LETTER OF TRANSMITTAL

Date: September 26, 2014

To: Dr. Thomas Boothby
TEBARC@engr.psu.edu

From: Nick Dastalfo
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Dear Dr. Boothby,

The enclosed documents include my Structural Technical Report 2 for AE481W – Senior Thesis. Technical Report 2 is a detailed structural analysis of 8621 Georgia Avenue in Silver Springs, Maryland.

This report includes a building abstract and site plans in addition to all necessary calculations for the gravity, wind, snow, and seismic load determinations for the building. There will be a detailed analysis of the floor loads, roof loads, and the horizontal distribution of lateral forces.

Thank you for taking the time to read and review my report. I am eagerly looking forward to discussing the project with you in the future.

Sincerely,

Nick Dastalfo

Technical Report 2

8621 Georgia Avenue
Silver Spring, Maryland

Nick Dastalfo | Structural

Advisor: Dr. Thomas Boothby

September 26, 2014

Executive Summary

The building at 8621 Georgia Avenue is proposed to be built on an existing 0.69 acre parking lot located in the downtown business district of Silver Spring, Maryland. The 17 story, 347,000 ft^2 project will create more downtown multi-family housing and parking for the booming region. The project has recently finished the permit phase of development and is nearly the start of construction.

The building will be the tallest of the surrounding buildings and will be clearly visible along specific urban view corridors and pedestrian heavy areas. Therefore, detailed focus was cast on the architectural impact of the form of the glass curtain wall clad building in these locations. Being the tallest building in the area came along with the challenges of remaining under the zoning height restriction of the area. Efforts were made to decrease the floor to floor height by using post tensioning in order to squeeze the most amount of floors into the building.

The first four stories used for parking, retail, and café have flat plate concrete slab floors with minimal use of concrete drop panelss and beams when necessary. The 5th through 17th floor utilize post-tensioned concrete flat plates with spans varying from 15'-10" to 24'-0" throughout these 12 floors of apartments. The variation in column locations and the use of transfer girders were eliminated due to strategic placing of columns in a regular grid that was appropriate for both the parking garage and the apartments.

The building was designed considering live loads, gravity loads, snow loads, wind loads, seismic loads, and lateral loads. The lateral force resisting system in the building is primarily made up of shear walls around the two stair towers of the structure.

The design for this building was governed by the International Building Code 2012 as well as the 'Minimum Design Loads for Buildings and Other Structures" (ASCE 7-10). These codes reference other standards that were integral in the design process and include ACI318-11 and parts 1-5 of the ACI Manual of Standard Practice, PTI's "Post Tensioning Manual, 6th Edition, the "Manual of Standard Practice" from CRSI, and AISC's Steel Construction Manual, 14th Edition.

This report will cover all of these features and many more, in greater detail.

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8621 Georgia Avenue
Silver Spring, Maryland

General Building Data:

Building Height: 161 feet
Number of Stories: 17 floors
Size: 347,009 ft^2
Cost: $51 million
Occupancy: Mixed Use
-Residential, Parking Garage, Retail

Architecture:

The façade of the building brings a refreshing modern addition to the skyline of the developing city of Silver Spring. The position of the building takes advantage of two major view corridors in the urban fabric and has an inviting present on the busy Georgia Avenue.

Structural Systems:

Construction:

MEP:

Lighting / Electrical:

Project Sponsor: Holbert Apple Associates

Nick Dastalfo I Structural
Advisor: Dr. Thomas Boothby

CPEP Link

Site and Location Plan

Documents Referenced for Report

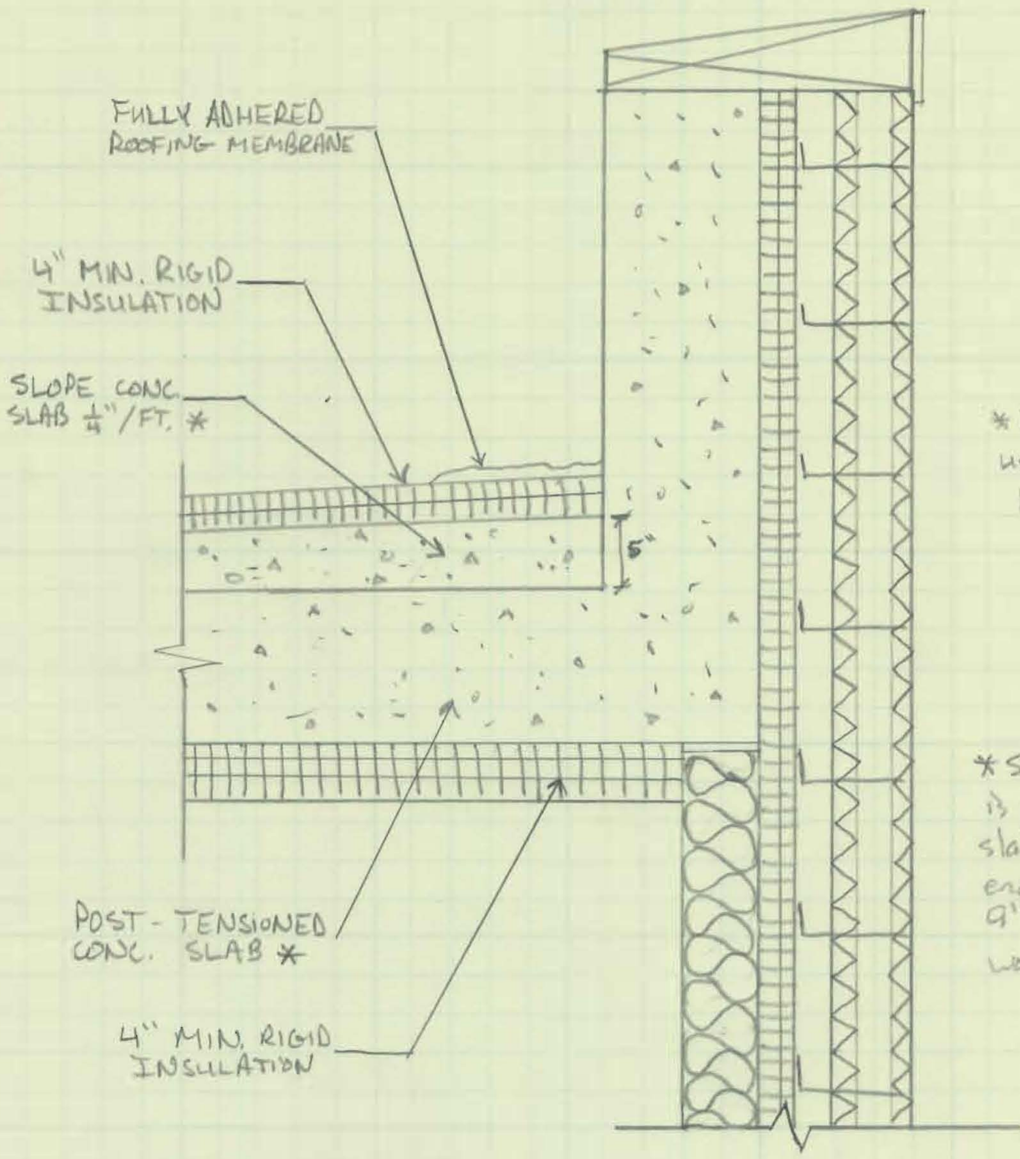
Shown below is a list of the design codes, standards or other references that were used in the structural analysis of 8621 Georgia Avenue for Technical Report 2.

- American Society of Civil Engineers
  - ASCE 7-10: Minimum Design Loads for Buildings and Other Structures
- Montgomery County Building Codes and Standards
- 8621 Georgia Avenue Silver Spring, MD
  - Construction Drawings
  - Specifications
  - Correspondence with Project Engineers
- American Institute of Steel Construction
  - AISC Manual of Load and Resistance Factor Design, 3rd ed.

Roof Loads

Typical Roof Dead Load on 17th Floor

Detail Cross-Section at Parapet



* Lightweight Concrete used for roof sloping LWC = 120 PCF

* Slab on south wing is 9", whereas the slab on the north end is 7½". Use 9" to determine worst case loading

Uniformly Distributed Dead Loads

- Rigid Insulation (2×4 = 8")	=	12 psf
- 5" LW Concrete	=	50 psf
- 9" NW Concrete	=	112.5 psf
- Roofing Membrane	=	2 psf
- Collateral	=	3.5 psf

Total = 180 psf

Typical Roof Live Loads

ASCE 07-10, Table 4-1: Minimum Uniformly Distributed Live Loads

Ordinary Flat Roof .... 20 psf

* See Snow loads, for controlling roof live load, where applicable.

* The Engineer also added an additional 30 psf superimposed dead load. This design decision may have been made for a number of foreseeable factors such as: snow accumulation, ponding, roof maintenance, etc.

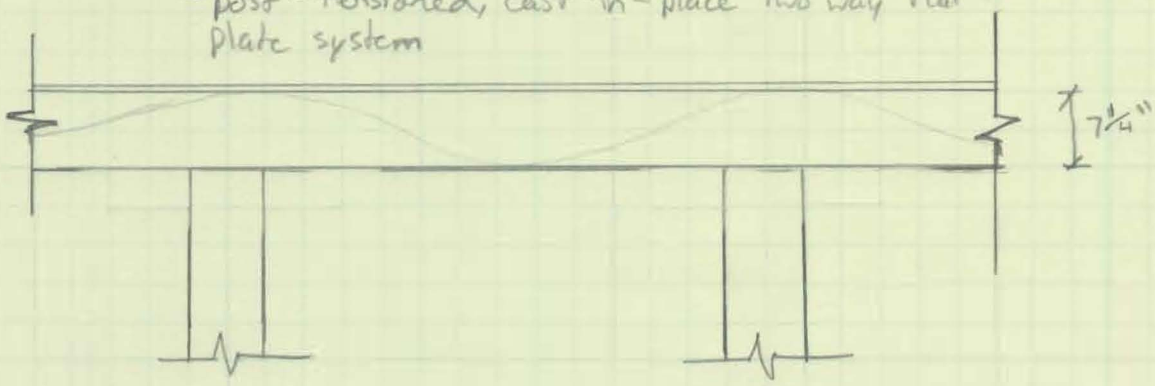
* The Engineer also chose to increase the minimum live load, provided in ASCE 07-10, to 30 psf.

Floor Loads

Floor Dead Loads

Apartment Levels

- post-tensioned, cast-in-place two way flat plate system



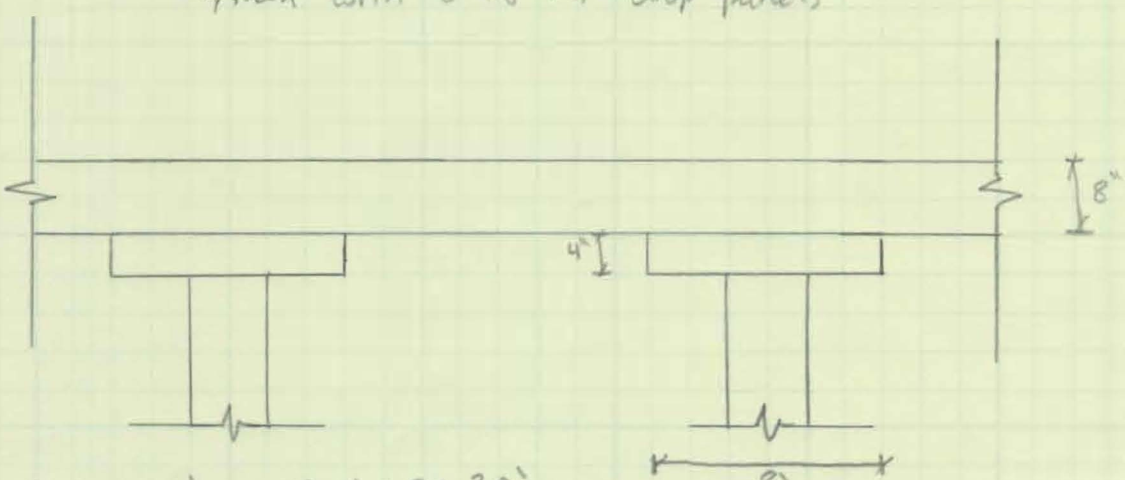
Uniformly Distributed Dead Loads

- 7¼ Concrete = 90.6 psf
- Floor Finish = 2 psf
- Collateral = 5 psf

Total = 97.6 psf

Parking Garage

- 8" mild-reinforced cast-in-place two way flat slab system with 8'×8'×4" drop panels



- average column spacing ≈ 20'

$$\frac{8'}{20'}(12") + \frac{12'}{20'}(8") = 9.6" \rightarrow 10" \text{ avg. thickness}$$

Uniformly Distributed Loads

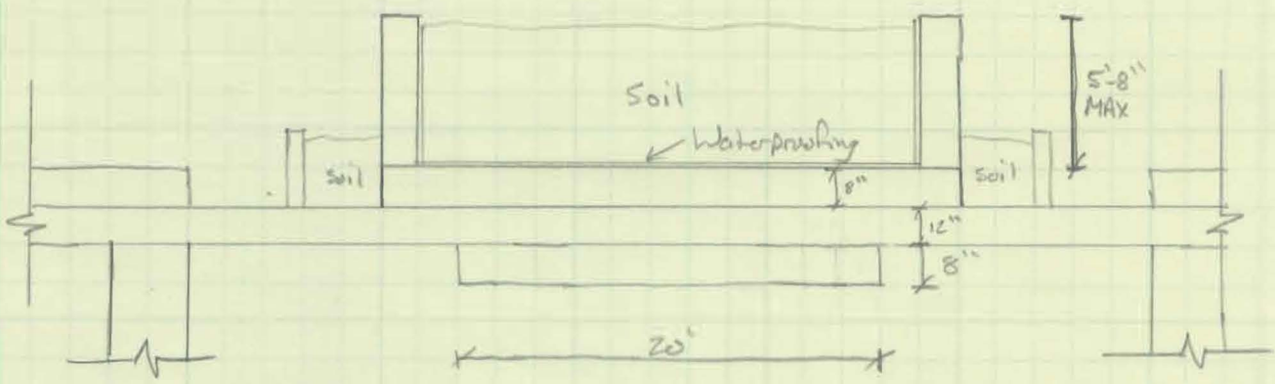
- 10" Concrete = 125 psf
- Collateral = 5 psf

Total = 130 psf

Floor Dead Loads Continued...

Under Bioretension Area:

- The bio-retension area is located on the 5th floor set back and is supported by mild reinforced concrete with a continuous drop panel.



- For worst case, assume planter is saturated $\gamma = 62.4$ pcf

Uniformly Distributed Load

- Avg 20" Concrete = 250 psf
- 5'-8" Water = 353.8 psf
- Water-proofing = 2 psf
- Collateral = 5 psf

Total = 610.8 psf

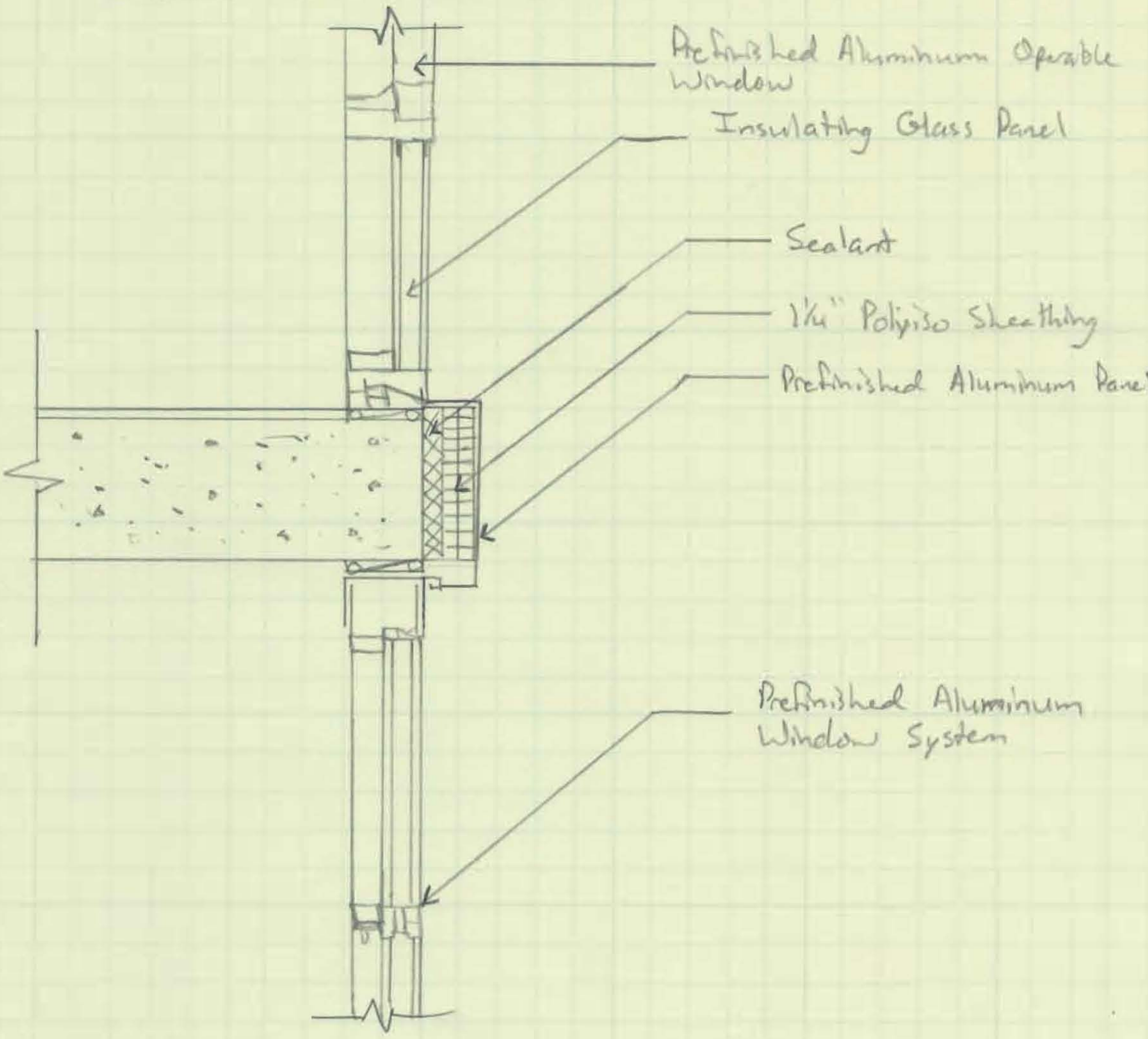
Floor Live Loads

Occupancy	Design Value	ASCE 7-10 Code Minimum
Residential	40 + 10 (partition)	40 psf + 10 (partitions)
Parking Garage	50 psf	40 psf
First Floor Retail	100 psf	100 psf
Public Space	100 psf	100 psf
Fitness Gym	100 psf	100 psf

Exterior Wall Loads

Wall Loads

Typ. Curtain Wall Slab - Operable Window



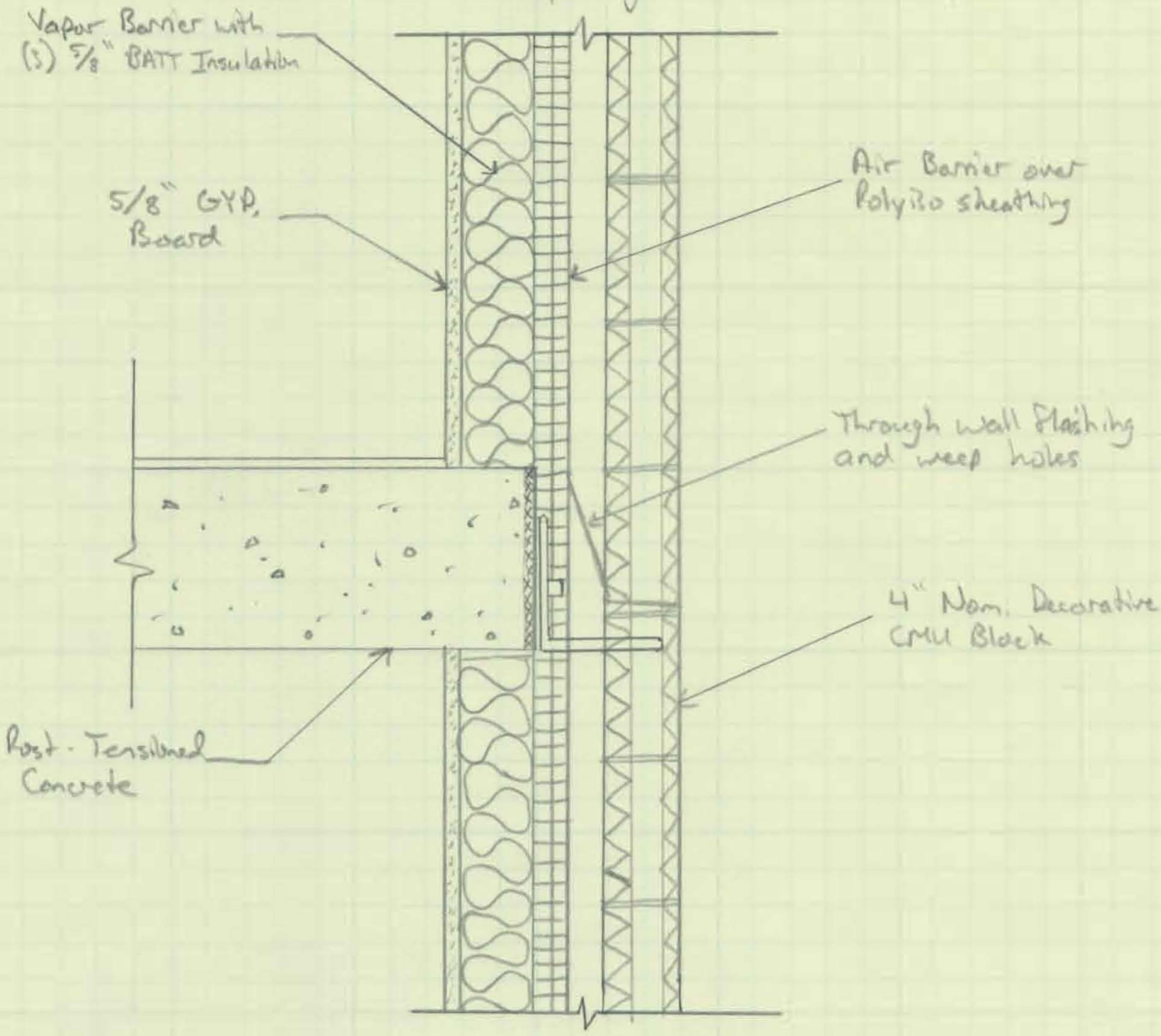
Window System .... 8psf × 10' = 80 plf
↳ AISC Manual

Fasteners .... 5 plf

Total = 85 plf

Typical Exterior Masonry Wall

- The facade is covered in windows but the greatest wall load will result from a fully masonry section.
- assume a 10' story height



Gypsum Board - 5/8" × 4 psf/in × 10' = 25 plf
Batt Insulation - 3(5/8) = 1 7/8" × 1 psf/in × 10' = 18.75 plf
Poly Iso - 1 pcf × 1/12' × 10' ≈ 1 plf
4" Nom. Concrete, 30 psf × 10' = 300 plf

Total = 345 plf

Snow Loads

Snow Loads

$$p_f = 0.7 \cdot C_e \cdot C_t \cdot I_s \cdot p_g$$

Exposure Factor, C_e - Table 7-2

$C_e = 1.0$

Thermal Factor, C_t - Table 7-3

$C_t = 1.0$

Importance Factor, I_s - Table 1.5-2

$I_s = 1.0$

Ground Snow Load, p_g - Figure 7-1

$p_g = 25$ psf → listed in ASCE
* Local code specifies a use of $p_g = 30$ psf

$p_f = 0.7(1.0)(1.0)(1.0)(30 \text{ psf}) = 21$ psf

$p_f = 21.0$ psf

Snow Drift Loads

Snow Drift

Is calculating snow drifts required?

If $\frac{h_c}{h_b} < 0.2$, snow drift loads are not applicable

$$h_b = \frac{p_s}{\gamma} = \frac{30}{0.13p_g + 14} = \frac{30}{.13(30)+14} = 1.68$$

γ = snow density
$\gamma = 0.13\,p_g + 14$, Equation 7.7-1

for parapet, $h_c = 4.33'$

$$\frac{h_c}{h_b} = \frac{4.33}{1.68} = 2.58 > 0.2 \quad \therefore \text{Consider Snow Drift}$$

- All roofs are flat or close to flat. The snow drifting will occur at the parapet or at the 16th/17th floor height difference, by the 16th floor pool.

Parapet Snow Drift

$l_u = 102'\text{-}2"$ $\quad p_g = 30$ psf

Leeward:

$h_d = 3.3'$ (Figure 7-9)

$h_d < h_c$, therefore $w = 4h_d = 13.2'$

Windward

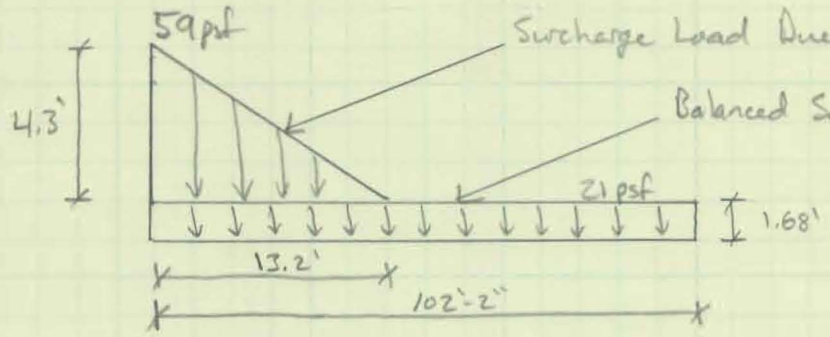
$h_d = \frac{3}{4}(3.3) = 2.48'$

$3.3' > 2.48'$ $\quad \therefore$ Use $h_d = 3.3'$ in design

Snow Drift Continued

$p_d = h_d \cdot \gamma = 3.3\left[0.13(30)+14\right]$

$p_d = 59$ psf



Snow Drift on 16th / 17th Floors

$h_b = 1.68'$, $h_c = 12.32'$

$\frac{12.32'}{1.68'} = 7.33 > 0.2$ ∴ Consider Snow Drift

Leeward:

$l_u = 33'$ $p_g = 30$ psf

$h_d = 1.5$ (Figure 7-9)

$h_c > h_d$, therefore $w = 4\frac{h_d^2}{h_c} = 4\frac{(1.5)^2}{1.68}$

$w = 5.36'$

Windward:

$l_u = 152'$ ← worst case distance, along CL 6 & 7

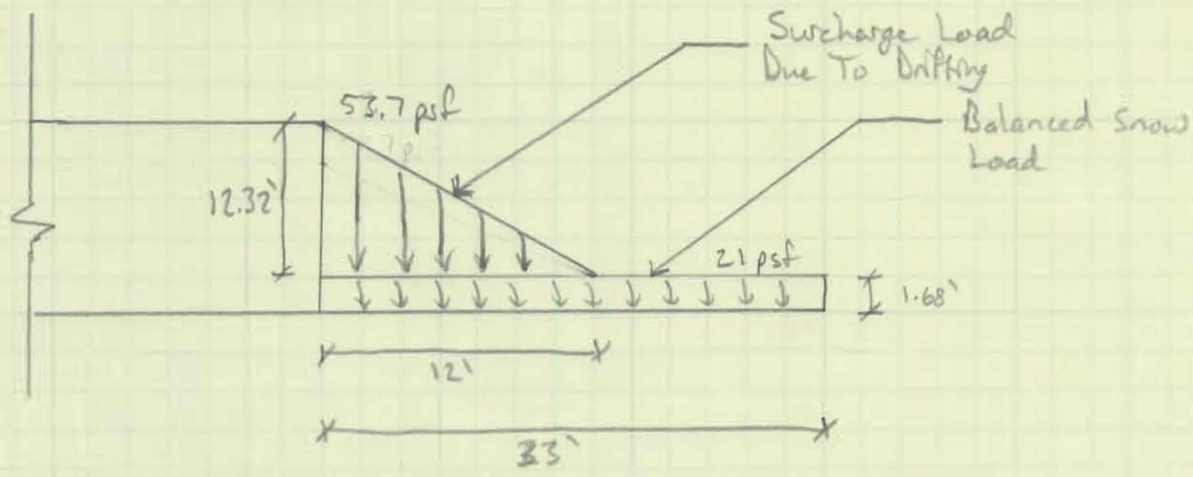
$h_d = 4.0\left(\frac{3}{4}\right) = 3.0$ $3.0 > 1.5$, ∴ Use 3.0 in design

$h_d > h_c$, therefore $w = 4h_d = 12'$

* windward snow drift produces worst case w

Snow Drift Continued...

$$p_d = 3.0\left(0.13(30)+14\right) = 53.7 \text{ psf}$$ $p_d = 53.7$ psf



Wind Loads

Wind Load Calculations

- ASCE 7-10 Chapters 26-30
  - Main Wind Force Resisting System (MWFRS) (Directional Procedure)

① Risk Category (Table 1.5-1)

Category II
- All buildings and other structures except those in Risk Categories I, III, and IV.

② Basic Wind Speed, V (Figure 26.5-1A)

$V = 115$ mph

③ Wind Load Parameters

A.) Wind Directionality Factor, K_d (Table 26.6-1)

$K_d = 0.85$
- For buildings with a MWFRS and Components and cladding.

B.) Exposure Category (Section 26.7)

Exposure Category C
- All cases where categories B and D don't apply

C.) Topographic Factor, K_{zt} (Table 26.8-1)

$K_{zt} = 1.0$
- Building is not located on a ridge, escarpment, or hill.

D.) Gust Effect Factor, G, (Section 26.9)

i.) Frequency Determination (Section 26.9.3)

① Building height < 300 ft.
② Building height < 4 L_{eff}

Meets requirements for alternative analysis in Section 26.9.3

i.) Frequency Determination Continued...

$$n_a = \frac{385(C_w)^{0.5}}{h} \quad \text{-for concrete shear wall buildings}$$

where,

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

Apply Summation to each Shear Wall...

Shear Wall 1 (at CL C, 3-4) E-W

floors 1-4

$h = 161'$

$h_i = 39'\text{-}10''$

$A_i = \frac{14''}{12} \times 18'\text{-}4'' = 21.39\ ft^2$

$D_i = 18'\text{-}4''$

$$\left(\frac{161}{39.83}\right)^2 \cdot \frac{21.39}{1 + .83\left(\frac{39.83}{18.33}\right)^2} = 71.05$$

floors 5-17

$h = 161'$

$h_i = 118'\text{-}4''$

$A_i = \frac{14''}{12} \times 28'\text{-}4'' = 33.06\ ft^2$

$D_i = 28'\text{-}4''$

$$\left(\frac{161}{118.33}\right)^2 \cdot \frac{33.06}{1 + .83\left(\frac{118.33}{28.33}\right)^2} = 61.2$$

Shear Wall 2 (at CL C, 5-6) E-W

$h = 161'$

$h_i = 158'\text{-}2''$

$A_i = \frac{14''}{12} \times 18'\text{-}4'' = 21.39\ ft^2$

$D_i = 18'\text{-}4''$

$$\left(\frac{161}{158.17}\right)^2 \cdot \frac{21.39}{1 + 0.83\left(\frac{158.17}{18.33}\right)^2} = 0.35$$

Shear Wall #3 (at CL 3, C-D) N-S

$h = 161'$
$h_i = 129'\text{-}4''$
$A_i = \frac{12''}{12} \times 9'\text{-}4'' = 9.33\ ft^2$
$D_i = 9'\text{-}4''$

$$\left(\frac{161'}{129.33}\right)^2 \cdot \frac{9.33}{1 + .83\left(\frac{129.33}{9.33}\right)^2} = 0.09$$

Shear Wall #4 (at CL 4, C-D) N-S

floors 1-4
$h = 161'$
$h_i = 28'\text{-}10''$
$A_i = \frac{12''}{12} \times 21'\text{-}6'' = 21.5\ ft.^2$
$D_i = 21'\text{-}6''$

$$\left(\frac{161}{28.83}\right)^2 \cdot \frac{21.5}{1 + 0.83\left(\frac{28.83}{21.5}\right)^2} = 269.02$$

floors 5-17
$h = 161'$
$h_i = 129'\text{-}4''$
$A_i = \frac{12''}{12} \times 9'\text{-}4'' = 9.33\ ft.^2$
$D_i = 9'\text{-}4''$

$$\left(\frac{161'}{129.33}\right)^2 \cdot \frac{9.33}{1 + 0.83\left(\frac{129.33}{9.33}\right)^2} = 0.09$$

Shear Wall #5 (at CL G, 5-6) E-W

$h = 161'$
$h_i = 169'\text{-}8''$
$A_i = \frac{12''}{12} \times 19'\text{-}6'' = 19.5\ ft.$
$D_i = 19'\text{-}6''$

$$\left(\frac{161.0}{169.67}\right)^2 \cdot \frac{19.5}{1 + 0.83\left(\frac{169.67}{19.5}\right)^2} = 0.28$$

Shear Wall #6 (at CL J, 4-6) E-W

$h = 161'$
$h_i = 169'-8''$
$A_i = \frac{12''}{12} \times 42'-4'' = 42.33 \text{ ft}^2$
$D_i = 42'-4''$

$$\left(\frac{161}{169.67}\right)^2 \cdot \frac{42.33}{1 + 0.83\left(\frac{169.67}{42.33}\right)^2} = 2.66$$

Shear Wall #7 (at CL 5.5, G-J) N-S

$h = 161'$
$h_i = 177'-8''$
$A_i = \frac{12''}{12} \times 32'-8'' = 32.67 \text{ ft}^2$
$D_i = 32'-8''$

$$\left(\frac{161}{177.67}\right)^2 \cdot \frac{32.67}{1 + 0.83\left(\frac{177.67}{32.67}\right)^2} = 4.87$$

Shear Wall #8 (at CL 6, G-J) N-S

$h = 161'$
$h_i = 177'-8''$
$A_i = \frac{12''}{12} \times 32'-6'' = 32.5 \text{ ft}^2$
$D_i = 32'-6''$

$$\left(\frac{161}{177.67}\right)^2 \cdot \frac{32.5}{1 + 0.83\left(\frac{177.67}{32.5}\right)^2} = 1.11$$

Shear Wall #9 (at CL 5, C-D) N-S

$h = 161'$
$h_i = 39'-10''$
$A_i = \frac{12''}{12} \times 21'-6'' = 21.5 \text{ ft}^2$
$D_i = 21'-6''$

$$\left(\frac{161}{39.83}\right)^2 \cdot \frac{21.5}{1 + 0.83\left(\frac{39.83}{21.5}\right)^2} = 91.28$$

Shear Wall #10 (at CL. 6, C-D) N-S

$h = 161'$
$h_i = 39'\text{-}10''$
$A_i = \frac{12''}{12} \times 19'\text{-}10'' = 19.83'$
$D_i = 19'\text{-}10''$

$$\left(\frac{161}{39.83}\right)^2 \cdot \frac{19.83}{1+0.83\left(\frac{39.83}{19.83}\right)^2} = 74.51$$

Shear Wall #11 (at CL C.5, 5-6) E-W

$h = 161'$
$h_i = 39'\text{-}10''$
$A_i = \frac{12''}{12} \times 18'\text{-}4'' = 18.33\ ft.^2$
$D_i = 18'\text{-}4''$

$$\left(\frac{161'}{39.83'}\right)^2 \cdot \frac{18.33}{1+.83\left(\frac{39.83}{18.33}\right)^2} = 60.89$$

Shear Wall #12 (at CL D, 5-6) E-W

$h = 161'$
$h_i = 39'\text{-}10''$
$A_i = \frac{12''}{12} \times 18'\text{-}4'' = 18.33\ ft^2$
$D_i = 18'\text{-}4''$

$$\left(\frac{161}{39.83}\right)^2 \cdot \frac{18.33}{1+0.83\left(\frac{39.83}{18.33}\right)^2} = 60.89$$

Shear Wall #13 (at CL. F.3, 7-8) E-W

$h = 161'$
$h_i = 44'\text{-}10''$
$A_i = \frac{12''}{12} \times 16' = 16\ ft.^2$
$D_i = 16'$

$$\left(\frac{161}{44.83}\right)^2 \cdot \frac{16}{1+.83\left(\frac{44.83}{16}\right)^2} = 27.46$$

Shear Wall #14 (at CL. H, 4-5) E-W

$h = 161'$
$h_i = 51'-4''$
$A_i = \frac{12''}{12} \times 22'-9'' = 22.75\ ft.^2$
$D_i = 22'-9''$

$$\left(\frac{161}{51.33}\right)^2 \cdot \frac{22.75}{1+.83\left(\frac{51.33}{22.75}\right)^2} = 42.83$$

• Find C_w

North - South

$$A_B = \left[(31'-7'' \times 118') + (134'-4'' \times 100'-1'') + (44' \times 118'-6'') + (16'-3'' \times 99'-4'')\right]$$
$$= 23,998.7\ ft.^2$$

$$\sum_{i=1}^{n} \left(\frac{h}{h_i}\right)^2 \frac{A_i}{1+0.83\left(\frac{h_i}{D_i}\right)^2} = 0.09 + 269.02 + 0.09 + 4.87 + 1.11 + 91.28 + 74.51 = 440.97$$

$$C_w = \frac{100}{23,998.7}(440.97) = 1.837$$

East - West

$A_B = 23,998.7\ ft.^2$

$$\sum_{i=1}^{n} \left(\frac{h}{h_i}\right)^2 \frac{A_i}{1+0.83\left(\frac{h_i}{D_i}\right)^2} = 71.05 + 61.2 + 0.35 + 0.28 + 2.66 + 60.89 + 60.89 + 27.46 + 42.83 = 327.61$$

$$C_w = \frac{100}{23,998.7}(327.61) = 1.365$$

• Frequency

$$n_{a,N\text{-}S} = \frac{385\, C_w^{0.5}}{h} = \frac{385(1.837)^{0.5}}{161} = 3.24\ Hz.$$

$$n_{a,E\text{-}W} = \frac{385\, C_w^{0.5}}{h} = \frac{385(1.365)^{0.5}}{161} = 2.79\ Hz.$$

$n_a > 1\ Hz$ therefore the structure can be considered rigid.

D). Gust Effect Factor Continued...

- For a rigid structure G can be taken as 0.85 or by the following formula (Section 26.9.4)

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

$$g_Q = g_v = 3.4 \text{ (Section 26.9.4)}$$

For exposure Category C: (Table 26.9-1)
$c = 0.2$
$\ell = 500$ ft
$\bar{\epsilon} = 0.2$
$z_{min} = 15$ ft.

$$\bar{z} = 0.6\,h = 0.6(161) = 96.6\text{ ft} > z_{min} = 15\text{ ft.}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.2\left(\frac{33}{96.6}\right)^{\frac{1}{6}} = 0.167$$

$$Q = \sqrt{\frac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1+0.63\left(\frac{134.33+161}{619.82}\right)^{0.63}}} = 0.847$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 500\left(\frac{96.6}{33}\right)^{0.2} = 619.82$$

$$G = 0.925\left(\frac{1+1.7(3.4)(0.167)(.847)}{1+1.7(3.4)(0.167)}\right) = 0.894$$

- The smallest B will result in the largest Gust Effect factor, Therefore the B from the North-South Direction was used to find the controlling Gust Effect Factor. This is greater than the alternate of 0.85 given in Section 26.9.4, use the greater of the two, which will produce a greater wind load.

$$G = 0.894$$

E). Enclosure Classification (Section 26.10)

Enclosed (Section 26-2).

F.) Internal Pressure Coefficient (Table 26.11-1)

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

④ Determine Velocity Pressure Exposure Coefficient, K_z (Table 27.3-1)

At floor 2 where $z < 15$ ft, $K_z = 2.01\left(\frac{15}{z_g}\right)^{2/\alpha}$, otherwise

$K_z = 2.01\left(\frac{z}{z_g}\right)^{2/\alpha}$

For Exposure Category C, $\alpha = 9.5$ $z_g = 900$ ft (Table 26.9-1)

$K_z\,(10\text{ft}) = 2.01\left(\frac{15}{900}\right)^{\frac{2}{9.5}} = 0.8489$

$K_z\,(19.5\text{ft}) = 2.01\left(\frac{19.5}{900}\right)^{\frac{2}{9.5}} = 0.8971$

$K_z\,(29\text{ft}) = 2.01\left(\frac{29}{900}\right)^{\frac{2}{9.5}} = 0.9753$

$K_z\,(40\text{ft}) = 2.01\left(\frac{40}{900}\right)^{2/9.5} = 1.0436$

$K_z\,(49\text{ft}) = 2.01\left(\frac{49}{900}\right)^{2/9.5} = 1.0891$

$K_z\,(59\text{ft}) = 2.01\left(\frac{59}{900}\right)^{2/9.5} = 1.1326$

$K_z\,(68\text{ft}) = 2.01\left(\frac{68}{900}\right)^{\frac{2}{9.5}} = 1.1669$

$K_z\,(77\text{ft}) = 2.01\left(\frac{77}{900}\right)^{2/9.5} = 1.1979$

$K_z\,(87\text{ft.}) = 2.01\left(\frac{87}{900}\right)^{2/9.5} = 1.2291$

$K_z\,(96\text{ft}) = 2.01\left(\frac{96}{900}\right)^{2/9.5} = 1.2548$

$K_z\,(105\text{ft}) = 2.01\left(\frac{105}{900}\right)^{2/9.5} = 1.2787$

$K_z\,(115\text{ft}) = 2.01\left(\frac{115}{900}\right)^{2/9.5} = 1.3034$

$K_z\,(124\text{ft}) = 2.01\left(\frac{124}{900}\right)^{2/9.5} = 1.3243$

$K_z\,(133\text{ft}) = 2.01\left(\frac{133}{900}\right)^{2/9.5} = 1.3439$

K_z Determination Continued...

$$K_z(146\text{ ft}) = 2.01\left(\frac{146}{900}\right)^{2/9.5} = 1.3706$$

$$K_z(158\text{ ft}) = 2.01\left(\frac{158}{900}\right)^{2/9.5} = 1.3936$$

$$K_z(161\text{ ft}) = 2.01\left(\frac{161}{900}\right)^{2/9.5} = 1.3991$$

⑤ Determine Velocity Pressure Exposure, q_z

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

K_z = varies by height
K_{zt} = 1.0
K_d = 0.85
V = 115 mph

$$q_z = 0.00256(K_z)(1.0)(.85)(115)^2 = 28.78\ K_z$$

q_z (10 ft) = 24.4 psf
q_z (19.5 ft) = 25.8 psf
q_z (29 ft) = 28.1 psf
q_z (40 ft.) = 30.0 psf
q_z (49 ft.) = 31.3 psf
q_z (59 ft.) = 32.6 psf
q_z (68 ft.) = 33.6 psf
q_z (77 ft.) = 34.5 psf
q_z (87 ft.) = 35.4 psf
q_z (96 ft.) = 36.1 psf
q_z (105 ft.) = 36.8 psf
q_z (115 ft.) = 37.5 psf
q_z (124 ft.) = 38.1 psf
q_z (133 ft.) = 38.7 psf
q_z (146 ft.) = 39.4 psf
q_z (158 ft.) = 40.1 psf
q_z (161 ft.) = 40.3 psf

*Also see attached spreadsheet for values of K_z and q_z *

⑥ Determine External Pressure Coefficient, C_p

North - South: $\frac{L}{B} = \frac{194'}{134'} = 1.45$

East - West: $\frac{L}{B} = \frac{134'}{194} = 0.69$

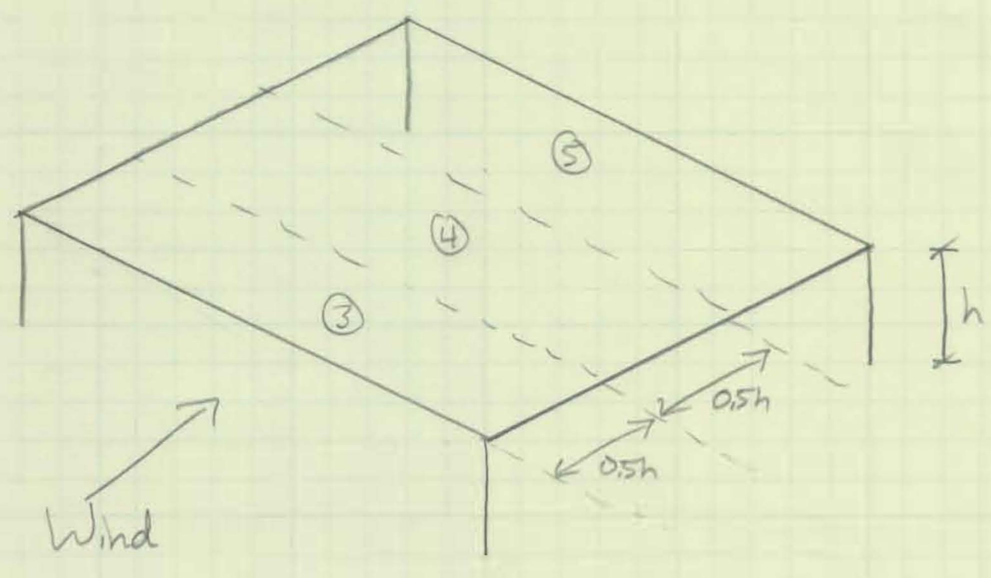
Walls:
- Windward => $C_p = 0.8$
- Leeward => $C_p = -0.3$ in N/S, $C_p = -0.5$ in E/W
- Side Wall => $C_p = -0.7$

Roofs:
- flat roof, $\theta = 0$

From Table 27.6-2 for h=160ft., V=115mph

Zone #	Pressure
1	0
2	0
3	-42.6 psf
4	-38.0 psf
5	-31.2 psf

For Flat Roofs, Table 27.6-2



$$p = q\,GC_p - q_i\,(GC_{pi})$$

$q = q_z$ for winward walls
$= q_z(161\text{ ft})$ for other walls
$G = 0.894$
$GC_{pi} = \pm 0.18$
$C_p = 0.8$ (windward)
-0.3 (N/S leeward)
-0.5 (E/W leeward)
-0.7 (side wall)

* See Excel Sheets for Pressure on Roof and Walls

Utilized Factors:

Zg = 900
α = 9.5
G = 0.894
$G_c p_i$ = +/- 0.18
C_p: windward = 0.8
N/S leeward = -0.3
E/W leeward = -0.5

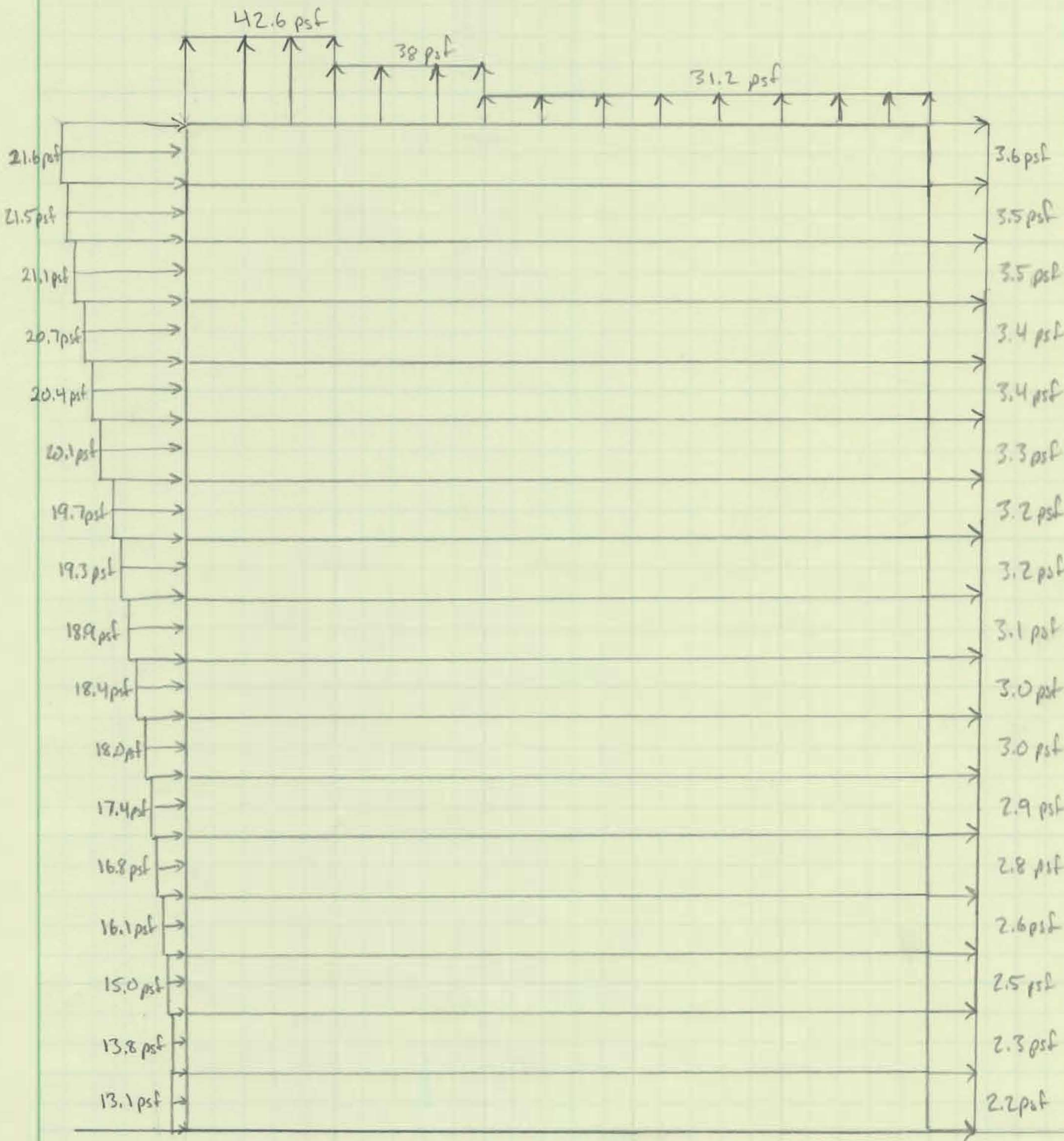
Building Dimension:

North/South = 135 ft.
East/West: Floors 1-4 = 192 ft.
Floors 5-17 = 184 ft.

Wind Force Determination - N/S							
Height	**Approx. Floor to Floor Ht.**	**kz**	**qz**	**Windward Pressure**	**Leeward Pressure**	**Trib Area**	**Force (K)**
10	9.5	0.849	24.429	13.074	-2.1546	1282.5	19.53112
19.5	9.5	0.897	25.816	13.817	-2.2770	1282.5	20.64026
29	11	0.975	28.066	15.021	-2.4754	1485	25.98193
40	9.5	1.044	30.032	16.073	-2.6488	1282.5	24.0107
49	9.5	1.089	31.343	16.775	-2.7644	1282.5	25.05877
59	9.5	1.133	32.592	17.443	-2.8746	1282.5	26.05793
68	9.5	1.167	33.581	17.973	-2.9619	1282.5	26.84852
77	9.5	1.198	34.472	18.449	-3.0404	1282.5	27.56036
87	9.5	1.229	35.369	18.930	-3.1196	1282.5	28.27801
96	9.5	1.255	36.110	19.326	-3.1849	1282.5	28.87017
105	9.5	1.279	36.798	19.694	-3.2455	1282.5	29.41999
115	9.5	1.303	37.509	20.075	-3.3083	1282.5	29.98887
124	9.5	1.324	38.109	20.396	-3.3612	1282.5	30.46838
133	13	1.344	38.675	20.699	-3.4111	1755	42.31316
146	12	1.371	39.442	21.109	-3.4788	1620	39.83271
158	12	1.394	40.103	21.463	-3.5371	1620	40.50063
161	2	1.399	40.262	21.548	-3.5511	270	6.776888
						SUM	472.1384

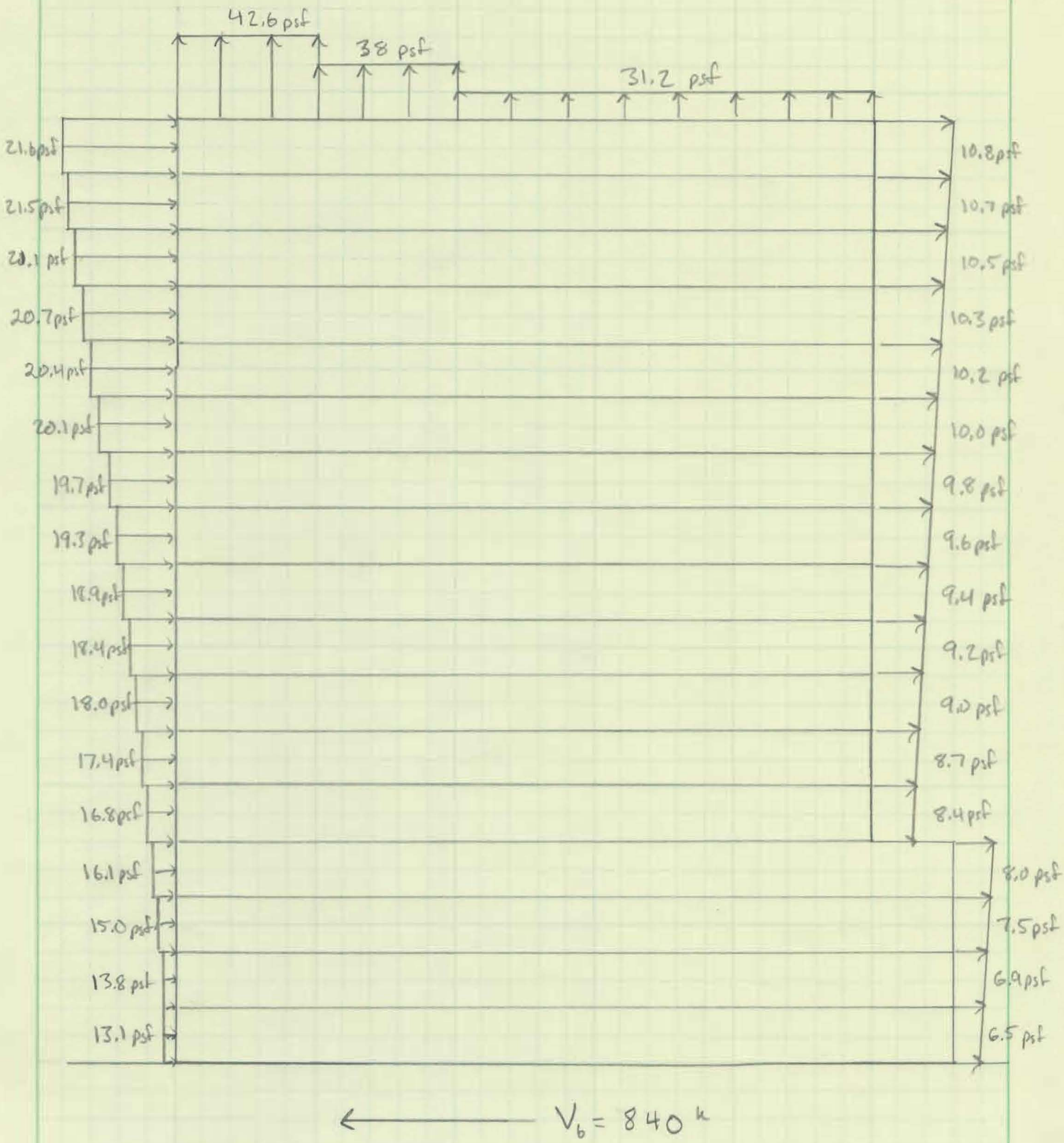
Wind Force Determination - E/W							
Height	**Approx. Floor to Floor Ht.**	**kz**	**qz**	**Windward Pressure**	**Leeward Pressure**	**Trib Area**	**Force (K)**
10	9.5	0.848884	24.42885	13.074	-6.5225	1824	35.74
19.5	9.5	0.897091	25.81613	13.817	-6.8929	1824	37.77
29	11	0.975267	28.06584	15.021	-7.4936	2112	47.55
40	9.5	1.043581	30.03175	16.073	-8.0185	1824	43.94
49	9.5	1.089133	31.34264	16.775	-8.3685	1748	43.95
59	9.5	1.13256	32.59236	17.443	-8.7022	1748	45.70
68	9.5	1.166921	33.5812	17.973	-8.9662	1748	47.09
77	9.5	1.19786	34.47155	18.449	-9.2039	1748	48.34
87	9.5	1.229052	35.36915	18.930	-9.4436	1748	49.60
96	9.5	1.254788	36.1098	19.326	-9.6413	1748	50.63
105	9.5	1.278686	36.79751	19.694	-9.8249	1748	51.60
115	9.5	1.303411	37.50904	20.075	-10.0149	1748	52.60
124	9.5	1.324252	38.10879	20.396	-10.1750	1748	53.44
133	13	1.343931	38.6751	20.699	-10.3263	2392	74.21
146	12	1.370577	39.44192	21.109	-10.5310	2208	69.86
158	12	1.393559	40.10329	21.463	-10.7076	2208	71.03
161	3	1.399089	40.26241	21.548	-10.7501	552	17.83
						SUM	840.89

Wind Pressure Elevation - N/S, in psf



$\longleftarrow \quad V_b = 472^k$

Wind Pressure Elevation - E/W, in psf



Seismic Loads

① Find Mapped Acceleration Parameters (Fig. 22-1-6)

$S_s = 0.125$
$S_1 = 0.055 > 0.04 \rightarrow$ Not assumed to be Seismic Design Category A

② Site Classification (Chapter 20)

Site Classification : C

③ Maximum Considered Earthquake Spectral Response Acceleration Parameters

$\left.\begin{array}{l} F_a = 1.2 \\ F_v = 1.7 \end{array}\right\}$ Table 11.4-1-2

$S_{ms} = F_a S_s = 1.2(0.125) = 0.15$

$S_{m1} = F_v S_1 = 1.7(0.055) = 0.094$

④ Design Spectral Parameters, Section 11.4.4

$S_{DS} = \frac{2}{3} S_{ms} = 0.10$

$S_{D1} = \frac{2}{3} S_{m1} = 0.062$

⑤ Importance Factor, Table 1.5-2

$I_e = 1.0$

⑥ Risk Category

Risk Category II $\rightarrow$ No special provisions (Section 11.6)

⑦ Seismic Design Category

Table 11.6-1 $\rightarrow$ SDC = A

Table 11.6-2 $\rightarrow$ SDC = A

∴ Use Seismic Design Category A

⑧ Exemption, Section 11.7

- Buildings with Seismic Design Category A are exempt from Seismic Design Criteria and must only comply with Section 1.4

Consider Section 1.4: General Structural Integrity...

Per Section 1.4.1-2

- A continuous load path has been provided for the building. The load path for the gravity system consists of mild reinforced and post-tensioned concrete slabs distributing the load to concrete columns and shear walls which transfer the loads down into the foundation. This load path is discussed in more detail in Technical Report #1.

The load path in the lateral system consists of a precast concrete or window glazing facade that transfers the lateral wind (controls over seismic) pressure the the concrete diaphragms at each floor. The lateral forces are distributed to 14 shear walls. The lateral load path is described in greater detail in Technical Report #1.

This section also requires adequate strength for the following load cases:

- $1.2D + 1.0N + L + 0.2S$
- $0.9D + 1.0N$
- $D + 0.7N$
- $D + 0.75(.7N) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R)$
- $0.6D + 0.7N$

- These cases and further analysis will be performed in the following Technical reports.

Per Section 1.4.3 - Lateral Forces

The story forces are given by

$$F_x = 0.01\, W_x$$

W_x = total dead load per story

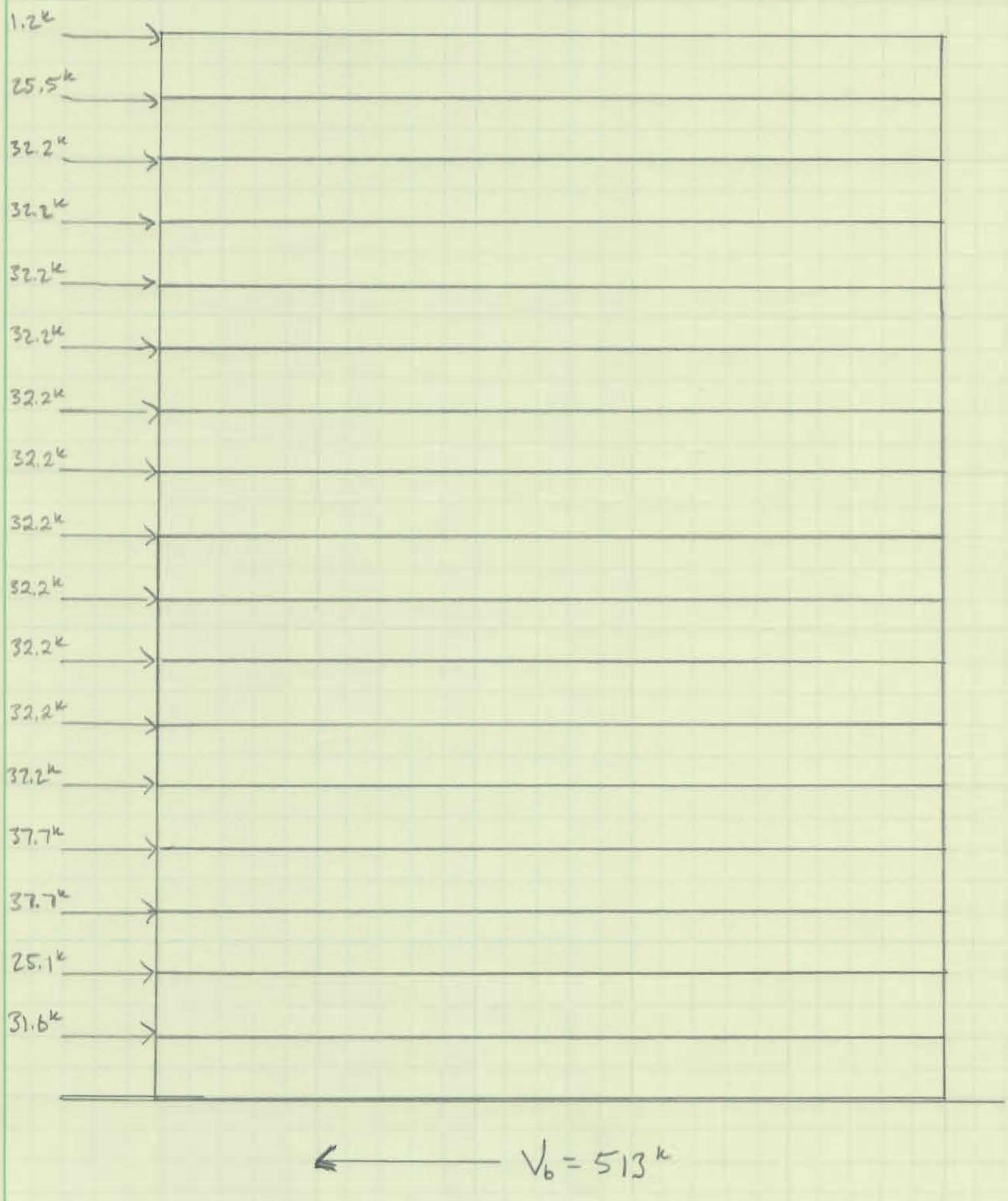
- See attached spreadsheet for story forces *

Seismic Force Determination				
Story	GSF	Story Weight (psf)		Force (kips)
1	21,076.00	150	0.01	31.614
2	16,746.00	150	0.01	25.119
3	25,136.00	150	0.01	37.704
4	25,136.00	150	0.01	37.704
5	21,479.00	150	0.01	32.2185
6	21,479.00	150	0.01	32.2185
7	21,479.00	150	0.01	32.2185
8	21,479.00	150	0.01	32.2185
9	21,479.00	150	0.01	32.2185
10	21,479.00	150	0.01	32.2185
11	21,479.00	150	0.01	32.2185
12	21,479.00	150	0.01	32.2185
13	21,479.00	150	0.01	32.2185
14	21,479.00	150	0.01	32.2185
15	21,479.00	150	0.01	32.2185
16	17,008.00	150	0.01	25.512
17	784.00	150	0.01	1.176
Fx = 0.01 * Wx			SUM	513.2325

Seismic Story Forces - N/S, in kips

Assumption:

- to estimate story weight, assume an average slab thickness of 10" concrete → 125 psf
- Add allowance for wall weights, columns, etc. + 25 psf
- Therefore, assume W_x = 150 psf × Area



Seismic Story Forces - E/W, in Kips

- With Simplified Method for buildings in a Seismic Design Category A, the seismic story forces are the same in both directions.