

11. Determine Story Forces

Find k:

0.5s	k = 1
T = 1.056s	k = ?
2.5s	k = 2

$$\frac{2-1}{2.5-0.5} = \frac{k-1}{1.056-0.5}$$

$$k = 1.278$$

* See excel sheet for the rest of the seismic force calculation. *
(Pg. 29 of this document)

SIESMIC LOAD CALCULATIONS

Floor Weight Calculation					
Floor Number	**Dead Load**	**Partition Load**	**Total Weight (PSF)**	**Floor Area (ft^2)**	**Weight (kip)**
Penthouse Roof	133	0	133	6704	892
Penthouse Floor	171	0	171	29703	5079
13	215	20	235	29703	6980
12	215	20	235	29703	6980
11	215	20	235	29703	6980
10	215	20	235	29703	6980
9	215	20	235	29703	6980
8	215	20	235	29703	6980
7	215	20	235	29703	6980
6	215	20	235	29703	6980
5	215	20	235	29703	6980
4	215	20	235	29703	6980
3	215	20	235	29703	6980
2	215	20	235	26494	6226
				Total Weight =	**88979**

Seismic Story Forces

T= **1.056** s

k= **1.278**

Vb= **6406.5 k**

Story Forces | North-South						
Floor Number	**hi (ft)**	**h (ft)**	**W (kip)**	**$w*h^k$**	**Cvx**	**Story Forces Fi (kip)**
Penthouse Roof	24.33	198.70	892	771405	0.0248	158.68
Penthouse Floor	14.50	174.37	5079	3718753	0.1194	764.93
13	13.17	159.87	6980	4573855	0.1469	940.83
12	13.17	146.70	6980	4097943	0.1316	842.93
11	13.17	133.53	6980	3633774	0.1167	747.45
10	13.17	120.36	6980	3182178	0.1022	654.56
9	13.17	107.19	6980	2744134	0.0881	564.46
8	13.17	94.02	6980	2320832	0.0745	477.39
7	13.17	80.85	6980	1913741	0.0614	393.65
6	13.17	67.68	6980	1524742	0.0490	313.63
5	13.17	54.51	6980	1156338	0.0371	237.85
4	13.17	41.34	6980	812063	0.0261	167.04
3	13.17	28.17	6980	497389	0.0160	102.31
2	15.00	15.00	6226	198271	0.0064	40.78
		SUM:	**88979**	**31145418**		**6406.5**

Base Shear [k] = 6406.5

Siesmic Load vs. Height Diagram